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Two driven pile load tests for use in Missouri LRFD guidelines

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TWO DRIVEN PILE LOAD TESTS FOR USE IN MISSOURI LRFD GUIDELINES

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ABSTRACT

A static pile load test program was initiated by the Missouri Department of Transportation (MoDOT) to evaluate the use of pile load tests in Missouri LRFD guidelines. The program's approach involves two phases to achieve the appropriate levels of reliability for driven piles in the state of Missouri. This thesis focuses on the data collection efforts of Phase 1. Two quick static pile load tests were performed to failure on test piles in the Southeast Lowlands geologic region of Missouri. The piles were dynamically monitored during installation and subsequent restrike tests performed. The results of the static and dynamic pile testing were evaluated and interpreted. Overall, the nominal resistances predicted by dynamic tests (CAPWAP) at beginning of restrike (BOR) compared well to the results of the static load tests evaluated using Davisson's method (at these specific sites). A comparison of the load transfer distributions from the dynamic and static load tests provided mixed results. The effects of pile set-up after driving are a significant factor to consider in determining the need for a restrike. The additional resistance available following pile setup can have a substantial effect on the nominal resistance determined using dynamic methods. When BOR capacities are measured using dynamic methods they can be used with confidence for the calibration of resistance factors with respective pile types and geologic units. Available pile load test data sets from Missouri's neighboring states and previous efforts conducted in Missouri were compiled as well. Two recently available pile load test databases were evaluated and considered for the upcoming phase to conduct calibration of resistance factors.

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1. INTRODUCTION

1.1. INTRODUCTION

Driven piles are the most common foundation system used in nearly 10,000 bridges encompassed within Missouri's state highway system. The geotechnical community in the United States has traditionally used the Allowable Stress Design (ASD) method to produce sufficient structural foundations (DiMaggio et al. 1999). ASD compares the actual forces estimated to be applied to the structure to the structure's available resistance, or strength, through a value known as the factor of safety (FS). The FS is a summary of the engineer's best estimate of the uncertainty associated with the project as a whole. Using the FS to determine the design loads of a foundation often reflect conservative estimates of a member's actual available resistances. Traditionally, different magnitudes of FS have been used to reflect the different levels of control in foundation design and construction, as well as past experience and engineering judgment (Paikowsky, 2004). However, it has long been recognized that standard bridge design specifications based on ASD do not promote a consistent reliability for design (AbdelSalam, 2010). Realizing this deficiency, extensive research efforts have been devoted to the development of a more rational design approach known as Load and Resistance Factor Design (LRFD). LRFD has been well established in design codes around the world for Structural Engineering, and was first adopted in North America through the American Concrete Institute (ACI) Code in 1953 (DiMaggio et al. 1999). The objective of LRFD is to produce engineering designs with consistent levels of reliability using procedures from probability theory to ensure a prescribed margin of safety (Paikowsky, 2004). Under LRFD, the uncertainties in loading are assessed separately from the uncertainties in resistance through load factors and resistance factors, respectively. The load factors and resistance factor are applied in such a way that the engineer is essentially over-estimating the loads on the structure and underestimating the structure's strength, thus assuring a consistent level of safety.

In 1994, the American Association of State Highway and Transportation Officials (AASHTO) published the first edition of LRFD bridge specifications. The new LRFD specification contains comprehensive design and construction guidance on both structural and geotechnical features. Initial use of the new specification, however, showed that the approach used in LRFD for structures is not fully compatible with geotechnical design needs (DiMaggio et al. 1999). As a result many geotechnical engineers reverted back to the ASD method of designing foundations that they were accustomed to using in the past. The structural engineers using the LRFD method to design the bridge's superstructure and the geotechnical engineers designing the substructure using ASD not only created uneconomical designs but also decreased the reliability of the designs.

In order to produce more reliable, consistent designs AASHTO and the Federal Highway Administration (FHWA) issued a policy memorandum on June 28, 2000, requiring all new bridges initiated after October 1, 2007, to be designed using the LRFD approach (Densmore, 2000). AASHTO included resistance factors in the LRFD specifications developed from a collection of Static Pile Load Test (PLT) data from around the U.S. However, these national resistance factors were overly conservative when applied to localized regions because of the variability in the geology and construction practices used to calibrate them. For this reason, AASHTO permitted state Departments of Transportation (DOTs) to develop their own resistance factors based on regional practices and geology to minimize the unnecessary conservatism built into a design. Following the authorization of regional resistance factors, many states such as Florida, Illinois, Washington, and Iowa have all published studies recommending LRFD resistance factors for driven pile foundations within their respective states.

