
# Design and Load Testing of Large Diameter Open-Ended Driven Piles

**DETAILS**

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

# NCHRP SYNTHESIS 478

# Design and Load Testing of Large Diameter Open-Ended Driven Piles

## *A Synthesis of Highway Practice*

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**Cover figure:** Pier 6 and 7 piles, 42-in. diameter open-ended steel piles for Hastings Bridge, Minnesota (Dan Brown and Associates, PC).

## FOREWORD

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

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

## PREFACE

*By Tanya M. Zwahlen*
*Consultant*
*Transportation*
*Research Board*

This report compiles and documents information regarding the current state of practice with respect to the selection, use, design, construction, and quality control of large diameter open-ended driven piles (LDOEPs) for transportation structures. This report is intended to provide agencies with information to develop guidance and methods to be incorporated into technical guides and design codes, as well as to identify gaps in knowledge to guide future research.

Information used in this study was acquired through a literature review, personal experiences, a survey of public agencies, consultations with private-sector experts, and documentation of case histories where piles have been employed for bridge construction.

Dan A. Brown and W. Robert Thompson III, Dan Brown and Associates, Sequatchie, Tennessee, collected and synthesized the information and wrote the report. The members of the topic panel are acknowledged on the preceding page. This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As progress in research and practice continues, new knowledge will be added to that now at hand.

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# DESIGN AND LOAD TESTING OF LARGE DIAMETER OPEN-ENDED DRIVEN PILES


## SUMMARY

As the design of bridge foundations has evolved to include issues of extreme event loadings (vessel collision, seismic event, liquefaction, scour, ice), large diameter piling has become a more attractive option because of the significant strength, ductility, and durability of these piles. Large diameter open-ended piles (LDOEPs) are steel or prestressed concrete cylinders 36 in. or larger in diameter that can provide large axial and lateral resistance even in relatively poor soil conditions. Steel LDOEPs have a long history of use for structural supports in the offshore oil and gas industry and have recently been employed for several high profile bridge projects, including the San Francisco–Oakland Bay Bridge, the Woodrow Wilson Bridge on the Potomac River, and the Tappan Zee Bridge in New York, which is currently under construction. Prestressed concrete LDOEPs are currently being used primarily for coastal structures in marine construction along the Atlantic and Gulf coasts because of their flexural strength and durability in harsh marine environments.

The design methodology for piling, along with testing and quality assurance procedures used in practice, is reflected in the AASHTO code; however, this code was not developed specifically for LDOEPs, and most transportation agencies do not have a robust base of experience with these piles. The AASHTO code has transitioned in recent years to the Load and Resistance Factor Design (LRFD) format to provide a more consistent basis for reliability of design, including extreme event loads and conditions; however, the reliability of the design and quality assurance procedures for LDOEPs has not been established. Uncertainties associated with LDOEPs are particularly important given that these piles are more likely to be employed on bridges that may be lifeline structures or are otherwise important.

The objectives of this synthesis are to present an overview of the current state of practice with respect to LDOEPs for transportation structures and to provide stakeholders in this industry with useful information supported by case histories. The information is also intended to provide agencies with a resource from which to develop guidance and methods into technical guides and design codes, as well as to identify gaps in knowledge that may provide direction for future research.

The information gathered in this synthesis includes background information from a literature review and personal experiences, a survey of public agencies within the United States, interviews with agency representatives as well as knowledgeable individuals from the private sector, and documentation of a range of case histories in which these piles have been employed for bridges in the United States. A survey was conducted of state geotechnical engineers (or equivalent) in all 50 state departments of transportation (DOTs) plus the District of Columbia and Puerto Rico, and 85% of these agencies responded to the survey. The synthesis also includes the findings from in-depth interviews that were conducted with seven of the 16 agencies that reported actively using LDOEPs. Additional interviews with private-sector participants involved engineers responsible for design, construction, and testing, including individuals with extensive experience in the offshore oil and gas industry, with a broad range of bridge projects, and with prestressed concrete LDOEPs. During the course of the survey and literature review, a number of interesting case histories were identified and summaries of 13 have been provided. These include bridge projects founded on LDOEPs, as well as case

histories of comparative testing programs that provide information of interest relating to the behavior and testing of LDOEPs.

The most important feature of LDOEPs affecting the behavior and understanding of these piles is the uncertainty related to the behavior of interior soil within the pile during installation and subsequent static loading by the permanent structure. When the hammer accelerates the pile rapidly downward, the soil inside tends to stay put because of the inertia associated with this large soil mass. As a result, there is a tendency for the pile to "core" during driving so that the pile does not incorporate the interior soil plug as a part of the pile even though this soil exerts some frictional resistance on the interior wall of the pile. For static loading long after the pile is installed, this behavior may be very different in that the frictional resistance on the interior pile wall may exceed the end-bearing resistance at the pile toe so that the pile advances as a "plugged" pile.

These behaviors affect the resistance of the pile during driving and thus the ability to predict pile hammer and equipment demands, and also affect our ability to predict the performance of the pile as a supporting element of the permanent structure. Since the driving resistance of the pile is used as an indication of long-term axial resistance for quality control and quality assurance, these different behaviors affect the reliability of our testing methods and thus the reliability of our completed design. The LRFD approach to design in the AASHTO code is intended to provide a reliability-based design methodology, and the resistance factors employed for LDOEPs are to logically reflect the reliability of this type of pile.

The agencies that use LDOEPs reported that large lateral and axial loads combined with certain favorable soil and rock conditions are the primary reasons for selecting these piles for design, with 12 agencies utilizing steel and seven utilizing prestressed concrete LDOEPs. Although the static prediction methods cited in the AASHTO code and FHWA *Driven Pile* manual were noted in this study to have little basis with respect to LDOEPs, these methods were by far the most used by agencies to estimate static axial resistance. Resistance factors for design were most typically selected to be consistent with AASHTO for other types of piles. A few states have developed their own procedures for estimating static resistance and selecting resistance factors, and two states use static methods from the American Petroleum Institute (API) design guidelines developed for offshore structures.

To verify the design in the field, most of the agencies rely on some type of driving criterion for final determination of pile length during installation, although three states at least sometimes install LDOEPs to a predetermined tip elevation without regard for driving resistance (most notably, Louisiana, where piles are often installed in deep, soft alluvial soils). Most of these agencies use high-strain dynamic testing to verify axial resistance and establish hammer operating procedures to minimize the risk of pile damage, although several agencies expressed a lack of confidence in the reliability of high-strain dynamic tests as an indication of static axial resistance for LDOEPs. Notably, the California Department of Transportation (Caltrans) has developed a system to apply loads of up to 8,000 kips in order to conduct static axial load tests on high-capacity piles such as LDOEPs. Some agencies employ rapid load tests that push the pile with lower inertial forces and many rely almost exclusively on high-strain dynamic tests for these high-capacity piles. In general, the states that make the greatest use of LDOEPs have also been most heavily engaged in testing, although a clear consensus has not yet emerged as to the most effective practices for testing.

The general consensus among the private practitioners who were interviewed was that LDOEPs do not tend to plug during installation, and there is a general lack of understanding in the industry related to the contribution of internal side resistance during installation as well as the plugging behavior during subsequent static loading. All acknowledged that there are unique challenges with dynamic testing for these types of piles, but all considered dynamic testing an important part of quality control/quality assurance for LDOEPs. The methods described in the API guidelines are most often employed for estimating static resistance by private industry.

The case histories demonstrate a varied use of LDOEPs for bridge projects, including steel pipe piles driven to bear on rock, long friction piles entirely in soil, and prestressed concrete cylinder piles for coastal structures where these piles allowed the use of pile bents and eliminated footings. Some of the case histories illustrate attempts to perform comparison tests with static, dynamic, and rapid load tests, reflecting a search for improved reliability in the design and testing of LDOEPs. A review of these tests contributes to improved understanding of pile behavior and the challenges, but do not yet portend a consensus solution to the problem.

Research needs were identified by all participants in an attempt to better understand the behavior of LDOEPs during installation and subsequent static loading, to improve the reliability and usefulness of testing, and to better quantify the reliability of these piles for LRFD-based design. The literature review documents more than 60 years of work in the offshore oil and gas industry to better understand the behavior of steel LDOEPs, which have been used extensively in that environment. The synthesis suggests that there is a knowledge gap in the transportation field with respect to the state of practice in piling design from this industry as it may apply to bridge foundations, but that there is also a need to adapt the general piling design practices from API to the specific reliability requirements for transportation infrastructure projects. Development of design procedures and resistance factors that are specific to LDOEPs for bridges is needed, and it is important that these reflect the reliability associated with testing for verification of axial resistance for these specific types of piles. Prestressed concrete LDOEPs differ significantly from the steel piles used by API and probably require considerations particular for these piles as compared with steel LDOEPs. Transportation agencies using LDOEPs recognize the need for guidance on testing requirements in order to achieve reliable and cost-effective foundations that meet the needs of modern transportation infrastructure.

CHAPTER ONE

# INTRODUCTION

## BACKGROUND

Synthesis Topic 45-05 gathered information on the current practices regarding the selection, use, design, construction, and quality control of large diameter open-ended piles (LDOEPs) for transportation structures. LDOEPs are cylinder-shaped piles driven open-ended; that is, with no plate or plug on the bottom of the pile, allowing soil to move up inside of the pile as it is driven into the ground. Piles of diameter 36 in. or greater were considered to be large diameter for this study.

Steel LDOEPs, often called steel pipe piles, are formed of steel plate that is bent into a tubular shape and welded to form the pipe. The most economical process for manufacturing typical pipe for use as LDOEPs is the spiralweld pipe, made by using a long coiled sheet that is twisted into a spiral and welded along the spiral seam in a continuous process. The welded tube is cut to individual pipe lengths. Alternatively, large diameter rolled and welded pipe can be made by rolling plate steel and welding the ends of the plate to form a tube, and then joining a series of individual tubes together to form a long pipe.

Concrete LDOEPs, often called cylinder piles, are typically manufactured using a spun-cast process whereby concrete with zero slump is placed within a rotating drum and spun to produce a very dense and durable concrete with low water-to-cement (*w/c*) ratio. Pile sections are typically 8, 12, or 16 ft in length. Each section contains ducts for post-tensioning after the concrete has cured. The sections are assembled, post-tensioned, the ducts grouted, and the cable ends trimmed at the precast yard prior to shipment. In recent years, some cylinder piles have been bed-cast in cylindrical forms using an interior form to create the void. Bed-cast cylinder piles may be prestressed in a manner similar to conventional concrete piling.

As the need to support larger lateral, seismic, and axial loads increases, designers and contractors are moving toward LDOEPs as one of the foundation types to consider resisting these loads. LDOEPs can be chosen over similar size drilled shafts and/or pile groups to address constructability and/or environmental concerns in certain circumstances or conditions. The same advances in construction equipment that have benefitted the installation of large diameter drilled shafts have also allowed LDOEPs to be a cost-effective solution for some of the same loading conditions. However, neither AASHTO nor FHWA design references provide any specific guidance for the selection, cost analysis, design, and construction of LDOEPs.

A significant issue with LDOEP design is that the current Load and Resistance Factor Design (LRFD) methods are based on small diameter piles, typically 24 in. or less. These design methods do not account for the different soil–pile–hammer interaction that occurs for large diameter driven piles as compared with the smaller diameter piles. It is important that design methods account for the influence of diameter and pile wall thickness, the degree of soil plugging or internal skin friction that exists, non-linear vibration effects, and scalability. There is also uncertainty as to whether the resistance factors used for small diameter piles are also valid for LDOEPs.

In addition to uncertainties in static design methods, significant questions have been raised by designers and contractors regarding drivability issues and the results of dynamic testing on LDOEPs. The potential for damage of LDOEPs driven to bear on rock is a concern and such damage at the toe of the pile is often difficult to detect or recognize. On some recent projects, dynamic tests have reportedly indicated much lower resistance values than were anticipated; alternative static or rapid load tests have sometimes demonstrated resistance values significantly higher than indicated by the dynamic testing. On rare occasions with piles in deep cohesive soils, the opposite pattern has been reported. Because static load tests on LDOEPs are expensive and therefore relatively uncommon, dynamic testing of these piles is of great interest. The same questions concerning the influence on size, wall thickness, plugging, etc., could be evaluated and accounted for in the analysis of dynamic testing results.

Although not as common in the transportation sector, other industries currently use LDOEPs for foundation systems. The offshore petroleum industry has a long history of using LDOEPs, including a body of research on estimating pile capacity and improving installation methods. The U.S. Army Corps of Engineers has a long history of using LDOEPs in flood control projects, most recently on several projects around New Orleans, Louisiana; repairing and improving the flood protection system of the city and surrounding areas. One other industry is the energy sector, where LDOEPs are used for transmission structures and other applications.

Although these other industries have developed practices based on their experiences, the applications to the transportation sector can be limited. The practices of these industries do not necessarily address the same concerns as for transportation projects, where it is primarily designed to drive LDOEPs to bear in some defined bearing stratum. The other industries typically have other major concerns, such as very high lateral loading and deep soft soils with no bearing. Practices or information seen as beneficial for the transportation sector will be addressed in this report. Proprietary products are mentioned in this report as they are essential to the subject of the report. No endorsement of these products is intended.

## STUDY PURPOSE AND OBJECTIVES

A study of current practices is needed as transportation agencies apply the LRFD platform for this type of deep foundation system (FHWA 2013). The project objectives are:

1. To locate and assemble documented practices and experiences on the use, selection, design, construction, quality control, and performance of LDOEPs;
2. To learn what practices have been used for estimating and verifying resistance of LDOEPs;
3. To learn what problems remain largely unsolved; and
4. To organize, evaluate, and document the useful information that is acquired.

This study will provide departments of transportation (DOTs), contractors, and private practitioners with useful information on how others select, design, and install LDOEPs, supported by case histories. The study will also be a resource for AASHTO and FHWA in developing specific guidance and methods into technical guides and design codes. The proposed research plan to accomplish these objectives is outlined here.

## KEY ISSUES TO BE ADDRESSED

Some of the key issues addressed by this report are:

- Conditions under which LDOEPs are or are not considered and selected (e.g., cost and design-build project delivery);
- Sizes, materials, and corrosion protection of piles (e.g., steel, concrete, and concrete-filled pipes);
- Typical manufacturing methods for LDOEPs;
- Type of static analysis method(s) considered during the initial design for LDOEPs;
- Assumptions made in design regarding plug formation;
- Type of methods considered to determine pile nominal resistance and displacements both axial and lateral;
- The resistance factors used in the design;
- Drivability analysis;
- Type of driving systems and hammers, field methods considered for quality control and pile resistance verification, problems that occurred during construction and how they were addressed;
- Availability of standard plans and indices regarding LDOEPs;
- Availability of construction specifications for LDOEPs;
- Availability of performance data of LDOEPs and the transportation structures they support during and after construction (e.g., settlement or displacement data);
- Pile set-up and relaxation;
- Availability of data for full-scale static, rapid, and dynamic load tests on LDOEPs;
- Research performed on design and/or load testing of LDOEPs; and
- Full-scale load tests of LDOEPs, including follow-up interviews with select DOT/Ministry of Transportation practitioners and/or private-sector experts to provide their first-hand experiences.

## STUDY APPROACH

This Synthesis project gathered relevant information through a comprehensive literature review plus surveys and interviews of practitioners in state DOTs and private practice. Case histories of the successful use of LDOEPs were obtained through the literature review, interviews, and the survey processes. The approach to gathering the information used the following methods:

- Literature review of state and international practice,
- Survey of state DOTs with response rates,
- Interviews with public-sector transportation practitioners,
- Interviews with practitioners in non-transportation industries, and
- Collection of project case histories and load test case histories.

## SYNTHESIS ORGANIZATION

The synthesis report presents the background information first, followed by the results of the surveys, and case history interviews. This chapter (chapter one) provides a brief introduction to the issues, the approach to the synthesis, and the basic outline of the report. Chapter two contains a summary of current practices and significant issues for design and installation of LDOEPs. The comprehensive literature review of U.S. and international sources is included in this chapter to establish background information for each of the significant issues. The literature review focused on the range of practices that have been, and are now being, pursued with respect to design and installation of LDOEPs.

Chapters three and four include the summary of experiences of public and private practice, respectively, obtained from the survey and interview process. Chapter three focuses

on the state transportation agencies, discussing the results of the survey and interviews arranged according to the topics in the survey. Several individuals in private practice with significant experience in the design and construction of LDOEPs were interviewed to include perspectives of those outside of the public sector. Chapter four presents the summary of the information gathered from those interviews.

The case histories are presented in chapter five. During the course of the literature review and the interviews with agency and private-sector entities several case histories of uses of LDOEPs were obtained. Many of the case histories included load testing of LDOEPs. The intent of this chapter is to illustrate successful testing and use of LDOEPs, as well as some of the lessons learned by owners, designers, and contractors.

The report concludes with a summary of the information gathered and identification of areas needing further study or research presented in chapter six. The appendices to the report contain a summary report of the survey data and notes from the telephone interviews.

CHAPTER TWO

# STATE OF PRACTICE AND LITERATURE REVIEW

## INTRODUCTION

LDOEPs are prefabricated tubular steel or prestressed concrete cylinder piles that are 36 in. outside diameter or larger and are driven into the subsurface to provide axial and lateral foundation support for the structure. These piles present a unique challenge for foundation designers owing to the combination of several factors:

- The tendency of the piles to "plug" during installation is uncertain and may affect the behavior during installation,
- The potential for installation difficulties and pile damage during driving is unlike other types of conventional bearing piles,
- The soil plug within the pile may behave differently during driving or dynamic testing compared with static loading,
- Axial resistance from internal friction, and
- The nominal axial resistance may be very large and therefore verification with conventional load testing is more challenging and expensive.

This chapter provides an overview of the state of the practice with respect to the use of these piles for transportation structures, including a review of published literature on the subject.

## SELECTION OF LARGE DIAMETER OPEN-ENDED PILES (LDOEPs) FOR TRANSPORTATION STRUCTURES

LDOEPs have primarily been used for bridge structures where one or more of the following conditions exist:

- Lateral load demands on the foundation are relatively high, often as a result of extreme event loading conditions such as vessel collision or seismically induced lateral forces.
- The piles are subject to a significant unsupported length as a result of scour, liquefaction, or marine conditions.
- Soils are relatively weak to a fairly substantial depth.
- Axial demand on the foundation is high.
- The use of a single large diameter pile can eliminate the need for a footing, such as to allow the use of a pile bent for a pier substructure.
- Marine construction conditions are implemented for pile delivery, handling, and installation.

LDOEPs are particularly favorable where large lateral demands must be resisted because of the significant flexural strength that is efficiently provided by a large diameter cylindrical shape formed of high-strength engineered materials. In addition, these piles provide the advantages of ductility where seismic stresses may be high. Bridges exposed to deep scour can result in long unsupported pile lengths and high bending stresses. Liquefaction conditions associated with seismic events increase flexural strength demand on piling. Vessel collision forces and other extreme event loadings can demand large lateral resistance from foundations and therefore favor the use of LDOEPs.

It is not unusual for LDOEPs and drilled shafts to be compared as alternatives in many such cases (S&ME 2008), because many of the conditions cited previously also favor the use of drilled shaft foundations. The factors that most favor LDOEPs over drilled shafts are the presence of deep weak soils and/or marine construction conditions. Drilled shafts are most cost-effective where a strong bearing stratum exists that can be engaged to provide resistance. Conditions where lateral resistance requires embedment into a hard stratum such as rock are not favorable for driven LDOEPs; however, if the rock is deep and lateral resistance is provided by overburdened soils, LDOEPs offer a potentially simpler and faster method of constructing deep foundations.

The use of prestressed concrete cylinder piles for transportation structures has been concentrated along the Gulf and Atlantic coasts, with cylinder pile sections of 36 in. to 66 in. outside diameter driven to bear in coastal alluvial sediments. A typical application for prestressed concrete LDOEPs might be to use these piles to construct pile bents for a viaduct across coastal marshlands or a shallow bay, where the flexural strength and durability of the concrete cylinder section provides a simple and repetitive means of constructing the bridge without the need for cofferdam structures and footings in the water. A marine environment with water access to the site is conducive to delivery and installation of large concrete cylinder piles, and the durability and corrosion resistance of these piles provides advantages in such an environment.

Steel LDOEPs have been used nationwide on transportation structures where the relative ease of installation combined with the ductility and flexural strength of these piles provide an advantage over alternative foundation types. A typical application for steel LDOEPs may be where an extreme

event loading such as a seismic event or vessel collision results in high foundation loadings. Several such examples are described in subsequent sections of this report.

Historically, the use of LDOEPs for offshore oil platforms provides a reference base for design of steel pipe piles, particularly for long friction piles in clay. Much of the understanding of the behavior of LDOEPs during driving and subsequent axial loading has come from the literature surrounding the offshore industry (Randolph 2003; Lehane et al. 2005b; Stevens 2010; API 2011). However, the use of LDOEPs for transportation structures differs from offshore applications in several ways. More favorable soil conditions for axial resistance most often exist at bridge sites compared with offshore, as offshore conditions often include deep deposits of soft clay. The relative costs, construction equipment availability, and schedule demands are different, as are the water depths and service life requirements. Offshore piles tend to be primarily shaft resistance piles so that base resistance is not as crucial, as often the case for transportation structures. Nevertheless, the offshore experiences are of great value in the design of LDOEPs for transportation structures.

## TYPES AND CHARACTERISTICS OF PILES

Prefabricated tubular steel or prestressed concrete cylinder piles represent virtually all of the LDOEPs used in transportation structures. The manufacture, specification, and handling of these two pile materials are described here.

### Steel Pipe Piles

Tubular steel LDOEPs are formed of steel plate that is bent into a tubular shape and welded to form the pipe. The most economical process for manufacturing typical pipe for use as LDOEPs is the spiralweld pipe, made by using a long coiled sheet that is twisted into a spiral and welded along the spiral seam in a continuous process. The ends of successive coils are straightened and joined before spiraling, and so the spiralweld pipe comes from the mill as essentially an endless pipe that is cut to individual pipe length (Figure 1). Alternatively, large diameter rolled and welded pipe can be made by rolling plate steel and welding the ends of the plate to form a tube, and then joining a series of individual tubes together to form a long pipe.

Spiralweld pipe is typically available in sizes up to 10 ft in diameter, with steel thickness of up to 1 in. and is most often specified by grade with reference to ASTM A252 (ASTM 2010). Grades 1, 2, and 3 have specified yield strength of 30, 35, and 45 ksi, respectively. A252 Grade 3 (modified) can also be obtained with yield strength of 50 to 80 ksi. Thickness of spiralweld pipe larger than 1 in. is not common because of spiral mill capabilities; therefore, rolled and welded pipe is more typically used for greater steel thickness and larger diameters.

FIGURE 1 Spiralweld pipe: straightening the coil (*top*); note coils in background; and welding the seam (*bottom*) (*courtesy*: Skyline Steel).

The seams of spiralweld pipe can be welded from both inside and outside in the manufacturing process and can achieve a full penetration weld that has a strength no less than the steel coil material itself. However, it should be noted that A252 does not specifically require a full penetration weld; therefore, an added note in the agency specification is required to ensure that the spiralweld pipe is manufactured with full penetration welds at the seams.

The A252 specification also includes a somewhat generous tolerance on permissible variations in weight and dimensions relative to currently available manufacturing tolerances such that the weight of a pile can be as much as 5% under the specified weight. This difference represents a significant quantity on a large contract. Therefore, a cost conscious manufacturer would read this requirement as a minimum weight that is 95% of the specified weight and supply the material accordingly. As a result, some agencies specify a *minimum* pile weight rather than defaulting to the A252 standard in this respect.

It is also worth noting that the ends of spiralweld pipe sections represent a cut in the pipe from the process of manufacturing a continuous pipe. When sections are to be subsequently spliced together in the field, it is advantageous to mark the cut ends so that these pieces can be re-joined at the same location and thus provide a better fit for field welding.

Rolled and welded pipe is less efficient to produce on a large scale than spiralweld and piles produced using this method typically are more expensive on a material basis. This process is often employed for manufacturing pipe for thicknesses that exceed 1 in., or very large diameter pipe as might be used for offshore piling or drilled shaft casing. Steel plate is rolled to produce a tubular shape as shown in Figure 2 and then welded at the ends to produce a straight seam.

As the individual cans are welded to form the pipe pile it is a relatively simple matter to modify the wall thickness with length so that greater wall thickness can be provided where needed. For instance, very long piling for offshore platforms often incorporate a greater wall thickness in the upper portion of the pile where flexural strength demand is greater. It is also possible to include a thicker bottom when driving LDOEPs to bear on rock.

Corrosion resistance is an issue with steel piles, especially where exposed to air and/or water above the ground surface. The splash zone or tidal fluctuation zone in a salt water environment is a particularly harsh environment for corrosion of exposed steel piling. Below the soil surface, the presence of high chloride, sulfate ion concentration, or low measured soil resistivity represents aggressive environments for corrosion of steel piling. The 2006 FHWA reference manual on driven piles (Hannigan et al. 2006) provides a summary of current practices with respect to corrosion of steel piling. *NCHRP Report 408* summarizes a research study of corrosion of steel piling in non-marine applications (Beavers and Durr 1998). The current standards for the evaluation of this subject are given in AASHTO Standard R 27-01 (2010), which provides a recommended assessment procedure for evaluating corrosion of steel piling in non-marine applications. Corrosion assessment for exposed steel piling in marine environments requires evaluation by corrosion specialists.

The most common means of addressing corrosion with steel LDOEPs is to provide some allowance of steel loss over the design life of the structure. Coatings can be considered, although the survival of a coating through the pile handling and driving process may pose a challenge. For the portion of steel piling that may be exposed above the soil surface it may be possible to remove soil from within the steel LDOEP and fill the interior with reinforced concrete to a suitable depth below grade.

#### Prestressed Concrete Cylinder Piles

The use of prestressed concrete cylinder piles for transportation structures has been concentrated along the Gulf and Atlantic coasts, with a typical wall thickness of 6 to 6.5 in., available in sizes ranging from 36 in. to 66 in. outside diameter. A typical application might be to use these

FIGURE 2 Rolling plate for pipe (*left*) and joining cans (*right*) (*courtesy*: Skyline Steel).

piles to construct pile bents for a viaduct across coastal marshlands or a shallow bay, where the flexural strength and durability of the concrete cylinder section provides a simple and repetitive means of constructing the bridge without the need for cofferdam structures and footings in the water. A marine environment with water access to the site is conducive to delivery and installation of large cylinder piles.

These piles are often manufactured using a spun-cast process (Figure 3), whereby concrete with zero slump is placed within a rotating drum and spun to produce a very dense and durable concrete with low *w/c* ratio. Concrete with 8,000 psi compressive strength is routinely produced with this process and the high-strength post-tensioning strands may be used to prestress the concrete to 1500 psi or greater. Confinement of the strands is typically provided by a spiral wire, and stainless wire may be used in the portion of the pile subject to marine environment. The concrete cylinders are typically 8-, 12-, or 16-ft long sections that contain ducts for post-tensioning after the concrete has cured. The sections are assembled, post-tensioned, the ducts grouted, and the cable ends trimmed at the precast yard prior to shipment. An adhesive is typically used (in combination with the post-tension forces) to seal each joint before pressure grouting the ducts. At the time of this writing (2014), there are two known facilities producing these piles: Gulf Coast Prestress in Pass Christian, Mississippi, and Bayshore Concrete Products Corp. in Cape Charles, Virginia. Cylinder piles fabricated using this technique have been installed in one piece with lengths exceeding 200 ft, the primary limitation being the availability of a crane to lift, set, and drive the pile.

In recent years, some cylinder piles have been bed-cast in cylindrical forms using an interior form to create the void. Although the concrete does not benefit from the density achieved by the spun-cast technique, the bed-cast method has been used to fabricate piles with wall thickness up to 8 in. and thus achieve greater cover. One challenge with this method of fabrication is to maintain alignment on the interior forms and achieve good filling in the spaces within the formwork. Bed-cast cylinder piles may be prestressed in a manner similar to conventional concrete piling. It may be noted that 36-in.-square prestressed concrete piles are cast in the same way with a 24-in. central void to form a type of LDOEP that has a square outer shape with a cylindrical void.

Corrosion resistance of concrete cylinder piles is generally considered to be very good, especially with the spun-cast concrete process, so long as pile damage is avoided during installation. Lau (2005) summarized the examination of three 40-year-old cylinder pile-supported bridges in Florida and found only minor or no corrosion distress of the spiral reinforcement or strand in the piles, in spite of small clear concrete cover values of only 0.4 to 1.5 in. Additional durability is provided by the grouted ducts surrounding the post-tensioning strands and stainless spiral confinement wire can also be used. The avoidance of cracking caused by pile driving appears to be a major factor in achieving durability with concrete cylinder piles.

FIGURE 3 Spun-cast concrete cylinder piles; during casting (*top*; courtesy: Gulf Coast Prestress), after casting and still in forms (*center*) and during post-tensioning (*bottom*).

## FACTORS AFFECTING DESIGN AND AXIAL RESISTANCE

This section provides an overview of those factors affecting the behavior of LDOEPs that are different from conventional smaller piling used in transportation structures. Besides the large diameter compared with most conventional piling, the

uncertainty in behavior associated with the soil plug within the pile during driving, testing, and subsequent static loading represent challenges that are unique to LDOEPs. Installation and performance of prestressed concrete LDOEPs present some unique conditions relative to steel, and an overview of these features is briefly described at the end of this section.

The installation of a large diameter pipe pile engages soil resistance to penetration on both the outside and the inside of the LDOEP. When a large pipe pile is driven into the soil, the hammer imparts a compression wave onto the pipe, which accelerates the pipe downward relative to the soil. For the pile to penetrate during the blow, it must overcome the frictional resistance at the pile/soil interface along the outside wall of the pipe. The soil within the inside of the pipe also resists the downward forces exerted by the pipe at the interior pile/soil interface, not only because of the base resistance near the pile toe, but more importantly because of the inertial resistance of the soil mass within the pipe. A simplified explanation of this effect is provided here.

### A Simplified Examination of the Dynamic Behavior of a Soil Plug

To understand the behavior of an LDOEP during installation and axial loading it is important to consider the behavior of the soil plug within the interior of the pile. Although there have been numerous papers analyzing the static behavior of the soil plug within a pipe pile, the behavior of the soil plug during installation involves some additional consideration of inertial effects. Because of the inertial resistance of the soil plug to downward acceleration, it is common that an LDOEP may advance without plugging during installation even though the pile may behave like a fully plugged pile during static loading, as illustrated on Figure 4.

The pipe is accelerated downward by the action of the hammer. The soil inside the pipe feels side resistance from the pipe as it moves downward and even without any force from below the inertia of the soil mass resists the forces applied by the pipe. For a unit length of pile, the side resistance force, $q_s$ is:

$$q_s = \pi d_i f_s \tag{1}$$

Where:

$d_i$ = inside diameter of pile, and
$f_s$ = unit side resistance at pile/soil interface on the inside of the pile.

The mass, $m$, of the soil plug per unit length is:

$$m = \frac{\pi d_i^2 \gamma_t}{4g} \tag{2}$$

Where:

$\gamma_t$ = total unit weight of soil, and
$g$ = acceleration of gravity.

Assuming zero net force acting on the top and bottom of the plug, the soil plug will then slip when the acceleration of the pile, $a_{slip}$, is such that:

$$F = ma_{slip} \tag{3}$$

And therefore:

$$\pi d_i f_s = \left(\frac{\pi d_i^2 \gamma_t}{4g}\right) a_{slip} \tag{4}$$

Rearranging Eqs. 2–4 to solve for acceleration, $a_{slip}$:

$$\frac{a_{slip}}{g} = \frac{4 f_s}{d_i \gamma_t} \tag{5}$$

A typical value for total unit weight is 0.125 ksf, therefore,

$$\frac{a_{slip}}{g} \cong \frac{32 f_s}{d_i} \tag{6}$$

Where: $f_s$ is in ksf and $d_i$ is in ft.

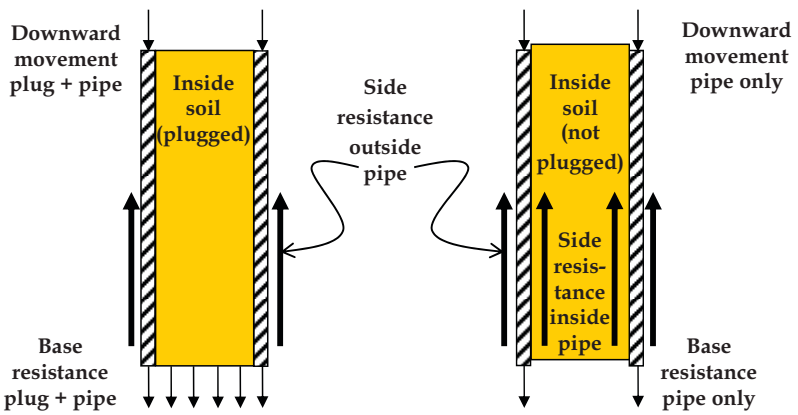


FIGURE 4 Schematic of a soil plug inside a pipe pile.

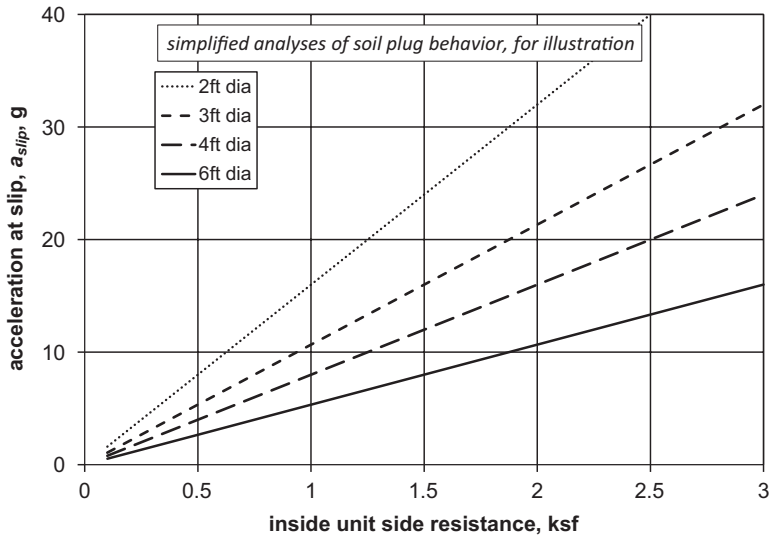


FIGURE 5 Acceleration that would cause slip of a soil plug inside an open pipe pile.

This equation provides the simple results illustrated in Figure 5. Since measurements indicate that the acceleration of a large diameter steel pipe pile during driving is likely to be higher than 30 g [Stevens (1988) reported accelerations averaging 178 g] and the unit side resistance on the inside would rarely be expected to be as high as 3 ksf, it is logical to expect that pipe piles larger than 3-ft diameter would rarely be expected to plug during driving. The larger the diameter, the less likely the pile will plug since the acceleration to cause slip is lower with increasing diameter.

The obvious conclusion of this simplified analysis is that plugging is unlikely for large diameter driven pipe piles. This conclusion is consistent with a point made in the 2003 Rankine Lecture by Randolph who noted: "the observations that, under dynamic conditions of pile driving, the soil plug does indeed appear to progress up the pile, with only small variations in the position of the top of the soil plug relative to the original ground surface." As a point of reference, consider that the area ratio (ratio of the pile cross section to the area within the outside diameter of the pile) of a 48-in.-diameter steel pipe pile with a ¾ in. wall thickness or a 72-in.-diameter steel pipe pile with a 1.125-in. wall thickness is around 6%; this value is less than that of a suitable thin-walled tube sampler according to ASTM D1587 (ASTM 2012) (about 8.5% for a 3-in. Shelby tube) used to obtain "undisturbed" soil samples for laboratory testing.

For static loading (zero acceleration), there is no inertial resistance to plugging and the only possible mechanism to push the soil plug into the pipe would be the base resistance mobilized at the pile toe.

It can be noted that the simplified analysis of pile plugging provided earlier is only intended to assist the reader in understanding some fundamental aspects of the problem, and not intended for use in design. The actual dynamic behavior of the plug is more complex than described, because the acceleration and inertia of the pile and plug vary with time as compression and tension waves move up and down the pile (Rausche and Webster 2007). It appears plausible that at least some transient penetration of the plug near the toe can occur during driving if sufficient downward traction is applied by the pile and the base resistance on the bottom of the soil plug is low; for example, a pile penetrating through a sand layer into a clay stratum below might have relatively high internal side resistance as a result of arching near the toe corresponding to low base resistance below the plug.

It is clear that plugging behavior in small diameter pipe piles and prestressed concrete LDOEPs (which have thicker walls and smaller inside diameter) may occur under circumstances in which plugging would not occur for large diameter pipe piles, and therefore observations of pile performance on smaller piles may not properly extrapolate to larger piles. A smaller diameter pile that behaves as a plugged pile during installation may displace a larger volume of soil relative to the pile volume compared with an LDOEP and this condition could influence the unit side resistance, the state of stress in the ground around the pile, the pore pressures generated, and the magnitude and time dependency of setup.

### Issues Affecting Behavior of Steel LDOEPs During and After Installation

A number of other factors have been observed or postulated to have a significant influence on the behavior of steel LDOEPs during and after installation. Many of these are related to the plugging effect and others are related to the size, shape, and length of LDOEPs, as described in the following paragraphs.

*Base Resistance of Steel LDOEPs on Rock and Driving Shoes*

The base resistance of driven steel pipe piles has been observed to be relatively low until the pile is installed to bear on rock or a similar hard bearing stratum, suggesting that the base resistance is largely dependent on the bearing area of the pile wall itself. Dasenbrock (2006) described observations of unexpectedly low axial resistance of 42-in.-diameter steel pipe piles that were intended to be driven to bear in sand, with the result that the piles were quite easily driven to refusal to a deeper bedrock stratum to achieve the required nominal resistance.

Where steel LDOEPs are driven to bear on rock or other hard materials it is quite common to employ a "driving shoe" composed of a ring of steel with greater thickness at the pile toe. If this thickness were to result in a large outside diameter, the effect would be that of a "friction reducer" (as might be used above a cone penetration probe to reduce rod friction), with potentially adverse effects on the nominal side resistance of the pile. Most engineers recognize this undesirable consequence and therefore use a driving shoe that matches the pile outside diameter and results in a reduced inside diameter. The reduced inside diameter has a similar friction reducing effect on the side resistance within the soil plug and, as a result, the pile is even more likely to drive in the unplugged condition. The long-term impact of this friction-reducing effect may also affect the tendency of the pile to plug during subsequent static loading; however, the use of a driving shoe is generally employed only where the pile is to bear on rock or other hard bearing strata, and so plugging is generally of little consequence in such circumstances.

The use of a driving shoe that is only a few inches tall may be ineffective in avoiding pile buckling at the toe of steel LDOEPs driven to bear on rock, as evident from Figure 6. A large diameter steel pipe pile driven to bear on rock can quite easily encounter rock on one small portion of the pile toe such that stress concentrations occur on the steel shell. One-dimensional analyses of a pile using wave equation techniques to predict pile stresses do not directly account for this non-uniform distribution of stress across the toe. Piles driven through soft soils to bear on a sloping rock surface probably represent the worst possible case for this condition, as a soil plug of weathered rock, till, or even very dense sand may help lessen the risk of buckling to some degree.

FIGURE 6 Buckling at the toe of steel LDOEP (*courtesy*: Bengt Fellenius).

One effective mitigation strategy that has been employed for steel LDOEPs bearing on rock include the use of a thickened bottom section of steel for a length of around 1.5 to 2 pile diameters (M. Holloway, personal communication with D. Brown, Dec. 2013). Another strategy is to "seat" the pile onto rock using a large number of relatively low energy blows from the hammer in an attempt to achieve more complete contact with the rock at the pile toe, followed by only a few hard blows to confirm bearing onto the rock (B. Fellenius, personal communication with D. Brown, Dec. 2013). These strategies have been successfully adopted for installation of 6-ft-diameter steel pipe piles at the new Tappan Zee Bridge after an initial observation of pile damage of a dynamic test pile (Palermo and Reichert 2014).