1.2. PILE DESIGN IN MISSOURI

Upon the inception of LRFD in Missouri, the Missouri Department of Transportation (MoDOT) adopted the resistance factors from the AASHTO LRFD Bridge Design Specifications (2010) for designing bridge pile foundations. However, due to the relatively low resistance factors associated with the analysis methods commonly employed by MoDOT, the acquired design loads continue to reflect conservative estimates of a member's available resistance. As a result, MoDOT is unable to gain from the advantages encompassed in LRFD design.

In 2008, MoDOT supported its first research program to develop a series of LRFD specifications based on the local geotechnical practices and geology within the state. Upon the project's completion in 2010, a newly developed set of resistance factors were calibrated using existing data from historical construction records of dynamic testing of piles. That is, Pile Driving Analyzer (PDA) and CAse Pile Wave Analysis Program (CAPWAP) software. Although the results of the program suggested the current resistance factors used should be increased, no records of static pile load test data were available to evaluate the actual ultimate capacity of the piles. Therefore, the newly calibrated resistance factors were developed under the strict assumption that dynamic testing methods provide the actual ultimate capacity values.

To validate this assumption a subsequent research project entitled, Evaluation of Pile Load Test for Use in Missouri LRFD Guidelines, was initiated. This thesis will discuss the current research efforts to evaluate the previously calibrated resistance factors based on high-strain dynamic testing methods for use in Missouri.

1.3. RESEARCH OBJECTIVES

The research provided herein is dedicated to allow MoDOT to produce more reliable and economically efficient design for pile foundations by accomplishing the following objectives:

- Evaluate MoDOT's current practice for pile foundations and provide recommendations for improvement in future practice, as well as for future research.
- Develop research grade, static pile load test data sets from previously characterized locations within the Missouri highway system.
- Evaluate the ability of high-strain dynamic testing to predict the actual nominal resistance measured by the static pile load tests, in hope of proving the accuracy of the 2008-10 developed resistance factors
- Compile the data collected from Missouri and it's neighboring states to assist in the establishment of a database and regional resistance factor calibration in a future phase.

- Propose recommendations to improve pile load testing procedures for future development of LRFD resistance factors in future research programs.

1.4. THESIS ORGANIZATION

The research provided herein consists of a literature review of driven piles summarizing: various methods for determining pile resistance, two methods used to design piles, and various states, including Missouri, efforts to accommodate LRFD design. MoDOT's state-of-practice and the multiple geologic regions found in Missouri are discussed followed by the methods, results, and data compilation of the current research effort. The thesis is organized as follows:

- Section 1 introduces the research effort.
- Section 2 describes piles in general, various methods for determining pile resistance (static analysis, wave equation analysis, high-strain dynamic testing, and static load testing). Two methods used to design piles (Allowable Stress Design and Load and Resistance Factor Design) are introduced and previous research programs devoted toward the development of regionally calibrated resistance factors are discussed.
- Section 3 discusses MoDOT's effort to implement LRFD, MoDOT's current state-of-practice and procedure for designing pile foundations including common types, sizes, and methods for determining resistance and length, together with an overview of Missouri's geological regions.
- Section 4 discusses the methods used throughout the pile load test program, including descriptions of the test equipment, instrumentation, and data acquisition system used, as well as an outline of the testing procedures and data reduction procedures.
- Section 5 discusses the results of two (2) pile load tests conducted at different sites within the Missouri highway system in the Southeast Lowlands of Missouri.
- Section 6 provides a summary and discussion of the results presented in Section 5.
- Section 7 discusses the effort established to compile datasets from projects completed in Missouri's neighboring states and previous projects completed in Missouri.

- Section 8 provides conclusions based on the research presented herein, as well as recommendations for the future practice for MoDOT and future research projects.
- The appendices include MoDOT bridge plans, MoDOT special provisions, static analysis results, GRL WEAP analysis results, dynamic testing reports (produced by Geotechnology, Inc.), unreduced static pile load data, and static pile load test results associated with each of the load tests performed during Phase I of this research project. A series of files containing pile load test data from other research projects are also included.