*Vibratory Driving and Splicing*

Where steel pipe piles are used with lengths greater than 100 ft, it is not unusual that a field splice will be required. Splices of steel piling may be accomplished with full penetration welds so that the strength of the splice is equal to that of the pile itself. However, the time required to make the splice may be several hours or more, and so most contractors prefer to stage this work to maintain efficient utilization of pile driving equipment. A common practice is to install the first section of piling with a vibratory hammer and use the impact hammer only to achieve final driving to the required driving resistance.

Because the contractor may wish to use the vibratory hammer to the maximum extent possible, the agency may be confronted with questions related to the hammer requirements for bearing piles and the suitability of the use of vibratory hammers for installation. In general, where steel LDOEPs are installed to achieve the required axial resistance primarily by base resistance on rock or a similar suitable hard bearing stratum, the use of vibratory hammers for most of the pile length may not be considered objectionable. Likewise, the uppermost soil strata around a very long pile that may be spliced is likely to be contributing a relatively small proportion of the total side resistance. However, where a substantial portion of the axial resistance is designed to be provided by side resistance in the soil, there is evidence to suggest that vibratory pile installation may result in lower axial resistance (Briaud et al. 1990; Mosher 1990; Canivan and Camp 2002). Most of these comparative studies have been performed on steel H and smaller diameter open-ended steel pipe piles, and at least some of the differences have been attributed to a reduced contribution to axial resis-

tance at or near the pile base. However, it appears plausible that more extensive remolding of soil near the pile wall occurs in cohesive soils with vibratory installation. The vibratory action is thought to liquefy the soil within a narrow zone adjacent to the pile wall during penetration in sands and therefore may result in different relative density and/or horizontal stresses at the pile/soil interface than conventional impact driving.

### *Effect of Pile Length on Behavior and Axial Resistance*

There are several factors related to the length of pile that may have important effects on behavior. Considerable evidence in recent years (e.g., Randolph 2003; Jardine et al. 2005; Lehane et al. 2005a) suggests that the unit side resistance in both clays and sands can be diminished with increasing length, possibly attributed to: (1) continued shearing of a particular soil horizon during pile installation, (2) progressive failure in strain softening soil, (3) reduction in radial stresses with increasing distance above the pile toe, and (4) degradation resulting from densification and/or grain crushing associated with the cyclic shearing action of pile installation. These effects are incorporated to varying degrees in some of the methods for estimating static resistance used for offshore piling summarized by Jeanjean et al. (2010). It is noted that Karlsrud (2012) holds a contradictory opinion (for clay), concluding that pile length or flexibility does not appear to affect the local ultimate shaft friction in clay.

A long pile can become quite flexible during compression loading or impact driving, such that the pile undergoes large vertical displacements. The elastic compression of a steel pipe pile that may be 150 to 200 ft in length could easily result in 1 to 2 in. of displacement at the pile top relative to the base, and the long travel time for a compression wave during driving can result in a large number of cyclic stress reversals. At a given point in the soil, perhaps 100 ft below grade, a 180-ft-long pile will result in the soil at the pile/soil interface at that elevation having been subjected to a large number of cyclic stress reversals associated with the penetration of the pile 80 additional feet beyond that elevation. The effects of these many cycles of stress reversal appear to contribute to strain softening behavior at the interface, possibly as a result of degradation of clay soils to residual shear strength conditions. If the unit side resistance at the pile/soil interface of a long flexible pile exhibits strain-softening behavior, then progressive failure along the length of the pile can occur during static loading and the axial resistance degrades toward a residual condition (Randolph 2003).

Reduction in radial stresses with increasing distance above the pile toe has been documented in experiments on jacked piles by Lehane and Jardine (1994). Similar behavior in sands is described by White and Lehane (2004), who refer to the effect as "friction fatigue" and conclude that the primary mechanism controlling friction fatigue is the cyclic history imparted to the soil elements at the interface during pile installation. Karlsrud (2012) reviewed data from a wide range of pile load tests in clay and concluded that open-ended steel pipe piles generated lower earth pressures against the pile than did closed-ended piles and that reduction of radial effective stress could occur with consolidation of soil around the pile, as excess pore pressures dissipate.

Experimental evidence of the degradation of sands has been documented by Yang et al. (2010), suggesting that the effects of pile installation on side resistance in sands extend beyond changes in radial stresses and relative density. Calcareous sands can be notoriously brittle and subject to grain crushing, and combined with the low relative density and cementation can result in very low axial resistance of open-ended steel pipe piles as documented by Murff (1987).

### *Time-Dependency of Axial Resistance*

A time-dependent increase in axial resistance (setup) is known to occur with LDOEPs as is generally the case with all types of driven piles. The increase in axial resistance is affected by soil type, the volume of soil displaced during pile driving, and many other factors. Most of the data available on pile setup are based on smaller piles or displacement piles and, therefore, some differences in the time dependency of LDOEPs are likely when compared with experiences with other pile types. It is widely accepted that soil disturbance, pore pressures, and time required for consolidation around the pile increase with increasing pile diameter, but that large diameter steel pipe piles may not displace a large volume of soil if plugging does not occur. Relaxation of arching around the pile, cementation, and other ageing effects in the soil at the pile/soil interface extend beyond simple dissipation of pore pressures and consolidations and result in time-dependent strength gain and setup (Axelsson 2000).

Measurements of setup are often based on repeated dynamic tests over time, sometimes even on the same pile. Since LDOEPs tend to have high capacity, it can often be the case that the driving system may be at or near the limits to mobilize the axial resistance and thus the full setup may not be measured (Stevens 2004). Repeated dynamic tests on the same pile can produce degradation issues at the pile/soil interface in calcareous sands, as noted previously, with the result that the measurement of setup is adversely affected by the pile testing history.

### *Driving Resistance and Dynamic Load Testing*

The use of driving resistance as an indication of pile axial resistance has a long history in foundation engineering, and the development of dynamic testing techniques based on stress wave measurements have come into wide acceptance over the last 30 years. However, LDOEPs present some unique

challenges compared with the use and interpretation of conventional dynamic measurements.

The modeling of the behavior and inertial resistance of the soil plug within a large diameter pipe pile during driving presents a challenge unique to LDOEPs. Conventional practice is often based on the simple assumption that base resistance acts only on the annular base of the pile and that the internal and external side resistance are lumped together and considered as external side resistance (Randolph 2003). However, the response of the soil plug is different than that of the external soil as the inertial mass of the soil plug affects the mobilization of internal resistance. Paikowsky and Chernauskas (2008) describe techniques for modeling the soil plug within a pipe pile.

The potential for differences in the behavior of the soil plug during impact driving compared with static loading has been described previously, and dynamic testing measurements are subject to the same differences compared with static pile behavior if the pile does not plug during a dynamic blow but plugs during static loading. Ordinarily one would expect that the static base resistance of a plugged pile in soil could contribute significant axial resistance that might not be observed during dynamic testing. For long friction piles with low base resistance, it is feasible that the contribution of side resistance derived from the inertial resistance of the soil plug could result in a greater contribution to axial resistance than that of the base resistance from a plugged pile during static loading (M. Holloway, personal communication with D. Brown, Dec. 2013). Static load testing provides the most direct means to measure the static behavior of a test pile, but with the magnitude of loads required to test high-capacity LDOEPs the costs of static tests are very high. The use of a longer duration and lower g force pulse such as the rapid load test method offers advantages in that the pile is more likely to exhibit plugged behavior during a test with lower inertial forces (Muchard 2005).

There have been a few attempts to promote plugging within LDOEPs by the use of a partial steel plate within the pile so that the plate allows water through, but engages the soil at some point and presumably makes the piles drive as a full displacement pile.

Another factor affecting dynamic measurements on long piles is the potential for residual stresses to affect behavior. Residual stress analysis in the wave equation analysis of piles is fairly well established, but not often performed in routine practice. However, where long, slender, and relatively flexible LDOEPs are analyzed, the use of residual stress analysis is significantly more realistic than a standard model (Rausche et al. 2010).

For long LDOEPs, where high base resistance is achieved on rock or other hard bearing strata, it may be quite easy to drive the pile to achieve good bearing on the rock, but the axial resistance that can be observed during dynamic testing can be limited by the ability of the hammer and driving system to mobilize the resistance. This limitation of the hammer can result in a misinterpretation of a dynamic measurement on a restrike blow to conclude that relaxation at the pile toe has occurred when the reality is that the setup in side resistance has diminished the energy reaching the pile toe and thus the mobilized base resistance is reduced upon restrike. In such a case, superposition may be justified (Hussein et al. 2002).

Dynamic measurements can be particularly valuable with respect to the detection and avoidance of pile damage during installation. With large diameter piles, hammer alignment on the top of a pile can be more of a challenge. Steel LDOEPs can easily be overstressed at the pile toe when driven to bear on an uneven rock surface as discussed previously. Dynamic measurements can be helpful in detection of damage at the toe of a steel pile; however, damage at the toe is notoriously difficult to detect right away because the reflection from the damage returns at almost the same time as the reflection from the pile toe anyway. Dynamic measurements can identify the onset of a strong base resistance as the pile toe encounters rock, and this identification can be very helpful in controlling the hammer operation to avoid damage.

### Issues Affecting Prestressed Concrete LDOEPs During and After Installation

Although many of the issues described previously apply generally to all LDOEPs, there are some specific issues unique to prestressed concrete LDOEPs, as described in the following paragraphs.

#### *Pile Volume and Prestressed Concrete LDOEPs*

The magnitude of the soil displaced by the pile has an effect on the axial resistance, particularly for piles installed in sandy soil profiles, and even an unplugged prestressed concrete LDOEP may displace a considerable volume of soil. As mentioned previously, the area ratio (ratio of the pile cross section to the area within the outside diameter of the pile) of a 48-in.-diameter steel pipe pile with a ¾-in. wall thickness is around 6%, which is less than that of a suitable thin-walled tube sampler according to ASTM D1587 (about 8.5% for a 3-in. Shelby tube) used to obtain "undisturbed" soil samples for laboratory testing. However, a 54-in.-diameter prestressed concrete cylinder pile has an area ratio of almost 40% and displaces 6.3 cubic ft of soil per foot of pile, even if it drives without plugging. This suggests that there could be significant differences in the frictional resistance behavior of these two types of LDOEPs, particularly in sandy soils.

There are potentially some significant differences in the behavior of concrete piles compared with steel in the behavior of the soil plug during driving. The downward acceleration of a concrete pile is generally much lower than that of a steel pile, the volume of soil displaced by the pile wall of a concrete cylinder is much greater, and the diameter of the

void smaller. The potential for sandy soil arching within the void is greater (owing to the large area ratio and displaced soil volume), potentially increasing the interior side resistance at the pile/soil interface within the plug. McVay et al. (2004) describe analysis of the potential plugging behavior in prestressed concrete cylinder piles similar to the discussion of the preceding section; however, these analyses lead to the conclusion that plugging of concrete LDOEPs during driving is a relatively unlikely occurrence. Rausche and Webster (2007) describe dynamic analyses using wave equation methods including plug soil mass, but conclude that soil plugs often do not develop during driving because of both the plug inertia and lack of internal friction.

The large volume displaced by prestressed concrete LDOEPs has been observed to result in the bulking of soil within the pile void such that it was necessary to remove material during installation. Kemp and Muchard (2007) describe longitudinal cracking in Florida believed to be from excessive hoop stress induced by mud and water buildup with the pile during driving, commonly referred to as "water hammer." Rausche and Webster (2007) and Muchard et al. (2009) describe occurrences of rising mud and water within prestressed concrete cylinder piles that necessitated removal of the hammer to remove soil from the pile interior; this issue was eventually mitigated in one case by predrilling.

Given the large volume of soil displaced by the pile wall of a prestressed concrete cylinder pile and the observations of bulking within the soil plug, it would appear that any cohesive soil within the plug is likely to be very much remolded by the pile driving process.

*Base Resistance of Concrete LDOEPs*

Concrete LDOEPs are not commonly driven to bear on hard rock strata, although there have been occasions in which concrete cylinder piles have been installed onto soft rock or hard bearing layers using a steel driving ring (M. Saunders, personal communication with D. Brown, 2011). This attachment was composed of a ¾ in.-thick-steel pipe that extended up through the inside diameter of a spun-cast cylinder pile, protruded 6 in. beyond the end of the pile, and was equipped with a flange to cover the end of the concrete with holes for the post-tensioning strands.

Most applications of concrete LDOEPs have been in soil with the pile designed as a long friction pile or else with bearing on a dense sand or weak limerock layer. For piles bearing in sand, driving aids such as jetting and/or predrilling are often employed to achieve penetration below the depth required to achieve lateral resistance. Since the area ratio of concrete piles is so large, it is often difficult to distinguish whether a test pile achieved base resistance by behaving as a plugged section or simply through the base resistance mobilized on the pile cross section. In many cases, it may be that the displacement required to mobilize the base resistance on the full plugged section may be so large that it is not observed during testing.

An interesting comparison was reported by S&ME (2008) between a pair of 54-in. diameter by 80-ft-long concrete LDOEPs bearing in a fairly dense calcareous sand near the South Carolina coast. Both piles were driven open-ended, but one of the piles had the soil plug removed over a large portion of its length and replaced with a concrete plug (although the concrete did not extend to the pile toe). The results of the load testing program detected no significant difference in the measured axial resistance between the two piles.

*Driving Resistance and Dynamic Load Testing*

Prestressed concrete LDOEPs are typically installed to a specified driving resistance and have many of the same issues related to general installation and dynamic testing as described previously. However, concrete piles have some additional unique considerations during installation in order to avoid potential damage to these piles.

Drivability analyses and dynamic measurements are effectively used to select pile hammers and cushions for concrete LDOEPs so that high tensile stresses can be avoided (Kemp and Muchard 2007; Rausche and Webster 2007). High tensile stresses can occur with high energy blows when relatively low base resistance is mobilized, particularly with hammers having a high-impact velocity (such as diesel or some hydraulic hammers). As with any prestressed concrete pile, the driving energy therefore needs to be managed as part of the installation criteria. The hammer must be carefully aligned onto the pile to avoid uneven stresses at the top of the pile that could produce localized overstress or spalling at the pile top, and the use of dynamic measurements with at least four gauges at 90° intervals around the pile can be helpful in verifying good hammer alignment. Hoop stresses in the pile wall may also be present as a result of radial stress from the soil plug (or water hammer, if a sufficient air void at the pile top is not maintained), and thus the spiral transverse reinforcement in the pile wall is an important component of the pile reinforcing.

Prestressed concrete cylinder piles are not typically spliced during installation and then subjected to additional driving. Because driving splices are not commonly used, the maximum length of these piles is limited by the contractor's ability to lift and drive a pile. For typical transportation structure projects this limits the maximum length to approximately 160 ft.

#### Structural Connection to the Top of an LDOEP

In general, the structural connection of the top of the LDOEP to the pier cap or footing is accomplished by installing a reinforcement cage into the pile void and casting a plug of concrete. This approach avoids the large obstruction caused by the extension of the pile wall into a footing or pier cap.

The depth of the concrete plug is controlled only by the need to achieve load transfer from the structure to the pile itself, although many agencies prefer to use concrete filling to a depth of a few feet below the scour elevation. This requirement is likely to necessitate the excavation of the soil plug within the pile to the appropriate depth.

For a steel LDOEP, the steel pipe itself may be structurally connected to the footing, as is illustrated by Figure 7. This detail is from the Lafayette Bridge across the Mississippi River in Minneapolis, for which the main piers are each founded on a $4 \times 6$ group of 42-in.-diameter steel pipe piles. The longitudinal reinforcement in the splice is composed of 20 number 8 bars extending into the pile cap.

Figure 8 provides a slightly different detail from the Hastings Bridge in Minnesota, an arch bridge with piers similarly founded on $3 \times 7$ groups of 42-in. steel pipe piles. In this case, shear studs are welded to the steel pipe itself to develop the structural strength of the steel pipe at the connection location.

Similar connection details are typical for prestressed concrete LDOEPs, although the diameter of the interior void is typically smaller relative to the outside diameter. Another consideration for prestressed concrete is that the prestressing strands of bed-cast piles also require some development length from the end of the pile for development of the full flexural strength of the pile.

## DESIGN OF LARGE DIAMETER OPEN-ENDED PILES

### Design for Axial Loading

Whereas the nominal axial resistance of a large diameter open-ended pipe pile is determined in the field based on driving resistance correlated with load test measurements, the static computations of axial resistance serve only as a guide to estimate the pile length before driving. Where LDOEPs are driven to bear on rock or other hard bearing strata, the pile length is particularly insensitive to the static computations as the final length will be determined by stratigraphy and the selection of driving equipment rather than static analysis methods. However, static computations of axial resistance are always needed to estimate the lengths of piles expected to terminate in soil in advance of construction. In some cases where driving resistance is not relied upon for determination of axial resistance (notably long friction piles in clay soils), the piles may simply be driven to a predetermined embedded length. In such cases, computed static resistance (perhaps correlated to static load tests) may serve as the basis for final design.

Static analysis methods outlined in the current AASHTO design specifications (2013) parallel the methods described in the most recent FHWA manual for driven piles (Hannigan et al. 2006). With the exception of the $\lambda$ method from 1972 (which was developed for offshore piling, but is no longer used offshore), none of the empirical methods described in these publications were developed specifically for LDOEPs or even based on data from load-tested LDOEPs. Most recent literature references do not give serious attention to the FHWA and AASHTO procedures for computing nominal axial resistance of LDOEPs.

It appears that the resistance factors for piles included in the current AASHTO guidelines do not specifically represent LDOEPs. The resistance factors are based largely on the work reported by Paikowsky (2004) in *NCHRP Report 507*, and the database of load tests used to develop the recommendations for LRFD resistance factors includes a very small number of open-ended pipe piles. LDOEPs are not documented separately from smaller open-ended pipe piles, but logically represent an even smaller portion of the data. The current AASHTO code (2013) does not distinguish the design of deep foundations on the basis of any of the unique characteristics of LDOEPs.

The most widely referenced procedure for the design of large diameter open-ended steel pipe piles is the API RP 2GEO (2011) procedures for offshore pile foundations. The American Petroleum Institute (API) also references other possible methods in the commentary.

The following paragraphs provide a brief summary of the computational methods described in the literature that are particularly relevant to LDOEPs in soil.

#### *Axial Resistance in Clay Soils*

The side resistance in clay soils determined using the API methods are based on correlations with undrained shear strength, $s_u$, using a dimensionless empirical correlation factor, alpha ($\alpha$), which typically is taken to be 1.0 or less. The API methods differ from other alpha methods in the approach used to determine alpha and the means of estimating $s_u$. Undrained shear strength is not an intrinsic material property, but rather a function of the test method used to measure it; in addition, the measurement of $s_u$ is subject to the effects of sampling disturbance and other factors. Because of the challenges of sampling and testing in an offshore environment, the API procedure includes suggestions for estimating $s_u$ as a function of Over Consolidations Rules and effective vertical stress ($p_0'$). This method is presented as follows:

$f(z)$, the unit shaft friction can be calculated by:

$$f(z) = \alpha s_u \tag{7}$$

Where:

$\alpha$ is the dimensionless shaft friction factor for clays; and
$s_u$ is the undrained shear strength of the soil at the point in question, in stress units.

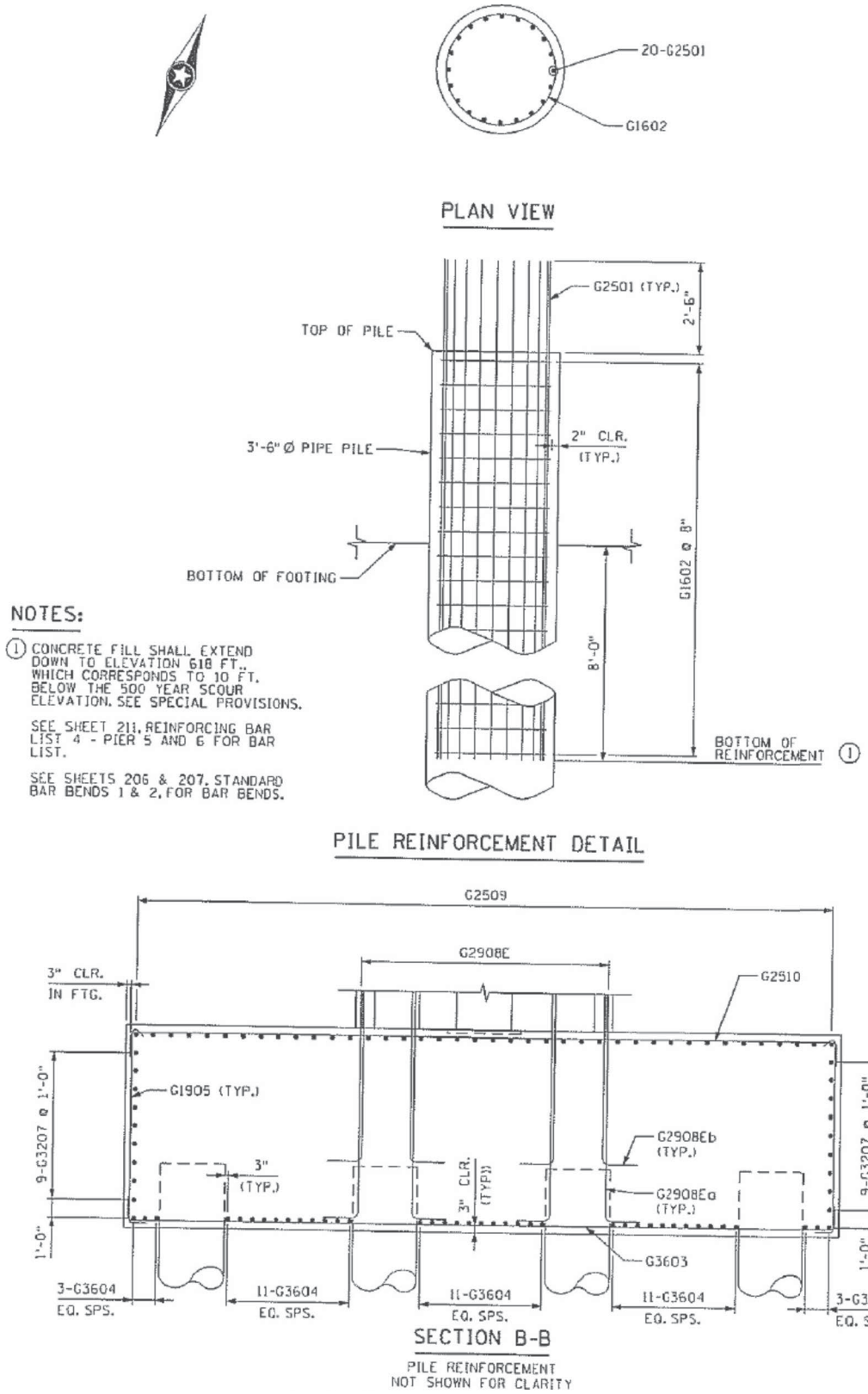


FIGURE 7 Pile to footing connection, Lafayette Bridge, Minnesota.

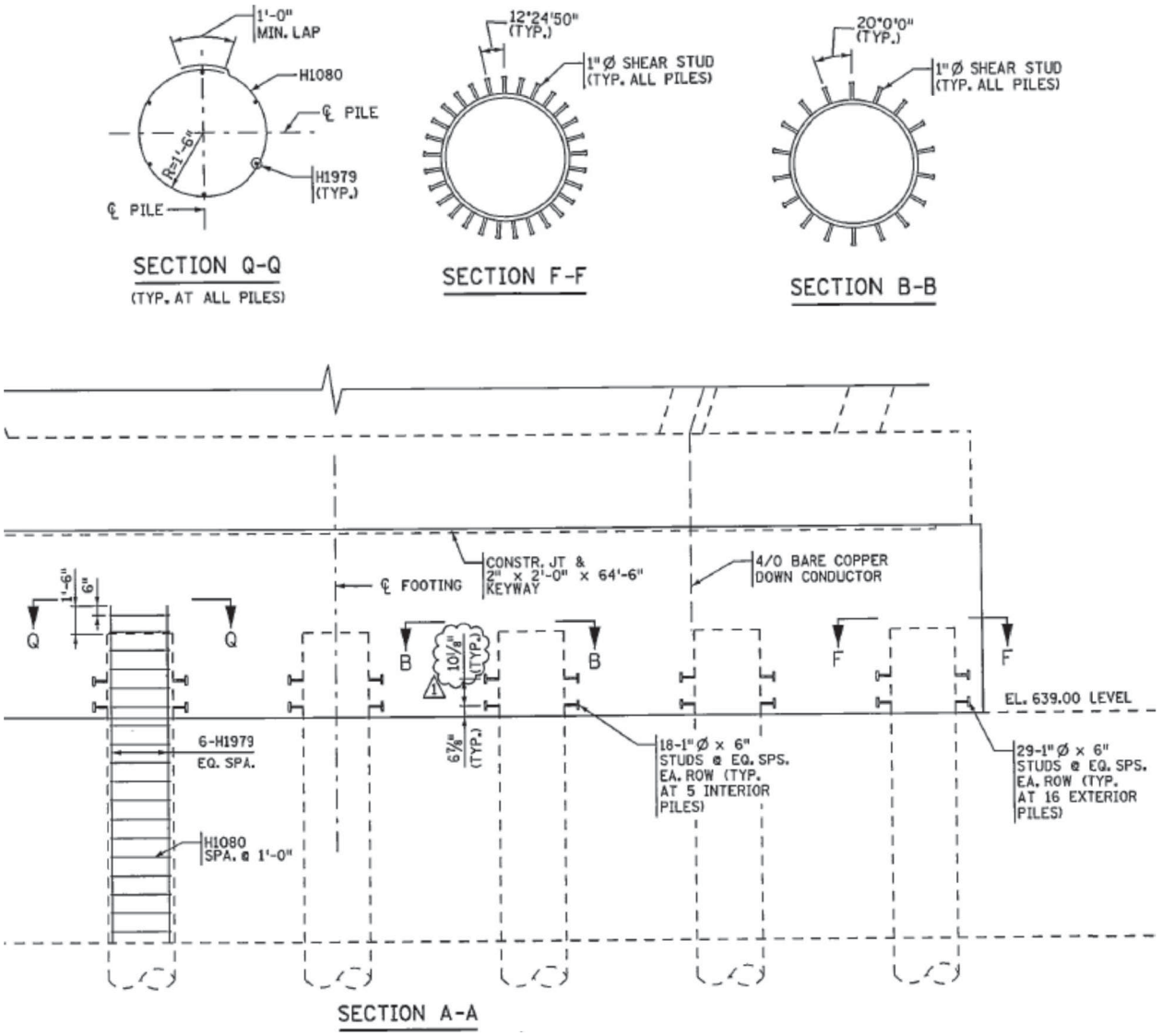


FIGURE 8 Pile to footing connection, Hastings Bridge, Minnesota.

The factor α can be computed by:

$$\alpha = 0.5\psi^{-0.5} \text{ for } \psi \le 1.0 \tag{8}$$

$$\alpha = 0.5\psi^{-0.25} \text{ for } \psi > 1.0 \tag{9}$$

with the constraint that $\alpha \le 1.0$, and where:

$$\psi = s_u/(p'_0(z)) \text{ at depth, } z \tag{10}$$

$$p'_0(z) = \text{effective stress at depth } z \tag{11}$$

Where the pile toe is in cohesive soils, the unit base resistance, $q$, is estimated as equal to $9s_u$, a value that typically represents a low proportion of resistance compared with the side resistance.

The side resistance is assumed to act on both the inside and outside of the pile, with the limitation that the resistance on the inside of the pile is limited by the base resistance of the plugged section below the toe. Because the base resistance of piles in clay is relatively low, the interior side resistance does not normally contribute. The API procedure provides discussion of the possible reduction in the computed nominal side resistance as a result of pile length effects, low Plasticity Index (PI) clays, and highly overconsolidated soils ($\psi > 3$), with reference to the commentary to the API code; however, discretion on these issues is left to the designer.

The "alpha method" approach to correlating unit side resistance with undrained shear strength of clay soil serves as a basis for some additional methods that follow this general methodology. Saye et al. (2013) provides a review of the issues of sample disturbance and the use of the normalized stress history (the "SHANSEP" approach) to address sam-

ple disturbance problems affecting side resistance using alpha methods. This method offers a potential way of addressing one of the most common problems with the use of undrained shear strength as a basis for design, namely the contamination of the design soil strength profile with data from "undisturbed" strength measurements that might be affected by sample disturbance. Karlsrud (2012) provided a recent review of available pile static load test data in cohesive soils leading to a modified procedure for estimating alpha as a function of PI to account for some unusually low axial capacity load test data in low PI soils.

*Axial Resistance in Sands*

The API methods in siliceous sands are based on the use of a shaft friction factor "beta" (β) and end-bearing factor $N_q$, which are multiplied by the effective vertical stress to obtain unit values of side and base resistance, respectively. This approach is fundamentally the same as the beta method described in the FHWA manual (Hannigan et al. 2006), but relies on a table of specific design parameters that are recommended for pipe piles based on the estimated relative density and grain size description of the soil. In general, it is considered that there is higher variability in computed nominal resistance in sands than in cohesive soils; however, dynamic measurements of driven piles in sands is likely to have somewhat greater reliability as an indicator of axial resistance; therefore, the overall reliability of piles in sands is not necessarily lower than for cohesive soils.

*Methods Utilizing CPT Data*

Other methods rely on cone penetration test (CPT) measurements in soils with adjustments to account for pile length and other effects; a summary of these methods is described in API RP 2GEO (2011). These include the ICP-05 methods promoted by the Institute of Civil Engineers (English) (Jardine et al. 2005), the UWA-05 methods promoted by the Australians (Lehane et al. 2005b), the NGI05 methods promoted by the Norwegians (Clausen et al. 2005), and the Fugro05 methods promoted by the Dutch (Kolk et al. 2005). Although a consensus approach has not emerged, these methods have many similarities. There appears to be merit and increased interest in the use of these approaches since they generally account for effects not included in the simplified API procedure; most of these methods also rely on CPT data, which may have advantages in terms of reliability and stratigraphic coverage relative to conventional methods based on laboratory tests.

*Methods Specific to Prestressed Concrete LDOEPs*

Similar methods to those described earlier may be employed for prestressed concrete cylinder piles, although none of these were developed specifically for prestressed concrete piles. McVay (2004) performed a study of axial resistance of cylinder piles for the Florida DOT (FDOT) that included load tests on 22 prestressed concrete cylinder piles from five separate sites; 19 of the 22 piles were from a site in the Florida panhandle and two sites in Virginia. McVay developed empirical correlations specifically for prestressed concrete LDOEPs with standard penetration test measurements ($N$, uncorrected for overburden pressure, and presumably $N_{60}$, although not stated) based on interpreted unit side ($f_s$) and base ($q_t$) resistance in units of tsf, as follows:

$$f_s = C_1 Ln(N) - C_2 \quad (12)$$

$$q_t = C_3 N \quad (13)$$

Where: $C_{1,2,3}$ = empirical constants for the range $5 < N < 60$, as listed in Table 1.

**Design for Lateral Loading**

The design of LDOEPs for lateral loading is fundamentally no different than that of any other deep foundation element. Some issues with respect to the pile itself include the structural connection to the pile cap or footing described previously and the effect on the foundation stiffness contributed by the potential concrete plug within the upper portions of the pile.

Where the pile is filled with concrete, the concrete adds considerable stiffness and likely generates behavior as a composite concrete/steel shell member in the same way that a permanent steel casing contributes strength and stiffness to a drilled shaft. In California, concrete-filled steel LDOEPs are sometimes referred to as "CISS" or cast-in-steel-shell piles.

TABLE 1
EMPIRICAL CONSTANTS $C_{1,2,3}$

| Soil | $C_1$ | $C_2$ | $C_3$ |
|---|---|---|---|
| Plastic Clays | 0.5083 | 0.634 | 0.2226 |
| Clay-Silt-Sand Mixtures | 0.3265 | 0.5404 | 0.4101 |
| Clean Sands | 0.0188 | 0.0296 | 0.5676 |

There exists some uncertainty about the distance from the ends of the pile required for the interior concrete to develop enough bond for full composite action, and it is understood that there are several research initiatives related to this issue for drilled shaft foundations that may have relevance to the structural behavior of concrete-filled LDOEPs.

#### Design for Settlement/Uplift/Serviceability

The design of groups of LDOEPs for settlement is fundamentally no different than that of any other deep foundation system, and general guidelines for estimating settlement of pile groups are provided by FHWA (Hannigan et al. 2006). The same is true for LDOEPs designed to resist uplift forces.

The axial stiffness of an LDOEP can be affected by the relative contribution of the base resistance if plugging behavior is anticipated and significant base resistance of the full plugged pile cross section is considered. Where a large diameter base contributes significantly to the axial resistance, the displacement required to fully mobilize that base resistance may be significant. For this reason, some agencies (e.g., Florida DOT) use a modified form of the Davisson offset method for interpretation of static load tests on piles that are 24 in. or larger, whereby the displacement at the strength limit is based on an offset of D/30 rather than D/120 as used for smaller piles. This greater value reflects the larger displacement to fully mobilize the base resistance. Where a computer model is used to replicate the stiffness of a group of piles, the "*t-z*" springs for the base resistance may need to be adjusted based on whether the LDOEP is to reflect the plugged behavior with associated larger displacement needed to fully mobilize the base resistance on the plug or whether the unplugged base resistance is anticipated at small displacements.

## SUMMARY

This chapter provides a summary of background information on the use of LDOEPs for transportation projects, outlining some of the important issues to be addressed in the selection and design of LDOEPs for this purpose. LDOEPs may consist of steel pipe or prestressed concrete cylinder piles and are defined for the purposes of this report as driven, open-ended piles that are of 36 in. outside diameter or larger.

LDOEPs provide advantages where large foundation loads may exist and/or the piles are subject to significant unsupported length as a result of scour, liquefaction, or very weak surficial soils. Marine construction conditions also favor the use of these piles, particularly where pile bents might be employed to eliminate footings.

Steel pipe is often specified by grade with reference to ASTM A252 (ASTM 2010) and may be economically manufactured as spiralwelded pipe using a long coiled sheet that is twisted into a spiral and welded along the spiral seam in a continuous process. Where piles larger than 10 ft in diameter or thicker than a 1-in. wall are required, rolled and welded straight seam pipe may be used.

Prestressed concrete cylinder piles have advantages of corrosion resistance and durability, which may be particularly important for coastal structures. These piles may be fabricated as spun-cast cylinders using low-slump concrete, which results in concrete with high strength (typically compressive strengths of 8,000 psi or greater), low permeability, and high density. The cylindrical sections are assembled and post-tensioned to fabricate piles with a length of up to 200 ft or greater. Concrete cylinder piles have also been cast in conventional horizontal prestressing beds using a form insert to create the center void.

A range of factors that are distinctive to LDOEPs as opposed to conventional piles are described, most notably the behavior during driving and the tendency of the interior soil to remain in place or even rise within the pile as the pile is driven. The failure of most LDOEPs to "plug" during initial installation is related to the inertial resistance of this soil mass as the pile is accelerated downward by the hammer. Plugging may be more likely during subsequent static loading where inertial forces do not contribute, and this difference between behavior during installation and subsequently is the source of some difficulty with the use of driving resistance or even high-strain dynamic load tests as an indicator of static axial resistance.

Steel LDOEPs have advantages when driven to bear on rock, because the "unplugged" behavior during installation allows the pile to penetrate to the rock with relatively less driving resistance until bearing is achieved. However, high and potentially nonuniform end-bearing stresses at the pile toe require consideration.

Installation of steel LDOEPs using a vibratory hammer can provide another advantage for ease of installation, particularly where a splice is required and the vibratory hammer may be used to install the first pile section. However, the uncertainty related to the effect of vibratory installation on subsequent axial resistance dictates that steel LDOEPs are typically driven to bear using conventional impact hammers.

The time dependency of axial resistance related to setup is a consideration with pile installation criteria as it is with any driven pile; however, LDOEPs have additional uncertainty related to the potential difference in behavior of the soil plug during dynamic penetration and static loading. Issues of differences between how dynamic and static loading is applied to the pile need to be included when evaluating setup with restrikes and/or load tests.

Prestressed concrete LDOEPs have many similar issues related to the interpretation of driving resistance, soil plugging, setup, etc.; however, a distinctive feature of these piles relative to steel is the large area ratio of the pile cross section that certainly affects the behavior of the soil plug and the available cross-sectional area of the pile to engage base resistance during installation. Prestressed concrete LDOEPs also require consideration of potential tensile stress during driving, as with any prestressed concrete pile.

The design of LDOEPs is distinctive from other types of driven piles primarily in terms of the computation of axial resistance. Although the AASHTO design codes do not distinguish these piles from other types of driven piles, the pile load test data that were used to establish resistance factors for design included very few examples of LDOEPs. The most commonly used computational procedures for estimating static axial resistance of steel LDOEPs in soil are found in the API guidelines, which have a history of use for the design of offshore platforms. The API procedures have expanded in recent years to include additional calculation methods based on the use of CPT and to account for length effects and other factors such as partial plugging. FDOT has sponsored one study to develop empirical design procedures for LDOEPs based on standard penetration test (SPT) measurements, which include data from a few prestressed concrete LDOEPs.

In conclusion, this summary of background information has identified many of the distinctive features affecting the design, construction, and testing of LDOEPs, and this information serves as a base of reference for the subsequent consideration of the synthesis of practice from transportation agencies described in the following chapters.

CHAPTER THREE

# AGENCY STATE OF PRACTICE FOR LARGE DIAMETER OPEN-ENDED PILES

This chapter provides detailed findings on how transportation agencies view and use LDOEPS based on the survey results and follow-up interviews. Some information from the literature review is also included.

## INTRODUCTION

To understand the perspectives and current practices of state agencies, an on-line survey was used to determine which agencies have experience with LDOEPs and to gather some preliminary information on how these piles are used by those agencies that currently use them. The goal of the survey was to identify those state agencies with experience using LDOEPs, obtain basic information on their design and construction techniques, and identify those that would be willing to provide detailed experiences through one-on-one interviews. Appendix A contains the survey.

The geotechnical engineers (or equivalent) in all 50 state DOTs plus the District of Columbia and Puerto Rico were invited to complete the survey. Of the 52 agencies surveyed, 44 (85%) provided responses. Figure 9 is a map of the responses, with green indicating states that have experience with LDOEPs, red states that do not use LDOEPs, and gray states that did not response to the survey.

The survey was structured such that the first question asked if the agency had experience within the last 10 years or currently uses LDOEPs. If the agency answered "No," the respondent was able to provide information on why LDOEPs are not used by the agency before jumping to the end of the survey. This allowed for the collection of reasons why LDOEPs are not used for inclusion and discussion in this report, as well as easier analysis of the answers provided by agencies that do use LDOEPs.

Of the 44 agencies that completed the survey, 18 (41%) indicated that they had experience with LDOEPs. Of this group of 18, two agencies [Maine DOT and North Carolina DOT (NCDOT)] reported that they are in the design phase of current projects where they are considering using LDOEPs, but have little to no past experience with LDOEPs. Most of the remaining agencies in this group have limited experience on only a few projects within the last 10 years. Some may have also had a few structures on these piles built predating the 10-year time frame.

Appendix B contains a summary report that focuses on the responses of the 16 agencies actively using LDOEPs. Two agencies, the Maine and North Carolina DOTs are only in the design stages of projects with LDOEPs and are not included in the summary report. Graphs and charts of the responses to questions with "Yes/No" or multiple choice answers are provided. Questions with written answers are shown in table form. It was believed that having "no answer" data in the totals would not be an accurate reflection of the state of practice.

To acquire more detail on the experiences of this group, telephone interviews were conducted with seven of the 16 agencies. The agencies with the most experience were selected to capture the range of experiences these agencies have with respect to pile types, sizes, and soil conditions. The notes from the telephone interviews are included in Appendix C.

## AGENCIES NOT USING LARGE DIAMETER OPEN-ENDED PILES

Twenty-six of the 44 agencies that responded to the survey (59%) indicated that they do not consider the use of or have not used LDOEPs for transportation structures. The survey provided the opportunity for respondents to provide comments on why their agency does not consider or use these piles. These responses could be grouped as follows:

- The agency responded simply that they do not use LDOEPs or did not provide any additional comment.
- The agency is evaluating LDOEPs for a project, but has not yet used them.
- LDOEPs are not cost-competitive with other deep foundation systems.
- Geologic and soil conditions are not suitable for LDOEPs, but more suited to drilled shafts, H-piles, or smaller diameter piles.
- There is a lack of expertise and equipment among the pool of contractors that typically perform foundation installation in the state.
- Smaller pile sizes are suitable for the typical structure size and loads.
- Design of these piles is not specifically addressed in AASHTO Design Specifications, leading to uncertainty in extrapolating the standard AASHTO axial resistance

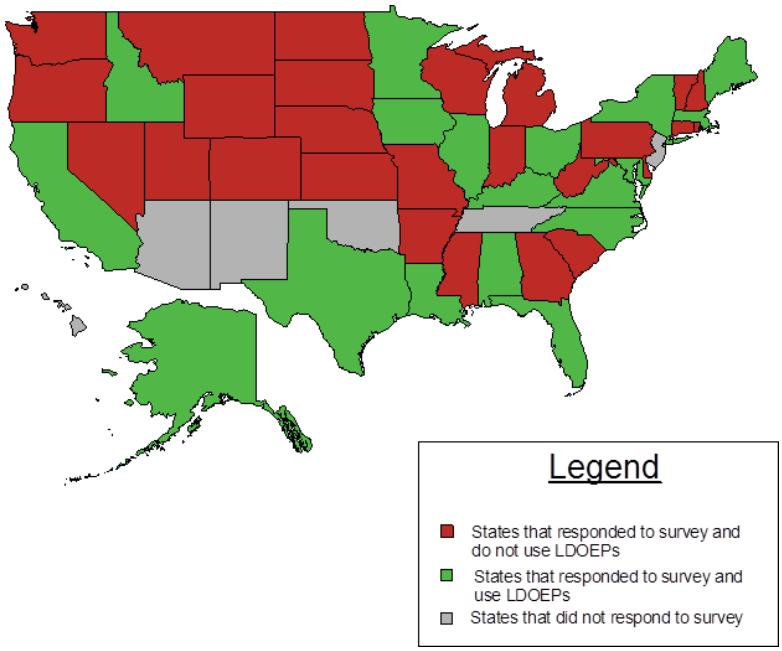


FIGURE 9 Map of survey responses.

prediction methods to larger pile sizes. Specific design issues and questions include:
  - Prediction and extent of plugging,
  - Determination of pile capacity/resistance,
  - Length of concrete infill,
  - Structural design of concrete–steel section, and
  - Resistance factor selection.
- Concerns over vibrations to adjacent structures.
- How to evaluate potential benefits against uncertain construction costs and risks.

The comments and concerns on design issues, as well as the benefits versus construction costs and risks, reflect some of the reasoning behind the undertaking of this synthesis project. The comments about unsuitable soil conditions, poor cost-competitiveness, and a lack of experienced contractors are related and are to be expected for some areas of the country. Not all locations are suitable for every foundation type.

## SURVEY RESPONSES

### Agencies That Responded

The 16 agencies that indicated they currently use LDOEPs are listed here. These agencies are spread across the country and represent a wide range of subsurface and geologic conditions. The level of experience is generally low, with all but Alaska Department of Transportation and Public Facilities (ADOTPF) and California DOT (Caltrans) noting that they have implemented no more than ten projects utilizing LDOEPs within the last ten years. Those agencies both noted that they have executed more than 50 projects with LDOEPs in the last ten years.

Alabama (ALDOT)
Alaska (ADOTPF)
California (Caltrans)
Florida (FDOT)
Idaho (ITD)
IDOT
Iowa DOT
Kentucky (KYTC)
Louisiana (LADOTD)
Maryland DOT
Massachusetts (MassDOT)
Minnesota (MnDOT)
New York (NYSDOT)
Ohio (ODOT)
Texas (TxDOT)
Virginia DOT (VDOT)

Each of these agencies reported that they use design consultants for LDOEPs. In addition to using consultants, most of the 16 said they also used one of the other three forms of design delivery: design by agency (75%), design-build (75%), and value engineering change proposal (VECP) by contractor (69%).

### Applications, Selection, and Pile Types

*Applications/Selection*

Several choices were provided to respondents to indicate the reasons that LDOEPs were selected for projects in their area, with respondents allowed to select more than one answer. The most common response (94%) was to resist large lateral loads. Slightly more than half also chose large axial loads (56%) and soil and rock conditions (56%). Cost-benefits (31%), special applications (25%), and other (19%) were also selected. Schedule and environmental considerations were given as "other." Figure 10 illustrates the reasons for selecting LDOEPs.

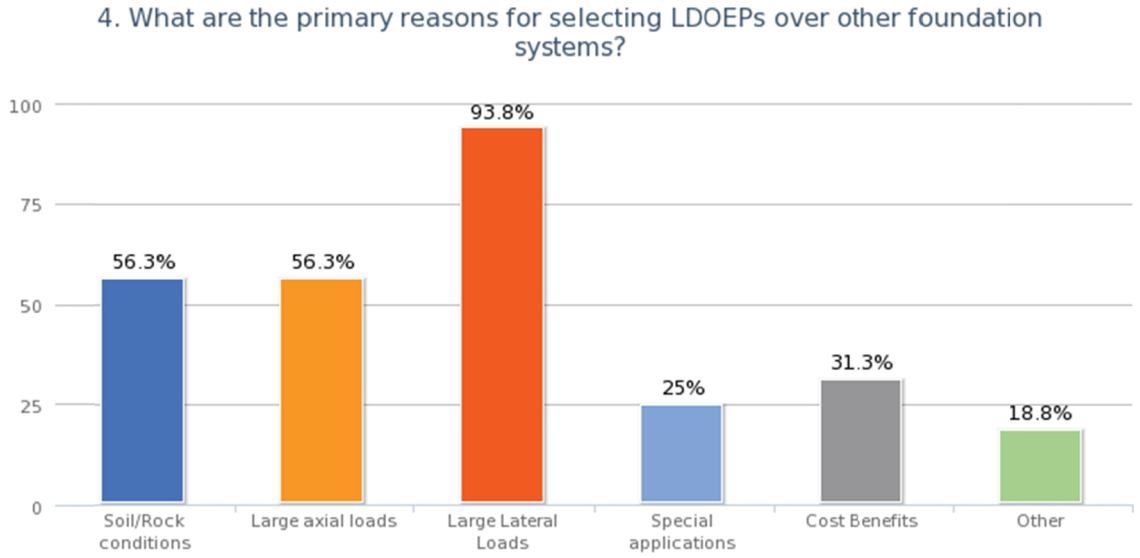


FIGURE 10 Primary reasons for selecting LDOEPs.

Common themes in the interviews were the ability to resist large lateral loads and the potential to avoid cofferdams and excavations for pile footings. By taking advantage of the efficient bending resistance of LDOEPs, a few larger piles can be used in a pile bent, eliminating the need for a pile footing and the associated cofferdam when using a large number of smaller piles. Also, fewer piles were reported to have construction schedule benefits and occasional environmental benefits.

Selection of LDOEPs is often heavily influenced by local practice—the availability of the piles and a qualified contractor pool that is capable of installing LDOEPs. Areas that have subsurface conditions favorable for LDOEPs tend to have piles readily available and experienced contractors to install them, making them a viable choice of foundation. Areas not as favorable tend to lack contractors experienced in LDOEP installation, making them less attractive to agencies to consider.

Where LDOEPs are typically used, the process of evaluating and selecting LDOEPs over an alternate foundation type is part of the typical bridge design process. The foundation that is ultimately selected is chosen based on many factors that usually include suitability of the foundation type for the specific project, costs, performance, and schedule.

ADOTPF specifically noted several additional benefits with LDOEPs that help with the selection process, including:

- Less environmental impact—pile bents with a few LDOEPs remove the need for excavation and cofferdams for large pile footings containing numerous H-piles or small pipe piles.
- Quick installation—ADOTPF is utilizing accelerated bridge construction as a result of the very short construction season in Alaska. There are also time restrictions because of the prevalence of wildlife. Fewer large piles can be installed more quickly and superstructure work started sooner.
- Seismic design—a benefit of adopting 48-in. pipe piles has been that no major changes are needed to designs to meet updated codes, since the pipe piles are suitable for the updated loadings.

*Concrete Piles*

Seven agencies use concrete LDOEPs, with five using spun-cast post-tensioned piles, one bed-cast piles, and one using both. The diameters range from 36 to 66 in., with 36 in. and 54 in. being the most common. Wall thickness ranges from 4 to 8 in., with concrete compressive strengths ranging from 6 to 7 ksi. Prestress values were reported to range from 0.7 to 1.6 ksi.

*Steel Piles*

Twelve agencies utilize steel LDOEPs with a variety of welded plate, straight seam welded, and spiralwelded pipes. The reported diameters ranged from 36 to 96 in., with 36 to 48 in., being the most common. Wall thickness values were ½ in. to 2 in., with yield strengths ranging from 36 to 75 ksi. Grade 50 steel appears to be the most typical.

#### Standard Plans and Specifications

Very few agencies (5) indicated that they have any standard plans or specifications for LDOEPs; most use their published standard specifications written for driven piles without regard to size. Some of the "Yes" answers were followed with descriptions of special provisions or modifications (such as reinforcement of testing requirements) rather than a completely separate specification. FDOT has settled on a standard design for two

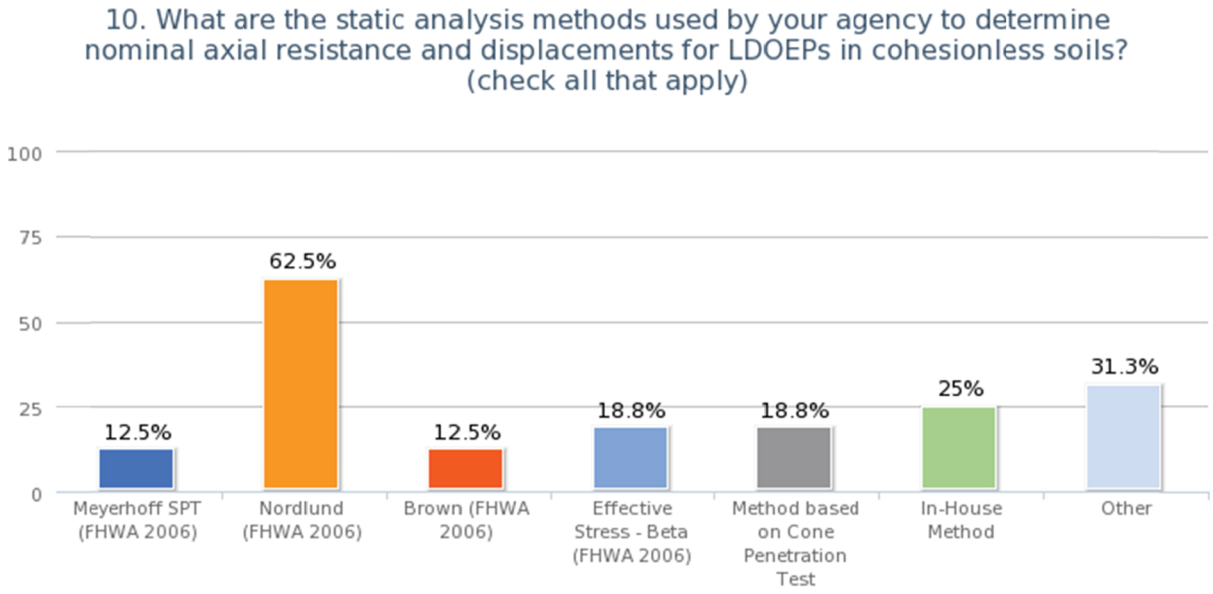


FIGURE 11 Static analysis methods in cohesionless soils.

types of concrete LDOEP: a 54-in.-diameter spun-cast post-tensioned pile and a 60-in.-diameter full-length pre-tensioned bed-cast pile. NYSDOT also has a standard specification for concrete cylinder piles that can be modified, if necessary, for each project.

### Design

*Piles Bearing in Soil*

To determine the methods used by agencies for determining static axial resistance and displacements, several common methods were listed for both cohesionless and cohesive soils. Respondents were asked to select all methods that they use and were provided an option to explain any "in-house" or other methods in use. The methods for both soil types are shown in Figures 11 and 12, with the percentage of the 16 agencies that selected each. The Nordlund and the alpha (Tomlinson) methods are the most often cited for cohesionless and cohesive soils, respectively. The "Other" and "In-House" methods included several items that are discussed here.

Louisiana Department of Transportation and Development (LADOTD), MnDOT, and NYSDOT all indicated that they generally follow the FHWA methods with little or no modification. MnDOT noted that most of its experience, however,

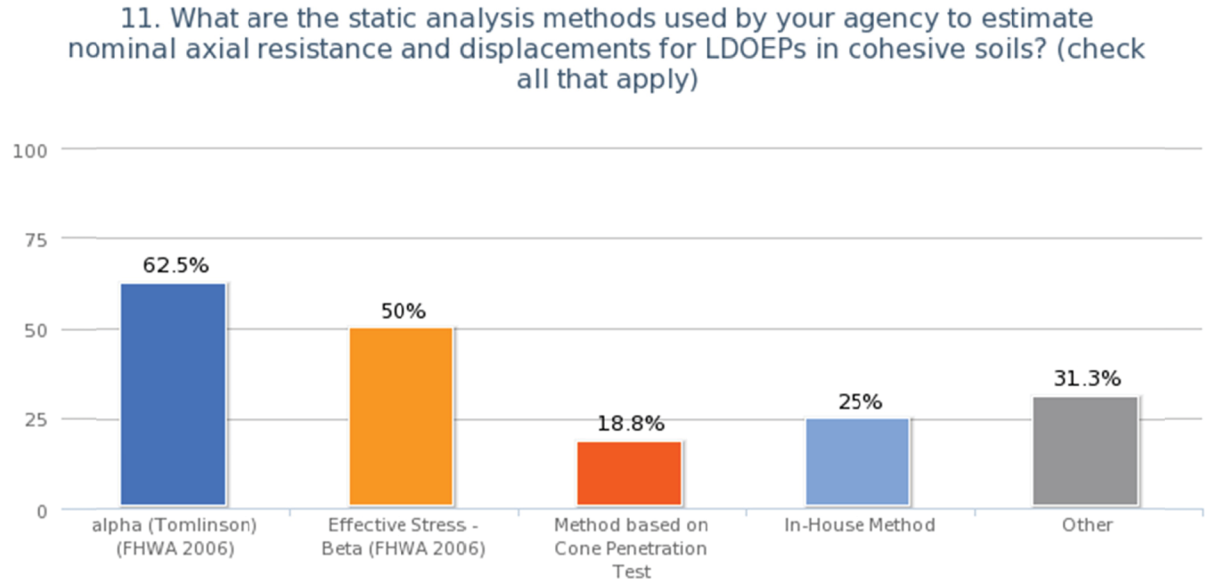


FIGURE 12 Static analysis methods in cohesive soils.

has been to drive pile to bear on rock, so that calculations of side resistance are not a significant issue for static resistance. NYSDOT noted that they temper the FHWA methods with their dynamic test experiences at other sites. VDOT reported that consultants on design-build projects are not limited to the method they can select and so can use methods other than those listed in the survey.

Several agencies reported that they have developed in-house methods for estimating pile axial resistance including Alabama DOT (ALDOT), ADOTPF, FDOT, Illinois DOT (IDOT), and (TxDOT). All of the methods apply correlations to historical installation data, primarily on small diameter piles.

ALDOT has an in-house computer program developed from test pile data performed in the 1960s and 1970s (K. Davis, personal communication with R. Thompson, July 2014). The program was originally written in Quick Basic to perform on DOS computers, but was recently upgraded to a Windows-based version. The program initially only accounted for ultimate tip and side resistance in sand and clay soils; it lacked the modeling of silty soil and the calculation of pile settlement. The latest version now has silty soil models and calculations of pile settlement, as well as the ability to perform calculations utilizing an LRFD resistance factor. The initial program was based on test pile data performed during the construction of the I-65 bridges through the river delta in Mobile County and the following references:

- NAVFAC DM-7, *Soil Mechanics, Foundations, & Earth Structures*, p. 7-13-9.
- *U.S. Steel Highway Structures Design Handbook*, Vol. 1, p. 1/10.10.
- Peck, Hanson, and Thornburn, *Foundation Engineering.*
- *Highway Research Record*, No. 333, p. 93.

ADOTPF has developed a modified beta method using case studies from historic PDA®/CAPWAP® results (Dickenson 2012). The study is based on 20 years of driving records and PDA® data for 36 to 48 in. LDOEPs in granular, cohesionless soils. This method uses region-specific empirical relationships to estimate unit side and base resistance, with an equivalent plug used for base resistance calculations. The methods have been used for some recent projects (2013 and 2014), where the dynamic test results matched very well with the modified static calculations. A summary of the report is included in chapter five. Prior to using this method, ADOTPF applied the unmodified FHWA methods as contained in the computer programs DRIVEN and Allpile. It believed that these methods were not modeling the plug correctly and that the methods were not properly "scaling up" for larger pile sizes; that is, the unmodified methods were based on smaller pile sizes that were not adequately modeling soil–pile behavior for larger pile sizes.

FDOT typically uses the computer program FBDEEP developed by the University of Florida for the agency, but available commercially through the Bridge Software Institute. The program allows for the use of either SPT or CPT data for pile design, with the SPT methodology based on empirical correlations between CPT and SPT tests for typical Florida soil types. The correlations are based on pile installations in Florida. Some consultants and others outside the state of Florida use FBDEEP for foundation design.

IDOT uses an in-house design method designated the Modified IDOT Static Method (IDOT 2011). The original IDOT static method was developed more than 40 years ago, correlating allowable pile resistance to the ENR (*Engineering News Record*) dynamic formula. In 2007, the method was updated to reflect the change to LRFD design, producing the Modified IDOT Static Method. The nominal pile resistance is calculated from unit side and base resistance that include correction factors for cohesive or noncohesive soils. The nominal unit side and tip resistance are both calculated from correlations to SPT $N_{(1)60}$ values for cohesionless soils and undrained shear strength, $q_u$, for cohesive soils. Factored resistance is calculated using a Geotechnical Resistance Factor, $\varphi_G$, of 0.55 applied to the nominal pile resistance, less reductions for liquefaction, scour, and downdrag.

TxDOT has design methods based on the Texas Cone Penetrometer (TCP) that is the standard in situ test method for TxDOT instead of the SPT or CPT. (Texas Department of Transportation 2012). The TCP is a cone similar to the CPT driven with a hammer similar to the SPT. The TCP blow counts, $N_{TCP}$, are used indirectly to estimate the in situ, undrained shear strength of soils. Correlations of $N_{TCP}$ to pile tip and side resistance are used for pile design. The correlations were initially developed in the 1950s and have been updated periodically (Vipulanandan et al. 2008). The data used for the correlations include piles of up to 24 in. in diameter.

The correlations for pile design are presented in the TxDOT *Geotechnical Manual* as design charts plotting "Allowable Skin Friction" or "Allowable Point Bearing" versus $N_{TCP}$. The different relationships are plotted for basic soil types. The plots include limiting values of maximum values of side and tip resistance for design.

Two agencies mentioned that they use the API design methods in API RP2 GEO (2011). Caltrans uses this method regularly; KYTC used this design guidance for the single project it has under design for which a load test program was performed (Terracon 2014). During the telephone interviews, one of the KYTC consultants noted that his experience, and that of the offshore industry, indicates very good agreement of API RP2 GEO with load test data. Significant differences between the API and FHWA methods that are beneficial when using the API method are:

- API evaluates friction inside of pile plug formation, rather than a more arbitrary selection by FHWA.
- API limits side resistance to a maximum mobilized value, whereas FHWA (Nordlund) does not include an upper limit by default.

KYTC noted that the soil properties and limiting values of pile resistance recommended or determined by the API method were adjusted based on KYTC and Terracon experience and the CPT data collected for the project. The CPT data were especially helpful in evaluating limits on mobilized pile resistance.

*Piles Bearing on Rock*

Six of the 16 agencies using LDOEPs drive the piles to bear on rock; four do not typically use special toe treatments, one does use special toe treatments, and one does either depending on the rock formation. The treatments tend to be strengthened rings or other special reinforcement added to the toe. ADOTPF has some special cases where if lateral support is needed the rock is cored to seat the pile in the socket. Concrete is sometimes used to set the pile.

For the design of rock-bearing piles, most agencies design based on the structural limits of the pile section and use wave equation analyses. The piles are monitored during driving with PDA® or driven to a required blow count. FDOT utilizes FBDEEP® with its method for soft Florida limestone.

MnDOT made special note that while static resistance analysis methods are reasonable estimates of the long-term resistance of the pile the resistance is not always easily demonstrated with dynamic testing methods. There is also concern with appropriate modelling of the pile and proper selection of associated damping/quake needed for analysis, considering the general low level of experience with dynamic testing of LDOEPs compared with smaller pile diameters.

*Resistance Factors*

Most of the agencies using LDOEPs utilize the resistance factors for driven pile design recommended in the current AASHTO Specifications, as noted in Figure 13. Eight agencies (50%) stated they use the AASHTO factors, whereas five (31%) use a combination of AASHTO and agency-specific factors for driven piles in general. Three agencies (19%) indicated that they use something other than AASHTO resistance factors. Caltrans uses factors that they have developed. ALDOT is currently evaluating resistance factors for LDOEPs as it does not have any projects in design since LRFD implementation. TxDOT's design method uses the TCP as an Allowable Stress Design-based method.

**Pile Plugging**

Most agencies (10) evaluate the potential for pile plugging during driving on a site basis, looking at the specific soil conditions, pile type, and pile size. Of the remaining six agencies, three [Idaho TD (ITD), Iowa DOT (Iowa DOT), and IDOT] usually assume that a plug will form, whereas the remaining three [FDOT, Maryland SHA (MSHA), and TxDOT] assume that the plug will not form. Note that the three "assume will form" agencies have had limited numbers of pile installations, including ITD having only one use of LDOEPs.

FDOT's basis for assuming that a plug will not form is from research on concrete cylinder piles by McVay et al. (2004). FDOT also had undertaken direct observations on two projects where the soil and water column inside the pile rose in the pile during driving, causing problems with pile cracking owing to increased stress from pressure inside the pile. A more detailed discussion of these occurrences is included in chapter five—Case Histories.

For KYTC's only LDOEP project to date plugging was not anticipated; therefore, the use of a constrictor plate in the pile was investigated in order to facilitate plug development in a specific target stratum (Terracon 2014). The key question was development of the plug—if the plug is being relied on to either achieve penetration into the target stratum or to achieve the nominal pile resistance, how could the certainty of the plug developing and its location be determined? Some of the test piles were fitted with steel constrictor plates inside the pile to encourage plug development. The evaluation started with the API method internal and external skin friction analyses, compared with a plug forming to determine the most effective point to set the plate to engage the plug. For the test piles, the plate was set at an elevation higher than estimated in order to have the piles penetrate further into the target stratum than determined for the design. This was done to account for variability of the top of the target stratum and to provide confidence that the piles did bear in the stratum. Of the two pile diameters evaluated, the 48-in.-diameter piles tended to develop a plugged condition at the target stratum, while the larger 72-in.-diameter piles did not. Additional discussion of the test program is included in chapter five—Case Histories.

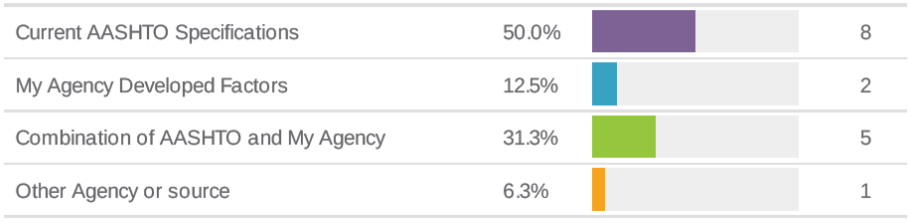


FIGURE 13 Source of resistance factors.

### Setup and Relaxation

Twelve of the 16 agencies (75%) evaluate setup and relaxation for each project, although most assume that relaxation will not occur. Dynamic testing on restrikes of piles is very common to evaluate or demonstrate that setup has occurred. VDOT stated that the coastal areas of Virginia routinely exhibit 200% to 300% setup. Production pile lengths are set based on a test pile program documenting the setup. High-strain dynamic tests with restrikes have significantly reduced the pile lengths being installed compared with when setup was not determined and accounted for.

### Drivability and Driving Criteria

All 16 agencies typically use wave equation analysis to assess drivability, although not all utilize it during design as part of the pile selection process. Some, such as MnDOT, will perform an initial check during design for larger diameters and special conditions (such as needing to penetrate a hard layer to get to minimum tip), where high driving stresses are anticipated to verify that the designed piles can be installed by using typical hammers. Most typically rely on the contractors' submittals to evaluate drivability for the specific hammer system and pile design. NYSDOT will evaluate drivability during design, including various plugging scenarios, as part of the drivability evaluation. The use of test piles to evaluate drivability was also noted by IDOT and MassDOT. IDOT includes comparison of the test pile to the WSDOT driving formula.

ADOTPF does not typically do full drivability analysis during design for most loading conditions, but will review driving stresses with low penetration as well as with penetration at the estimated tip. If a specific project has very high axial loads, or high driving stresses are anticipated, a wave equation analysis will be performed to verify that the designed piles can be installed with the use of typical hammers. Contractors are required to submit wave equation analysis with equipment submittal as is typical with most agencies.

Figure 14 shows the agency responses to the question concerning driving criteria. The use of high-strain dynamic tests on test or indicator piles for setting driving criteria is used by 12 of the 16 agencies (75%). Verification of the required resistance with restrikes is also common [10 of 16 (63%)]. Load tests and wave equation analyses were used by almost half of the agencies [7 of 16 (44%)]. Driving to a specified tip elevation (routine practice for LADOTD) to practical refusal or to a specified resistance using wave equation analyses are also used to some degree among the agencies using LDOEPs. All of these agencies contributed to *NCHRP Synthesis 418: Developing Production Pile Driving Criteria from Test Pile Data* (Brown and Thompson 2011). Details on how each develops and uses driving criteria can be found in that report.

### Testing

Most of the 16 agencies use high-strain dynamic testing to measure or demonstrate pile resistance, as well as to monitor driving stresses to reduce pile damage. Some use it exclusively (e.g., ADOTPF), whereas others use it alone or with rapid and/or static load tests (e.g., FDOT). Most use restrikes to develop setup curves and establish driving criteria (e.g., LADOTD). VDOT noted that all projects with LDOEPs will have a comprehensive test program of high-strain dynamic testing during driving, often supplemented with static or rapid load tests.

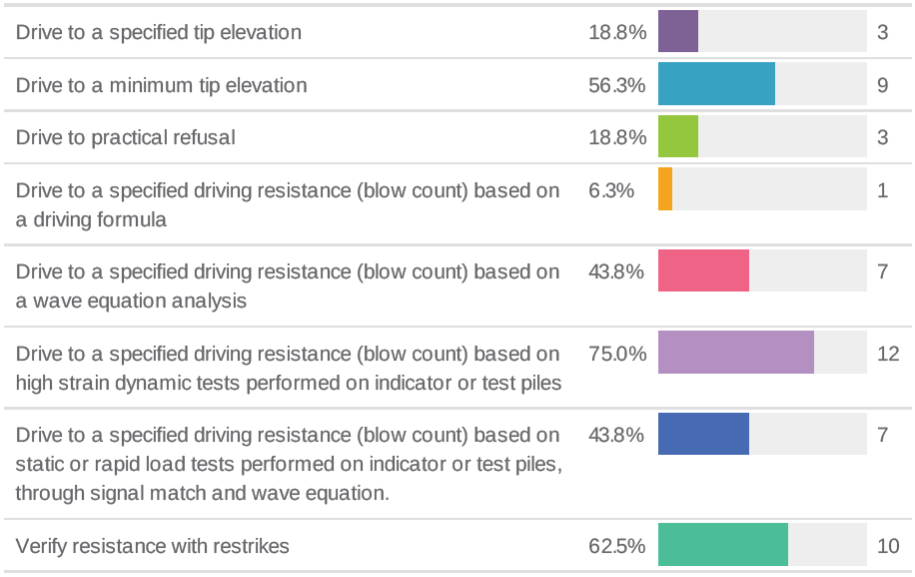


FIGURE 14 Driving criteria practices.

FDOT relies heavily on dynamic testing to verify resistance, set tip elevations, and establish final order lengths. The agency has had no significant issues with dynamic testing; however, load test programs did indicate that pile resistance determined by CAPWAP® was conservative compared with static/Statnamic® testing (Muchard 2005; Kemp and Muchard 2007).

The test program on steel LDOEPs executed by KYTC (Terracon 2014; see also chapter five—Case Histories) included an extensive program of dynamic, static axial, Statnamic® axial, and Statnamic® lateral tests on piles with wall thicknesses ranging from 1 to 2 in. KYTC concluded that dynamic testing was underestimating the static pile resistance as determined by the static and Statnamic® tests. In addition to comparisons of test methods, a detailed evaluation of dynamic pile testing records was done to consider methods of improving the match quality and the estimation of static pile resistance when considering plugging. Comparisons of analyses using a single-toe pile model and a double-toe model were made, as well as application of radial or radiation damping models. Among other findings, it appeared that the radiation damping models provided better match quality as well as estimating base resistance when the constrictor plates were engaged.

MnDOT considers problems with demonstrating the required pile resistance by means of dynamic testing methods to be a significant issue. There is reluctance by MnDOT, as with many other state DOTs, to use a higher resistance than can be verified though testing. The experience of MnDOT has been that it is often difficult to provide a large enough hammer to move the pile enough on restrike to demonstrate the required resistance once the pile is firmly bearing on rock or any setup has occurred. Having adopted the approach to perform rapid load tests (Statnamic®) to better assess the static pile resistance and help correlate and/or calibrate PDA® data has resulted in more confidence in designs utilizing LDOEPs.

NYSDOT used a combination of static and dynamic load tests 25 to 30 years ago to evaluate the resistance of concrete piles. Currently only dynamic testing is used, both for measuring resistance and setup and for quality control (monitoring stresses to reduce cracking and damage to concrete). Dynamic tests are sometimes done on pre-production test piles to set order lengths, while on some projects tests are on production piles only. Dynamic tests always include signal match analysis using CAPWAP® software. Interestingly, NYSDOT reported that in practice they will use the superposition method (Hussein et al. 2002) noted in chapter two, adding base resistance from the end-of-initial drive (EOID) with the side resistance from restrike blows to estimate the static pile resistance. For very long piles, the side resistance from several blows is superimposed to estimate the side shear resistance for the pile.

#### Driving Aids

Driving aids have been used by only five of the 16 agencies and they highlighted the following useful practices:

- Where concrete LDOEPs are used in sandy soils, such as by ALDOT and NYSDOT, jetting is allowed under certain circumstances. Jetting involves using water delivered under pressure in a pipe with special nozzles to the pile tip to loosen the soil to make driving easier. Environmental issues related to turbidity usually require special measures or can prohibit jetting at specific project sites.
- FDOT allows jetting if environmental permits can be obtained, but tends to use pre-drilling or pre-forming holes no deeper than minimum tip elevations. Pre-drilling is allowed in the FDOT specification, but is not currently practiced with LDOEPs.
- IDOT allows piles to be set and started with vibratory hammers, with completion or verification by impact hammer.
- Caltrans has several allowable driving aids available to contractors, including driving shoes, pre-drilling, center relief drilling (drilling out the plug in the center of the pile), and vibration.

#### Observations, Challenges, and Lessons Learned

In the survey and during the interviews, respondents were given the opportunity to offer specific observations, lessons learned, or challenges from their experiences. Some of those not highlighted in the previous sections of this chapter are noted here.

*ADOTPF*

- Do not use static analysis methods alone to predict resistance. They are too conservative as a result of not scaling up to larger diameters.
- Have observed piles reaching a maximum resistance and then not gain additional resistance with increased depth. One idea is that the soil is liquefying close to the pile as it is being driven, causing reduction or loss of side resistance. Some gain occurs after driving has been completed, but not to the level expected.
- Cleaning out the center of piles (center relief drilling) is effective for overcoming hard driving or obstructions in gravely soils.
- If pile resistance is lower than the anticipated pile resistance from ADOTPF static analysis methods, increasing the frequency of high-strain testing to increase the resistance factor has been effective. Increased resistance factor decreases the required driving resistance.

*Caltrans*

- How to demonstrate and/or verify nominal axial resistance:
  - PDA® alone for large diameter piles does not appear to adequately measure axial resistance.

  - Pile driving formulas for large diameter piles are not sufficient.
  - Static pile load testing in conjunction with PDA® is needed according to the Caltrans LRFD Amendment to the AASHTO design code (California Department of Transportation 2012).
  - Load tests are needed to be taken to failure to better calibrate resistance factors; however, this is difficult.
- Potential for effects of vibrations from installations in highly urbanized areas. More monitoring data and research is needed for LDOEP installations since most information is for small diameter piles.

*FDOT*

- Proper quantity, size, and location of vent holes in the sides of concrete cylinder piles are very important to reduce potential for longitudinal cracking as a result of stresses in the void space of the pile.
- When driving concrete cylinder piles, the pile cushion needs to have a void with the same size as the void of the pile. Using a solid pile cushion may result in it being pushed into the void, generating radial stresses that initiate longitudinal cracking or spalling at the pile head.
- Careful consideration of the configuration of a driving helmet is important to avoid cracking resulting from misalignment and/or radial stresses at the top of the pile.
- Evaluations of installed concrete cylinder piles that experienced significant longitudinal cracking showed these piles performing very well for resisting corrosion (Lau 2005).

*LADOTD*

- Attention to the details of vent hole placement (for stress relief) and reinforcing at the top of concrete piles (for driving stresses).
- Based on work for the LA 1 project, unit side resistance appears to be less for the larger diameter piles compared with small diameter piles.

*VDOT*

- The length and weight of LDOEPs requires good construction control to meet installation tolerances.
- Installing LDOEPs as batter piles adds to the difficulty of meeting installation tolerances.
- Good templates by the contractor help reduce the problems of maintaining control of the pile during installation.

## RESEARCH NEEDS IDENTIFIED BY AGENCIES

Through the survey and the interviews agencies using LDOEPs were asked what they perceived as areas needing research to better utilize these types of piles. These general areas included calculating axial resistance and the issues of appropriate resistance factors and installation methods. Specific suggestions included:

- Developing new methods or improving existing methods for calculating static resistance by accounting for the large pile sizes.
- Developing appropriate resistance factors.
- Better understanding of the mechanism of pile plugging, both during driving and under static loading. This also includes research on the effectiveness of forcing a pile to plug.
- Investigating the impact on pile axial resistance if a vibratory hammer is used.
- Correlations of soil resistance during driving to nominal axial static resistance.
- Wave equation modeling of LDOEPs with insert plates or other devices to force the formation of a plug.
- Effects on the formation of the plug when vibratory hammers are used.
- Evaluating the time for setup of LDOEPs compared with closed-end piles or other pile types.
- Determining the most appropriate or applicable failure criteria or mechanism.
- Calibrating resistance factors and static analysis methods to dynamic testing.
- Guidance on how to adequately perform signal matching and wave equation analysis for LDOEPS as compared with smaller piles.
- Better understanding of the effects that the hammer impact on the pile has on pile resistance, particularly in cases of soils where piles do not increase resistance with depth resulting from soil remolding or other phenomena.
- Increasing understanding and reliability of field verification of pile resistance.

CHAPTER FOUR

# PRIVATE-SECTOR PERSPECTIVES ON LARGE DIAMETER OPEN-ENDED PILES

This chapter outlines some of the perspectives of several private-sector individuals on the design, installation, and testing of LDOEPs. The information discussed is based primarily on interviews. Those requesting to be interviewed represent consultants and contractors. A brief introduction of the participants is included first, followed by summaries of the topics discussed. The summaries presented are based on all of the interviews unless specifically noted. Detailed notes of each interview are included in Appendix D.

## PARTICIPANTS

The private-sector participants included:

- Dr. D. Michael Holloway, P.E.—Consulting Engineer
- Mr. Mike Muchard, P.E.—Applied Foundation Testing, Inc.
- Mr. Steven Saye, P.E.—Kiewit
- Dr. Robert Stevens, P.E.—Fugro–McClelland Marine Geosciences, Inc.
- Mr. Scott Webster, P.E.—GRL Engineers, Inc.

Holloway is in a private consulting practice based in the San Francisco Bay area. He has provided on-site consultation and testing services addressing foundation engineering problems for more than 40 years. Holloway specializes in testing and analysis of deep foundation systems, with emphasis on driven pile foundations, marine design and construction, instrumentation, and static and seismic soil–structure interaction. He provides expertise for design, construction, and forensic investigations on assignments nationwide and overseas.

Muchard is a founder of Applied Foundation Testing, Inc., a firm that specializes in deep foundation testing, including dynamic, Statnamic® (rapid), static testing, and instrumentation. Muchard has 25 years of geotechnical engineering, geo-structural engineering, and heavy civil construction experience. His LDOEP experience includes steel piles, spuncast concrete, and bed-cast cylinder piles for deepwater offshore structures, waterfront, and bridge structures. His expertise is in instrumentation, Statnamic® (rapid), dynamic, and static load testing of LDOEPs. This expertise extends into the numerical analysis and interpretation of LDOEP load test measurements. In addition to the testing and instrumentation, he often provides consulting related to the design, drivability, and installation problems associated with LDOEPs.

Saye is a Senior Geotechnical Engineer and Design-Build Geotechnical Technical Lead for Kiewit, supporting projects across North America. Saye has 38 years of experience and is a recognized expert in the implementation of geotechnical engineering for design-build projects, as well as the design of soft soil ground improvements. Key design-build projects involving LDOEP foundations include the Permanent Canal Closure and Pumps project in New Orleans, Louisiana; the Port Mann Bridge Project in Vancouver, British Columbia; and the Pitt River Bridge project in Vancouver, British Columbia.

Stevens has 40 years of experience as a geotechnical engineer, primarily in deep foundation design and testing for offshore projects. He joined the Special Projects Group of McClelland Engineers, Inc. in March 1978, and has since worked on offshore projects with LDOEPs throughout the world. He currently chairs the in-house advisory group for pile driving monitoring and analysis at Fugro–McClelland Marine Geosciences, Inc. He is a member of the Pile Foundations Standards Committee of ASCE and also the Geotechnics Chair for the ASCE Coasts, Oceans, Ports, and Rivers Institute (COPRI) Marine Renewable Energy committee.

Webster has 33 years of experience in construction and testing of driven piles for both offshore and on-shore structures. Working for both STS Consultants and GRL Engineers has allowed him to develop a strong background in dynamic testing for driven piles. Webster has worked extensively since 1986 with dynamic testing and analysis techniques on a variety of driven pile projects. Since about 1994 most of his work has focused on drivability analyses and dynamic pile testing for offshore projects within the United States and abroad.

## ISSUES AND EXPERIENCES

### Pile Plug Behavior

A common issue or theme among the participants was the issue of pile plugging. As noted in chapter two, this is one of the most significant topics of LDOEP behavior that is not completely understood. The participants all mentioned that plugging or the absence of plugging dominates the driving behavior of the pile, so that understanding if it is plugging or not is key to understanding how the pile will drive. Because plugging is not well understood, it is difficult to predict and thus is often treated as if the choice is one or the other: plugged or unplugged. The actual behavior of the pile

is actually somewhere in between. Specifically, how plugging affects driving is not just a function of the soil, but is related to pile diameter and hammer selection.

Although plugging behavior is not well understood and is difficult to predict with certainty the general consensus is that LDOEPs tend to not plug during driving. The general observation is that the piles are advancing as a "cookie cutter" into the soil. Stevens' experiences on more than 250 offshore platforms indicate that plugging is rare. He has witnessed LDOEPs driven up to 300 ft into clay soils with no plugging. This is not to say that plugging during driving does not occur; however, most observations tend to be unplugged. Saye emphasized the need for additional research to add to the understanding of this behavior.

A common question when assessing drivability and plugging is what is the proportion of skin friction development between the outside and the inside surfaces of the pile? There is no clear understanding of how the soil inside the pile behaves and how much of the friction resistance is derived inside the pile and how much outside. Designers often make an assumption based on local experience or rules of thumb. Holloway noted that the old rule of thumb was to assume two-thirds of the friction is outside the pile and one-third inside the pile.