2. LITERATURE REVIEW

2.1. INTRODUCTION

Foundations are the structural components that distribute a structure's load to the soil. Composed of concrete, steel, wood, or a combination thereof, these elements are most commonly characterized as either shallow foundations or deep foundations. Shallow foundations (spread footings, wall footings, and mat foundations) transfer loads to near-surface soils. In contrast, deep foundations (both piles and drilled shafts) transmit some or all of the loads to a depth at which adequate support becomes available (Prakash, 1990). Whenever possible, shallow foundations are used because they are both cost effective and simple to construct. However, when the construction of shallow foundations is not feasible (i.e., when the required loads cannot be adequately supported at shallow depths), deep foundations provide an alternative solution. Based on the objectives of this research, driven piles will be the only foundation type discussed herein. The following sections will provide a brief overview of pile foundations, discuss various methods for determining pile resistance, and introduce two methods used to design piles.

2.2. DRIVEN PILE FOUNDATIONS

Piles are long, slender, prefabricated structural elements that are typically installed by either hammering or driving them into the ground. Pile foundations are generally used when proper bearing stratum are unavailable at shallow depths. They may also be used for structure's with large structural loads that would make shallow foundations would either uneconomical or infeasible (Das, 2007). Deep foundations provide resistance through mechanisms known as end-bearing and side friction. End-bearing is the resistance contributed by the area of the tip (or toe) of the pile; side friction is the development of resistive forces along the pile's length due to the friction/adhesion between the soil and pile during driving (Prakash, 1990).

When bedrock is located within a reasonable distance from the ground's surface, piles are commonly driven until they come into contact with the underlying bedrock. As a result, the pile's nominal resistance is significantly dependent on the bedrock material's load-bearing capacity (Das, 2007). Piles that obtain their resistance in this manner are classified as end-bearing piles. When bedrock is located at great depths and the installation of end-bearing piles is uneconomical, driven piles must rely largely on their side friction for resistance. Naturally, these piles are categorized as friction piles.

Piles are available in a variety of materials, diameters, and lengths, each depending on their application within a project. The following sections will present some of the common types of piles, as well as, each type's most common size and use.

2.2.1. Timber Piles. Throughout history, timber piles have been the most widely used form of piling. Derived from trunks of trees, timber piles are still a common option for use today due to their low construction cost. Timber piles can be fabricated from a variety of acceptable trees. Both Southern Yellow Pine and West Coast Douglas Fir are most commonly used today because they are tall, straight, and relatively abundant (Coduto, 2001). The dimensions of a timber pile are dependent on the specific tree being used. Diameters between 6 and 18 inches and lengths of up to 60 feet are, however, most typical (Das, 2007). Timber piles can be spliced together, though this process usually increases the cost of construction significantly. If the required length cannot be achieved with a single timber pile, an alternative material is typically chosen. Timber piles can carry design loads of up to 100 kips. They are best suited for light driving conditions, however, because they are more susceptible to damage during driving than piles made of other materials. Timber piles are most commonly used as friction piles in either loose sand or soft to medium clays (Prakash, 1990).

2.2.2. Steel Piles. Steel piles are commonly used in practice for projects with either difficult ground conditions or heavily loaded structures. The high strength and ductility of steel makes them ideal for driving in hard soils. Steel's high tensile strength also makes steel piles the common choice for tensile loaded applications. Steel piles are often the primary pile choice in areas with variable bedrock depths because they are easy to both splice and cut (Prakash, 1990). Disadvantages of steel piles include cost, noise during installation, and susceptibility to corrosion (Coduto, 2001). The most common steel piles used in engineering practice are pipe piles and H-piles.

2.2.2.1 Pipe piles. Pipe piles are available in a variety of diameters and wall thicknesses; diameters between 8 and 36 inches and wall thicknesses of up to ½ inch are typical (Coduto, 2001). These long cylinders can be driven open-ended or closed-end by welding a thick plate to the end of the pile. Closed-end pipe piles are commonly used as friction piles due to the increase in resistance created by the closed end. Consequently, the closed end causes a larger displacement of soil to occur making driving more difficult.