How much skin friction develops inside the pile is probably related to the relative acceleration of the pile to the soil mass inside. Stevens' early work included investigating analyses of plugging behavior by evaluating the acceleration of the soil mass in the pile with respect to the acceleration of the pile. The inertia of the pile is almost always greater than the soil mass under the large forces from the hammer on the pile. Webster noted that the blow of a hydraulic hammer results in higher pile acceleration than that from a diesel or steam hammer, commenting that the hydraulic hammer acceleration is somewhat similar to the action of a vibratory hammer with respect to the acceleration helping to drive the pile with each blow (the action is similar, but the acceleration of vibratory hammers is much smaller). It is thus important that inertial forces be carefully evaluated in the wave equation analyses to attempt to adequately model the pile–soil interaction along the inside of the pile.

Although there is general agreement that most LDOEPs usually do not plug during driving, a majority also agree that the behavior under static loading will usually involve plugged behavior to some degree. There is still enough uncertainty among designers, however, that considering the pile to behave as a friction pile (unplugged) or a displacement pile (plugged) can affect the approach the designer takes when calculating the static resistance of the pile. Plug behavior for long-term static capacity still requires more investigation to be fully understood for pile design.

There have been some recent cases of LDOEPs driven with devices installed inside the pile to encourage partial or full plugging of the pile in an attempt to ensure plugged behavior for long-term static resistance (Muchard et al. 2009; Terracon 2014). Stevens has observed such behavior only a few times, but believed it was relatively successful for achieving a plugged condition.

#### Dynamic and Static Testing

Dynamic testing of LDOEPs is a common method for evaluating pile resistance. However, there is concern in the industry that dynamic testing does not adequately indicate the available pile resistance (see also discussion in chapter two). With the high loads that LDOEPs are capable of, it can be difficult to have a hammer of the appropriate size to verify or test the pile resistance. The lower soil resistance during driving allows for the use of a smaller hammer for installation than may be needed to demonstrate the full available resistance of the pile.

The effect of plug behavior on dynamic testing is not fully understood, implying that it can be very difficult to accurately assess plugging through dynamic testing. Improved instrumentation means that reliable strain measurements on pipe piles can now be obtained to help answer some of the questions about soil plugging. Holloway made special note on evaluating pile plugging through testing, explaining that static or pulse loading tests should be considered to help with plug evaluation either by doing an uplift/pullout test or drilling out the soil in the center of the pile and then doing dynamic testing to quantify friction distribution along the pile shaft. If an internal plate is used to fix the plug in place, soil mass needs to be added to the pile in the WEAP® model for the portion below where the plug is expected to form as well as increasing toe quake.

All of the participants emphasized that proper use of PDA® equipment to monitor and test LDOEPs is essential to proper assessment of pile resistance and pile driving behavior. Variables to be considered when testing LDOEPs include:

- Instrumentation location and quantity. The larger diameters require the use of two sets of gauges to better average the stress–time history across the pile section. For small diameter piles, one set of gauges consisting of two strain transducers and two accelerometers set 180 degrees apart on the pile perimeter is sufficient. For LDOEPs, two sets consisting of four transducers and accelerometers, each placed 90 degrees apart around the perimeter of the pile are needed. Using only one set on an LDOEP as is common on small diameter piles usually yields poor quality data, especially with spiral-weld pipe, where the placement of the instruments can affect the interpretation of the results. Recent experience by Saye on hurricane protection projects in New Orleans, Louisiana, has confirmed the necessity of this practice for LDOEPs.

- The durability of the pile during driving. High required nominal pile resistance can lead to aggressive driving and an increasing risk of pile damage.
- Temperature of piles for instrumentation before and after installation, especially steel pipe piles.
- Accounting for residual stresses from manufacturing in spiralweld pipe piles and concrete piles.
- Sizing the hammer large enough to move the pile sufficiently to mobilize the target nominal resistance to be demonstrated.
- How plugging is interpreted in the data.

Owing to the large size of the pile, static tests are not as common for LDOEPs as for smaller piles, although several case histories are available in the literature, some of which are summarized in chapter five. In some cases, there is significant disagreement between dynamic testing and static testing, with dynamic testing usually under predicting the static pile resistance. Although this is conservative with respect to design, if dynamic testing significantly under predicts that static resistance, designs become inefficient. Some experiences, however, such as those of Stevens generally observe very good agreement (within 5%) of pile resistance determined from CAPWAP® analyses of dynamic test results and that of static load tests.

Saye has observed that load test data for LDOEPs appear to be limited with respect to information on the distribution of resistance with depth. Instrumentation in the lower portions of piles can have difficulty surviving the pile installation process. Until recently, Muchard experienced the same problems with instrument survivability. He reports that significant advances in strain gauge designs for LDOEPs have been made, allowing for greater survival of the instruments. For example, a 280-ft-long LDOEP was strain-instrumented along its length, successfully measuring load distribution during rapid load testing on the Tappan Zee Bridge Pile Demonstration Project (still to be published). Similar recent strain instrumentation successes on LDOEP rapid load tests have been reported by MnDOT. Embedded strain instrumentation has also been installed and monitored in spuncast concrete LDOEPs (Muchard 2005). In one case study, the strain gauges were monitored during the pile posttensioning process, during driving, and then during subsequent load testing.

Muchard emphasizes that the actual static load distribution along the pile has proven beneficial to the understanding of the performance of LDOEPs and is an extremely useful design and verification tool. More of this type of data is needed.

#### Static Axial Resistance Calculations

From the interviews and literature review there emerged a general opinion that the more widely used methods of axial analysis significantly underestimate the pile resistance of LDOEPs. These methods do not appear to adequately capture the influence on axial resistance that the construction practices for these larger piles have. Most of the design methods used by transportation agencies are based on tests performed on small diameter piles. For piles above 36 in. in diameter, there are not as many well-documented case histories of the evaluation of static analysis methods, especially cases where the geotechnical limit state was evaluated. In many cases, significantly more resistance is available than considered in design.

Webster's and Stevens' offshore experiences with regard to why higher resistance values are not taken advantage of are typical of the comments of the others interviewed, as well as the experiences of the authors and others in the practice. Reasons often cited are related to schedule risks rather than a lack of knowledge, although it still occurs:

- Owners and/or designers are unwilling to investigate higher resistance. Owners are willing to trade lower pile resistance for the reduced risk of construction problems or claims (e.g., "We have never had problems in the past doing it this way").
- The project schedule or other constraints do not allow the time or access needed to demonstrate and use higher resistance resulting from soil setup.

In both the agency and private practice interviews, those that realize analysis methods are conservative for LDOEPs generally use static analysis methods for a rough estimate of the pile resistance. These estimates are tempered by experience, with dynamic testing and wave equation analysis interpretations providing better estimates of pile resistance. In some instances, for example temporary works such as trestles, a contractor requires installation of a pile to a certain resistance to be available soon after driving. Being able to calculate the long-term static resistance of the pile is of less consequence than knowing that the pile has sufficient resistance at the end of driving to be placed into service almost immediately.

Although the methods available to transportation structure designers are included in the various FHWA and AASHTO design documents, the offshore industry uses the API RP2 GEO design guide and its associated methods as noted in chapter two. Stevens believes that estimating static resistance with API RP2 GEO is probably the most effective approach and entirely applicable to transportation structures. He has consulted on bridge projects using the API methodology (Bay Bridge in San Francisco and Trans Tokyo Bay Bridge in Japan). API regularly updates the procedures based on data and experiences from the field and ongoing research. Recent updates include modifications to the design of piles in sands based on CPT test data. This procedure provides better estimates of pile resistance in very dense sands; however, he

cautions that care be exercised when estimating the mobilized end bearing for drivability analysis.

One major issue in practice noted by Holloway is the use of different methods for estimating the base and side resistance. For example, using Meyerhoff for side resistance and Nordlund for base resistance. The approach to the pile–soil behavior for each method is different; therefore, mixing methods can lead to poor predictions of total pile resistance.

An issue noted by both the industry and agency interviews is how to account for setup or long-term increase in side resistance over time. Testing LDOEPs after significant setup can be a challenge, as noted in some of the case histories in chapter five. Earlier in this chapter, it was noted that load tests are often not taken to failure; therefore, measuring the full gain with time is not accomplished. In some project environments, such as temporary works or design-build delivery, allowing for setup to occur may create significant negative schedule (and thus cost) impacts. The piles are thus intentionally designed to be less efficient to accommodate the lower resistance without setup.

Another design issue that Holloway notes is that many designers do not account for residual stress in the pile analysis. This leads to overestimating side resistance and underestimating base resistance. In many cases load tests are essentially "proof" tests (without reaching failure) confirming that the structure load can be supported. Muchard adds that this practice can lead to non-conservative designs if the pile lengths are optimized (shortened) on the basis of overestimated side shear resistance. However, this does not help provide a clear understanding of the true resistance and how the soil–pile interaction actually behaves, which can lead to significant inaccuracy using unit resistance values in making adjustments to the pile toe elevations.

#### Driving Behavior

Holloway discussed how it is important that designers recognize that how an open pile drives is highly dependent on how the stresses are reaching the toe of the pile—the failure mechanism at the toe. With relatively thin-wall steel piles, wave equation analysis will sometimes indicate that toe stresses are not very large relative to the yield stress, but that piles still have problems with collapsing during driving, usually because of poor driving alignment resulting in ovaling and the collapse of the piles owing to transverse and eccentric stresses. He recommends that stresses at the toe need to be less than half of the yield stress of the steel to accommodate the eccentric forces encountered at the toe when significant end-bearing or potential obstructions are anticipated.

Saye discussed some experiences of Kiewit in clay soils with significant setup that indicated long delays in driving for splicing have the potential for damaging the long-term pile resistance. A common practice is for a contractor to set the first section of many piles, weld the next section on all of the piles started, and then return to drive the piles after all have been spliced. This practice can sometimes require weeks or even months between the driving of the first section and the re-start of the driving of the first piles that were set. It is possible that re-driving after such a significant time of setup could have a negative effect on the pile shaft resistance as the remolding of the soils along the pile could result in lower pile resistance than expected or would be available had splicing and the re-start of driving occurred within a short time period rather than weeks.

In the practice of "sticking" a lower section of pile before splicing, it is common for a contractor to use a vibratory hammer to install the first section. It is also common for a vibratory hammer to be used to set a pile that is installed in one section. The design of piles for axial loading does not always take into account any vibratory installation, rather it assumes that the piles are impact-driven the entire length. The effect of vibratory installation on pile resistance is not well-understood, with very little comparison of the actual effect of the vibratory hammer on pile installation in clay available. Significant differences in opinion exist as to whether or not vibratory hammer installation has a negative influence on pile resistance, especially in clays. Saye noted that the U.S. Army Corps of Engineers specifications for some of the hurricane protection projects in New Orleans allowed vibrating up to 50% of the pile length in clay before impact driving. Most standard specifications do not address the use of vibratory hammers.

#### Driving Piles to Bear on Rock

Holloway and Stevens provided some insights when driving steel LDOEPs to bear on rock. Holloway has observed instances where hard driving into sedimentary rocks can result in the breakdown of the composition of the rock. Relaxation of base resistance tends to occur as a result of the breakdown of the rock structure, resulting in less long-term base resistance than estimated (substantial toe relaxation). Restrikes frequently show a decrease in base resistance, with piles being driven 1 to 1.5 m into the rock in order to observe base resistance increase again in some cases. Holloway believes that careful evaluation of the impact of the pile at the rock interface is necessary for these piles.

Stevens' experiences offshore have included piles driven through layers of soft rock (very hard clay, shale, siltstone, gypsum, etc.) to achieve resistance. In such conditions, the toe stresses and toe displacements must be carefully monitored. Driving is to be halted if toe stress reaches 80% of the yield stress of pile. If the toe displacement turns negative, it indicates the toe is being damaged or crushed. It is also important to check drivability using 90% end bearing (modeling a sudden fixed-end condition) and evaluating the resulting stresses at the pile toe.

## RESEARCH NEEDS IDENTIFIED BY PRIVATE SECTOR

Areas that were identified by the practitioners as needing additional research and investigation included:

- Comparing dynamic and static load tests to better correlate dynamic testing and wave equation analyses with static resistance.
- Defining the pile movement corresponding to the selected pile load test capacity for the LDOEP static load tests.
- Evaluating the influence of vibratory pile installation on the capacity of piles, including LDOEPs.
- Evaluating the effect of delays in pile installation for splicing on the side resistance capacity of LDOEPs.
- Assembling good quality instructive case histories or databases of LDOEP behavior with cone penetration tests to characterize the soil conditions.
- Investigating the specific differences between steel and concrete LDOEPs comparing the two types on the same site to investigate the differences in driving behavior, plugging, and pile resistance in the same conditions.
- Further investigating pile plugging during driving.
- More load tests investigating distribution of resistance with depth (e.g., strain instrumentation installed along the length of LDOEPs).

CHAPTER FIVE

# CASE HISTORIES

During the course of the literature review and the interviews with agency and private-sector entities, several case histories of uses of LDOEPs were obtained. Many of the case histories included load testing of LDOEPs. Table 2 lists the case histories included in this chapter with basic information for each. This chapter illustrates successful testing and use of LDOEPs, as well as some lessons learned by owners, designers, and contractors. As such, the case histories included here are meant to be a select sample of the many case histories and reports available in the published literature. A complete list of all available case histories was not within the scope of this report. It is important that the reader be aware that the summaries in this report are brief by design and cannot include many of the details of how tests were interpreted. The results presented here are those presented by the authors of the papers reporting each project. The reference for each case history can be consulted for details on test interpretation, site conditions, etc.

As mentioned earlier in this report, one issue among state DOTs is selecting resistance factors for design. Most of these case histories did not include much or any discussion on resistance factor selection, focusing instead on testing and installation.

## PROJECT EXAMPLES

### Hastings Bridge (Hastings, Minnesota)

Dan Brown and Associates, PC (DBA) was the Foundation Engineer for the recently completed Hastings Bridge spanning the Mississippi River in Hastings, Minnesota (Dan Brown and Associates 2010). The key items in this case history include: (1) the increased reliability of the foundation design through demonstrated pile resistance; (2) the issues of vibrations on existing structures; (3) consideration of the limitations of dynamic tests to demonstrate fully mobilized pile resistance for piles driven to refusal on rock; (4) the use of a lateral load test for design; and (5) designing a test program for more than just "verification" of the design.

The Hastings Bridge was constructed adjacent to an existing bridge structure built in the 1950s. The old bridge was founded on large groups of 50-ft-long timber piles that were tipped primarily in fine-grained soils, but with medium-dense granular soils beneath the pile tips. The piles were driven as deep as practical for the era, particularly because they were being installed over water. To maintain the necessary navigation channel and to reduce the required span length the substructure locations for the new bridge were close to those for the old bridge.

The new bridge includes five piers founded on groups of 42-in-diameter open-ended pipe piles. Information regarding the piers where LDOEPs were used is provided in Table 3 and Figure 15; Figures 16 and 17 show pile installation. The geology in this area of Minnesota includes some interesting foundation challenges, including up to a several hundred feet of highly organic and compressible very soft silts and clays to very dense sand and gravel overlying sedimentary bedrock.

Experiences from five previous bridge projects using the same pile sections and constructed in the last 15 years in Minnesota and in similar geologic conditions suggests that these piles penetrate even dense granular materials easily to achieve bearing on rock. In all of these previous instances, a Delmag D125 diesel hammer (Rated Energy = 314,000 ft-lbs; Ram Weight = 27.6 kips) was used. The dynamic test results during initial drive and re-strikes did not fully capture the available geotechnical resistance under static loading conditions in either the soil or for piles bearing on rock.

Initially, the potential impact of vibrations and vibration-induced settlements to the adjacent bridge during pile driving for the new bridge was a significant concern. Some piles were located within 20 ft of the existing bridge and the installation did produce some vibration that was noticeable, but nothing of any consequence and no damage whatsoever to the existing bridge, which was old and in poor condition. An automated and remotely operated instrumentation system was attached to the existing structure at several locations to continuously monitor displacement, tilt, and vibration and provide immediate alarm should any thresholds be exceeded. This system was augmented by manual optical survey measurements at specified time increments throughout the foundation construction duration. Based on these measurements, no adverse effects to the existing structure were observed. It can be postulated that open-ended pipe piles are probably advantageous in that regard because, despite a little bit of vibration associated with driving, there is no appreciable soil displacement as the pile cuts through the soil without plugging during installation.

TABLE 2
CASE HISTORY SUMMARY

| Name | Location | Type | Pile Type | Pile Diameter | Hammer Type | Soil Type | Testing Method | Notes | Reference |
|---|---|---|---|---|---|---|---|---|---|
| Hastings Bridge | Minnesota | Project | Steel pipe | 42 inch | Diesel | Rock | Dynamic; Statnamic® | Test method comparison; design | Dan Brown and Associates (2010) |
| Stoney Creek Bridge | California | Project | Steel pipe | 96 inch | Diesel; hydraulic | Dense sand, stiff clay, gravel | Dynamic, Static | Test method comparison; design | Liebich (2009) |
| Woodrow Wilson Bridge | Virginia/ Maryland | Project | Steel pipe | 54, 42, 36 inch | Hydraulic | Stiff clay; dense sand | Dynamic; Static; Statnamic® | Test method comparison; design | Ellman (2009) |
| St. George Island Bridge | Florida | Project; installation problems | Concrete cylinder (spun-cast) | 54 inch | Not listed | Florida limestone | Dynamic; Static; Statnamic® | Test method comparison; design; pile damage | Kemp and Muchard (2007) |
| Cross Bay Boulevard over North Channel | New York | Project | Concrete cylinder (spun-cast) | 54 inch | Hydraulic | Alluvium; sense sand | Dynamic; Static | Test method comparison; design verification | NYSDOT (1996) |
| Rigolets Pass Bridge Replacement | Louisiana | Project | Concrete cylinder (spun-cast) | 66 inch | Hydraulic | Clay | Dynamic; Statnamic® | Load test method comparison; design | Robertson and Muchard (2007) |
| Trout River Bridge | Florida | Installation problems | Concrete cylinder (bed-cast) | 54 inch | Hydraulic | Florida limestone | Dynamic | Pile damage | Kemp and Muchard (2007) |
| CAPWAP®-Based Correlations | Alaska | Research report | Steel pipe | 12 to 48 inch | Diesel; hydraulic | Dense sand; glacial deposits | Dynamic | Design method developed from CAPWAP® data | Dickenson (2012) |
| Kentucky Lake Bridge | Kentucky | Load test program | Steel pipe | 48 and 72 inch | Hydraulic | Chert residuum (Fort Payne) | Dynamic; Static; Statnamic® | Constrictor plates used to force plug | Terracon (2014) |
| Axial Pile Capacity of Prestressed Concrete Cylinder Piles | Florida | Research report | Steel pipe; concrete cylinder (spun-cast and bed-cast) | 36 to 84 inch | Diesel; hydraulic | Sand, clay, Florida limestone | Dynamic; Static; Statnamic® | Design methods developed | McVay (2004) |
| Oregon Inlet | North Carolina | Load test program | Concrete cylinder (spun-cast) | 66 inch | Hydraulic | Sand; silt | Dynamic; Static | Design | Keaney and Batts (2007) |
| Comparison of Dynamic and Static Tests | Offshore | Research report; load test program | Steel pipe | 36 to 78 inch | Diesel; hydraulic | Sand; clay; silt | Dynamic; Static | Test method comparison; design | Stevens (2013) |
| US-378 Bridge over Pee Dee River | South Carolina | Load test program | Concrete cylinder (spun-cast) | 54 inch | Hydraulic | Clayey sand; sandy silt (Pee Dee) | Dynamic; Statnamic® | Plug formation; test method comparison | S&ME (2008) |

TABLE 3
HASTINGS BRIDGE LDOEP GROUP INFORMATION

| Pier | Group Configuration | Wall Thickness (in.) | Length Prior to Cut-Off (ft) | Embedded Length in Soil (ft) |
|---|---|---|---|---|
| 6 | 3 x 7 Rectangle | 1 | 175 | 135 |
| 7 | 2 x 7 Rectangle | 1 | 185 | 165 |
| 8 | 2 x 7 Rectangle | 1 | 185 | 170 |
| 9 | 4 x 2 x 4 (see Figure 15) | 7/8 | 190 | 175 |
| 10 | 4 x 2 x 4 (see Figure 15) | 7/8 | 185 | 175 |

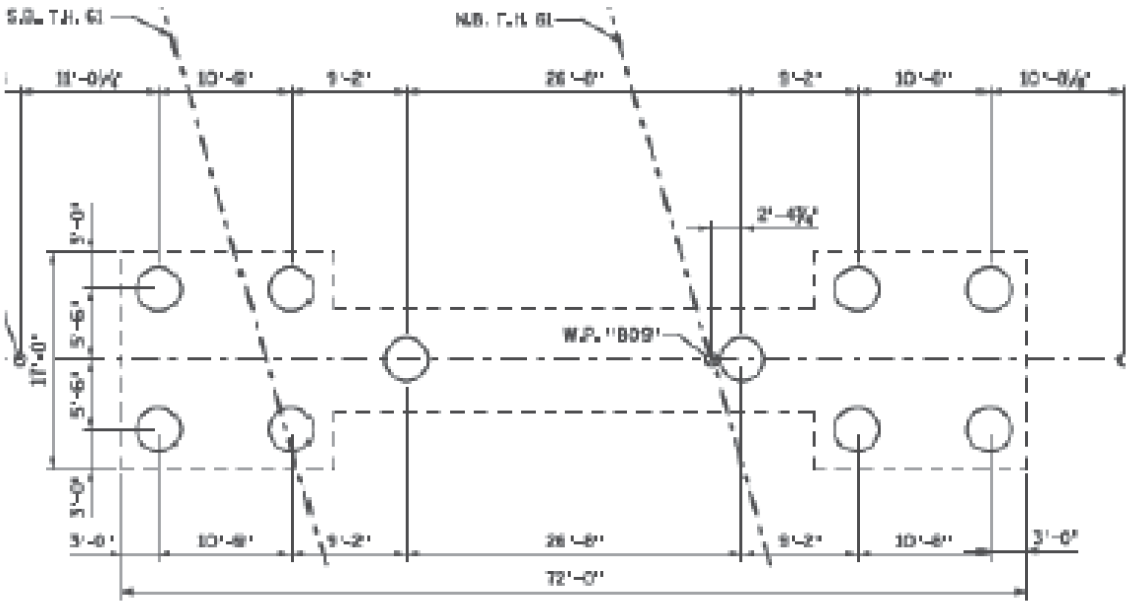


FIGURE 15 Foundation plan view at Hastings Piers 9 and 10.

The pile design was an iterative process between the foundation engineer and the structural engineer. The open collaboration was necessary to optimize the design by utilizing the full structural and geotechnical capacity available. Considering the lateral demand in addition to the axial demand was necessary to optimize the pile section. Ultimately, the design was optimized to a point where both lateral and axial considerations produced controlling conditions under various load combinations.

The 42-in.-diameter pipe pile was selected because it was considered to be the largest section that could be driven with the equipment available to the design-build contractor. Additional evaluations were performed considering a greater number of smaller piles; however, the larger diameter piles were more attractive given the efficiency in resisting lateral demand. In addition, the plan dimension size of the required pile cap and coffercell could be reduced by using fewer larger diameter piles.

Upon arriving at the conclusion that 42-in.-open-ended pipe piles represented the preferred pile, the necessary wall thickness was determined. Accordingly, for the substructures exposed to large vessel collision forces transverse to the bridge, the necessary wall thickness was determined to be


FIGURE 16 Driving Pier 6 test pile.

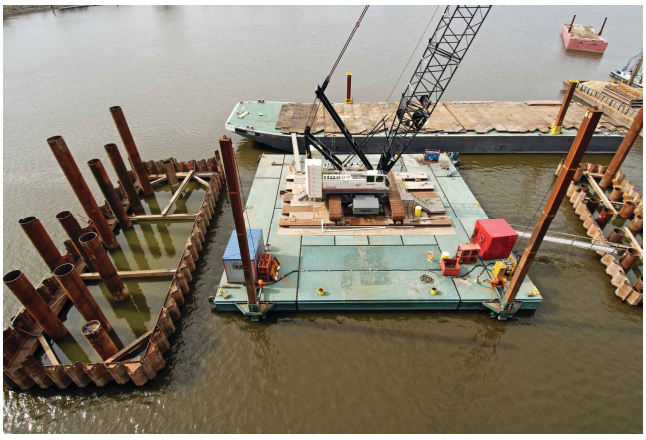

FIGURE 17 Piers 6 and 7 under construction.

1 in. At other locations not exposed to the fully loaded vessel forces, a 7/8-in. wall thickness was necessary as controlled by the strength limit state load combination in the longitudinal direction of the bridge. The pile wall thickness was determined in order to resist the flexural demand, as well as the large axial demand on the outer piles resulting from group action under large lateral loading events.

The amount of soil lost to scour was another important design consideration for this transportation structure. The large diameter pipe piles provided the necessary structural capacity in flexure to resist elevated bending moments following various scour events that had to be associated with corresponding load combinations. A lateral load test was performed on a test pile in the river to calibrate the soil model used for lateral load design.

Finally, the potential for corrosion was evaluated in the design given the required 100-year design life specified by the owner. To account for the potential loss of section as a result of corrosion, a slightly larger wall thickness was specified than needed for structural resistance alone, as recommended by the project corrosion engineer to satisfy the minimum 100-year design life requirement in the Request for Proposal (RFP).

Both dynamic and rapid (Statnamic®) load tests were performed for verification of the large axial resistance available for these piles bearing on rock (Dan Brown and Associates 2010). The implementation of Statnamic® load testing also allowed a higher resistance factor to be used in design at the Strength Limit State (when the factored load exactly equals the factored resistance) in accordance with the project RFP and at the discretion of the designer. Two axial Statnamic® tests were performed using a 10,000-kip gravel-catch device. One test was performed in the river on a pile with a 1 in. wall and the other on the north bank on a pile with a 7/8 in. wall. The results indicated approximately 4,600 kips and 4,200 kips of nominal resistance for the 1-in. and 7/8-in. wall piles, respectively. As anticipated, the behavior of the piles during the load testing was essentially elastic, as the pile heads deflected a maximum of about 2½ in. and rebounded almost completely with permanent sets of around ¼ in.

The dynamic tests indicated about 3,000 to 3,500 kips of resistance with the piles bearing on rock at hammer blow counts indicating practical refusal. Therefore, by definition, that resistance is all that the hammer was capable of demonstrating and the nominal resistance used in design was that determined by Statnamic® testing. Dynamic testing was utilized on many of the production piles to demonstrate: (1) that the piles were driven to a good seating on rock; (2) that the piles were not damaged during installation; and (3) that the hammer was performing as intended.

#### Stoney Creek Bridge (California)

Caltrans published a case history for the Stoney Creek Bridge Project in "High-Capacity Piles at the Stony Creek Bridge Project" (Liebich 2009). This paper presents Caltrans' experience with the advantages and limitations of LDOEPs for a specific project. It presents a successful case of utilizing both dynamic and static testing to realize significant cost savings by using LDOEPs rather than smaller piles. The case also presents a comparison of field tests with design calculations that were based on unplugged behavior under static loading: a portion of the side resistance on the interior of the pile was included in the geotechnical resistance.

As described in this paper, Caltrans uses the terminology "high-capacity piles (HCPs)," which are defined as piles that are larger than 3 ft in diameter or have an axial capacity of greater than 2,000 kips. The piles for the Stoney Creek Bridge Project are 8 ft in diameter, have a wall thickness of 1.75 in., and are embedded 170 ft below grade.

Stoney Creek Bridge is located approximately 100 miles north of Sacramento on Highway 32. The original bridge was completed in 1976, but by 1992 unanticipated scour conditions had partially exposed the bridge foundations. A significant retrofit was performed, including structural elements to protect bridge piers and extensive placement of rip-rap. By 2000 however the structure was found to again require remediation. It was then determined that, as a result of nearby mining operations, the potential scour depth could eventually reach 50 ft.

For the new design, a single LDOEP was located under each of the seven single-column bents. The required pile nominal geotechnical resistance was calculated as 7,800 kips utilizing the full external side resistance along the perimeter of the pile, the base resistance using the wall area of the pile, and the internal side resistance gained from the soil in the lower third of the pile. The soil profile generally consisted of layers of sand, clayey sand, clayey gravel and fat clay with hard to very stiff fine sandy clay, or sandy silt to below expected tip.

The first production pile was installed as a test pile for a static axial load test. To verify the pile resistance for the maximum design scour condition, the test pile was installed within a sheet pile cofferdam excavated to the approximate scour elevation to isolate the pile from side resistance in the scour zone. The pile was initially driven using an APE D100-13 diesel hammer (Rated Energy = 248,000 ft-lbs; Ram Weight = 22 kips), but was completed using an APE HI 750U hydraulic hammer (Rated Energy = 750,000 ft-lbs; Ram Weight = 120 kips), owing to high penetration resistance encountered by the smaller hammer. Liebich (1999) reported that wave equation analyses before construction indicated that the larger hammer would not be sufficient to demonstrate the required resistance after pile setup; therefore, a static test load was planned.

A static axial load test was conducted 28 days after installation, attaining a resistance of 7,860 kips before experiencing a plunging geotechnical failure (Figure 18). Liebich notes in the paper that the load test matching so well with the calculated resistance is more the result of favorable coincidence than repeatable skill.

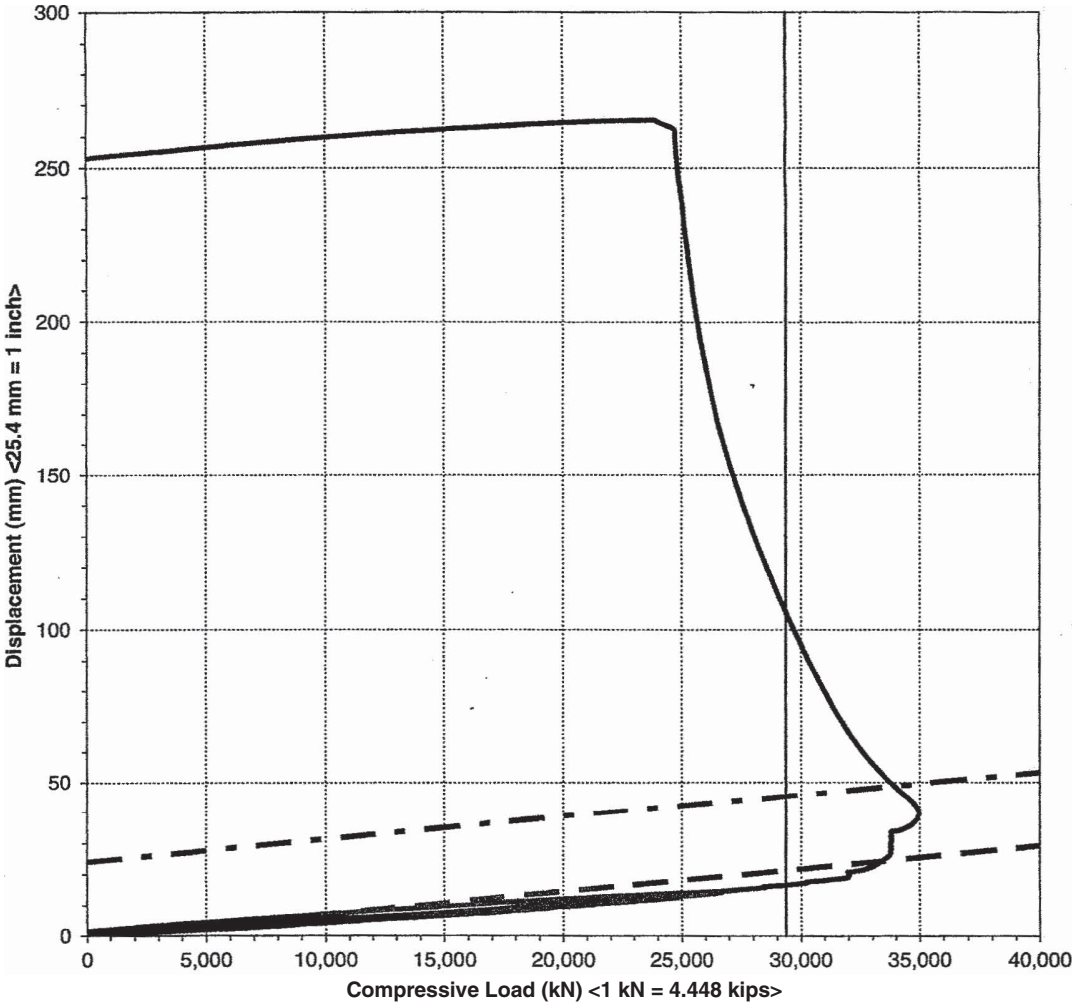


FIGURE 18 Load-displacement plot from Stoney Creek Bridge load test (Liebich 2009).

**Woodrow Wilson Bridge (Virginia/Maryland)**

The replacement of the Woodrow Wilson Bridge between Alexandria, Virginia, and Prince George's County, Maryland, was completed in 2009. A major test pile program was executed to enable efficient design of the planned driven pile foundations (Ellman 2009). The program included load tests on three 54-in.-diameter, three 42-in.-diameter, and one 36-in.-diameter steel pipe piles. All piles had 1-in.-thick walls and all were installed with a ICH S-280 (Rated Energy = 206,000 ft-lbs; Ram Weight = 30 kips) or a ICE-275 (Rated Energy = 110,000 ft-lbs; Ram Weight = 27.5 kips) hydraulic hammer. Testing included PDA® monitoring during driving as well as 7-day and/or 14-day restrikes. Signal matching analyses using CAPWAP® were performed for all dynamic tests. Static load tests were performed on one of each of the three pile sizes. Statnamic® rapid load tests were performed on one 54 in. and one 42 in. pile.

The soil conditions at the bridge site included very deep soft alluvium along much of the alignment, especially at the area of the main channel where the new bascule span was to be located. For the bascule span, the piles would be driven through the alluvium to bear in a stiff to hard clay. Parts of the approaches would bear in the same clay, while others would terminate piles in a dense sand stratum.

Measurements of the side resistance by dynamic testing during driving indicated to the design team that the piles did not plug during driving. Side resistance values computed from CAPWAP® analyses were relatively uniform and low (as expected) for the soft alluvium. The CAPWAP® calculated side resistance in the underlying clay was much more variable than expected. The data from all of the test piles was used to develop an average profile from which select values of side resistance were selected for each stratum. These values were used in wave equation analyses to predict blow counts during driving for comparison with the observed pile driving. However, as noted later, the observed blow count ended up not being the pile acceptance criteria.

The static and Statnamic® load test piles were instrumented to measure the distribution of side resistance and the proportion of side versus base resistance. The total axial resistance and unit side resistance measured by the Statnamic® tests were significantly higher than the values measured by the static tests

TABLE 4
WOODROW WILSON BRIDGE TEST PILE RESULTS

| Location (Hammer) and Pile No. | Pile No. | Pile Diameter (in.) | Total CAPWAP® Resistance (kips) | Static Test Resistance (kips) | Statnamic® Test Resistance (kips) |
|---|---|---|---|---|---|
| PL-1 (IHC S-280) | B | 54 | 4250 | | 5300 |
| PL-1 (IHC S-280) | C | 54 | 3133 | 2929 | |
| PL-2 (IHC S-280) | E | 42 | 2948 | | 4360 |
| PL-2 (IHC S-280) | F | 42 | 2786 | >2000 | |
| PL-3 (ICE-275) | I | 36 | 1324 | >1900 | |

as shown in Table 4. Because of the significant differences, the project team decided that there was poor correlation between the two test methods at this site; therefore, the team conservatively used the lower static test result values.

Ellman noted that the team believed the static test data indicated the pile did behave in a plugged manner under static loading. He also noted that unit side resistance values in the stiff to hard clays appeared low (0.9 to 2.3 ksf) based on the relatively high (7 to 8 ksf) undrained shear strength of the clays. Published test data would indicate values of 3 to 4 ksf for similar strength clays. Ultimately a design value of 1.7 ksf for unit side resistance was selected based on all of the test data and engineering judgment of the designers. Back-calculation of alpha values from the selected unit side resistance values was not included in the paper.

After analysis of all of the test data and performing drivability studies, the design team decided to use a "Specified Tip Criteria" for production pile installation. These criteria required piles be installed to a set tip, not a minimum demonstrated resistance. The relative uniformity of the subsurface conditions for the water spans led the team to select this method to simplify pile installation. The criteria still included dynamic testing to perform quality assurance/quality control testing and check driving stresses.

The design phase testing was deemed to be successful in allowing the optimization of the foundations for the bridge. The results of the test data were extrapolated to design 72-in.-diameter open-ended pipe piles for the bascule span foundations. The Maryland approach for foundations consisted of 48-in., 54-in., and 66-in.-diameter piles (the Virginia approaches were designed for 24-in.-square precast concrete piles). The use of the Specified Tip Criteria was reported to have worked well for the pipe pile installations.

#### St. George Island Bridge (Apalachicola, Florida)

This project utilized segmented spun-cast post-tensioned concrete piles for a new bridge over Apalachicola Bay (Kemp and Muchard 2007). Comparisons of static, rapid (Statnamic®), and dynamic tests were performed on four test piles that included embedded strain instrumentation.

FDOT uses standard pile designs for segmented spun-cast post-tensioned concrete piles (see FDOT interview notes). One of the standard designs consists of a 54-in. outside diameter pile with 8-in.-thick walls, and a specified concrete strength of at least 6,000 psi at the time of application of the pre-stressing force. A 28-day compressive strength of 7,000 psi was used for this particular project. The advantage of using spun-cast cylinder piles for the project was that it allowed the casting yard to start production of pile segments while still awaiting the test pile results to determine the final order lengths of the post-tensioned assembled piles. Casting segments early in the project helped lessen material escalation costs as well as pile driving rig down time for the 646 piles planned for the bridge.

The subsurface conditions at the site generally consisted of fine silty sands over weathered Florida limestone (also called "limerock"). The top of the limestone into which the piles were embedded varied in elevation from −60 ft to −80 ft National Geodetic Vertical Datum (NGVD). The factored load for the piles ranged from 600 to 1944 kips.

The load test program included four static load tests, six Statnamic® load tests, and 50 dynamic tests on production piles. Figure 19 shows the static and Statnamic® test setups. The static and Statnamic® test piles were also subjected to dynamic tests, including CAPWAP® analyses of restrikes.

Reported test results indicated that the static and Statnamic® tests were in reasonable agreement, with the Statnamic® resistance reporting 2% less than the static test resistance for three of the four test piles, and 9% less for the fourth pile. It was also reported that the pile resistance estimated by signal match (CAPWAP®) on the load test piles was 9% to 42% less than the static test resistance, although the time between re-strike and static test, and the details of the signal match models (e.g., plug, radiation damping) were not specified. Table 5 shows the total resistance values.

#### Cross Bay Boulevard over North Channel (Queens County, New York)

NYSDOT compiled a case history of the test pile program for a replacement bridge for Cross Bay Boulevard over the northerly section of Jamaica Bay in Queens County (NYSDOT 1996). The purpose of the case history included an evaluation and correlation of the static and dynamic pile load tests performed on the project in order to develop recommendations for projects utilizing similar foundations in similar soil conditions.

FIGURE 19 Static (*left*) and Statnamic® (*right*) load tests at St. George Island Bridge [Photos by Applied Foundation Testing (Copyright 2000)].

Construction of the replacement bridge structure began in 1988. The northbound half of the bridge was completed first and opened to traffic in 1991. This allowed for the demolition of the existing bridge and construction of the southbound half, which was completed in 1993. The six-lane bridge is 2,842 ft long, supported on two abutments and 33 piers. The approach spans are supported on pile bents, each with six 54-in.-cylindrical spuncast concrete piles. The main channel span Piers 7 and 8 are each supported on sixty 14-in.-square piles. The maximum reported design load on the 54-in. cylinder piles estimated during design was 660 kips. Only air or steam hammers were allowed to drive the piles.

According to the report, "The bridge is located on the south shore of Long Island, an area where the glacial outwash soils (fine to coarse sands) from the Long Island terminal moraine have been modified by wave and wind action. The density of the sands ranges from very loose near the surface to dense and very dense below elevation –65 ft to –75 ft. Soft organic clayey silt, about five to ten feet thick, deposited during post-glacial times in the sheltered waters of Jamaica Bay, covers the sandy soils in the North Channel. The channel is connected to the open sea and is subject to tidal water level fluctuations."

Pile 3 of Pier 26 was subjected to dynamic and static testing. The test pile was jetted to elevation –54 ft and then driven with a CONMACO 5300 hydraulic hammer (Rated Energy = 150,000 ft-lbs; Ram Weight = 30 kips), using a 3 ft stroke, to elevation –75 ft. The pile was then driven an additional 6 in. using a 5 ft stroke. Dynamic testing with

TABLE 5
SUMMARY OF TEST RESULTS FOR ST. GEORGE ISLAND BRIDGE

| Pile No. | Static Load Test Maximum Capacity (kN/tons) | Statnamic® Load Test Maximum Capacity (kN/tons) | CAPWAP® Restrike Maximum Capacity (kN/tons) |
|---|---|---|---|
| LT-1 | 9,493/1,068 | 9,627/1,083 | 8,667/975 |
| LT-2 | 13,813/1,554 | 13,564/1,526 | 8,960/1,008 |
| LT-3 | 13,600/1,530 | 13,831/1,556 | 8,089/910 |
| LT-5 | 12,836/1,444 | 11,689/1,315 | 9,013/1,014 |

*Source*: Kemp and Muchard (2007).

TABLE 6
RIGOLETS PASS BRIDGE TEST PILE RESULTS

| Test Pile | Pile Diameter (in.) | Total CAPWAP® Resistance (kips) | Statnamic® Test Resistance (kips) |
|---|---|---|---|
| TP-2 | 66 | 1245 | 2966 |
| TP-3 | 66 | 2030 | 3077 |
| TP-4 | 66 | 2166 | 3315 |

CAPWAP® analysis indicated a resistance of 659 kips at the EOID. Based on these test results, driving criteria were established that incorporated the jetting as well as both the 3 ft and 5 ft stroke settings.

After completion of the restrike test, a static load test was performed on the pile. The "failure load," or maximum resistance, as interpreted by NYSDOT was 1548 kips, 2.35 times the estimated resistance of 660 kips. The piles did not include any embedded strain gauges to measure the distribution of side and base resistance.

To further investigate the pile behavior, NYSDOT performed a more detailed CAPWAP® analysis after the completion of the project. For this analysis, NYSDOT made two different assumptions regarding the pile plugging behavior: (1) The pile and plug act separately (pile "cookie cuts" through the soil); and (2) the soil plug in the pile is driven together with the pile and adds to the mass of the pile. Pile resistance was calculated separately for each assumption. From these analyses, NYSDOT determined that the detailed CAPWAP® analyses showed relatively good agreement with the static load test at pile displacements of about 0.4 in. This was not initially the case immediately at the conclusion of the test program, when the dynamic testing appeared to be underestimating the total pile resistance. Only after evaluating the data and accounting for plug behavior did NYSDOT develop an approach that provided good agreement between the dynamic and static test results.

Although no direct correlation between dynamic testing and anticipated static pile resistance was developed, NYSDOT applied the knowledge gained to future projects in the form of better understanding the level to which dynamic testing can under-predict pile resistance in similar conditions.

#### Rigolets Pass Bridge Replacement (Slidell, Louisiana)

This project was for the Rigolets Pass Bridge on US-90 near Slidell, Louisiana (Robertson and Muchard 2007). Statnamic® load testing was performed on three 66-in.-diameter, 6-in.-thick wall spun-cast, post-tensioned, concrete cylinder piles. The case history focused on comparison of the Statnamic® results with the dynamic testing.

The soil conditions consisted of sands overlying intermittent layers of dense fine sand and hard clay transitioning to dense fine sand, with the piles tipped into the hard clay layer. The piles were cast with strain instrumentation at four locations along the length of the pile. A Bruce SGH-3013 hydraulic hammer was used to install the test piles (Rated Energy = 282,000 ft-lbs; Ram Weight = 66 kips). Dynamic testing with PDA® was performed on all three test piles during installation, including restrikes.

The pile resistance values calculated from the Statnamic® tests and the CAPWAP® analysis of the re-strikes are shown in Table 6. Analysis of the test data suggested that the dynamic testing did not fully mobilize the side shear resistance as well as the Statnamic® testing. Their conclusions concerning the differences in CAPWAP® and Statnamic® resistance values for this project were the result of (1) the hammer may have had sufficient energy to install the piles but not sufficient energy to fully mobilize the nominal static resistance during re-strikes, and (2) soil plugging behavior that is present during Statnamic® testing may not exist during dynamic testing.

### REPORTED INSTALLATION PROBLEMS AND ADOPTED SOLUTIONS

#### St. George Island Bridge (Apalachicola, Florida)

In addition to the load testing, this case history (Kemp and Muchard 2007) also highlights the investigation and mitigation of longitudinal cracking that occurred in some of the piles during driving. About two-thirds of the way into production pile installation longitudinal cracks were observed in some piles, usually within three to four weeks after driving. The cracks were noticed primarily from evidence of calcium efflorescence (or leaching) that developed. Eventually 7% of the driven piles were determined to have cracks. Observation and mapping of the cracks was performed to further evaluate the problem.

The investigation of the cracking included a review of the pile structural design and the performance of the driving system. Neither of these was determined to be out of the ordinary. Searching available engineering literature on the subject revealed that the problem was not unique, but that very little had been done to isolate the cause or causes. The paper references two significant resources: a report by the Louisiana Transportation Research Center (Avent and Mukai 1998) and a report by the *PCI Journal* (PCI 1993). The Avent and Mukai report (1998) investigated cracks post-construction; therefore, it did not have specific conclusions as to the causes

of the cracks. The report did note that no significant corrosion had occurred, even at ages of up to 40 years. The PCI reported that (1993) "states when driving piles in semi-fluid soils, a soil plug may come up through the hollow of the pile building up a 'hydraulic ram effect' and produces high circumferential tensile forces exerted on the pile walls. This internal loading condition occurs only during installation. It is also known as 'water hammer' and results in what is sometimes referred to as 'hoop stress.' These little understood forces apparently can exceed the pile's hoop stress resistance causing the cracks."

Solutions to reduce the potential for these forces to build up include increasing the number and size of vent holes in the piles, increasing the lateral reinforcement of the pile, or periodically cleaning out the plug. Since the pile segments were already cast for the project the contractor elected to clean out the plug. This resulted in no further cracking observed during installation of the remaining 250 piles on the project.


FIGURE 20 Inside driving bell (after Kemp and Muchard 2007).

#### Trout River Bridge (Jacksonville, Florida)

The Trout River Bridge project (Kemp and Muchard 2007) utilized 54-in. bed-cast concrete piles. The case history focused on the issue of severe spalling of the pile top during installation. Dynamic testing was utilized to monitor pile stress and confirm nominal pile resistance, but no static tests were performed.

The factored design load for the piles was 550 kips. To achieve the required resistance the contractor opted to use an APE 400 hydraulic hammer (Rated Energy = 320,000 ft-lbs; Ram Weight = 80 kips). The rated energy exceeded the minimum requirement of 240,000 ft-lbs included in the project plans. For this project, FDOT used a pile reinforcement arrangement that had an increase in spiral ties throughout the pile length, compared with previous designs.

Despite the increased reinforcement in the piles, the first three piles experienced substantial cracking and spalling problems near the top three to five feet of the pile. The first pile had such sever spalling that the pile head had to be cut off below the damaged portion in order to continue driving.

The investigation of the root cause of the cracking eventually focused on the driving system and the geometry of the custom-made helmet, which was suspected of causing localized large driving stresses. The particular hammer used by the contractor had been special ordered for the project and included a driving bell to maintain alignment of the pile within the driving system. The driving bell had an inner ring to hold the pile cushion in place above the pile. This ring extended a relatively small distance into the pile void when the hammer was set on the pile. The driving bell also had ribs along its skirt in four places that fit closely to the outside of the pile. Up to ½ in. of free movement of the pile was possible between the inner and outer restraints. The free movement allowed the mechanism to be out of vertical alignment, causing the pile to get "pinched" in the bell during each blow of the hammer. The pinching action from the tilted bell resulted in cracks that eventually led to spalling after sufficient hammer blows.

The reported solution was to add shims between the inner and outer alignment ribs within the driving bell (Figure 20) to reduce the free play that had developed. After adding wedge plates as shims and removing the inner diameter pilot ring the extent of cracking and spalling was greatly reduced. Only infrequent minor cracking was observed after the modifications.

## LOAD TEST CASE HISTORIES AND RESEARCH REPORTS

During the literature review for this report several published case histories highlighting load test programs on LDOEPs were found, as well as research reports investigating pile behavior. The following is a list of some of the notable case histories with a brief summary of the project conditions and pile configurations. These are presented to allow the reader to quickly identify published histories or reports that may be useful for a specific project or condition.

#### CAPWAP®-Based Correlations for Estimating the Static Axial Capacity of Open-Ended Steel Pipe Piles in Alaska

This report (Dickenson 2012) compiled CAPWAP® analyses of data from PDA® monitoring of open-ended steel pipe piles at 32 bridge sites across Alaska. The piles ranged in diameter from 12 in. to 48 in., with depths ranging from 23 ft to 161 ft in mostly cohesionless soils. The database contains analyses for 68 piles, 33 of which include data for EOID and beginning of restrike. The report presents region-specific, empirical relationships for unit shaft resistance and unit toe resistance at BOR.

The predominate soil type for bridge projects in Alaska, and hence the predominate soil type in the database, is cohesionless soil deposits. The deposits in the state can be broadly categorized as Class 1 (normally consolidated) and Class 2 (highly overconsolidated). Class 1 is the most common, with Class 2 being found primarily in the Anchorage area. For estimating unit side resistance the report provides a CAPWAP®-based relationship of stress-normalized unit shaft resistance ($f_s/\sigma'_v$) with depth developed from the database. The relationship for Class 1 soils is shown in Figure 21. For unit side resistance in cohesive soils, the report recommends the alpha and beta methods in the current FHWA design manual (i.e., Hannigan et al. 2006).

For unit toe resistance, the report recommends the methods outlined in the FHWA manual supplemented with the range of "equivalent" unit toe resistance values presented in the report computed from CAPWAP® analyses in the database. Dickenson designates the toe resistance values as "equivalent" owing to the impact on how the PDA® data are evaluated with respect to assumptions of pile plugging.

The proposed relationships were used to predict pile resistance on a project with 29 monitored piles driven into silt-rich deltaic deposits. Dickenson reported "Overall, the agreement between the predictions and the CAPWAP® results was good to excellent, and the proposed method provided much more reliable ranges of estimated pile resistance than obtained using widely adopted, standard of practice procedures."

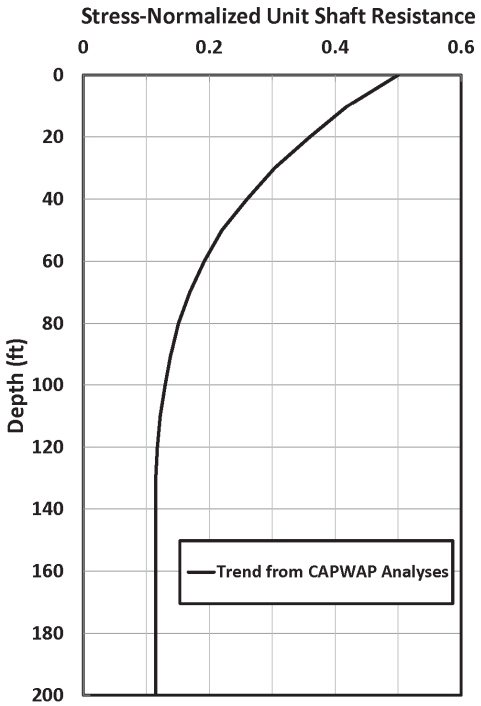


FIGURE 21 Trend of stress-normalized unit shaft resistance ($f_s/\sigma'_v$) with depth for site Class 1 soils (Dickenson 2012).

**Kentucky Lake Bridge Pile Load Test Program**

A design-phase pile load test program was performed to study the constructability and design of 48-in. and 72-in. steel LDOEPs for the Kentucky Lakes Bridge project in Marshall and Trigg counties, Kentucky (Terracon 2014). The piles were installed with a Menck MHU 800S hydraulic hammer (Rated Energy = 604,000 ft-lbs; Ram Weight = 100 kips). The testing included dynamic, static axial, Statnamic® axial, and Statnamic® lateral tests on piles with wall thicknesses ranging from 1 to 2 in. Vibration monitoring was also performed.

There were several design issues that led to considering LDOEPs and having a test pile program for this particular project. The two main factors were the efficiency of fewer, larger piles providing greater stiffness for seismic and impact loads, and the difficult soil and/or geologic conditions for drilled shafts and smaller piles at the site. The site was underlain by residuum of the Fort Payne Formation—a chert residuum that behaves as a dense gravel, with some silt and clay layers. This formation creates potential difficulties with keeping drilled holes open, usually requiring full-depth temporary casing. The formation is known to create difficulties in achieving clean shaft bottom conditions and is also known to cause excessive wear on drilling tools, driving up costs for contractors. Other pile types were problematic because closed-end piles would not be able to reach the bearing strata and there was concern that small diameter open-ended piles would plug easily and not reach the bearing strata.

An important question for the project was how could the certainty of the plug developing and its location be better determined? The location of a significant chert stratum directed the design details in that the plug would be relied on to either achieve penetration into the chert or to achieve the nominal pile resistance. The team wanted to keep the pile open to get it down relatively easily, but needed it to plug to get the pile to drive the minimum distance into the chert required for fixity. It was believed that having the piles plug and thus act as a displacement pile when penetrating the chert would allow the piles to more easily penetrate the chert than if the piles cut into it. There was concern on the part of the design team that the pile toe would be damaged if the piles attempted to cut into the chert rather than displace the material. The team used the load test program to experiment with where to create the plug, designing steel constrictor plates to be installed inside the pile to attempt to form the plug at a fixed location. A schematic of the one of the plates is shown in Figure 22. The design evaluations included discussions between the design team and Paikowsky and his work on the Sakonnet River Bridge in Rhode Island (Paikowsky 2011).

Another aspect of the testing program was a detailed evaluation of dynamic pile testing records to consider methods of improving the match quality and the estimation of static pile resistance when considering plugging. Comparisons

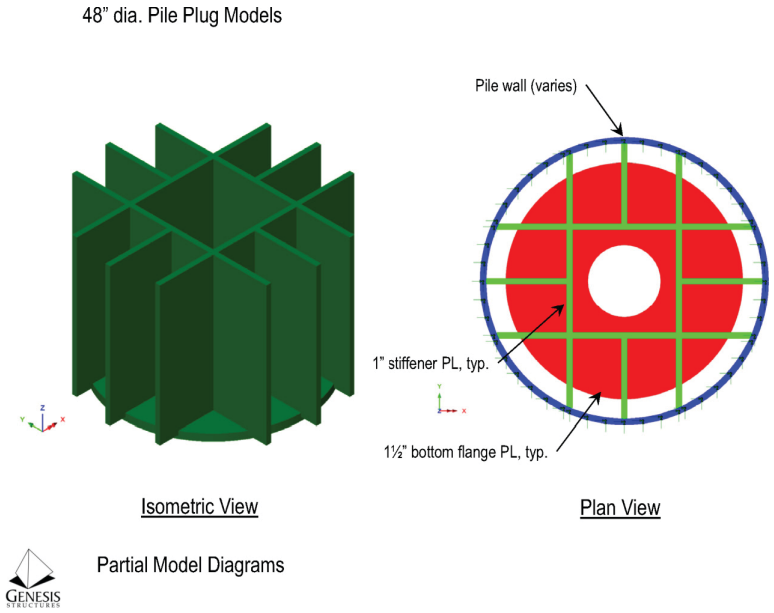


FIGURE 22 Constrictor plate schematic (*courtesy*: Genesis Structures, Inc. and KYTC).

of analyses using a single-toe pile model and a double-toe model were made, as well as application of radial or radiation damping models. The radiation model assumes some energy is radiated away from the pile tip instead of being completely confined to static and dynamic responses of the soil shear along the pile and at the pile toe. The following concerning the analyses can be reported:

- For the dynamic records where radiation damping was applied the model generally resulted in a significantly better signal match quality, indicating that the radiation damping allows CAPWAP® to better model the signals recorded by the dynamic pile testing equipment.
- The pile resistances calculated with CAPWAP® using the radiation damping model also generally produced higher end-bearing resistance values than the CAPWAP® models without the radiation damping. It appears that the radiation damping model is better suited for estimating the end-bearing component of the piles when less pile set is experienced per hammer blow. This is the case when the constrictor plates are engaged on the dense granular soils.
- Wave equation analyses indicated that plugged piles would have high stresses. In addition, there was concern that localized high stresses might be encountered owing to the presence of the chert. Testing on the piles typically did not approach as high values as expected.

Terracon and KYTC used the API RP 2A method for their comparative analysis of static resistance calculations to the load test results. The experience of Terracon with this method for similar piles is that it provides a better prediction of static resistance than the FHWA methods. An example of their analyses is provided in Figure 23, which shows the load test and calculated static resistance for the 48-in.-diameter piles in the shallow water test location. Table 7 lists the reported CAPWAP® and estimated static resistance of the test piles from the static or Statnamic® tests. Some of the key conclusions reported for the testing program were:

- The tests indicated that the 48-in. piles were more likely to achieve plugging with the plates at the target stratum for the load test program than were the 72-in. piles. The results of the test program were used to revise the target stratum for positioning the plates for the production piles.
- Vibrations generated by driving the selected piles would not likely be damaging to the adjacent existing bridge; however, monitoring was recommended for all adjacent existing bridge piers during production pile installation.
- Dynamic testing was generally underestimating the static pile resistance as determined by the static and Statnamic® tests. The use of radiation damping models generally reduced the magnitude of the underestimates. The driving criteria for production piles can be developed based on the lower dynamic resistance correlated to the static resistance.

### Determination of Axial Pile Capacity of Prestressed Concrete Cylinder Piles

This research (McVay 2004) evaluated steel and concrete LDOEPs under driving and static loading conditions, including special attention to plug behavior under both conditions. Load test data for 35 tests (22 concrete piles and 13 steel piles) on pile diameters ranging from 36 to 84 in. from 11 projects were assembled and analyzed. The data were obtained from

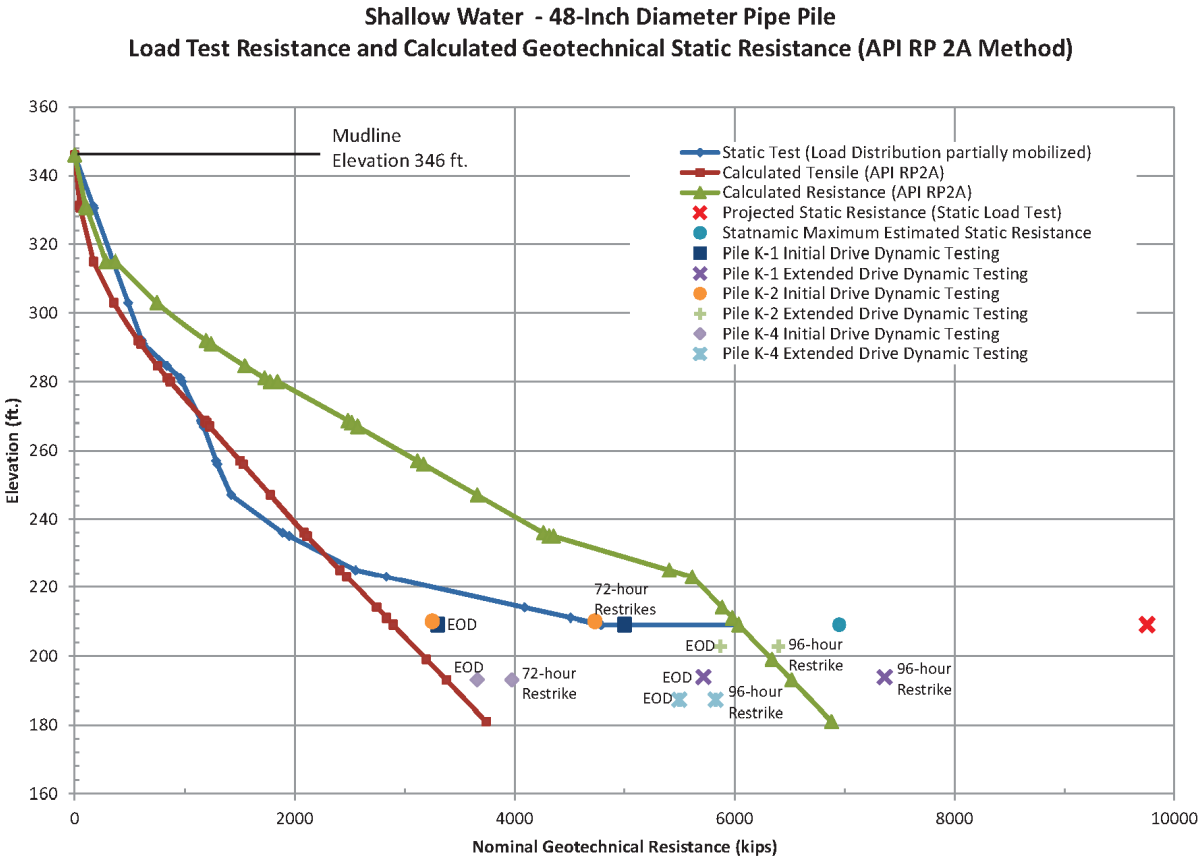


FIGURE 23 Load test and calculated geotechnical resistance analysis (Terracon 2014).

FDOT, Caltrans, VDOT, NCDOT, and MSHA. A summary of the work was also presented in Lai et al. (2008).

The load test analyses focused on developing equations for unit side resistance and unit end bearing as a function of SPT *N* values. Two approaches were used depending on the data available for each test: a Direct Method using strain gauge data from instrumented piles and an Indirect Method plotting load versus deflection data on arithmetic scale to determine the Davisson's Capacity ("failure" load) and on a log–log scale to use DeBeer's Method to determine the distribution of side and tip resistance. The report provided relationships for unit side and base resistance as a function of SPT *N* for different soil types

Analysis of pile plug behavior during driving was evaluated by a parametric study of pile diameter on inertia force and required soil column height inside the pile to generate enough friction to cause plugging to occur. This study was done for one soil type (silt) and one pile length (80 ft) for inside pile diameters of 54, 38, 24, 20, 18, and 12 in. For each diameter a critical g-force was determined for the soil column. For pile accelerations below this level, the soil plugs the pile and the soil column moves down with the pile. For pile accelerations above the critical g-force the pile does not plug and "cookie cuts" into the soil. In general, the report noted that prestressed concrete piles driven in accordance to FDOT specifications typically have accelerations from 40 g to 60 g, well above the critical g-force values for typical concrete pile diameters (15 g for a 54-in.-diameter pile).

To evaluate potential plugged behavior under static loading, finite element modeling was performed. The analysis suggested that for typical pile wall thicknesses, soil strengths,

TABLE 7
KENTUCKY LAKES BRIDGE TEST PILE RESULTS

| Test Pile | Pile Diameter (in.) | EOD CAPWAP® Resistance (kips) | 72-hour Restrike CAPWAP® Resistance (kips) | Estimated Static Resistance (kips) |
|---|---|---|---|---|
| K-1 | 48 | 3300 | 5000 | 9550 (Static test) |
| K-2 | 48 | 3250 | 4730 | 6952 (Statnamic® test) |
| K-3 | 72 | 3800 | 5200 | 8511 (Statnamic® test) |

and rate of loading for FDOT projects, plugged behavior during static loading was feasible.

LRFD resistance factors for design of concrete cylinder piles were also developed during this research. McVay used the advanced first order second moment method to perform the reliability calibrations and derive recommended resistance factors from the load test data. Two approaches were investigated; Approach 1 utilized only the "ring area" of the pile in the resistance calculations—only the cross section of the pile material was used, neglecting the void in the center. Approach 2 utilized the full cross-section area of the pile. Using a target reliability index, $\beta_T = 2.75$ based on previous FDOT LRFD calibration work; a resistance factor of $\varphi = 0.76$ was obtained for Approach 1 and $\varphi = 0.61$ for Approach 2.

#### Oregon Inlet (North Carolina)

In 1996, a 66-in.-diameter, 140-ft-long prestressed post-tensioned concrete cylinder test pile was driven and load tested at Oregon Inlet, North Carolina, for the design of a new replacement bridge (Keaney and Batts 2007). The pile was driven using an HPSI 3505 hydraulic hammer (Rated Energy = 176,000 ft-lbs; Ram Weight = 35 kips). The pile was jetted and driven while monitored with PDA®, including a restrike test 16 hours after the EOID. Difficulties with maintaining hammer alignment were observed with the eccentric stresses measured by the PDA® during driving, but were deemed to not affect the pile penetration.

At the EOID, the pile tip was stopped about 10 ft short of the planned tip elevation. During the restrike test, no additional movement of the pile was observed, even with the jets engaged at full pressure. The contractor ended up cutting off the top 10 ft of the pile to accommodate the designed reaction frame system for the static axial load test.

The static load test was performed to a maximum test load of 1962 kips (220% of the pile design load). The authors reported a resistance of 1266 kips calculated from CAPWAP® analysis of the restrike data, but noted that the restrike did not fully mobilize the pile resistance. There was no mention of an attempt to use superpositioning of EOID and restrike data to calculate the pile resistance. The static test resistance was about 50% greater than the CAPWAP® calculated resistance, indicating that the restrike did not fully mobilize the pile resistance.

### A Comparison of Dynamic and Static Pile Test Results

A paper presented at the 2013 Offshore Technology Conference compared dynamic and static load tests performed on three offshore projects in mixed sand and clay profiles (Stevens 2013). The LDOEPs installed were steel pipe ranging from 36 in. (914 mm) to 78.7 in. (2,000 mm) in diameter. The observed and predicted pile resistance values were in reasonable agreement, including use of CAPWAP® analyses to evaluate contributions of side resistance.

The first project included two 48-in.-diameter steel pipe piles driven with a Junttan HHK 14S hydraulic hammer (Rated Energy = 152,000 ft-lbs; Ram Weight = 31 kips) up to 65 ft into a sandy soil profile. Restrikes were performed 2 and 27 days after initial drive. CAPWAP® analyses of both initial drive and restrike showed little change in the shaft resistance and end bearing. The shaft resistance obtained from the CAPWAP® analyses was 1290 and 2530 kips at 40 and 65 ft penetration, respectively. These values were compared directly with the results of pull-out tests performed 52 and 46 days after restrike. The piles were reported to have a tension resistance of 1180 and 2530 kips at 40 and 65 ft penetration, respectively. The test results were considered to be in good agreement with the resistances predicted from the CAPWAP® analysis.

The second project utilized 36-in.-diameter steel pipe pile initially vibrated with an HPSI Model 1600 vibratory driver and then impact-driven using a Berminghammer B-6505 diesel hammer (Rated Energy = 203,000 ft-lbs; Ram Weight = 17.6 kips) in dense to very dense silty sand to sand, stiff to very stiff clay, and very dense silty sand. The pile was not monitored during initial drive. CAPWAP® analyses were performed at the beginning and end of restrike to estimate the pile resistance as 4337 kips. A static load test was performed 5.5 months after driving the pile; however, it was only tested to 3400 kips (twice the design load of 1700 kips) with a maximum displacement of 1.5 in. The test was not conducted past the specified maximum test load in order to compare with the predicted CAPWAP® resistance.

The third project included 78-in.-diameter steel pipe pile driven with a Menck MRBS 5000 steam hammer (Rated Energy = 542,000 ft-lbs; Ram Weight = 110 kips) to a penetration of 96 ft in interlayered very stiff clay and very dense sand. The test program was designed to evaluate pile setup. CAPWAP® analysis of PDA® data for the EOID and the beginning of restrike from one of the reaction piles was used to predict the test pile pull-out resistance for a static load test performed 52 days after driving. A delay of 43 hours occurred between EOID and restrike. An estimated dissipation of pore pressures was used to calculate the potential setup 52 days after driving from the 2-day setup determined from the CAPWAP® analysis. A compressive static load test was performed with a test pile instrumented with strain gauges to determine the load distribution along the pile. The shaft resistance calculated from the load test was 5875 kips. This compared favorably with the authors predicted 5950 kips from the CAPWAP® data and pore pressure dissipation model.

#### US 378 Bridge over Pee Dee River (South Carolina)

The South Carolina DOT executed a comprehensive foundation test program for this bridge replacement project (S&ME 2008). The program included dynamic and Statnamic® load

testing of two 54-in.-diameter prestressed concrete cylinder piles, an 18-in.-square prestressed concrete pile, and a 5-ft-diameter drilled shaft. The two 54-in.-cylinder piles (designated Cylinder Pile A and Cylinder Pile B) included embedded instrumentation. Both were installed with an APE 400u hydraulic hammer (Rated Energy = 400,000 ft-lbs; Ram Weight = 80 kips). The piles were installed through sandy overburden into the Pee Dee Formation, a calcareous clayey sand to sandy silt.

To monitor the formation of a plug in the interior of the pile, a simple device called a pile plug monitoring device (PPMD) was constructed. The PPMD consisted of lead weights attached to a 100-ft fiberglass measuring tape. The weights would fall to the top of the soil column inside of the piles, allowing the distance to the soil to be computed. Access to the interior of the pile was made through a vent hole near the top of the pile. The PPMDs were read intermittently throughout test pile installation. The data showed that soil was rising inside both piles during driving indicating that disturbed soil and water was accumulating in the pile rather than a pile plug forming and traveling down with the pile.

Dynamic testing of the cylinder piles was performed during installation and restrikes, followed by Statnamic® testing. Cylinder Pile A was dynamically monitored during a restrike 3 days after installation and again 20 days after installation (1 day after Statnamic® testing). Cylinder Pile B was only monitored during a 21-day restrike (1 day after Statnamic® testing). CAPWAP® analyses were performed on all dynamic tests.

Statnamic® tests were performed on both test piles using a maximum derived static load of 2800 kips for Cylinder Pile A and 2500 kips for Cylinder Pile B. The strain gauge instrumentation was used to calculate the load distribution and the unit resistance values for each pile.

Figure 24 was presented in the report to summarize the dynamic and Statnamic® test results, including the calculated unit side and base resistance values. It was determined that for Pile A the dynamic resistance was 20% lower than the resistance from the Statnamic® test, while the dynamic resistance was 10% higher than the Statnamic® resistance for Pile B.

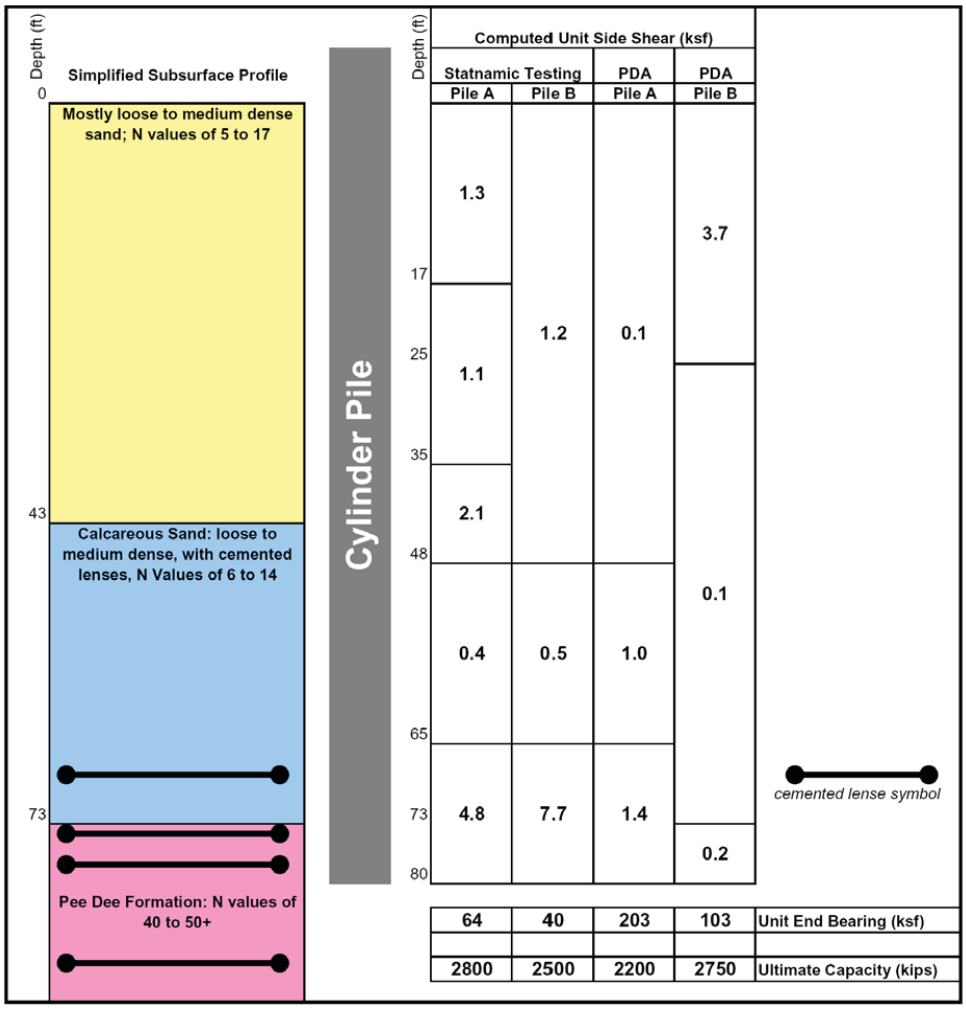


FIGURE 24 Test results summary for Pee Dee River Bridge load test (S&ME 2008).

CHAPTER SIX

# CONCLUSIONS

Synthesis Topic 45-05 gathered information on the current practices regarding the selection, use, design, construction, and quality control of large diameter open-ended piles (LDOEPs) for transportation structures. Although owners, designers, and contractors are considering LDOEPs more often as a foundation type to resist large lateral, seismic, and axial loads, there is no guidance in either AASHTO or FHWA design references specific to LDOEPs. The objectives of this study were to locate and assemble documented practices and experiences on the use, selection, design, construction, quality control, and performance of LDOEPs, and to then identify what problems remain largely unsolved that need further research.

The conclusions presented in this chapter are based on the information gathered from the literature review, the survey of state agencies, interviews of selected state agencies, and interviews of industry practitioners. The discussion has been organized according to four major topics that were found to be key factors in the selection, use, design, and construction of LDOEPs; pile behavior, pile modeling, estimating static resistance, and testing.

## PILE BEHAVIOR

The uncertainty in behavior associated with the soil plug within the pile during driving, testing, and static loading represents challenges that are unique to LDOEPs. Many agencies and private designers have an understanding of pile plugging based on small diameter open-ended piles and can make assumptions on behavior that are not true for LDOEPs (i.e., assume that a plug will form during driving). The general consensus of the private practitioners was that LDOEPs usually do not plug during driving. This is borne out in much of the research done specifically on LDOEPs, particularly in the offshore industry.

During driving of LDOEPs, the soil plug or column inside of an LDOEP has a much greater mass with respect to the pile than for a smaller diameter pile, so there is more inertial resistance against downward movement for the soil mass than there is for the pile. It is therefore common in dense granular soils for an LDOEP to advance without plugging during installation, even though the pile may behave like a fully plugged pile during static loading and only then develop substantial additional end bearing. When LDOEPs are subject to static loads, the pile and plug tend to move downward together since the inertia of the plug under the slower rate of static loading is overcome by the internal friction of the pile against the soil.

In many cases, steel LDOEPs are driven to bear on rock or other hard strata. Since plugging in soil is unlikely, plugging during driving is typically not an issue for steel LDOEPs driven to rock or similarly hard bearing material. Analyses of the mechanics of plug behavior in this report and in the reviewed literature indicate that with properly sized driving systems plugging of the pile during installation is an unlikely event.

Where steel LDOEPs derive resistance from soil, it is likely that the magnitude of base resistance in soil during initial installation is likely to be low, and the resistance to penetration comes from side resistance from friction or adhesion on the outside of the pile as well as from the friction or adhesion of the soil plug acting on the inside surface. The relative magnitude of the two components of side resistance is uncertain and potentially complicated by soil remolding, transient pore water pressures, or features such as a pile shoe or splice.

The survey indicates that most transportation agencies have relatively little experience with these types of piles and those who do have significant experience identified uncertainty relating to the soil plug and pile behavior during installation as a significant issue.

Concrete LDOEPs are not commonly driven to bear on hard rock strata, but are typically designed as a long friction pile or bearing on a dense sand or weak rock layer. Because of their thicker walls and smaller voids, the potential for a concrete LDOEP to experience plugging during driving is somewhat greater as a result of the slower downward acceleration of a concrete pile, the larger volume of soil displaced by the pile wall, and the smaller void diameter increasing the potential for sandy soil arching within the void. However, the plugging of concrete LDOEPs during driving is still a relatively unlikely occurrence based on the studies reviewed for this report. Some bulking of material inside the pile void has been observed, sometimes requiring removal to advance the pile, but actual plugging of the pile still remains relatively rare.

An aspect of behavior that can be a major issue for both steel and concrete LDOEPs is setup, or the increase in resistance with time, and most agencies recognize this to be an

important component of design and testing. The large size and high required test loads can make measuring the full setup of an LDOEP difficult and expensive; therefore, the full potential for setup is not known. The large pile diameters also introduce differences in soil–pile interaction with respect to soil disturbance, remolding, and pore pressure dissipation that all affect friction resistance. The larger volume of soil displaced by a concrete LDOEP can have significantly different effects on the frictional resistance and potential setup when compared with steel LDOEPs.

## MODELING

Modeling of an LDOEP during driving and during static loading is important to the successful use of LDOEPs and is closely related to understanding pile behavior, especially plugging. The modeling of the pile and the soil plug is very important for wave equation analyses and CAPWAP® analyses. The agencies and individuals interviewed have a few different approaches, depending on their base assumption of plugging or lack of plugging during driving. The approaches to dynamic modeling of the pile typically include some adjustments to the mass of the pile or the soil to achieve the desired effect—plugged or unplugged behavior. Since the testing methods and computer software for analysis were developed with small piles, usually not open-ended piles, it can be difficult for an agency, designer, or contractor to be sure that the models are reflecting actual behavior in the field. Uncertainty in modeling can lead to less confidence in the reliability of the design.

How pile setup is approached in modeling is also important to how LDOEPs are designed and installed. As noted later in this chapter, testing of LDOEPs can be limited as to how much resistance can be measured cost-effectively. Without an understanding of the magnitude of setup designs can often not take full advantage of the robust load resistance available from LDOEPs, leading to less efficient designs than could potentially be achieved with better understanding of LDOEP behavior.

## ESTIMATING STATIC AXIAL RESISTANCE

Most agencies and private practitioners involved in the design of LDOEPs for transportation structures rely on the methods and guidance contained in the AASHTO codes and FHWA design manuals. The empirical methods for computing axial resistance in the current AASHTO and FHWA guidance were not developed specifically for LDOEPs or even based on data from load-tested LDOEPs, rather they are based on small diameter mostly closed or solid pile.

Although there is still concern among most agencies about the applicability of methods based on load tests from smaller diameter mostly closed end piles, the agencies realize that these may be the "best available" until more is understood about LDOEPs. Some agencies undertake the use of dynamic testing data to make adjustments to the FHWA methods during design; utilizing their previous experience to help better estimate static axial resistance. Several agencies reported that they have developed in-house methods for estimating pile axial resistance, utilizing correlations to historical installation data, but still mostly on small diameter piles.

Outside of the transportation sector, most references in the recent literature do not give serious attention to the FHWA and AASHTO procedures for computing nominal axial resistance of LDOEPs. The most widely referenced procedure for the design of large diameter open-ended steel pipe piles is the API RP 2GEO (2010) procedures for offshore pile foundations. Within API RP 2GEO are several different methods for estimating axial resistance including several based on cone penetration test data. Some of the approaches in American Petroleum Institute are similar to the FHWA methods, but include consideration of some of the factors unique to LDOEPs. The extensive experience and large body of research of the offshore industry that has gone into the development of the methods contained in API RP 2GEO provide a sound basis for design that can be readily adapted from offshore structures to transportation structures.

Concerning the selection of Load and Resistance Factor Design (LRFD) resistance factors for LDOEPs, agencies are typically following AASHTO guidance for driven piles. As with the methods for estimating axial resistance, the various resistance factors for driven piles included in the AASHTO code do not specifically distinguish between small diameter piles and LDOEPs, being developed from a database of load tests from mostly small diameter piles and few open-ended piles of any diameter. As noted in this report, the behavioral differences of LDOEPs directly affect the driving resistance of the pile, which is used as an indication of long-term axial resistance and directly affects the reliability of our testing methods and thus the reliability of our completed design. Because the LRFD approach to design is intended to provide a reliability-based design methodology, it is important that the resistance factors employed for LDOEPs reflect the reliability of this type of pile and all of the behaviors unique to LDOEPs. This means that different resistance factors may be needed for LDOEPs versus small diameter or solid piles.