In the United States pipe piles are often filled with concrete after driving (Prakash, 1990). Once concrete has been placed in a pipe pile, it is referred to as a cast-in-place (CIP) pile. The placement of concrete provides the advantages of increased uplift resistance due to the additional dead-weight, greater shear and moment resistance due to the concrete's strength, and a longer service-life in corrosive environments (Coduto, 2001). The design resistance of CIP piles can be as high as 250 kips. However, when lengths surpass 80 feet, the cost of CIP piles generally becomes uneconomical (Prakash, 1990).

2.2.2.2 H-piles. H-piles are steel members manufactured specifically to be used as piles. Their shape resembles wither wide flange (WF) beams or I-beams. The primary difference is the web and flange thicknesses of H-piles are equal (the web thickness of both WF beams and I-beams is thinner than the flanges) (Prakash, 1990). H-piles are suitable for use in hard driving conditions because they displace a relatively small amount of soil during driving. Thus, H-piles are typically used as end-bearing piles and are driven to bedrock (Coduto, 2001). They may be damaged or deflected from vertical during driving through hard layers or past major obstructions. As a result, hardened steel points are regularly welded to the pile toe to provide protection during driving (Das, 2007).

2.2.3. Concrete Piles. Concrete piles are pre-cast, reinforced concrete members designed to withstand damage from not only handling and driving but also service loads (Prakash, 1990). Concrete piles are typically wither square or orthogonal in shape. Reinforcement is provided within the pile using lateral bars and ties, pre-tension, or post-tension methods. In the past, conventionally reinforced concrete piles (lateral bars and ties) were very common. Today, however, pre-stressed methods (pre-tension or post-tension) are almost always used in the U.S. (Coduto, 2001). Although concrete piles are more susceptible than steel piles to damage in hard driving conditions, they cost less than steel piles and can be used in corrosive environments (where steel is susceptible to degradation). Concrete piles can be used as either end-bearing or friction piles, although they are more difficult to cut and splice than steel piles. They are best suited for use in either end-bearing when bedrock depths are well defined or as friction piles that will not reach refusal (Coduto, 2001).

2.3. DETERMINING PILE RESISTANCE

An engineer must consider a number of options when designing a foundation with piles. These options include: pile type, length, diameter, shape, number and spacing. While the selection of these qualities is often determined by not only previous experience but also the availability of materials, the end result of all pile designs are the same: they must provide the required load-bearing resistance needed to support the structure. Although the nominal load of a structure is usually well-defined by the structural engineer, determining the actual nominal resistance available from the geotechnical engineer's design is not as straightforward. The uncertainties in the geotechnical design are primarily attributed to the prediction of the strength-deformation behaviors of soil and the overall performance of the soil-foundation system (Goble, 1996).

The maximum load a pile can carry before failing is known as the pile's nominal resistance (in LRFD design. It should be noted that piles provide axial, lateral, and pullout (or tension) resistances and although each of these modes can be evaluated separately, axial resistance will be the only form discussed herein. Furthermore, the term "resistance" throughout the remainder of this thesis will be in reference to the nominal resistance in the axial direction. The nominal axial resistance of a pile is a combination of the resistances provided by the end-bearing and the skin friction. The nominal resistance of an axially loaded pile is expressed in the following equation:

$$Q_u = Q_{eb} + Q_t \tag{2.1}$$

where Q_{eb} represents the end-bearing resistance and

Q_t represents the skin friction resistance.

The following sections will discuss the various methods for determining pile resistance including: static methods, wave equation analysis, high-strain dynamic testing, and low-strain static testing.

2.3.1. Static Methods. Static methods are empirical equations that use measured strength parameters from subsurface materials to predict the available side-friction and toe-bearing resistances of a pile during driving. Because in-situ tests are both subjective and highly-variable, the correlations provided by static methods have been viewed as less precise and conservative (Fang et al., 1975). Because geotechnical investigations are performed before construction is initiated, static methods are attractive because the geotechnical data needed for their calculation are usually readily available.

Static methods are most often used to initiate a preliminary design because they are the quickest and cheapest way to predict a pile's nominal resistance. These methods, however, require an engineer to both recognize and understand their limitations. Unlike shallow foundations, the installation of deep foundations causes changes to the local soil conditions. For example, as piles are driven into the ground, the displaced soil induces large horizontal stresses which consolidate the soil, changing its engineering properties (Coduto, 2001). As a result, the strength parameters measured before installation (in the geotechnical investigation) are not necessarily representative of the soil's strength parameters after installation.