## TESTING

Test methods and equipment have developed significantly over the last several decades, making testing of LDOEPs less of a challenge than in the past. However, there is still a lack of confidence in the reliability of high-strain dynamic testing of LDOEPs as an indication of axial resistance. In many respects, potential differences in fundamental behavior affect the interpretation of dynamic measurements with respect to predicting static response. The understanding of dynamic testing and analysis is still based predominately on data obtained from small diameter piles, although the body of data from

LDOEPs is increasing. Even with the general lack of confidence, most agencies apply high-strain dynamic testing to verify axial resistance as well as to establish hammer operating procedures to minimize the risk of pile damage. Agencies have a significant reliance on the driving resistance of piles to assess the effectiveness of the design; therefore, having a better understanding of the behavior of LDOEPs subject to high-strain dynamic testing will improve the confidence in the test results and thus improve reliability of the designs.

A key concern of agencies with dynamic testing of LDOEPs is that often the axial resistance that can be observed during dynamic testing is limited by the ability of the hammer and driving system to mobilize the resistance. This limitation often means that the nominal resistance of the pile cannot be measured, thus reducing the reliability or efficiency of the design. Acceptance of a pile often includes demonstrating the nominal resistance relied on for design. If that value cannot be measured or otherwise demonstrated, lower nominal values than what is truly available are used for design. Hammer limitations also can result in a misinterpretation of a dynamic measurement on a restrike blow to conclude that relaxation at the pile toe has occurred when actually the setup in side resistance has reduced the energy reaching the pile toe, reducing the mobilized base resistance.

Most of the agencies surveyed use high-strain dynamic testing to measure or demonstrate pile resistance as well as to monitor driving stresses to reduce pile damage. Some use it exclusively (e.g., Alaska Department of Transportation and Public Facilities), whereas others use it alone or with rapid and/or static load tests (e.g., Florida DOT). Most use restrikes to develop setup curves and establish driving criteria (e.g., Louisiana Department of Transportation and Development). However, the full value of setup available is often not measured because of the limitations of the hammer to mobilize the resistance of the pile.

The industry participants emphasized that proper use of PDA® equipment to monitor and test LDOEPs is essential to proper assessment of pile resistance and pile driving behavior. Some of the key points they made included instrument location and quantity, potential for aggressive driving to achieve high nominal resistance, accounting for residual stresses in the piles from manufacturing, and proper sizing of the hammer.

Static load testing provides the most direct means to measure the static behavior of a test pile; however, with the magnitude of loads required to test high-capacity LDOEPs, the costs of static tests can be very high when including the robust reaction frame and reaction piles needed to apply the load. When static load tests are performed, they are rarely taken to failure. This implies that the full nominal resistance and the full setup with time are not measured. There is also a general lack of measurements of load distribution with depth because of a lack of fully instrumented load tests.

Many of the agencies interviewed have used rapid load tests as an economical solution for load testing LDOEPs. A rapid load test can often be done for the high loads needed for an LDOEP at significantly less cost than a static test, although differences in soil resistance associated with rate of loading effects must be estimated and accounted for in the interpretation of measurements.

## RESEARCH NEEDS

Although relatively little research has been done by the transportation sector on LDOEPs, the offshore industry has an almost 60-year history of research on steel LDOEP design and installation practices. Although based on the conditions found offshore, the data and information available in the published literature from the offshore industry can be incorporated and adapted by the transportation industry to meet the reliability-based designs required for transportation structures. It is not necessary to research and develop entirely new design or testing methods; however, field research utilizing the types of LDOEPs and the subsurface conditions typical for transportation structures is needed to adapt the extensive data available from the offshore industry to transportation structure design.

Lists of specific research needs identified by the agencies or private practitioners are included in chapters three and four of the report. Many of these items relate back to the basic understanding of the behavior of the pile during driving and the effectiveness of measuring pile resistance. The consensus of both groups is that development of design procedures and resistance factors that are specific to LDOEPs for bridges is needed and could be incorporated in the AASHTO code. The procedures would reflect the reliability associated with testing for verification of axial resistance for these specific types of piles. This implies that a better understanding of what testing measurements mean and how they are to be interpreted is needed through full-scale testing.

Since the offshore industry has utilized steel LDOEPs almost exclusively, the adaption of API design methods, data, and pile behavior observations from these piles will need special consideration when applied to prestressed concrete LDOEPs. Work performed by the University of Florida and Florida DOT on prestressed concrete piles has been noted in this report; however, this work is based on limited data from only a few sites. Prestressed concrete LDOEPs differ significantly in their application and behavior from steel LDOEPs and there is a need for additional research to better understand the behavior of these piles and develop guidelines for design, testing, and quality assurance.

# GLOSSARY

*Beginning of restrike* (*BOR*): The first few restrike blows after a period of time.

*Drivability analysis*: An analysis of the maximum driving resistance and the installation equipment in order to evaluate whether a hammer and driving system will likely install the pile to the required depth or resistance in a satisfactory manner.

*Dynamic analysis method*: Using dynamic formulas and/or test data to calculate pile resistance.

*Dynamic monitoring*: A measure of the behavior of the pile during one or more hammer blows in which instrumentation on the pile is used to obtain measurements of strain and acceleration. The Pile Driving Analyzer® (PDA) is a commonly used apparatus for dynamic monitoring.

*Dynamic pile driving formula*: A closed form equation, such as the Gates or *Engineering News* formulas, used to relate pile hammer characteristics and penetration to the axial static resistance of the pile.

*End-of-initial driving* (*EOID*): The last few blows during the initial installation of a driven pile.

*High-strain dynamic test*: The procedure for monitoring dynamically loaded deep foundations and estimating static axial resistance as described by ASTM D4945.

*LDOEP* (*large diameter open-ended pile*): An open-ended pipe pile made of either steel or concrete with a diameter that is 36 in. or greater. Also referred to as cylinder piles.

*Maximum driving resistance*: The maximum amount of axial resistance that must be overcome in order to install the pile to the minimum pile penetration and to achieve the nominal bearing resistance. The maximum axial resistance that must be overcome includes the nominal bearing resistance plus any axial resistance present at the time of driving within zones of soil that may be removed in the future by scour or that may be subject to downdrag.

*Nominal unit base resistance*: The resistance to static axial compression loading on the base of the pile, at the strength limit state.

*Nominal unit side resistance*: The resistance to static axial compression or tension loading along the exterior or interior surface along the length of a pile, at the strength limit state.

*Penetration resistance*: A measure of the resistance to penetration of the pile during driving. May be expressed as blows per foot (b/f or blow count), blows per inch (bpi), or set per blow (inches).

*Pile driving criteria*: A specific set of requirements used to define the conditions that must be met during the installation of a production pile. Usually involves some combination of minimum embedment and/or penetration, the latter related to specific installation equipment.

*Plugged condition or plug*: A condition when the soil column inside of the interior of the pile moves downward with the pile, closing the end of the pile. This causes the pile to behave as a closed-end pile during driving or during static loading.

*Production pile*: A pile that will become part of the permanent foundation for the structure.

*Rapid load test*: The application of a force pulse to perform a load test of a deep foundation element as described by ASTM D7383. The Statnamic® (STN) loading device is a commonly used method for performing a rapid load test.

*Relaxation*: A reduction in the axial resistance after a period of time.

*Restrike*: A hammer blow or series of hammer blows applied to a pile after a period of time ranging from hours to days during which the pile is not actively driven. Restrike blows are applied to provide a measure of setup or relaxation after the initial driving of the pile.

*Setup*: An increase in the axial resistance of a pile that develops over time.

*Signal matching*: Numerical analysis of the pile based on the results of a high-strain dynamic test to determine static axial resistance. The CAPWAP® (Case Pile Wave Analysis Program) is an example of a computer code used for signal matching.

*Soil column*: The soil that is inside the interior of the open-ended pile. The soil column may or may not move with the pile as the pile is driven into the ground.

*Static analysis method*: Using static formulas, correlations, and/or static load test data to calculate pile resistance.

*Static load test*: The application of a static force to perform a load test of a deep foundation element as described by ASTM D1143.

*Test pile*: A pile that is installed for the primary purpose of performing a test of the pile, including the behavior during installation and/or during subsequent testing to determine the axial resistance. A test pile may or may not be incorporated into the permanent foundation as a production pile.

*Wave equation analysis*: Numerical model of the specific pile, soil conditions, and installation equipment used to evaluate behavior of the pile and driving equipment for a specific project.

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# APPENDIX A

# Survey Questionnaire

# NCHRP Synthesis Topic 45-05: Design and Load Testing of Large Diameter Open-end Driven Piles

## Introduction

**Page description:**
Opening and Instructions

Dear State Geotechnical, Bridge, or Construction Engineer

The Transportation Research Board (TRB) is preparing a synthesis on **"Current Practices for Design and Load Testing of Large Diameter Open-end Driven Pipe Piles"**. This is being done for NCHRP, under the sponsorship of the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration.

Synthesis Topic 45-05 will gather information on the current practices regarding the selection, use, design, construction, quality control, and performance of large diameter open-end steel pipe piles and concrete cylinder piles (LDOEPs) in transportation infrastructure. As the need to support larger lateral, seismic, and axial loads increases, designers and contractors are moving towards LDOEPs as one of the foundation types to consider resisting these loads. LDOEPs can be preferred over similar size drilled shafts and/or pile groups to address constructability and/or environmental concerns in certain circumstances or conditions. The same advances in construction equipment that have benefitted the installation of large diameter drilled shafts have also allowed LDOEPs to be a cost-effective solution for some of the same loading conditions. Yet, AASHTO nor FHWA design references provide no specific guidance for the selection, cost analysis, design and construction of LDOEPs.

A significant issue with LDOEP design is that the current Load and Resistance Factor Design (LRFD) methods are based on small diameter piles, typically 24 inches or less. These design methods do not account for the different soil-pile-hammer interaction that occurs for large diameter driven piles as compared to the smaller diameter piles. Design methods need to account for the influence of diameter and pile wall thickness, the degree of soil plugging or internal skin friction that exists, non-linear vibration effects and scalability.

We request your assistance in completing this survey, which is being sent to all State Departments of Transportation. Your cooperation in completing the questionnaire will ensure the success of this effort. **If you are not the appropriate person at your agency to complete this questionnaire, please forward it to the correct person.**

**We request that this survey be completed by March 4, 2014.** If you are completing this survey outside of Survey Gizmo, please email the completed survey to Mr. Robert Thompson, P.E. (Project Manager) at Dan Brown and Associates (consultant for the project) at rthompson@danbrownandassociates.com

If you have any questions about the survey, you may contact Dr. Dan Brown (Principal Investigator) at dbrown@danbrownandassociates.com or by phone at 423-942-6861, or Mr. Thompson at the email above or 334-239-3135.

QUESTIONNAIRE INSTRUCTIONS

1. To view and print the entire questionnaire, Click on the following link and print using "control p" http://surveygizmolibrary.s3.amazonaws.com/library/64484/survey_1439205.pdf
2. To save your partial answers and complete the questionnaire later, click on the "Save and Continue Later" link at the top of the page in the center of your screen. A link to the incomplete questionnaire will be emailed to you from *SurveyGizmo.* To return to the questionnaire later, open the email from *SurveyGizmo* and click on the link. We suggest using the "Save and Continue Later" feature if there will be more than 15 minutes of inactivity while the survey is opened, as some firewalls may terminate due to inactivity.
3. To pass a partially completed questionnaire to a colleague, click on the on the "Save and Continue Later" link at the top of the page in the center of your screen. A link to the incomplete questionnaire will be emailed to you from *SurveyGizmo.*" Open the email from *SurveyGizmo* and forward it to a colleague.
4. To view and print your answers before submitting the survey, click forward to the page following question 24. Print using "control p."
5. To submit the survey, click on "Submit" on the last page.

Thank you very much for your time and expertise.

Please enter the date (MM/DD/YYYY).

Please enter your contact information.

First Name *

Last Name *

Title

Agency/Organization *

Street Address *

Suite

City

State *

Zip Code *

Country

Email Address *

Phone Number *

**Definition of Terms**

**Page description:**
Defines common terms associated with LDOEPs

**Definition of terms**

For purposes of this questionnaire, the following definitions of terms are provided:

*LDOEP (Large Diameter Open-End Pipe):* An open-end pipe pile made of either steel or concrete with a diameter that is 36inches or greater. Also referred to as cylinder piles.

*Production pile*: A pile which will become part of the permanent foundation for the structure.

*Test pile*: A pile which is installed for the primary purpose of performing a test of the pile including the behavior during installation and/or during subsequent testing to determine the axial resistance. A test pile may or may not be incorporated into the permanent foundation as a production pile.

*Nominal bearing resistance*: The resistance of a pile to static axial compression loading at the geotechnical strength limit state.

*Nominal unit side resistance:* The resistance to static axial compression or tension loading along the exterior or interior surface along the length of a pile, at the geotechnical strength limit state.

*Nominal unit base resistance:* The resistance to static axial compression loading on the base of the pile, at the geotechnical strength limit state.

*Maximum driving resistance*: The maximum amount of axial resistance which must be overcome in order to install the pile to the minimum pile penetration and to achieve the nominal bearing resistance. In addition to the nominal bearing resistance, the axial resistance which must be overcome may include axial resistance within zones of soil that may be removed by scour or that may be subject to downdrag.

*Pile driving criteria*: A specific set of requirements used to define the conditions which must be met during the installation of a production pile. Usually involves some combination of minimum embedment and/or driving resistance, the latter related to specific installation equipment.

*Wave equation analysis*: Numerical model of the specific pile, soil conditions, and installation equipment, used to evaluate behavior of the pile and driving equipment for a specific project.

*Pile driving formula*: A closed form equation, such as the Gates or Engineering News formulas, used to relate pile hammer characteristics and driving resistance to the axial static resistance of the pile.

*Driving resistance*: A measure of the resistance to penetration of the pile during driving. May be expressed as blows per foot (b/f or blow count), blows per inch (bpi), or set per blow (inches).

*Drivability analysis*: An analysis of the maximum driving resistance and the installation equipment in order to evaluate whether a hammer and driving system will likely install the pile to the required depth or resistance in a satisfactory manner.

*End-of- driving (EOD)*: The last few blows during the installation of a driven pile.

*Restrike*: A hammer blow or series of hammer blows applied to a pile after a period of time ranging from hours to days during which the pile is not actively driven. Restrike blows are applied in order to provide a measure of setup or relaxation after the initial driving of the pile.

*Setup*: An increase in the nominal axial resistance of a pile that develops over time.

*Relaxation*: A reduction in the axial pile resistance after a period of time.

*Beginning of redrive (BOR)*: The first few restrike blows after a period of time.

*Dynamic monitoring*: A measure of the behavior of the pile during one or more hammer blows in which instrumentation on the pile is used to obtain measurements of strain and acceleration. The Pile Driving Analyzer® (PDA) is a commonly used apparatus for dynamic monitoring.

*High strain dynamic test*: The procedure for using dynamic monitoring to test deep foundations and determine static axial resistance is described by ASTM Standard D 4945-00.

*Signal matching*: The use of numerical modeling of the pile and pile driving system, back-calibrated to the results of a high strain dynamic test to determine static axial resistance. The CAPWAP (CAse Pile Wave Analysis Program) is an example of a computer code used for signal matching.

*Rapid load test*: The application of a force pulse to perform a load test of a deep foundation element as described by ASTM Standard D-7383-08. The Statnamic® (STN) loading device is a commonly used method for performing a rapid load test.

*Static load test*: The application of a static force to perform a load test of a deep foundation element as described by ASTM Standard D-1143.

**General Experience with LDOEPs**

**Page description:**
Click Here For Definitions of Terms

**Page exit logic:** No LDOEPs
**IF:** Question (ID 313) is answered **THEN:** Jump to page 9 - Thank You!

1. What is your general geographic area (location & extent) of practice? (e.g. state, district, geographic area, etc.) *

2. Does your agency consider the use or has used LDOEPs for transportation structures?

- Yes
- No

If Yes, continue to Question 3. If No, please briefly describe why your agency does not use LDOEPs, including (but not limited to) things such as soil conditions, cost, availability, etc. Once complete, please click "Next" at the bottom of the page to complete the survey.

3. Are LDOEPs considered for the following type of projects: (check all that apply)

- [ ] Design by Agency
- [ ] Design by Consultant
- [ ] Design-Build
- [ ] Value Engineering Change Proposal (VECP) by Contractor

4. What are the primary reasons for selecting LDOEPs over other foundation systems?

- [ ] Soil/Rock conditions
- [ ] Large axial loads
- [ ] Large Lateral Loads
- [ ] Special applications
- [ ] Cost Benefits
- [ ] Other

Please provide a brief description for Other:

5. On how many projects has your agency installed LDOEPs within the last 10 years?

- None (no use)
- 1 to 10
- 10 to 25
- 25 to 50
- 50 or more

6. Please list significant projects that utilized LDOEPs with the pile diameter, maximum length, year completed, and primary reason(s) that LDOEPs were selected.

7. What is your agency's experience with sizes and materials of concrete LDOEPs? Please give ranges of values for each item below.

| | |
|---|---|
| Outside Diameter (inches) | |
| Wall Thickness (inches) | |
| Concrete Strength (psi) | |
| Pre-or Post Tension Stress (ksi) | |
| Manufacturing Methods | |

8. What is your agency's experience with sizes and materials of steel LDOEPs? Please give ranges of values for each item below.

| | |
|---|---|
| Outside Diameter (inches) | |
| Wall Thickness (inches) | |
| Yield Strength (ksi) | |
| Manufacturing Method | |

9. Does your agency have standard plans and/or construction specifications specifically for LDOEPs?

- ○ Yes
- ○ No

If yes, please briefly describe the most significant differences between plans/specifications for LDOEPs and those for other driven piles.

**Design Methods**

**Page description:**
Click Here For Definitions of Terms

10. What are the static analysis methods used by your agency to determine nominal axial resistance and displacements for LDOEPs in cohesionless soils? (check all that apply)

- ☐ Meyerhoff SPT (FHWA 2006)
- ☐ Nordlund (FHWA 2006)
- ☐ Brown (FHWA 2006)
- ☐ Effective Stress - Beta (FHWA 2006)
- ☐ Method based on Cone Penetration Test
- ☐ In-House Method
- ☐ Other

Please briefly describe "In-House" or "Other"

11. What are the static analysis methods used by your agency to estimate nominal axial resistance and displacements for LDOEPs in cohesive soils? (check all that apply)

- ☐ alpha (Tomlinson) (FHWA 2006)
- ☐ Effective Stress - Beta (FHWA 2006)
- ☐ Method based on Cone Penetration Test
- ☐ In-House Method
- ☐ Other

Please briefly describe "In House" or "Other"

12. Do you ever use LDOEPs bearing on rock?

- ☐ No
- ☐ Yes - Without special treatment at the pile toe
- ☐ Yes - With special treatment at the pile toe

Please briefly list or explain any special treatments at the pile toe used by your agency for rock bearing piles.

13. What is your agency's procedure for design of LDOEPs bearing on rock or a bearing stratum expected to provide refusal to penetration through base resistance?

14. What source do you use for the resistance factors used in design?

- Current AASHTO Specifications
- My Agency Developed Factors
- Combination of AASHTO and My Agency
- Other Agency or source

**Special Design Considerations**

**Page description:**
Click Here For Definitions of Terms

15. What assumptions are made during design concerning plug formation during driving?

- We usually assume that a plug WILL form
- We usually assume that a plug WILL NOT form
- We evaluate each site based on soil conditions, pile type, and pile size

Please add any additional discussion concerning your assumptions of plug formation during driving.

16. What assumptions are made during design concerning setup?

- We usually consider that significant setup WILL occur
- We usually consider that significant setup WILL NOT occur
- We evaluate each site based on soil conditions, pile type, and pile size

Please add any additional discussion concerning your assumptions of pile setup after driving.

17. What assumptions are made during design concerning potential relaxation after driving?

- We usually assume that relaxation WILL occur
- We usually assume that relaxation WILL NOT occur
- We evaluate each site based on soil conditions, pile type, and pile size

Please add any additional discussion concerning your assumptions of potential relaxation after driving.

**Driving Criteria and Driving Aids**

**Page description:**
Click Here For Definitions of Terms

18. What techniques do you use to evaluate driveability of LDOEPs? (Check all that apply)

- [ ] Pile Driving Formula
- [ ] Wave Equation Analysis
- [ ] Other

Please provide any additional comments concerning drivability analysis

19. What are the driving criteria used for installation? (Check all that apply)

- ☐ Drive to a specified tip elevation
- ☐ Drive to a minimum tip elevation
- ☐ Drive to practical refusal
- ☐ Drive to a specified driving resistance (blow count) based on a driving formula
- ☐ Drive to a specified driving resistance (blow count) based on a wave equation analysis
- ☐ Drive to a specified driving resistance (blow count) based on high strain dynamic tests performed on indicator or test piles
- ☐ Drive to a specified driving resistance (blow count) based on static or rapid load tests performed on indicator or test piles, through signal match and wave equation.
- ☐ Verify resistance with restrikes

Please provide any additional comments concerning your agency's use of driving criteria for LDOEPs

20. Do you use or allow jetting or other driving aids when installing LDOEPs?

- ○ Yes
- ○ No

Please list or briefly explain any driving aids that are allowed.

21. Briefly describe any typical recurring installation problems and how they have been addressed.

**Performance, Load Testing, Research**

**Page description:**
Click Here For Definitions of Terms

22. Do you have any project case histories (published or unpublished) that would illustrate concepts such as driving systems, filed quality control, resistance verification, or problems during construction (and how they were addressed)?

- Yes
- No

23. Do you have full scale static, rapid, or dynamic load test data for LDOEPs that you would be willing to share?

- Yes
- No

24. Have you performed or sponsored any research on design and/or load testing of LDOEPs?

- Yes
- No

25. Do you have any post-construction performance case-histories or instrumentation data for the long-term performance of LDOEP supported structures?

- ○ Yes
- ○ No

26. Would your agency be willing to participate in a telephone or web video interview to discuss your experiences in more detail?

- ○ Yes
- ○ No

**Thank You!**

Thank you for taking our survey. Your response is very important to us. If you have any questions or comments, please feel free to contact Dr. Dan Brown (Principal Investigator) or Mr. Robert Thompson at the contact information below:

Dr. Brown

- E-mail: dbrown@danbrownandassociates.com
- Phone: 423-942-6861

Mr. Thompson

- E-mail: rthompson@danbrownandassociates.com
- Phone: 334-239-3135

# APPENDIX B

## Summary Report of Survey Responses of States Using LDOEPs

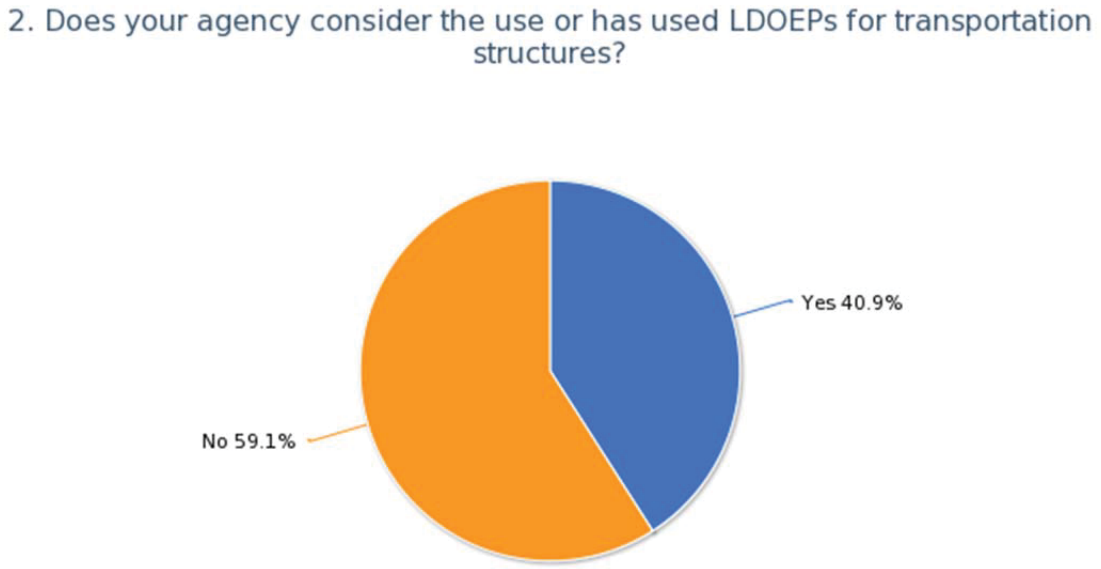
surveygizmo

Online Surveys, Data Collection and Integration
www.SurveyGizmo.com

# Summary Report

Survey: NCHRP Synthesis Topic 45-05: Design and Load Testing of Large Diameter Open-end Driven Piles

## 2. Does your agency consider the use or has used LDOEPs for transportation structures?

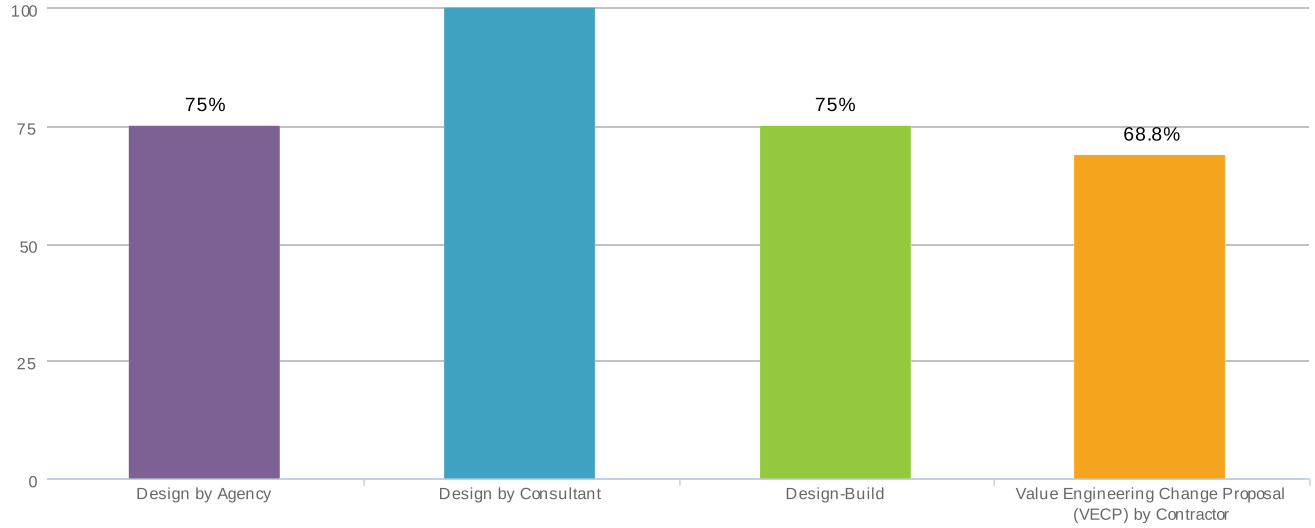



| Value | Count | Percent |
|---|---|---|
| Yes | 18 | 40.9% |
| No | 26 | 59.1% |

| Statistics | |
|---|---|
| Total Responses | 44 |

NOTE: Two of the 18 agencies that answered “Yes” were in the process of evaluating LDOEPs for some projects, but did not have experience as of the time of the survey. These two agencies only answered some of the remaining questions. Their responses are not included in the rest of the summary report. Only answers from the 16 agencies that have experiences with LDOEPs are included.

3. Are LDOEPs considered for the following type of projects: (check all that apply)

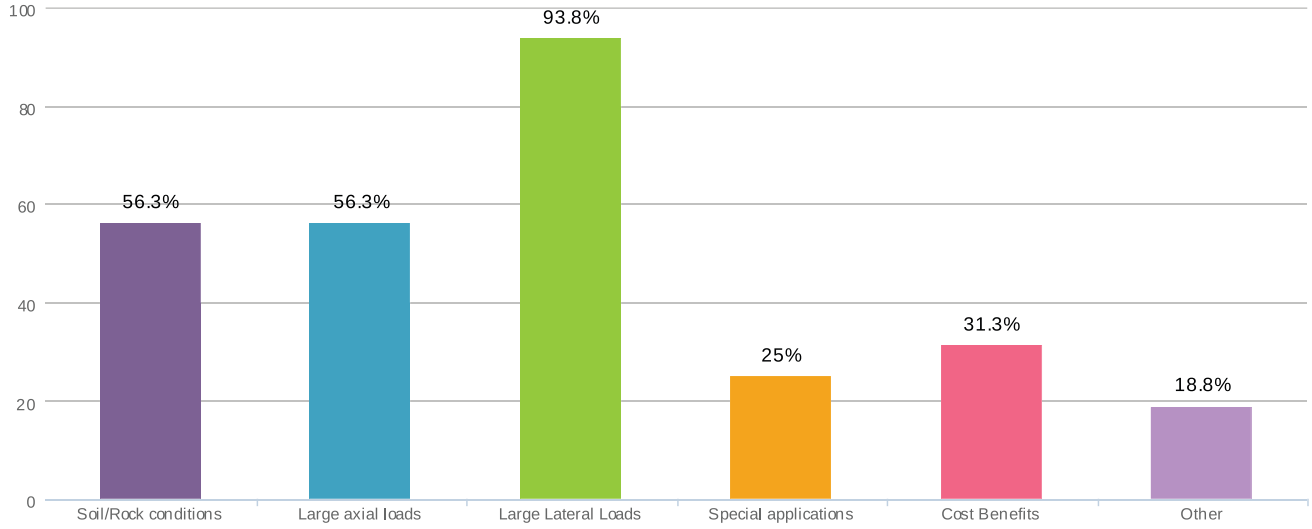



| | | | |
|---|---|---|---|
| Design by Agency | 75.0% | | 12 |
| Design by Consultant | 100.0% | | 16 |
| Design-Build | 75.0% | | 12 |
| Value Engineering Change Proposal (VECP) by Contractor | 68.8% | | 11 |
| | | Total | 16 |

**Statistics**

| | |
|---|---|
| Total Responses | 16 |

4. What are the primary reasons for selecting LDOEPs over other foundation systems?




| | | | |
|---|---|---|---|
| Soil/Rock conditions | 56.3% | | 9 |
| Large axial loads | 56.3% | | 9 |
| Large Lateral Loads | 93.8% | | 15 |
| Special applications | 25.0% | | 4 |
| Cost Benefits | 31.3% | | 5 |
| Other | 18.8% | | 3 |
| | | Total | 16 |

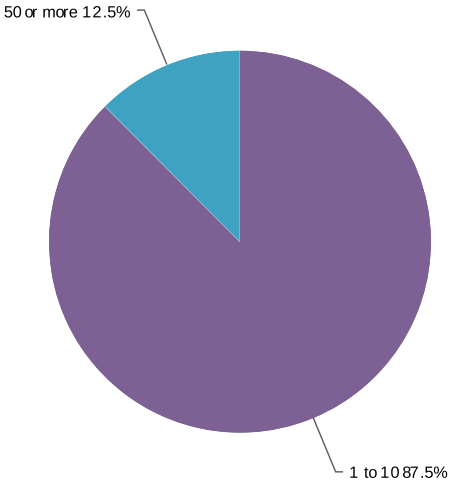
**Statistics**

| | |
|---|---|
| Total Responses | 16 |

Please provide a brief description for Other:

| Count | Response |
|---|---|
| 1 | Construction aspects (time), size of cofferdams, etc... |
| 1 | environmental issues |
| 1 | speed of construction |

5. On how many projects has your agency installed LDOEPs within the last 10 years?




| | | | |
|---|---|---|---|
| None (no use) | 0.0% | | 0 |
| 1 to 10 | 87.5% | | 14 |
| 10 to 25 | 0.0% | | 0 |
| 25 to 50 | 0.0% | | 0 |
| 50 or more | 12.5% | | 2 |
| | | Total | 16 |

| Statistics | |
|---|---|
| Total Responses | 16 |
| Sum | 114.0 |
| Average | 7.1 |
| StdDev | 16.2 |
| Max | 50.0 |

6. Please list significant projects that utilized LDOEPs with the pile diameter, maximum length, year completed, and primary reason(s) that LDOEPs were selected.

| Count | Response |
|---|---|
| 1 | 54 inch, 160 feet, 2006 Anticipate greater capacities |
| 1 | Projects can be discussed during the teleconference interview |
| 1 | TUS-Tuscarawas Ave. (2009) Diam=48" STA-21-05.25 (2012) Diam=48" |
| 1 | Wakota Hastings Dresbach Lafayette |
| 1 | Wantagh Parkway over Sloop Channel - 60 inch, 140 feet, 2005, large axial and lateral loads. Also used 54 inch concrete cylinders, 160 feet long. Rte 9P over Saratoga Lake - 36 inch concrete cylinders, 70 feet, to utilize precast pier caps because of tight window for construction. |
| 1 | SH 35 @ Copano Bay, 54" od, 120', under construction, lateral loading, environmental conditions, soil profile |
| 1 | I-270 chain of rocks cannel, 36", 120ft, 2013, redesign due to obstruction encountered installing the proposed drilled shaft. Marten Luther King Bridge connector Ramp, 48", 100', 2014, speed const. by leave LDOEP steel above ground to at as stay in place forms for pier columns. |
| 1 | Snake, Gakona, Barnette, many more bridge projects. Pipe diameters are 18", 24", 36" or 48". Typical pipe pile embedment length varies from about 40 feet to 150 feet. Alaska DOT has been using open ended pipe piles for over 30 years. Mendenhall River Bridge is a significant project recently awarded but not yet constructed, where we have 48" pipe piles extending down to around 300 feet. |
| 1 | • US 68/KY 80 over Kentucky Lake - Design Phase Pile Load Test Program Marshall & Trigg Counties, KY • 48” & 72” Diameter Open-Ended Steel Pipe Piles • Max. Pile Length = 210 ft. (Max. Penetration = 177 ft.) • Completed 2013 • The primary reason for selecting LDOEP’s were to penetrate into a dense bedded chert layer in order to provide sufficient nominal axial resistance and sufficient penetration for lateral resistance. • This design phase load test program is the only completed project where we have used LDEOP’s. However, the contract for the bridge has been let and awarded but construction has not started. |
| 1 | BR-0182(502) Baldwin County SR 182 over Little Lagoon Pass 36" Spun Cast Concrete Piles. Bridge in construction now |
| 1 | - 36" OD, 135' long, 2006, to support high axial load in soft soil. - 42" OD, 75' long, 2003, support high axial load in dense sand and gravel. |
| 1 | Woodrow Wilson bridge, up to 72", 200+ feet, 2003, lateral and axial loading. Dover River Bridge, up to 48", 150+ feet, out to bid, lateral and axial loading. |
| 1 | Only project with LDOEP's is the new US 34 bridge over Missouri River. Diameter = 4 ft. Max Length = 135 to 140 ft. Year Completed: LDOEP's installed 2013, bridge not yet complete. Reason selected: Lateral loads from barge impact. |
| 1 | VA Rte 5 over Chickahominy River: 66" dia, Lmax:160', 2007, long unsupported lenghts. VA Rte 175 at Chinoteaque, VA: 36" dia, Lmax:120', 2012,long unsupported lenghts |
| 1 | MassDOT has 2 projects in active construction with 36 inch steel OEP piles. One project has 250-300 foot long piles after failure to achieve nominal resistance at a higher elevation. The other project is being changed to a CEP and has acheived nominal resistance with a 90 foot CEP. |

7. What is your agency's experience with sizes and materials of concrete LDOEPs? Please give ranges of values for each item below.:Outside Diameter (inches)

| Count | Response |
|---|---|
| 2 | 36 |
| 1 | 36 to 66 |
| 1 | 36" to 66' |
| 2 | 54 |
| 1 | 54" |

7. What is your agency's experience with sizes and materials of concrete LDOEPs? Please give ranges of values for each item below.:Wall Thickness (inches)

| Count | Response |
|---|---|
| 1 | 4 |
| 1 | 5 |
| 1 | 5" to 6.5" |
| 1 | 6 |
| 1 | 6" |
| 1 | 8 |

7. What is your agency's experience with sizes and materials of concrete LDOEPs? Please give ranges of values for each item below.:Concrete Strength (psi)

| Count | Response |
|---|---|
| 1 | 6000 |
| 1 | 6000 psi |
| 2 | 7000 |
| 2 | 7000 psi |
| 1 | > 6000 |

7. What is your agency's experience with sizes and materials of concrete LDOEPs? Please give ranges of values for each item below.:Pre-or Post Tension Stress (ksi)

| Count | Response |
|---|---|
| 1 | 92 k per strand |
| 1 | 0.87 |
| 1 | 1 ksi |
| 1 | 1.6 ksi |
| 1 | 700 |
| 1 | Pre |
| 1 | Unknown |

7. What is your agency's experience with sizes and materials of concrete LDOEPs? Please give ranges of values for each item below.:Manufacturing Methods

| Count | Response |
|---|---|
| 1 | Centrifugal casting process |
| 1 | Spin cast |
| 1 | Spun Cast Cylinder Post Tensioned Cylider piles |
| 1 | Spun cast post tensioned segments, bed cast pre-stressed |
| 1 | spun cast |
| 1 | static cast |

8. What is your agency's experience with sizes and materials of steel LDOEPs? Please give ranges of values for each item below.:Outside Diameter (inches)

| Count | Response |
|---|---|
| 1 | 14-96 |
| 1 | 36 |
| 1 | 36 to 60 |
| 2 | 36-48 |
| 1 | 42 |
| 1 | 42 inch |
| 1 | 42" |
| 1 | 48 |
| 1 | 48 - 72 |
| 1 | 48 inches |
| 1 | 72 |

8. What is your agency's experience with sizes and materials of steel LDOEPs? Please give ranges of values for each item below.: Wall Thickness (inches)

| Count | Response |
|---|---|
| 1 | .75 to 1.75 |
| 1 | 0.375-1.5 |
| 1 | 0.5 |
| 1 | 0.75 |
| 1 | 0.75 inch (variable) |
| 1 | 0.75" |
| 1 | 0.75-1.0 |
| 1 | 1 inch |
| 1 | 1.0 - 2.0 |
| 1 | 1/2-3/4 |
| 1 | 2 |
| 1 | 3/4 to 7/8 |

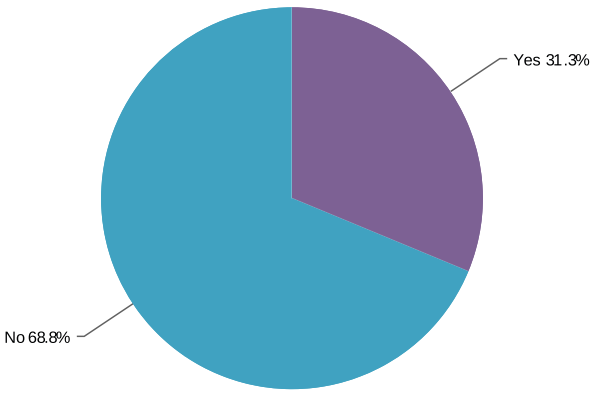
8. What is your agency's experience with sizes and materials of steel LDOEPs? Please give ranges of values for each item below.: Yield Strength (ksi)

| Count | Response |
|---|---|
| 1 | 36 to 50 |
| 3 | 45 |
| 1 | 45 ksi |
| 3 | 50 |
| 1 | 50-75 |

8. What is your agency's experience with sizes and materials of steel LDOEPs? Please give ranges of values for each item below.: Manufacturing Method

| Count | Response |
|---|---|
| 1 | Spiral welded |
| 1 | Variable |
| 1 | spiral weld |
| 1 | straight seam and spiral |
| 1 | welded plate |
| 1 | welded steel plate |
| 1 | The more important parameter that we look at is the relationship between diameter and pipe wall thickness. The ratio of diameter to thickness must be less than 48 (usually) for structural reasons. |

9. Does your agency have standard plans and/or construction specifications specifically for LDOEPs?



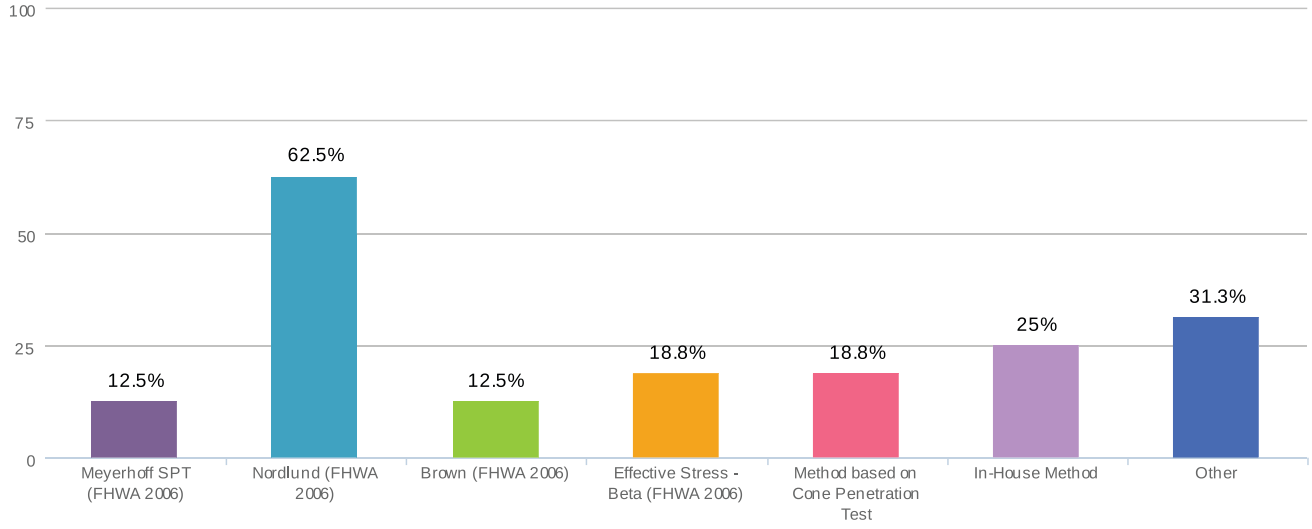

| | | |
|---|---|---|
| Yes | 31.3% | 5 |
| No | 68.8% | 11 |
| We do not distinguish between LDOEPs and smaller pipe piles in our standard documents | 0.0% | 0 |
| | Total | 16 |

| Statistics | |
|---|---|
| Total Responses | 16 |

If yes, please briefly describe the most significant differences between plans/specifications for LDOEPs and those for other driven piles.

| Count | Response |
|---|---|
| 1 | No significant difference. |
| 1 | Refer to Caltrans' specifications and standards |
| 1 | Shape specific reinforcement |
| 1 | higher capacities, concerns for plug rising in the pile |
| 1 | Requires larger less common more expensive hammers and cranes, Requires more rigorous construction inspection to establish driving acceptance criteria. |
| 1 | We use a Special Specification for concrete LDOEPs and is not included in our Standard Specifications. |

10. What are the static analysis methods used by your agency to determine nominal axial resistance and displacements for LDOEPs in cohesionless soils? (check all that apply)



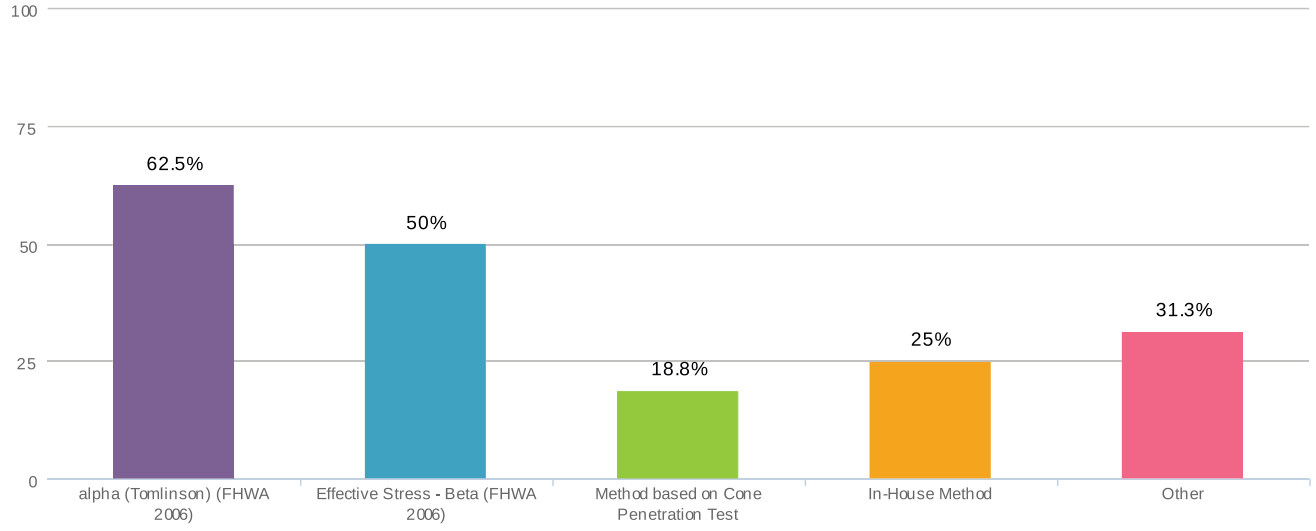

| | | | |
|---|---|---|---|
| Meyerhoff SPT (FHWA 2006) | 12.5% | | 2 |
| Nordlund (FHWA 2006) | 62.5% | | 10 |
| Brown (FHWA 2006) | 12.5% | | 2 |
| Effective Stress - Beta (FHWA 2006) | 18.8% | | 3 |
| Method based on Cone Penetration Test | 18.8% | | 3 |
| In-House Method | 25.0% | | 4 |
| Other | 31.3% | | 5 |
| | | Total | 16 |

| Statistics | |
|---|---|
| Total Responses | 16 |

Please briefly describe "In-House" or "Other"

| Count | Response |
|---|---|
| 1 | API RP 2A |
| 1 | Consultants and design builders are not limited to these methods |
| 1 | Design by consultant. |
| 1 | Drive to rock; also methods in A-PILE, DRIVEN and UniCone software packages |
| 1 | FBDEEP software |
| 1 | In house program developed from test pile data performed in the 1960's and 1970's. |
| 1 | Modified beta method using case studies from historic PDA/CAPWAP results |
| 1 | Nordlund for small diameters, API for large diameters |
| 1 | available on IDOT web site: http://www.dot.il.gov/bridges/Modified%20IDOT%20Pile%20Length.xls |
| 1 | TxDOT developed the Texas Cone Penetrometer (TCP) method of design back in the 1950's this is the method that is used. |

11. What are the static analysis methods used by your agency to estimate nominal axial resistance and displacements for LDOEPs in cohesive soils? (check all that apply)



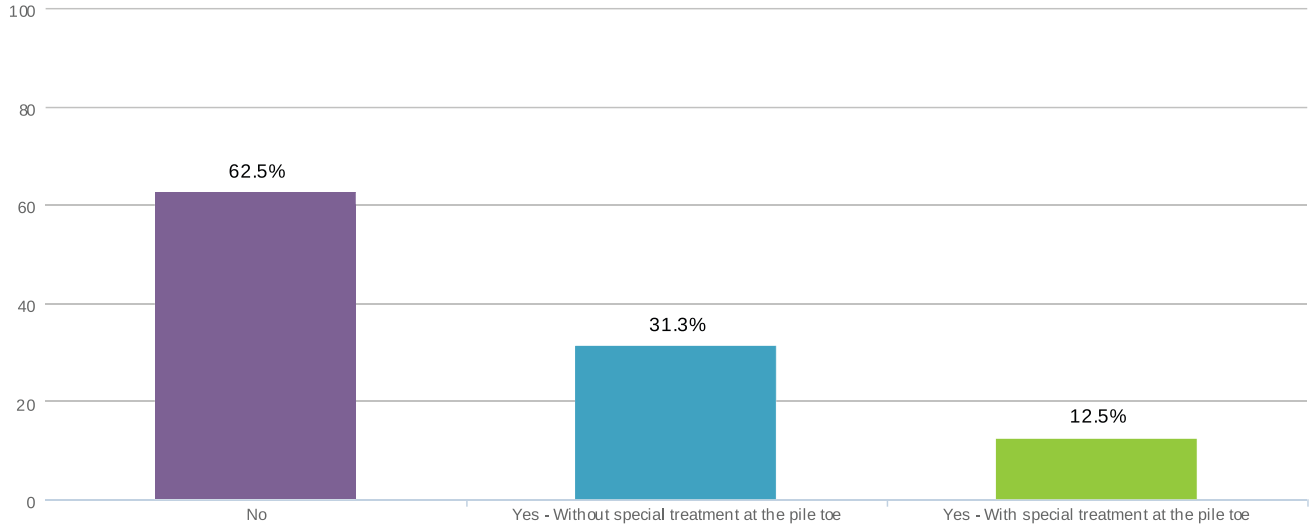

| | | | |
|---|---|---|---|
| alpha (Tomlinson) (FHWA 2006) | 62.5% | | 10 |
| Effective Stress - Beta (FHWA 2006) | 50.0% | | 8 |
| Method based on Cone Penetration Test | 18.8% | | 3 |
| In-House Method | 25.0% | | 4 |
| Other | 31.3% | | 5 |
| | | Total | 16 |

| Statistics | |
|---|---|
| Total Responses | 16 |

Please briefly describe "In House" or "Other"

| Count | Response |
|---|---|
| 1 | API RP 2A |
| 1 | Alpha for small diameters, API for large diameters |
| 1 | Consultants and design builders are not limited to these methods |
| 1 | Design by consultant. |
| 1 | FBDEEP software |
| 1 | Modified beta method using case studies from historic PDA/CAPWAP results |
| 1 | available on IDOT web site: http://www.dot.il.gov/bridges/Modified%20IDOT%20Pile%20Length.xls |
| 1 | TxDOT developed the Texas Cone Penetrometer (TCP) method of design back in the 1950's this is the method that is used. |

12. Do you ever use LDOEPs bearing on rock?




| | | | |
|---|---|---|---|
| No | 62.5% | | 10 |
| Yes - Without special treatment at the pile toe | 31.3% | | 5 |
| Yes - With special treatment at the pile toe | 12.5% | | 2 |
| | | Total | 16 |

| Statistics | |
|---|---|
| Total Responses | 16 |

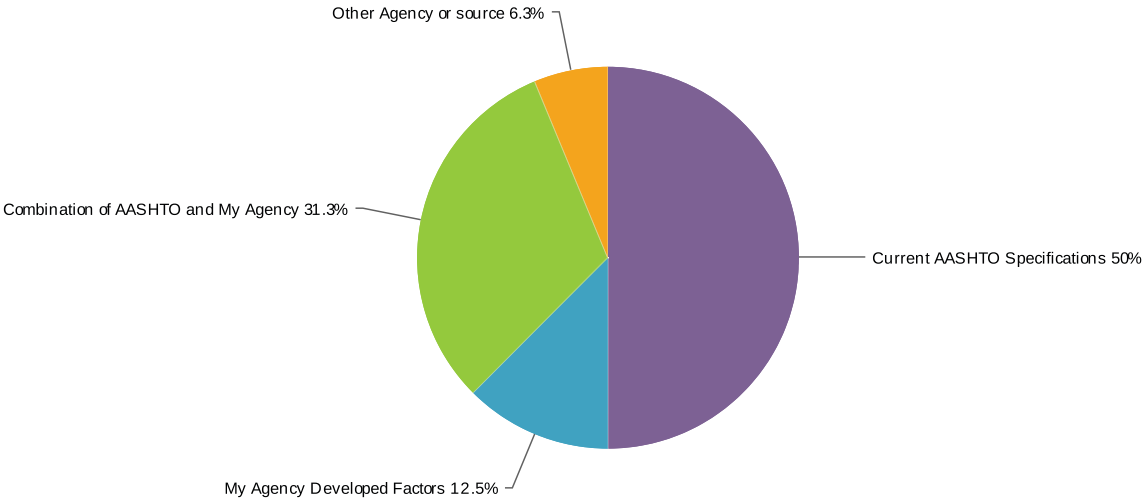
Please briefly list or explain any special treatments at the pile toe used by your agency for rock bearing piles.

| Count | Response |
|---|---|
| 1 | Strengthened ring on toe of LDOEP's. |
| 1 | Tip reinforcement dependent on rock type |
| 1 | depending on depth, if lateral support is needed we core out rock and seat pile in socket, sometimes we use concrete to set the pile. if no lateral is required (deep pile) then we use reinforcing tips. |

13. What is your agency's procedure for design of LDOEPs bearing on rock or a bearing stratum expected to provide refusal to penetration through base resistance?

| Count | Response |
|---|---|
| 1 | Depends on steel strength, end area of the pile, rock type and strength. |
| 1 | FBDEEP software |
| 1 | No definitive procedure -- have used LDOEP's on only one bridge. |
| 1 | Perform WEAP analysis to ensure driveability and acceptable driving stresses in pile at refusal |
| 1 | Test with PDA for driving stresses and capacity at refusal. |
| 1 | design for structural capacity of pile elements, with appropriate resistance factors. |
| 1 | If the bearing stratum is deep enough such pile fixity has been achieved, we drive until the blow counts indicate required driving resistance has been achieved. |

14. What source do you use for the resistance factors used in design?

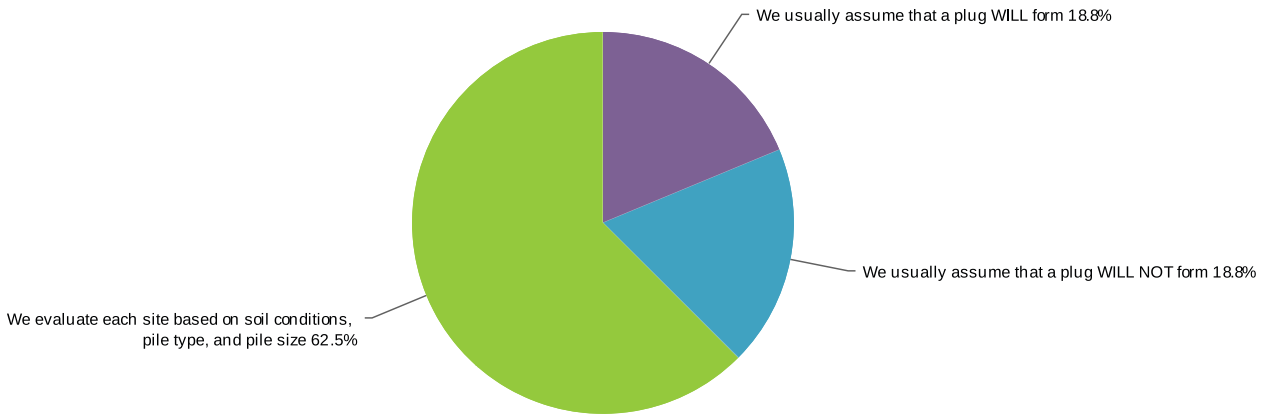



| | | | |
|---|---|---|---|
| Current AASHTO Specifications | 50.0% | | 8 |
| My Agency Developed Factors | 12.5% | | 2 |
| Combination of AASHTO and My Agency | 31.3% | | 5 |
| Other Agency or source | 6.3% | | 1 |
| | | Total | 16 |

**Statistics**

| | |
|---|---|
| Total Responses | 16 |

15. What assumptions are made during design concerning plug formation during driving?




| | | | |
|---|---|---|---|
| We usually assume that a plug WILL form | 18.8% | | 3 |
| We usually assume that a plug WILL NOT form | 18.8% | | 3 |
| We evaluate each site based on soil conditions, pile type, and pile size | 62.5% | | 10 |
| | | Total | 16 |

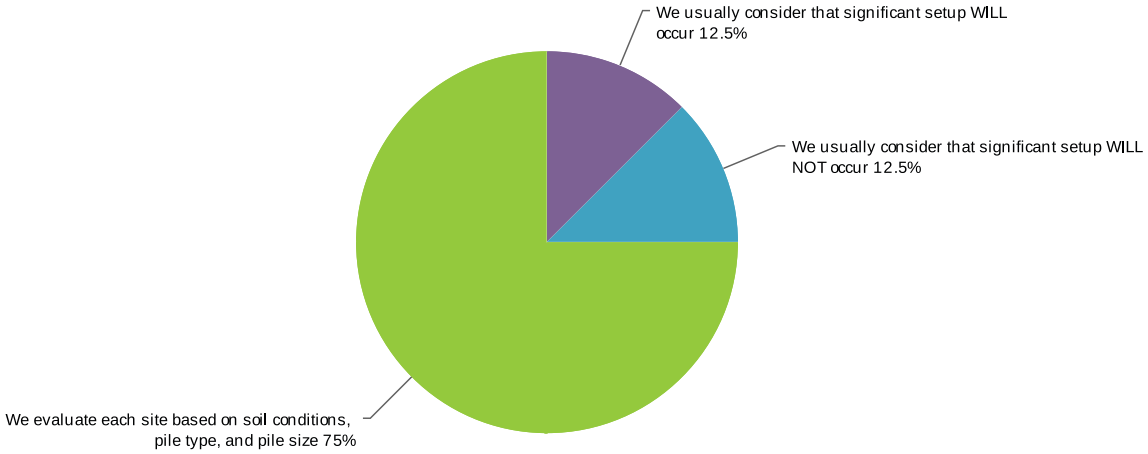
**Statistics**

| | |
|---|---|
| Total Responses | 16 |

Please add any additional discussion concerning your assumptions of plug formation during driving.

| Count | Response |
|---|---|
| 1 | As stated above, have used only 1 time. |
| 1 | Depending on site conditions we may assume a plug will form. |
| 1 | We used an artificial plug aka constrictor plan to help the pile form a plug. |
| 1 | http://www.dot.state.fl.us/research-center/Completed_Proj/Summary_GT/FDOT_BC354_60_rpt.pdf |
| 1 | During driving a plug is assumed not to form. Where time is allowed for set-up (and running load tests) a plug is assumed to form later, and used in capacity determinations. |
| 1 | Larger piles in looser materials tend to not get as much plug formation. as far as using plug formation to predict capacity, we look at case histories and related previously observed capacities, when available. |

16. What assumptions are made during design concerning setup?



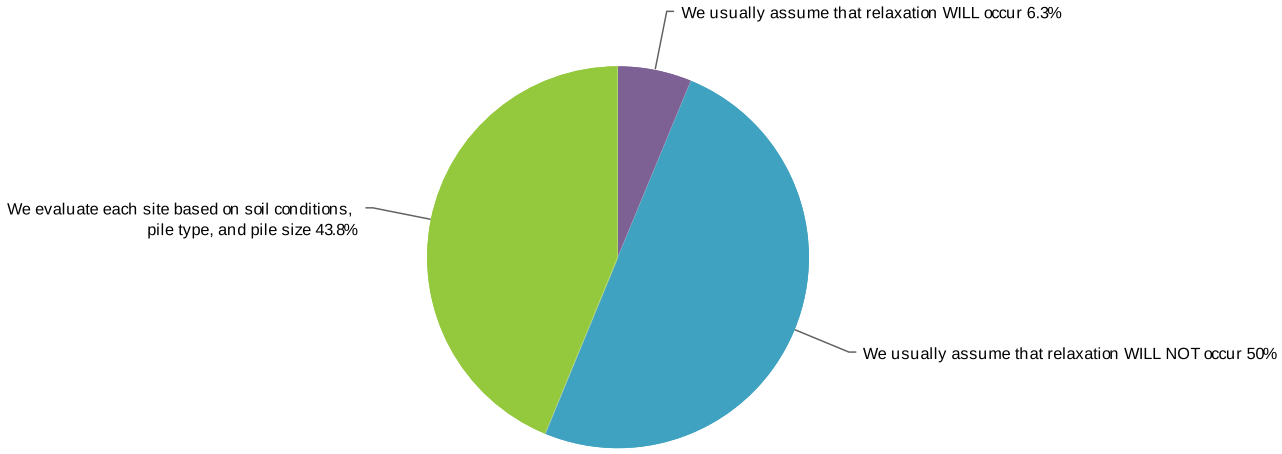

| | | |
|---|---|---|
| We usually consider that significant setup WILL occur | 12.5% | 2 |
| We usually consider that significant setup WILL NOT occur | 12.5% | 2 |
| We evaluate each site based on soil conditions, pile type, and pile size | 75.0% | 12 |
| | Total | 16 |

| Statistics | |
|---|---|
| Total Responses | 16 |

Please add any additional discussion concerning your assumptions of pile setup after driving.

| Count | Response |
|---|---|
| 1 | As stated above, have used only 1 time. |
| 1 | Both projects where located in granualr soils and no setup was anticipated. |
| 1 | Measured using PDA testing. |
| 1 | We delay the static and dynamic load testing in order to try and capture some of the pile setup. |
| 1 | We will always do restrike testing with the PDA. |
| 1 | set-up based on soil conditions rather than pile type/size |
| 1 | Generally do not expect significant setup, since most of our soil deposits are cohesionless, but we will do restrikes if we are not at capacity yet have reached expected pile tip elevation and additional pile lengths are not readily available. |
| 1 | We performed several restrike dynamic tests to evaluate the change in nominal resistance over time. |
| 1 | Virginia coastal areas routinely exibit 200% to 300% setup. Adjustments after a driving test program is finalized to set production pile lenghts. PDA with restrikes has significantly reducted our pile lenghts. |
| 1 | Some testing shows that set-up occurs; it is reviewed at each site (note this is less of a factor if piles driven to rock). Require PDA/CAPWAP *and* Quasi-static/static testing. |

17. What assumptions are made during design concerning potential relaxation after driving?



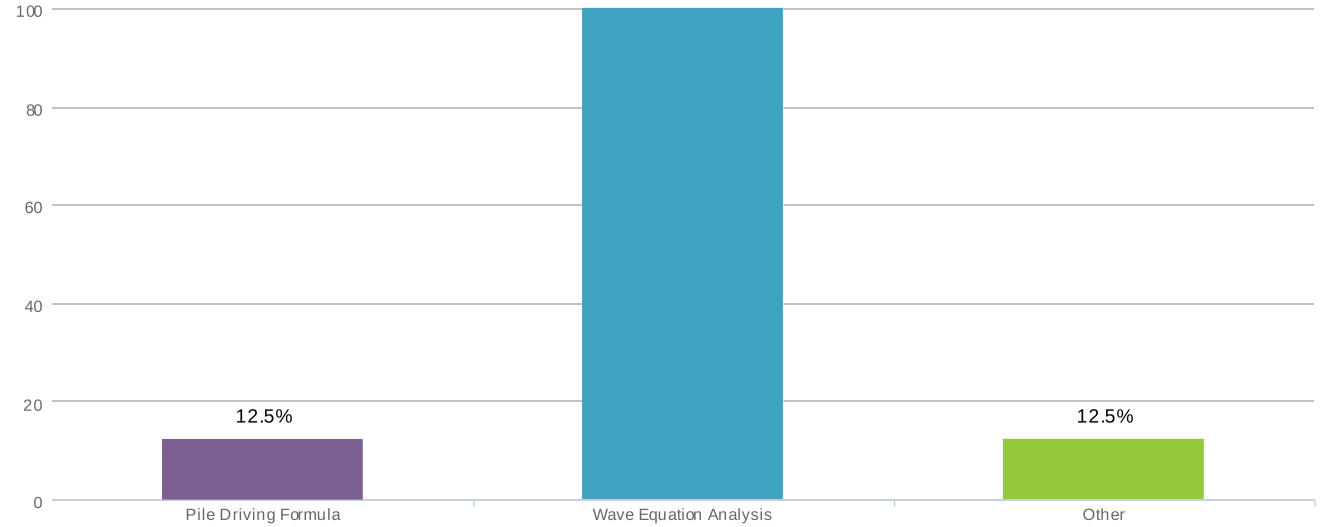

| | | | |
|---|---|---|---|
| We usually assume that relaxation WILL occur | 6.3% | | 1 |
| We usually assume that relaxation WILL NOT occur | 50.0% | | 8 |
| We evaluate each site based on soil conditions, pile type, and pile size | 43.8% | | 7 |
| | | Total | 16 |

| Statistics | |
|---|---|
| Total Responses | 16 |

Please add any additional discussion concerning your assumptions of potential relaxation after driving.

| Count | Response |
|---|---|
| 1 | As stated above, have used only 1 time. |
| 1 | Assumptions of relaxation based on regional experience with other piles |
| 1 | WE will always do restrike testing with the PDA. |
| 1 | We performed several restrike dynamic tests to evaluate the change in nominal resistance over time. |
| 1 | We seldom test for relaxation. If capacity is met at end of drive, we move on. Occasionally we have done restrikes when capacity was not there and observed minimal relaxation, however. |

18. What techniques do you use to evaluate driveability of LDOEPs? (Check all that apply)



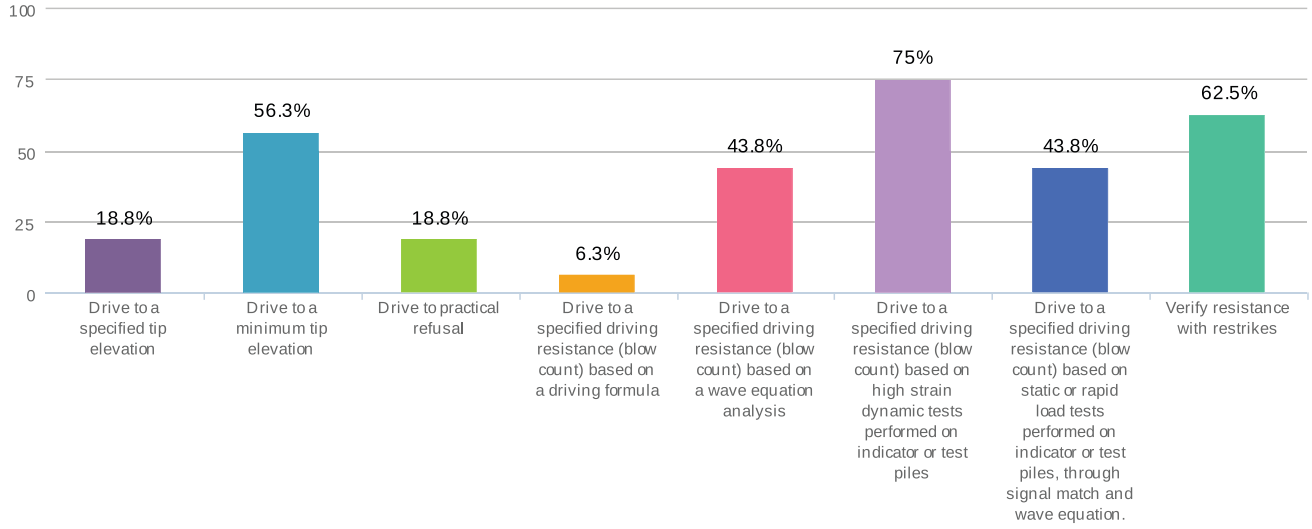

| | | | |
|---|---|---|---|
| Pile Driving Formula | 12.5% | | 2 |
| Wave Equation Analysis | 100.0% | | 16 |
| Other | 12.5% | | 2 |
| | | Total | 16 |

| Statistics | |
|---|---|
| Total Responses | 16 |

Please provide any additional comments concerning drivability analysis

| Count | Response |
|---|---|
| 1 | Drivability analysis will include a check on different degrees of plugging. |
| 1 | Project had a test pile. |
| 1 | sacrificial test pile. |
| 1 | Require test pile as first production pile, PDA & capwap, compare to WSDOT driving formula to establish driving acceptance critera |

19. What are the driving criteria used for installation? (Check all that apply)



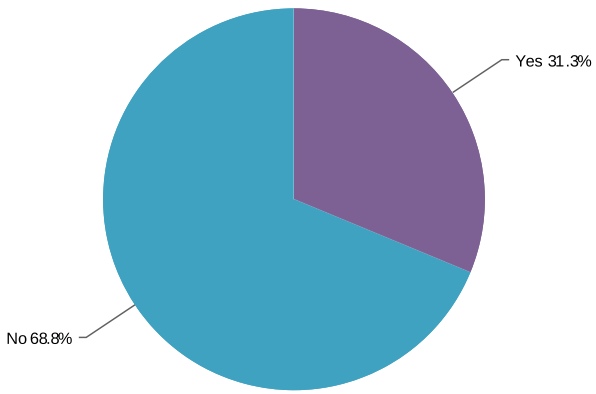

| | | | |
|---|---|---|---|
| Drive to a specified tip elevation | 18.8% | | 3 |
| Drive to a minimum tip elevation | 56.3% | | 9 |
| Drive to practical refusal | 18.8% | | 3 |
| Drive to a specified driving resistance (blow count) based on a driving formula | 6.3% | | 1 |
| Drive to a specified driving resistance (blow count) based on a wave equation analysis | 43.8% | | 7 |
| Drive to a specified driving resistance (blow count) based on high strain dynamic tests performed on indicator or test piles | 75.0% | | 12 |
| Drive to a specified driving resistance (blow count) based on static or rapid load tests performed on indicator or test piles, through signal match and wave equation. | 43.8% | | 7 |
| Verify resistance with restrikes | 62.5% | | 10 |
| | Total | | 16 |

| Statistics | |
|---|---|
| Total Responses | 16 |

Please provide any additional comments concerning your agency's use of driving criteria for LDOEPs

| Count | Response |
|---|---|
| 1 | As stated above, have used only 1 time. |
| 1 | Estimated Tip also listed on plans. |
| 1 | PDA/CAPWAP calibrated to Statnamic testing. |
| 1 | Piles must be driven to or below the depth to provide fixity, as well as driven to meet axial loading requirements. We always use either wave equation alone or high strain dynamic testing to determine blow count. |
| 1 | Projects with LDOEPs will have a comprehensive driving test program including PDAs. Load teats are also often required. |
| 1 | We often used results of high strain dynamic tests to verify the pile driving criteria developed by WE analysis. |
| 1 | We used the results of the load testing program to establish a dynamic pile testing protocol for the bridge construction contract. The protocol will require dynamic testing at the end of the initial drive and nominal 3-day and 7-day restrikes. |

20. Do you use or allow jetting or other driving aids when installing LDOEPs?




| | | | |
|---|---|---|---|
| Yes | 31.3% | | 5 |
| No | 68.8% | | 11 |
| | | Total | 16 |

| Statistics | |
|---|---|
| Total Responses | 16 |

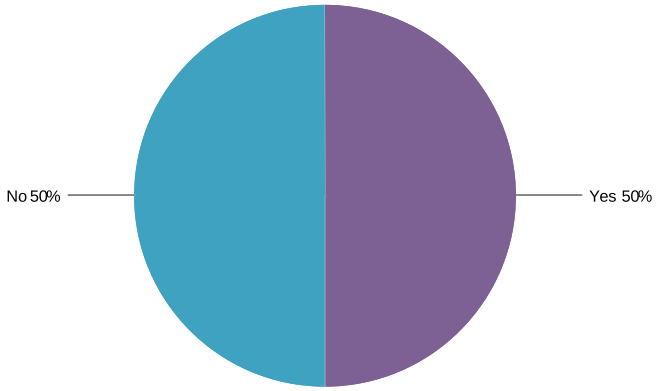
Please list or briefly explain any driving aids that are allowed.

| Count | Response |
|---|---|
| 1 | As stated above, have used only 1 time. |
| 1 | Driving shoe, pre-drilling, center relief drilling, vibration aid |
| 1 | Environmental concerns routinely do not allow jetting. |
| 1 | Jetting |
| 1 | Jetting is commonly used for the concrete LDOEP's in sandy soil. |
| 1 | NA |
| 1 | Vibratory allowed to be verified by impact. |
| 1 | pre-drilling or pre-forming as needed to achieve minimum tip for lateral stability |

21. Briefly describe any typical recurring installation problems and how they have been addressed.

| Count | Response |
|---|---|
| 1 | As stated above, have used only 1 time. |
| 1 | I project over ran length considerable but did achieved driving criteria. |
| 1 | We experienced pile cracking during installation. |
| 1 | Longitudinal cracking due to void filling with soil and water. Increased venting to maintain sufficient air cushion in void, and/or stop driving and partially remove soil/water from void when water exits vent holes. Spalled concrete at pile head due to pinching between inner and outer alignment features in the helmet. Inner alignment ring removed and wedges added to the guiding ribs. |
| 1 | Hitting obstructions prior to reaching minimum penetration. We then need to clean out the pile, or try to remove obstruction. If piles run long, we either add or increase frequency of high strain testing in order to increase resistance factor, which will decrease required driving resistance. |
| 1 | Subsurface condition different from anticipated, re-evaluation and reset driving criteria, hard-driving condition Center relief drilling |
| 1 | The long, heavy piles require good construction control to meet tolerances. Batter piles add to the difficulty. Good templates reduce the problems. |
| 1 | Dynamic testing with PDA has typically underpredicted capacity, so we require statnamic testing for axial capacity. |
| 1 | During jetting of concret LDOEP's where the channel debris (stone filling etc.) will get washed in the prejet hole causing obstructions to driving. Have put notes in contract plans to clean channel bottom and use oversized steel casing if necessary. |

22. Do you have any project case histories (published or unpublished) that would illustrate concepts such as driving systems, filed quality control, resistance verification, or problems during construction (and how they were addressed)?

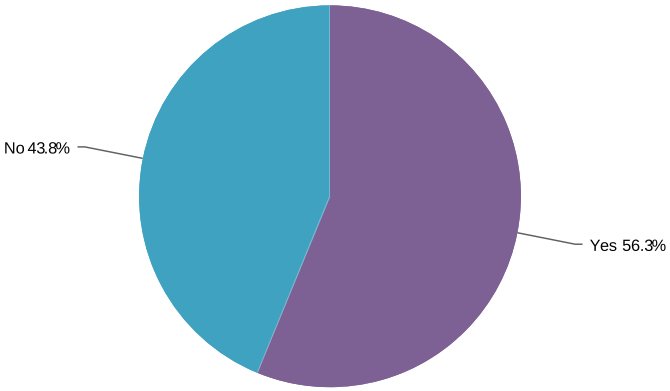



| | | | |
|---|---|---|---|
| Yes | 50.0% | | 8 |
| No | 50.0% | | 8 |
| | | Total | 16 |

**Statistics**

| | |
|---|---|
| Total Responses | 16 |

23. Do you have full scale static, rapid, or dynamic load test data for LDOEPs that you would be willing to share?

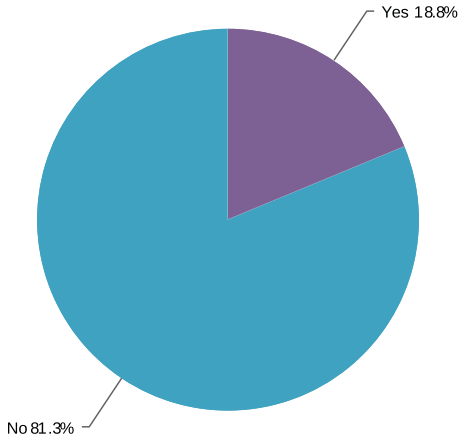



| | | | |
|---|---|---|---|
| Yes | 56.3% | | 9 |
| No | 43.8% | | 7 |
| | | Total | 16 |

**Statistics**

| | |
|---|---|
| Total Responses | 16 |

24. Have you performed or sponsored any research on design and/or load testing of LDOEPs?

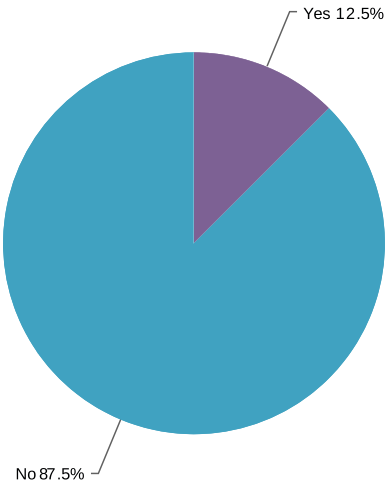



| | | | |
|---|---|---|---|
| Yes | 18.8% | | 3 |
| No | 81.3% | | 13 |
| | | Total | 16 |

**Statistics**

| | |
|---|---|
| Total Responses | 16 |

25. Do you have any post-construction performance case-histories or instrumentation data for the long-term performance of LDOEP supported structures?

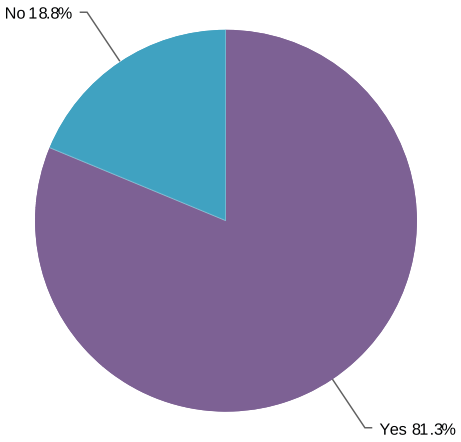



| | | | |
|---|---|---|---|
| Yes | 12.5% | | 2 |
| No | 87.5% | | 14 |
| | | Total | 16 |

**Statistics**

| | |
|---|---|
| Total Responses | 16 |

26. Would your agency be willing to participate in a telephone or web video interview to discuss your experiences in more detail?




| | | | |
|---|---|---|---|
| Yes | 81.3% | | 13 |
| No | 18.8% | | 3 |
| | | Total | 16 |

**Statistics**

| | |
|---|---|
| Total Responses | 16 |

# APPENDIX C

## Interview Notes of Selected States Using LDOEPs

*NCHRP 45-05*
*Interview Notes – Alaska Department of Transportation* *Page 1 of 3*

| | |
|---|---|
| *Agency:* | Alaska Department of Transportation and Public Facilities |
| *Participants:* | Dave Hemstreet, P.E. |
| *Interview Date:* | April 22, 2014 |

*Primary interview was with Mr. Helmstreet. A follow-up phone call was made to Mr. Elmer Marx, P.E. in Bridge Design to supplement two points. Mr. Marx's comments are in italics below.*

Design

- Piles are steel pipe, 48in diameter is largest used to date.
- Typically granular, cohesionless soils.
- Use FHWA Beta method with modified beta factors.
    - Modifications are based on recent research report "CAPWAP-Based Correlations for Estimating the Static Axial Capacity of Open-Ended Steel Pipe Piles in Alaska, December 26, 2012."
    - Study based on 20 years of driving records and PDA data.
    - Use an equivalent plug to estimate base resistance.
    - Have applied the results of the research to a couple of recent projects and found that the dynamic tests matched well with the modified static analysis calculations.
- Previous design method was to use unmodified FHWA methods in the computer programs DRIVEN and Allpile. Shortcomings of this approach believed by ADOTPF:
    - The plug was not being modeled correctly.
    - The calculations did not properly "scale up" to larger sized piles.
    - Idaho DOT did a study and determined that if the SPT N value was scaled down, the Nordlund method gave reasonable results.
- Piles are concrete filled (with rebar) at least down to point of fixity required for seismic response.

*Notes from Mr. Elmer Marx, P.E., Bridge Design*

- Use of LDOEPs began in the 1970s during and after Alaskan Pipeline project. Steel pipe piles 48in in diameter were used on that project. ADOTPF saw many of the benefits and adopted them for transportation, using the piles for pile bents to replace large pile footings.
    - Less environmental impact – pile bents remove need for excavation and cofferdams for large pile footings containing numerous H-piles or small pipe piles.
    - Quick installation. ADOTPF is utilizing accelerated bridge construction due to the very short construction season in Alaska. Also restrictions for fish, wildlife, etc. Fewer large piles can be installed quickly and superstructure work started sooner.

  - To help reduce environmental issues with impact hammers, piles are vibrated in to extent possible, and then completed with impact hammer.
  - Benefit of adopting 48in pipe piles has been seen with changes in seismic design codes. No major changes needed to meet updated codes since the pipe piles were suitable for the updated loadings.
- Due to supply issues and seismic design issues, ADOT has stopped using ASTM A252 steel pipe and adopted three types of pipe:
  - ASTM A53 for small diameter piles
  - API5L PSL2 (pipeline pipe) – very detailed specifications for chemistry and tolerances on manufacture; designed for seismic applications.
  - Spiral weld pipe – worked with Skyline Steel to develop and transitioning to this for more projects.