The Federal Highway Administration (FHWA) provides a compilation of static methods to predict pile resistance through the computer program DRIVEN. This program is commonly used by the Missouri Department of Transportation (MoDOT) to create the preliminary design and follows both the methods and the equations presented by Thurman (1964), Meyerhof (1976), Nordlund (1963, 1979), Tomlinson (1980, 1986), Cheney and Chassie (1982), and Hannigan et al. (1997). The pile's nominal resistance is determined at selected depth intervals from the soil profile once the entire soil profile is input into the program. At each interval, DRIVEN distinguishes how much of the nominal resistance is contributed by skin-friction and how much is contributed by end-bearing. DRIVEN also has the ability to analyze multiple water tables, negative skin friction, and scour (Cravens, 2011).

2.3.2. Wave Equation Analysis. The wave equation is a numerical model that simulates the pile driving process by applying the theory of one-dimensional stress wave propagation (Rausche et al., 2012). Smith (1962) used a series of masses, springs, and dashpots to model all of the aspects influencing pile driving, including hammer mass and travel, combustion in a diesel hammer, helmet mass, cushion stiffness, hammer efficiency, soil strength, elastic properties of the pile, and so forth. The wave equation analysis then calculates the velocities, displacements, and resulting forces as a result of the impact per time for all of the elements in the system (Fang et al., 1975).

Many companies have commercially produced computer software to simplify use of the wave equation. The Wave Equation Analysis of Piles (GRLWEAP), produced by Pile Dynamics, Inc. is one of the most commonly used of these programs. When performed before driving, a WEAP analysis can be used to estimate the driving resistance, pile stresses, and hammer performance.

2.3.3. High-Strain Dynamic Testing. High-strain dynamic testing involves recording stress wave measurements at the pile head, under dynamic loading, to estimate the nominal resistance of a pile foundation (Uddin, 2001). Both the cost and the duration of this testing are much smaller than the cost and duration of an ordinary static load test. High-strain dynamic testing has become a common pile testing procedure for estimating not only pile resistance but also evaluating pile integrity for the driven pile (Rajagopal, 2012).

A series of instruments are installed approximately two pile diameters below the pile to measure the stress wave produced by the pile-driving hammer during impact. Two strain gages measure the induced strain and two accelerometers are installed to measure the induced acceleration. Both measurements are transmitted through a cable or wireless transmitter to a data acquisition system known as a Pile Driving Analyzer (PDA). This PDA (provided by Pile Dynamics, Inc.) is used to record, digitalize, and process both the force and the acceleration signals measured at the pile head.

2.3.3.1 PDA. The signals received on the PDA screen are given in plots of the measured force and velocity with respect to time. These plots are known as "wave traces" and provide valuable qualitative information on the distribution and magnitude of the soil's resistance (Fang et al., 1975). The PDA uses these wave traces to estimate the pile's nominal resistance through a simplified field procedure known as the Case Method (the uses of wave traces for the CAPWAP procedure will be discussed in Section 2.2.3.2.). Pile driving stresses, structural integrity, and hammer/driving system performance can also be evaluated from the received data (Coduto, 2001).

2.3.3.2 Wave equation/Case method analysis remarks. Although the Wave Equation and Case Method analyses are useful in practice, an engineer must be aware of their limitations. A wave equation analysis contains a more powerful numerical model than the Case method analysis. The parameters used in WEAP (or any other Wave Equation software) to estimate the hammer performance and transferred energy to the pile, however, are really variables with certain value ranges. Without knowing the actual energy delivered by the hammer and the resultant reaction of the soil-pile system, an analysis is only qualitatively correct; it is not necessarily quantitatively correct unless corrected by observation (Fang et al., 1975).

In contrast, the Case method analysis uses the actual energy delivered to the pile to produce the computation of some 40 dynamic variables in real time. However, it also contains an empirical value known as a damping factor (commonly represented as JC) (Coduto, 2001). This damping factor calibrates the analysis by considering the energy loss that takes place during driving. Because it is a function of the interaction between the soil-pile system, the numerical magnitude of the damping factor is specific to the soil conditions at the site. While the damping factor can be determined by on-site static or dynamic load tests, this value is most often determined from empirical correlations developed from sites with similar subsurface conditions, thus simplifying the true dynamics of pile driving (Coduto, 2001). Thus, the accuracy of the results determined from a Case method analysis are dependent on the engineer's ability to select the proper damping factor value and the quality of the collected data.