Plug Formation

- Evaluate plug formation for each site, looking also at previous work nearby
- No set procedure or depth of plugging

Drivability Analysis

- Typically do not do full drivability analysis for most loading conditions. Will check driving stresses with low penetration as well as with penetration at estimated tip, but don't typically do a full driveability analysis showing expected blowcount and stresses for the entire depth.
- If a project has very high loads, or high driving stresses are anticipated, will do wave equation analyses to verify the designed piles can be installed with typical hammers used.
- Contractors are required to submit wave equation analysis with equipment submittal.

Dynamic Testing

- Frequently used, especially for friction piles
- Take advantage of increased resistance factors for higher load demands.
- No static load tests
- No pre-production testing.
- Dynamic test at least one pile per bent/substructure. Bridges over 100 feet wide may get two tests. If capacities are not coming up, will do either a restrike or test more piles within that bent (in order to use a higher resistance factor), or combination of the two.

Pile Damage

- Not frequent, but sometimes have toe damage for piles driving into rock, particularly sloping bedrock.

Lessons Learned

- Do not use static analysis methods alone to predict resistance. They are too conservative due to not scaling up to larger diameters.
- Have observed piles reaching a maximum resistance and then not gain additional resistance with increased depth. One idea is that the soil is liquefying close to the pile as it is being driven, causing reduction or loss of side resistance. Some gain occurs after driving has completed, but not to level expected.
- Interested in comparing other agencies experiences with dynamic testing in granular materials to see how they compare with ADOTPF.

*NCHRP 45-05*
*Interview Notes – California Department of Transportation* *Page 1 of 1*

| | |
|---|---|
| *Agency:* | California Department of Transportation |
| *Participants:* | Sharid K. Amiri, Ph.D., P.E. |
| *Interview Date:* | May 6, 2014 |

*Dr. Amiri shared experiences in design oversight capacity, specific to the projects in his region of Caltrans and are not for the entire Caltrans. A geotechnical manual for the department is currently being developed, with some modules available on-line. The deep foundations module is still in development.*

Design

- LRFD was used per AASHTO (4th edition) guidelines and Caltrans Amendment dated 2008 in geotechnical design of the deep foundation for these projects.
- Two types of LDOEPs
  - "Pipe Piles" – does not include concrete infill (with or without reinforcement)
  - "Cast in Steel Shell Piles" (CISS) – includes concrete and reinforcement steel. CISS piles were 48 inch in diameter with Shell thickness of ¾ inch. These piles are typically designed to withstand significant seismic and lateral loading. They also contributed to a reduction in the footprint during construction.

Issues

- How to demonstrate/verify nominal axial resistance
  - PDA alone for large diameter piles does not appear to adequately measure axial loading.
  - Pile driving formulas or large diameter piles are not sufficient
  - Pile load testing (PLT) in conjunction with PDA is needed. ( Ref: Caltrans LRFD Amendment)
  - Load tests need to be taken to failure to better calibrate resistance factors, but this is difficult
- Vibrations from installation
  - Installations in highly urbanized areas impacting nearby residences
  - Need more monitoring data for LDOEP installations since most information is for small diameter piles.
  - Pile vibration monitoring was performed for these projects including specific data for these large diameter CISS piles. This information is being gathered at the present time.

*NCHRP 45-05*
*Interview Notes – Florida Department of Transportation* *Page 1 of 2*

| | |
|---|---|
| *Agency:* | Florida Department of Transportation |
| *Participants:* | Larry Jones, P.E.<br>Rodrigo Herrera, P.E. |
| *Interview Date:* | May 2, 2014 |

General

- FDOT has had three recent bridges built using concrete LDOEPs:
  - Hathaway Bridge Replacement (Bay County)
  - St. George Island Bridge Replacement (Franklin County)
  - Trout River Bridge (Duvall County)
- Many details of the contractors' experience summarized in the following papers:
  - Muchard, M. (2006). "Statnamic Load Testing of High Capacity Marine Foundations", 54th Annual Geotechnical Engineering Conference, University of Minnesota.
  - Kemp, J.J. and Muchard, M. (2007). "Recent Experiences with Concrete Cylinder Piles in Florida", GSP 158 Contemporary Issues in Deep Foundations, Geo-Denver 2007, Geo-institute of ASCE.

Design and Construction

- Two standard concrete pile types:
  - 54 inch diameter spun-cast post-tensioned. Piles are cast in 16-foot segments.
  - 60 inch diameter full-length pre-tensioned.
- Steel pipe piles (42in diameter) only used once on minor bridge on state land.
- Pile tip elevations and pile resistance are estimated during design using FBDEEP software.
- Dynamic testing used to verify resistance, set tip elevations, and establish final order lengths. No significant issues with dynamic testing. Load test programs did indicate that pile resistance determined by CAPWAP was conservative compared to static/Statnamic testing.
- FDOT has standard pile design detail drawings for both pile types.
- Have also had static and Statnamic load testing as detailed in the papers by Muchard.
- Do not assume plug formation during driving and did not observe it on the first two projects, in fact the opposite was observed, with a column of soil and water raising inside the void.
- Jetting is allowed if sufficient measures are in place to prevent turbidity and the project location is not in a sensitive waterway.

Lessons Learned

- Proper quantity, size and location of vent holes in the sides of the piles is very important.

  - Insufficient venting led to longitudinal cracking in several piles.
  - As the pile is driven, the pile experiences axial shortening during a hammer blow. The column of water within the void section is incompressible, applying excessive uniform stress against the pile cylinder, resulting in longitudinal cracking.
  - Contractor used an airlift to remove water/soil from inside the upper portion of the pile void to reduce the potential for excessive stress, and driving continued thereafter.
  - Additional vents and larger vents added to the piles reduced/eliminated the cracking issues related to poor venting.
- When driving concrete cylinder piles, the pile cushion needs to have a void with the same size as the void of the pile. Using a solid pile cushion may result in it being pushed into the void, generating radial stresses that initiate longitudinal cracking or spalling at the pile head. This has also been found to be true by FDOT for square piles with voids extending through the top of the pile.
- Have also had experience where contractor welded a steel ring inside the helmet to help hold and align the pile cushion. This ring apparently ended up contributing cracking due to misalignment and/or radial stresses at the top of the pile.

Research

- University of South Florida (USF) "Corrosion Performance of Concrete Cylinder Pile" Final report submitted to FDOT on 2005. USF inspected the piles at various bridge sites that experienced vertical cracking and found the piles to be performing very well with regards to resisting corrosion.
- University of Florida "Determination of Axial Pile Capacity of Pre-Stressed Concrete Cylinder Piles" Final report submitted to FDOT on 2004. The research was used as the basis for updates to FBDEEP software.
- University of Florida "Development of Modified P-y Curves for Large Diameter Piles/Drilled Shafts in Limestone for FBPIER" Final report submitted to FDOT on 2004. The research was used to develop a lateral resistance model for Limestone, and implementation in design software (currently known as) FB-Multipier.

*NCHRP 45-05*
*Interview Notes – Kentucky Transportation Cabinet* *Page 1 of 3*

| | |
|---|---|
| *Agency:* | Kentucky Transportation Cabinet (KYTC) |
| *Participants:* | Darrin Beckett, P.E. – KYTC Transportation Engineer Specialist<br>Ron Ebelhar, P.E. – Terracon, Inc. (Consultant to KYTC)<br>Jeff Dunlap, P.E. – Terracon, Inc. (Consultant to KYTC) |
| *Interview Date:* | April 21, 2014 |

*KYTC has had very little recent experience with LDOEPs. The experience that is the subject of the interview is from a test pile program conducted for the project US 68 / KY 80 over Kentucky Lake, report dated January 23, 2014. Items in italics are excerpted from the report.*

Design issues that led to considering LDOEPs and having a test pile program

- Seismic loading due to proximity of New Madrid fault
- Barge impact loads
- Difficult soil/geologic conditions for drilled shafts
    - Residuum of Fort Payne Formation – chert residuum that behaves as a dense gravel, some silt and clay layers.
    - Concern about potential difficulties with keeping drilled holes open, and that full depth temporary casing would be required.
    - Not confident that a clean shaft bottom could be achieved.
    - Chert formation hard on drilling tools, driving up costs for contractors
- Transition to LRFD provided opportunity to evaluate resistance factors with load tests
- Experience of geotechnical consultant (Terracon) with LDOEPs for marine structures
- Concern of small diameter open-end piles plugging and not reaching the bearing strata
- Concern with closed-end piles not reaching the bearing strata
- Efficiency of fewer, larger piles providing stiffness for seismic and impact loads

Design Process and Methods

- Mr. Ebelhar's experience with offshore structures supported on LDOEPs includes use of API RP 2A method for design. His experience, and that of industry, indicates very good agreement with load test data. He has designed several projects around the world in similar soil/rock conditions.
- Significant differences between API and FHWA methods that are beneficial when using API:
    - API evaluates friction inside of pile plug formation, rather than a more arbitrary selection by FHWA
    - API limits side resistance to a maximum mobilized value whereas FHWA (Nordlund) does not include an upper limit.
- The soil properties and limiting values of pile resistance recommended/determined by API method were adjusted based on KYTC and Terracon experience and the CPT data collected for the project. CPT data were especially helpful in evaluating limits on mobilized pile resistance.

- Key question was development of the plug – if the plug is being relied upon to either achieve penetration into the chert or to achieve the nominal pile resistance, how could the certainty of the plug developing and its location be determined?
    - Wanted to keep the pile open to get it down relatively easily, but needed it to plug to get the pile to drive the minimum distance into the chert required for fixity.
    - Experimented with where to create the plug, designing insert plates to form the plug at a fixed location. Design evaluations included discussions with Dr. Sam Paikowsky, P.E. and his work on the Sakonnet River bridge in Rhode Island.
    - A significant concern was related to piling quantity – be able to reliable estimate where the plug would form to get the needed penetration of pile and nominal resistance.
    - Also concerned with schedule impacts if piles did not achieve required resistance and need to be driven deeper.

Drivability and Plug Development

- Started with the API method internal and external skin friction, compared to plug forming to determine best point to set the plate to engage the plug.
- For the test piles, the plate was set higher to have the piles penetrate further into the chert than determined for the design. Wanted to be sure that the piles were bearing in the chert.
- The test pile results indicated a silty sand with gravel layer that helped to increase the pile resistance if the plug could be engaged at this stratum. Fine sands and silt/lean clay zones were not adequate to develop end bearing on the plate.
- *Axial pile load testing indicated that the 48-inch-diameter piles were more likely to achieve a plugged condition with the steel constrictor plates located 98 feet above the design pile tip than the 72-inch-diameter piles at the test locations. When the piles were driven to or near the design pile tip elevation at the test locations, the 72-inch-diameter piles did not achieve a fully plugged condition with the steel constrictor plates located 98 feet above the pile tip.*

Dynamic Testing and Wave Equation Analysis

- *Selected dynamic pile testing records were reviewed by a sub-consultant to Terracon with extensive pile testing experience to consider other methods of improving the match quality and the estimation of static pile resistance. These analyses used a single-toe pile model, and while the initial analyses performed by the contractor's testing consultant used a double-toe model.*
    - *Radial or radiation damping models were applied to the signal matching analysis. The radiation model assumes some energy is radiated away from the pile tip instead of being completely confined to static and dynamic responses of the soil shear along the pile and at the pile toe.*
    - *For the dynamic records where radiation damping was applied, the model generally resulted in a significantly better signal match quality, indicating the*

*radiation damping allows CAPWAP to better model the signals recorded by the dynamic pile testing equipment.*

- *The pile resistances calculated with CAPWAP using the radiation damping model also generally produced higher end bearing resistance values than the CAPWAP models without the radiation damping. It appears that the radiation damping model is better suited for estimating the end bearing component of the piles when less pile set is experienced per hammer blow. This is the case when the constrictor plates are engaged on the dense granular soils.*

- Wave equation analyses indicated that plugged piles would have high stresses. Additionally there was concern that localized high stresses might be encountered due to the presense of the chert. Testing on the piles typically did not approach as high values as expected.

- Experimented with wall thickness ($t_w$) during wave equation analysis and the testing program
- 48" dia. pile – $t_w$ = 1.5" and 1"(48" piles were not selected for final design of the bridge)
- 72" dia. pile – $t_w$ = 2" and 1.5" (selected 2" for final design of the bridge)

<u>Lessons Learned</u>

- It is important to be certain that the plug forms where it is counted on. Using plates can reduce risk of plug not forming as intended.

*NCHRP 45-05*
*Interview Notes – Louisiana Department of Transportation and Development* *Page 1 of 2*

| | |
|---|---|
| *Agency:* | Louisiana Department of Transportation and Development |
| *Participants:* | Ching Tsai, P.E.<br>Chris Nickel, P.E. |
| *Interview Date:* | April 21, 2014 |

General

- Typically use concrete cylinder piles rather than steel pipe. Concrete are less costly.
- Piles used mostly in coastal areas.
- 54in diameter spun cast is very common.
- Had some issues with pile cracking, especially near the top of the pile. Adjustments to the detail for strands and number of turns reduced cracking.

Design

- Use FHWA methods for design, but no set "standard" for assumptions with regard to plug formation.
- For LA 1 project, started by assuming pile will plug 50% for static analysis, but always look at each case separately.
- One project with 54in diameter, 160ft long piles had test piles instrumented with strain gauges near the bottom. Using Davisson's method, the end bearing is 55% of fully plugged capacity. If plunging is the criterion, then 83% plug efficiency. Try to balance size of pile, capacity, and quantity of piles.
- Available driving equipment and barge access (draft) tent to limit the pile size that can be used on a particular project.
- Setup is accounted for in design. Typically do a restrike program up to 14 days after initial drive. A paper was published in the proceedings of the 2014 GeoCongress (Geo-Institute of ASCE) including LA 1 test program and investigation of size effect on setup.

Load Tests

- Very few static tests, relying on statnamic and dynamic testing
- Pre-production test piles will be specific to a project and not always included.
- Test piles driven during production, including restrikes to develop setup curves and establish driving criteria.
- Number of piles tested depends on site variability, not a minimum number per bent, etc.
- For LA 1, barge access dictated test pile locations.

Drivability Assessment

- Not usually evaluated during design.
- Wave equation analyses performed based on contractor equipment submittals.
- Test piles are monitored and adjustments made to WEAP models based on field results.

Driving Aides

- Evaluated on a case by case basis
- Jetting is sometimes allowed, usually no more than through the scour zone where no side resistance is counted on for design.

Lessons Learned

- Comfortable with spun cast concrete piles, paying attention to details such as reinforcement at the top (for driving stresses) and vent hole placement to relive pressure inside the pile during driving.
- At LA 1 project, comparing the cylinder piles to other test piles at the same location, the unit skin friction appears to be slightly smaller for the cylinder piles. When combining the effects of larger than calculated tip resistance, the results are good total resistance prediction, but not as good for individual components.

| | |
|---|---|
| *Agency:* | Minnesota Department of Transportation |
| *Participants:* | Gary Person, P.E.<br>Derrick Dasenbrock, P.E. |
| *Interview Date:* | April 24, 2014 |

MnDOT began using LDOEPS about 10 years ago. Their experience to date is with 42in diameter steel pipe piles.

Design

- Steel pipe piles, 42in diameter on all projects to date; though have one project tout for bid with 30in diameter piles and an upcoming river crossing that may have piles larger than 42in diameter
- Use FHWA design methods (Meyerhoff, Nordlund) and CPT based methods when designing for non-rock bearing with no modifications. Some checks were conducted with A-Pile (but not principal design). Hastings Crossing was Design-Build procurement with analysis by consultant. Most projects have been to rock, as rock has been 'reasonably close' and therefore cost effective.
- Believe that the static resistance methods are reasonable estimates of the long term resistance of the pile, but not always easily demonstrated with dynamic testing methods. Also: lack of reliability and repeatability. Due to infrequency of use, there is also concern with the PDA expertise and appropriate modelling of the pile and proper selection of associated damping/quake needed for analysis.
- Most piles have been driven to refusal on rock, so side calculating static side resistance not as important as the limiting structural resistance of the pile.

Plug Formation

- So far have not experienced significant plug formation. Large piles have thus far exclusively been used at 'sandy' fluvial sites (river crossings).
- Currently believe that could be that the large velocities generated by driving result in the pile "cookie cutting" through the soils at the project sites.
- Also have granular materials so could be some local liquefaction of the sands at the pile-soil interface allowing free movement of the pile relative to the soil inside.

Drivability Analysis

- Perform initial check during design for larger diameters and special conditions (such as needing to penetrate a hard layer to get to minimum tip) where high driving stresses are anticipated to verify the designed piles can be installed with typical hammers used.
- Contractors are required to submit wave equation analysis with equipment submittal.

Dynamic and Load Testing

- Wakota Bridge provides a good case history of early use and testing of LDOEPs by MnDOT.
- Main issue is demonstrating required resistance via dynamic testing methods.
- In MnDOT experience, initial drive PDA/CAPWAP appears to consistently under-predict capacity when compared with restrikes, static methods, and rapid load test methods. Having recognized this, rapid load testing has been required on all projects following to Wakota.
- Difficulty with dynamic testing is being able to provide larger enough hammer to move the pile to demonstrate the required resistance once pile firmly bearing on rock or any setup has occurred.
- Have adopted the approach to perform rapid load tests (Statnamic) to better assess the static pile resistance and help correlate/calibrate PDA data to provide comfort level with results.
- Partly a design/construction process issue where bridge designer has criteria of demonstrating required resistance at end of drive. Continued use of rapid load tests with PDA testing should increase confidence that end of drive resistance can be correlated with long term static resistance.

Lessons Learned

- We have found the under-prediction of the dynamic formula at time-of-drive to be problematic as it 'spooks' the field engineers who are very worried that the measured capacity is sometimes even below the factored capacity (and therefore well under the required nominal capacity). Restrikes should be included to assist in demonstrating that there are 'driving effects.' Statnamic or static load tests should be used to verify, particularly as an agency is beginning use of LDOEPs.
- Existing FHWA design methods appear to provide reasonable estimates of static (long-term) pile resistance.
- Use of LDOEPs has been very successful and now the first driven piling option considered for large river crossings.

*NCHRP 45-05*
*Interview Notes – New York State DOT* *Page 1 of 3*

| | |
|---|---|
| *Agency:* | New York State DOT |
| *Participants:* | Steve Borg, P.E. – Civil Engineer II (Geotechnical Engineer) |
| *Interview Date:* | April 17, 2014 |

*Note: Items in italics are from previous interview for Synthesis Project 41-10 (Developing Production Pile Driving Criteria from Test Pile Data) dated August 31, 2010*

General Experience

- Typically use concrete cylinder piles with 36in to 54in diameter. Steel pipe is usually less than 36in, though have had a few projects with 36in pipe piles. Pile lengths of up to 160ft are common on Long Island.
- Soil conditions Upstate are typically sands, silts, gravels and clays, and other glacial deposits. Long Island is typically sands and some clay.
- Gave a summary of a few example projects.
    - Near Saratoga – 36in cylinder piles, could not get shaft resistance in the gravels, but achieved adequate setup in the clays. For piles in gravel that were not getting capacity, ended up driving H-Piles through the void of the cylinder to rock.
    - Chautauqua Lake – 36in cylinder piles in lacustrine deposits and till
    - Pedestrian Bridge north of NYC – all precast bridge with single column piers. Drove 36in cylinder piles with no jetting.
    - Jamaica Bay and others on Long Island – 54in cylinder piles using pre-jetting
- LDOEPs usually selected to avoid cofferdams for pile footings while providing good bending resistance. Cylinder piles used for pile bents, all plumb piles. On Long Island, cylinder piles selected for corrosion resistance, tidal scour considerations. Seismic forces sometimes also play into the considerations.
- No problems with cylinder pile manufacturing. All piles come from Bayshore.

Design

- Design approach is to use AASHTO/FHWA (methods listed in survey) tempered by experience and dynamic tests of previous projects.
- Lateral design by LPILE and GROUP . No uplift.
- For plug formation – use recommended GRL WEAP method for increasing mass of soil at tip of pile, not increasing stiffness of pile. Will usually do a sensitivity analysis by varying plug length.
- For setup – NYSDOT has history of dynamic tests with restrikes documenting setup in areas where LDOEPs are typically used. Generally see 40% to 50% setup, even in the Long Island sands. Restrikes typically done 24 hours after driving.
    - *The biggest challenge for NYSDOT is developing good estimates of set up in the clays soils found in certain parts of the state. Experience indicates that setup times can range from 24 hours to one month. The project schedule is typically set such that there is not time to allow a test program to fully investigate pile set up .*

*Piles tend to be over driven (driven to higher resistance than necessary if setup in clay was better defined) in order to meet the schedule.*

- No relaxation. (We sometimes see relaxation when H-piles are driven to shale bedrock)

Load tests

- Used static + dynamic 25 to 30 years ago. Now only dynamic tests, both for measuring resistance/setup and for damage control (cracking and damage to concrete). Have not uses Statnamic.
- Dynamic tests are sometimes done on pre-production test piles to set order lengths, others on production piles only.
- Some projects give contractor the option to have sacrificial test pile or use production pile.
- Have not performed lateral load tests.
- Dynamic tests always include signal match analysis using CAPWAP software.
- *NYSDOT will use the base resistance from the end of initial drive with the side resistance from restrike blows to estimate the static pile resistance. For very long piles, the side resistance from several blows is superimposed to estimate the side shear resistance for the pile.*

Drivability Analysis

- Drivability analysis performed during design using an assumed driving system. Most contractors tend to use one of two hammers: Conmaco 5300 or Raymond 60X. (Diesel and hydraulic hammers have also been used on cylinder pile projects.)
- Wave equation analysis also performed using contractor submittal once received during construction.

Construction and Quality Control

- *Driving Criteria*
  - *Wave equation analysis is typically used to evaluate the contractor's hammer system submittal and set the driving criteria. The inspectors are provided an acceptance blow count and minimum hammer energy or stroke criteria. Restrike blows are only used if piles do not achieve the desired resistance at the estimated drive length*
  - *For soils conditions or large projects where High Strain Dynamic Test (HSDT) are more suitable, the HSDT are used to set the driving criteria. One test pile (a production pile) per substructure is tested at initial drive and with a 24-hour restrike. The inspector is provided the acceptance blow count and hammer performance criteria based on the HSDT results.*
  - *With pre-cast pre-stressed cylinder piles, pre –production HSDT is performed to set pile lengths and determine the driving criteria. Evaluation of tensile and*

*compressive stresses in the piles during driving is also a major part of dynamic testing of these piles.*

- Pre-Jetting
  - On Long Island, pre-jetting is typically used. No pre-jetting when LDOEPs used Upstate.
  - With pre-jetting on Long Island, temporary casing is also required to prevent debris falling into the pre-jet hole. The area has a lot of riprap and such for scour protection. it often ends up in or near channels, etc. Had a project that debris kept falling in to the pre-jet holes, obstructing piles. Typical process is to vibrate casing, jet, drive pile, pull casing.
- Dynamic tests are done on production piles to monitor driving stresses, prevent damage, and check resistance. Can be one per bent or one every other bent (especially when constructing bridge in two phases).
- On one project had a sacrificial pile that was intentionally overstressed to measure maximum compressive stress in the pile. The results showed that 4.5ksi was a good limit of concrete strength – started to fail/crumble beyond that.
- No pile points or other toe protection used.
- NYSDOT has a special specification for concrete cylinder piles. It can be modified for a specific project or used as written.
- No experience with splicing cylinder piles – either pre-planned mechanical splices or unplanned field splices.

<u>Lessons Learned</u>

- Pre-jetting with debris – learned to use temporary casing, include a contingency amount in the contract for debris removal.
- Measurement of 4.5ksi stress for concrete piles mentioned above. It was 36in diameter pile.
- It is crucial to take measurements early in the driving after jetting to measure and monitor tensile stresses.
- Experience that fine cracking will occur in the cylinder piles, but usually terminate at the horizontal joints between the pile segments.
- Need to have pile lengths ordered to have sacrificial length at top of pile to be cut off to provide sound pile at top for connection to bent cap. This avoids need for buildups.
- Have tried using wood plugs to plug the pile, but have not had any success with this technique.
- Obstructions are always an issue in locations NYSDOT uses these pile.

# APPENDIX D

## Interview Notes of Private Practice

*NCHRP 45-05*
*Interview Notes – D. Michael Holloway, Ph.D., P.E. - Consultant* *Page 1 of 2*

| | |
|---|---|
| *Agency:* | Industry |
| *Participants:* | D. Michael Holloway, Ph.D., P.E. – Consultant |
| *Interview Date:* | April 22, 2014 |

The discussion focused on the key issues that Dr. Holloway believes are important for the industry to consider or solve with respect to LDOEPs. Many of his observations are from his extensive practice in California.

General Background

- Adoption of widespread use of LDOEP (pipe pile mostly) in California did not really begin until after the 1989 Loma Prieta earthquake. One design change after the quake was to design pile foundations to not fail – to remain elastic. The pile-structure connection could be damaged, but the foundations needed to survive undamaged.
- When applying the new design methodology, CALTRANS adopted load factors that were overly conservative.

Driving to Rock

- Many sedimentary rocks where hard driving of LDOEPs occurs can result in breakdown of the composition of the rock.
- We tend to see relaxation of base resistance due to the breakdown of the rock, resulting in less base resistance than estimated (substantial toe relaxation).
- Restrikes frequently show a decrease in base resistance under these circumstances.
- Piles needed to be driven 1 to 1.5 meters in restrike into the rock to see base resistance increase again in some cases.
- Careful evaluation of the impact at the rock interface is necessary for these low displacement piles.

Driving Behavior

- Need to recognize that how an open pile drives is dependent on how the stresses are getting to the toe of the pile – the failure mechanism at the toe.
- With thin wall pile, wave equation analysis will sometimes indicate that stresses are not very large relative to the yield stress, but piles still have problems with collapsing during driving. This is usually due to poor driving alignment, resulting in ovaling and collapse of the piles due to transverse/eccentric stresses.
- Stresses at the toe need to be less than half of the yield stress of the steel to accommodate the eccentric forces encountered at the toe.

Plug Behavior

- Plugging or absence of plugging dominates the behavior, so understanding if it is plugging or not is key to understanding how the pile will drive.

- Adding toe treatments to make driving easier (say an inner ring to cut and loosen soil and reduce plugging) will reduce nominal base resistance of the pile. If the pile is to derive significant base resistance, care must be taken when adding means to reduce the plug to make driving easier.
- There is not a clear understanding of how the soil inside the pile behaves and how much of the friction resistance is derived inside the pile and how much outside.
- An old rule of thumb is to assume 2/3 of the friction is outside the pile and 1/3 inside the pile, as detected in high strain testing.
- The "devil is in the details". Plugging must be addressed wisely. Don't reduce plugging if it is needed for meaningful toe resistance.

Dynamic Testing and Wave Equation MOdeling

- Very difficult to accurately assess plugging effects using dynamic testing.
- Static or pulse loading tests should be considered to help with plug behavior evaluation.
- A worthwhile approach would be to either do an uplift test, or drill out the soil in the center of the pile, and then do dynamic testing to quantify friction distribution along the pile shaft.
- There are some investigators who assume that larger diameter piles made of higher strength steel will have a greater wave speed than the typical 16,800 ft/ sec wave speed customarily used for steel piles. Improper use of higher wave speed can lead to missing damage near the toe that is actually occurring.
- To evaluate plugging when using an internal plate to fix the plug in place, add soil mass to the pile in the WEAP model for the portion below where the plug is expected to form.

Static Axial Capacity Calculations

- Dr. Holloway tends to use static analysis methods for a rough estimate of axial resitance/capacity. Experience and dynamic testing/wave equation analysis interpretations tend to provide better estimates of pile resistance.
- A major problem in practice is using different methods for the base and side resistance. For example, using Meyerhoff for side resistance and using Nordlund for base resistance. The approach to the pile-soil behavior for each method is different, so mixing methods can lead to poor predictions.
- Many designers do not account for residual stress in the pile analysis. This leads to over-estimating side resistance and under-estimating base resistance. In many cases load tests are essentially "proof" tests confirming that the structure load can be supported, neglecting to apply axial loads to failure. However, this does not help provide a clear understanding of the true resistance and how the soil-pile interaction really behaves, which can lead to significant inaccuracy in making adjustments to the pile toe elevations.

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| *Agency:* | Industry |
| *Participants:* | Mike Muchard, P.E. –Applied Foundation Testing, Inc. |
| *Interview Date:* | April 24, 2014 |

The discussion focused on Mr. Muchard's experiences with testing LDOEPs for both offshore and on-shore structures.

- Soil plug behavior biggest issue for open end piles not driven to rock.
- Will pile plug or remain unplugged - high pile acceleration during driving will allow pile to cookie cut and not form a plug.-
- Uncertainty in predicting design capacity
    - If unplugged, pile acts as friction pile
    - If plugged, pile acts as a displacement pile
    - Is friction same on open end piles as other pile types
- Predicting drivability always a huge question
- Will you need a driving shoe when bearing on rock
- Is pile capacity verification with dynamic testing reliable? Dynamic testing may under predict or over predict due to plug behavior.
- For successful testing of LDOEPs, it is important to understand pile behavior.
- Things to consider for the testing program include
    - Durability of pile and testing equipment
    - Waterproofing of sensors and equipment
    - Protection system for sensors
- Interpretation issues
    - In concrete piles, accounting for residual stresses from manufacturing
    - Spiral Welded Pipe – placement of instrumentation and interpretation of results
    - Temperature before and after installation - thermistors must be incorporated
- Static or Statnamic tests tend to be more reliable for evaluating pile capacity than dynamic testing methods.
- Improved instrumentation means we can now obtain reliable strain measurements on pipe piles to help answer some of the questions about soil plugging.

*NCHRP 45-05*
*Interview Notes – Steve Saye, P.E. - Kiewit Engineering Co.* *Page 1 of 2*

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| *Agency:* | Industry |
| *Participants:* | Steven Saye, P.E. – Kiewit Engineering Co. |
| *Interview Date:* | April 21, 2014 |

The discussion focused on the key issues that Mr. Saye believes are important for the industry to consider or solve with respect to LDOEPs

Ability to Calculate Axial and lateral resistance is poor

- There is much we don't understand about the behavior of LDOEPs.
- Analysis methods are thought to significantly underestimate pile resistance of LDOEP's.
- Our methods of axial analysis don't appear to adequately capture the impact of construction practices for these larger piles.
- For piles above 36 in diameter, there are not very many well documented cases.
- Assembling good, well documented case histories of these piles should be helpful to DOT's and Industry.

Weld Quality and Inspection

- Mr. Saye is aware of a case where welding of splices for 42 inch diameter pipe resulted in significant quality issues. Further guidance to DOT's regarding the inspection and details required for welding of splices for steel LDOEP's is merited.

Pile Resistance in Sand (Cohesionless Soil)

- Generally observe LDOEP piles not plugging – the piles are advancing as a "cookie cutter" into the soil.
- Design methods based on smaller piles don't adequately predict resistance of LDOEP's.

Vibratory Hammers

- For very long, spliced pipe piles, a common practice is for a contractor to install the first section with a vibratory hammer to set the pile.
- Design does not always take this method of installation into account.
- The effect of vibratory installation on pile resistance is not well understood. Significant differences in opinion occur regarding the damaging impact of vibratory hammer installation of piles in clay, or not. No good comparisons of the actual effect of the vibratory hammer on pile installation in clay are available.
- The Corps of Engineers Specifications allow vibrating up to 50% of the pile length on some of the hurricane protection projects in New Orleans. Many other standard specifications do not address the use of vibratory hammers.

Spiral Weld Pipe

- PDA procedures need special attention for spiral weld pipe. Experience where PDA tests were run with 2 accelerometers and 4 strain gauges. The static load test and 14 day restrike (after load test) were significantly different. Another project showed better comparison of PDA and static load test results when 4 accelerometers and 4 strain gauges were used.

Effects of Delays for Splicing in Clay

- Potential damaging impact on long-term pile resistance from significant delays for splicing.
- If contractor sets first section of many piles, allowing weeks between the driving of the first section and the re-start of driving of the first piles that were set, does the re-driving have a negative impact on the pile shaft resistance in that first section? In clay, the remolding of the soils along the pile could result in lower pile resistance than expected or would be available had splicing and re-start of driving occurred within a short time period rather than weeks.

Differences between Concrete Cylinder and Steel Pipe

- Very few cases of comparing the two types of LDOEP's on the same site to investigate the differences in driving behavior, plugging, and pile resistance in the same conditions.
- Efforts need to be made to get data made available from the few recent projects, such as the Inner Harbor Closure project in New Orleans.

Research Needs

- An assembly of good quality, instructive case histories or database of LDOEP behavior with cone penetration tests to characterize the soil conditions.
- Comparisons of dynamic and static load tests to better correlate dynamic testing and wave equation analyses with static resistance.
- Assemble current pile inspection and acceptance criteria.
- Defining the pile movement corresponding with the selected pile load test capacity for the LDOEP static load tests.
- Evaluate the impact of vibratory pile installation on the capacity of piles, including LDOEP's.
- Evaluate the effect of delays in pile installation for splicing on the side resistance capacity of LDOEP's.

*NCHRP 45-05*
*Interview Notes – Bob Stevens, Ph.D., P.E. – Fugro-McClelland* *Page 1 of 2*

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| *Agency:* | Industry |
| *Participants:* | Bob Stevens, Ph.D., P.E. – Fugro-McClelland Marine Geosciences, Inc. |
| *Interview Date:* | April 24, 2014 |

The discussion focused on the experiences of Dr. Stevens based on his 36 years in design and construction of LDOEPs for both offshore and on-shore structures.

- One of the key concepts is to determine if the pile will plug or not.
- Dr. Stevens early work included investigating analyses of plugging behavior by evaluating the acceleration of the soil mass in the pile with respect to the acceleration of the pile. Inertial forces need to be evaluated in the wave equation analyses.
- His offshore experience (over 500 sites including 250 platforms) indicates that plugging rarely occurs during driving.
    - He has seen piles driven 200 to 300 feet in clay with no plugging
    - The acceleration of the pile is almost always greater than the soil mass under the large forces from the hammer on the pile.
    - Very few times he has seen the use of plates or other devices to cause the pile to plug at a set depth, he believes they have been successful.
    - While the pile usually won't plug during driving, the behavior under static loading will usually be plugged behavior.
- Proper use of PDA equipment to monitor and test LDOEPs is essential.
    - Larger diameters require 4 transducers and 4 accelerometers to better average the stress-time history across the pile.
    - The transducers are very robust and can withstand the long driving times and heavy impacts of these piles.
- Estimating static resistance with API RP2 method is probably the best approach and entirely applicable to transportation structures. Dr. Stevens consulted on bridge projects using the API methodology (Bay Bridge in San Francisco, Trans Tokyo Bay Bridge in Japan).
    - API regularly working to update the procedures based on data from the field.
    - Recent updates include modification to the design of piles in sands based on CPT test data. This procedure provides better estimates of pile resistance in very dense sands.
    - Care does need to be exercised when estimating the mobilized end bearing for drivability analysis.
- While static tests are not common for LDOEPs, when they are performed, Dr. Stevens usually sees very good agreement (within 5%) of pile resistance determined from CAPWAP analyses of dynamic test results and static load tests.
- When driving through soft rock (very hard clay, shale, siltstone, gypsum, etc.), the toe stresses and toe displacements must be very carefully monitored.
    - Stop driving if toe stress reaches 80% of yield stress of pile.

  - Stop driving if toe displacement turns negative – indicates toe is being damaged/crushed.
  - Important to check drivability using 90% end bearing, looking at stresses at toe, modeling a fixed-end condition.
- Current state of practice appears to not consider the long-term strength gain of piles in clay.
  - Pore pressures can continue to dissipate over years, slowly increasing pile resistance.
  - Did restrikes on a 24in diameter pile 30 months after load test and had significant increase in resistance.
  - Also did restrikes on 96in diameter piles 24 months after driving.
  - The 14 day restrike or static test of a pile is not the maximum or nominal resistance of the pile.
  - Current practice disregards a lot of resistance available for piles in clay.

*NCHRP 45-05*
*Interview Notes – Scott Webster, P.E. – GRL Engineers, Inc.* *Page 1 of 2*

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| *Agency:* | Industry |
| *Participants:* | Scott Webster, P.E. – GRL Engineers, Inc. |
| *Interview Date:* | May 2, 2014 |

The discussion focused on the experiences of Mr. Webster based on his 33 years in construction and testing of LDOEPs for both offshore and on-shore structures.

- While the majority of Mr. Webster's experience has been with construction and testing, his observations from the numerous offshore projects he has been involved with indicate that design methods are conservative to the actual pile resistance available. In many cases, significantly more resistance is available than considered in design.
    - Sometimes the willingness of the owner/designer or the time needed to demonstrate and use higher resistance is not available.
    - With offshore construction, the time window for completing foundations is very narrow (sometimes as little as 2 to 3 days). Thus, the time to verify long-term resistance is not always available.
    - Another consideration of the offshore industry is that the difficulty of delivering materials to a platform location means that the provided pile lengths need to achieve the needed resistance with no margin for splicing, etc. Thus, estimates of resistance tend to be conservative to avoid problems during construction.
- The increased use of LDOEPs is a result of the significant improvements in pile driving equipment making installation of these piles possible. The hydraulic hammer provides the ability to install the larger piles. Some pile sizes may not be possible with diesel hammers.
- Design loads are increasing, especially for extreme events such as vessel impact, scour, and in the case of offshore structures, cyclic loading and fatigue.
- Pile plugging during driving is a behavior we do not fully understand and needs more investigation to fully understand it.
    - It is often treated as if the choice is one or the other: plugged or unplugged.
    - The actual behavior of the pile is somewhere in between. Whether a pile will plug during driving or not is extremely difficult to predict.
    - One debate in the offshore industry is the proportion of skin friction development between the outside and the inside surfaces of the pile.
    - How plugging affects driving is related to pile diameter, soil type, and the hammer selection, not one single factor.
    - The blow of a hydraulic hammer results in higher pile acceleration than that from a diesel or steam hammer. The hydraulic hammer acceleration is almost similar to that of a vibratory hammer.
- Plug behavior for long term static capacity also still needs to be looked at to fully understand the behavior.

- Due to the higher loads required, the limit of the ability of some hammers is being pushed to install these piles.
- With high loads it can be difficult to have a hammer of the appropriate size to verify or test resistance. The lower soil resistance during driving allows for the use of a smaller hammer than may be needed to demonstrate the full available resistance of the pile.

Abbreviations used without definitions in TRB publications:

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| A4A | Airlines for America |
| AAAE | American Association of Airport Executives |
| AASHO | American Association of State Highway Officials |
| AASHTO | American Association of State Highway and Transportation Officials |
| ACI–NA | Airports Council International–North America |
| ACRP | Airport Cooperative Research Program |
| ADA | Americans with Disabilities Act |
| APTA | American Public Transportation Association |
| ASCE | American Society of Civil Engineers |
| ASME | American Society of Mechanical Engineers |
| ASTM | American Society for Testing and Materials |
| ATA | American Trucking Associations |
| CTAA | Community Transportation Association of America |
| CTBSSP | Commercial Truck and Bus Safety Synthesis Program |
| DHS | Department of Homeland Security |
| DOE | Department of Energy |
| EPA | Environmental Protection Agency |
| FAA | Federal Aviation Administration |
| FHWA | Federal Highway Administration |
| FMCSA | Federal Motor Carrier Safety Administration |
| FRA | Federal Railroad Administration |
| FTA | Federal Transit Administration |
| HMCRP | Hazardous Materials Cooperative Research Program |
| IEEE | Institute of Electrical and Electronics Engineers |
| ISTEA | Intermodal Surface Transportation Efficiency Act of 1991 |
| ITE | Institute of Transportation Engineers |
| MAP-21 | Moving Ahead for Progress in the 21st Century Act (2012) |
| NASA | National Aeronautics and Space Administration |
| NASAO | National Association of State Aviation Officials |
| NCFRP | National Cooperative Freight Research Program |
| NCHRP | National Cooperative Highway Research Program |
| NHTSA | National Highway Traffic Safety Administration |
| NTSB | National Transportation Safety Board |
| PHMSA | Pipeline and Hazardous Materials Safety Administration |
| RITA | Research and Innovative Technology Administration |
| SAE | Society of Automotive Engineers |
| SAFETEA-LU | Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005) |
| TCRP | Transit Cooperative Research Program |
| TEA-21 | Transportation Equity Act for the 21st Century (1998) |
| TRB | Transportation Research Board |
| TSA | Transportation Security Administration |
| U.S.DOT | United States Department of Transportation |

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