2.3.3.3 CAPWAP. The CAse Pile Wave Analysis Program (CAPWAP) uses the method of characteristics to solve the one-dimensional wave equation (PDI, 2006). The CAPWAP analysis can use the force, velocity, or wave-up values by the PDA at the end of drive (EOD) (or beginning of restrike [BOR]) to complete a more rigorous evaluation of the nominal resistance. The CAPWAP model divides the pile and soil into a series of segments which the user can adjust the damping, quake, and soil resistance variables to calculate a resulting force, velocity, or wave-up trace. By trial and error, the variables are adjusted until the calculated force, velocity, or wave-up trace plots on top of the traces measured during driving.

2.3.4. Static Pile Load Tests. A static pile load test (PLT) is the only method available to determine the actual pile nominal resistance. The objective of a PLT is to directly measure nominal pile resistance by slowly increasing an applied load until the member fails. Note that each of the methods previously mentioned estimate nominal resistance in an indirect, less precise manner. PLTs can be performed on both production piles that will remain in service or on "sacrificial" piles installed for load testing purposes only and removed after testing is complete. During a PLT, the applied load and the resulting settlement are measured to develop a load-settlement curve. This curve is used to determine the pile's nominal resistance. ASTM D-1143 (2007) contains the standard specifications of various arrangements and various methods for conducting a PLT under axial compressive loads.

2.3.4.1 Loading procedures. PLTs are categorized as either controlled stress tests or controlled strain tests (Coduto, 2001). Controlled stress tests apply predetermined loads to the test pile and measure the corresponding displacement. Controlled strain tests are simply the opposite. Because controlled stress tests are most common in practice, they will be the only type of loading procedure discussed herein. The following sections will discuss the various types of PLTs and multiple methods for determining the pile's nominal resistance from collected data.

2.3.4.1.1 Slow Maintained Load (ML) method. The Slow Maintained Load (ML) method is considered the traditional or "standard loading procedure." During this method, the test pile is loaded in eight equal increments up to a maximum load. Increments of 25, 50, 75, 100, 125, 150, 175, and 200 percent of the predetermined factored resistance are typically used (Fang et al., 1975). It is not uncommon for any load test to be performed past the 200 percent value. The most important aspect, however, is that both the skin-friction and the end-bearing resistance become fully mobilized to ensure failure has occurred.

Each increment is maintained until a minimum movement is reached. This movement is commonly referred to as the "zero movement." Zero movement is usually defined as either 0.01 in/hr or .002in/10min; it may be required to maintain each load 1 to 2 hours to meet this criterion (Fellenius, 1990). The maximum load, equal to 200 percent or greater, is always held for a duration of 24 hours. Overall, a Slow ML Test is very time consuming and can require between 30 to 70 hours to complete (Fang, 1975).

2.3.4.1.2 <u>Quick Maintained Load (ML) method.</u> The Quick Maintained Load (ML) Test, or, more simply, the Quick Test, is similar to the Slow ML Test. Unlike the Slow ML Test , however, each load increment in the Quick Test is held for a predetermined time interval before the next loading, regardless of the rate of pile movement (Coduto, 2001). For most Quick Tests, a maximum load of 200 percent of the predetermined allowable load is still used, though, in most cases, the number of loading increments is increased. A typical Quick Test arrangement may consist of 10 percent load increments held between 5 and 15 minutes each. When only the applied load and the movement of the pile head are monitored, time intervals of 5 minutes will typically suffice (Prakash, 1990). ASTM standards permit intervals of time between load increments as short as 2 minutes. Time intervals shorter than 5 minutes, however, may not be practical unless a data acquisition system is used (Fellenius, 1990).

A Quick Test can usually be completed within 3 to 6 hours, depending on the interval each load is held. The use of Quick Tests in practice has significantly increased due to their technical, practical, and economical advantages.

2.3.4.2 Interpretation of test results. As previously mentioned, data collected during PLTs is used to develop the load-settlement curve. Once this curve has been obtained, the engineer must determine when the pile's nominal resistance occurred. A number of methods have been proposed to interpret the nominal resistance (or failure load) from load-settlement curves. Choosing one method for use over another, however, is difficult; it is often heavily dependent on one's past experience and one's definition of failure. The following presents the procedures for five separate methods for determine the nominal resistance from PLT results.

2.3.4.2.1 Davisson (1972) method. Davisson's Method, also known as the offset limit, was developed in conjunction with the wave analysis of driven piles and dynamic measurements. This method is defined as the load corresponding to the movement that exceeds the elastic compression of the pile by a value of 0.15 inch, plus a factor equal to the diameter of the pile divided by 120 inches (Fellenius, 1990). The procedure for Davisson's (1972) Method, as outlined by Prakash (1990), is given as the following:

- Plot the load-movement curve.
- Plot the line of elasticity as:

$$\Delta = \frac{(Q_{va})*L}{A*E} \qquad (2.2)$$

where Q_{va} is the applied load,

L is the pile length,

A is the pile cross-sectional area, and

E is the modulus of elasticity of the pile material.

- Plot a parallel line and offset a distance of x from the line of elasticity:

$$x = 0.15 + \frac{D}{120\ inches} \qquad (2.3)$$

where D is the pile diameter in inches.

- The failure load is at the intersection of offset line and the load-movement curve.

The primary advantage of Davisson's method is that it can be used as acceptance criteria for proof-tested contract piles because both the line of elasticity and the offset line can be plotted before testing begins (Prakash, 1990).

2.3.4.2.2 Chin (1970) method. Chin (1970) proposed a method applicable for either Slow ML or Quick ML Tests as long as equal time increments are used between loadings. Under Chin's (1970) Method, each settlement reading is divided by its corresponding applied load value. The resulting value is then plotted versus the recorded settlement values. In general, the plot should result in a straight line with limited slope charges as the load is increased (Fang et al., 1975). The inverse slope of the resulting line is defined as the Chin failure load. The Chin Method allows the engineer to continuously monitor the readings being recorded. Particularly, sharp changes in slope can indicate a problem with either the pile or the test arrangement (Chin, 1978).

2.3.4.2.3 De Beer (1967) method. The De Beer (1967) Method plots the load-settlement values in a log-log diagram. This diagram, in turn, produces in two approximate straight lines. The De Beer (1967) failure load is then defined as the load that falls at the intersection of these two straight lines. De Beer's (1967) Method was proposed for Slow ML Tests, though it is often used for Quick ML Tests as well because of its simplicity.

2.3.4.2.4 Brinch Hansen (1963) 90 Percent Criterion. The Brinch Hansen (1963) Method defines the failure load (Q_{va}) as the load and corresponding deformation (Δ_u) that yields twice the movement of the pile head as obtained for 90 percent of the applied load (Fellenius, 1990). The method is applied as follows:

- Plot the load-movement curve.
- Using trial and error, find the load (Q_{va}) that yields twice the movement of the pile head (Δ_u) as that obtained for 90 percent of the load (Q_{va}):

$$Q_{va} = \frac{\Delta_u}{\Delta_{u\,@\,90\%\,Qva}} = 2 \qquad (2.4)$$

2.3.4.2.5 Mazurkiewicz (1972) method. The Mazurkiewicz (1972) Method, also known as "the method of intersections," consists of the following steps:

- Plot the load-movement curve.
- Choose a series of equal pile head movements, and draw vertical lines that intersect on the curve. Draw horizontal lines from these intersection points on the curve to intersect (and extend past) the load axis.
- Draw 45° line to intersect with the succeeding load line at the intersection of each horizontal line and the applied load axis.

- These intersections fall, approximately, on a straight line. The line of these intersections drawn back towards the load axis defines the failure load.

It is important to note that not all of these line intersections fall on a straight line. Therefore some judgment may be required in drawing the straight line to define the failure load (Prakash, 1990).

2.4. PILE DESIGN METHODS

All of the available information about the proposed structure, subsurface conditions, anticipated loading, and so forth must be compiled and analyzed to determine a suitable foundation design. The ideal foundation effectively transfers structural loads to the subsurface in a way that minimizes cost without sacrificing either safety or performance (Salgado, 2008). The difficulty in determining the ideal foundation lies in effectively evaluating the physical uncertainties associated with geotechnical practice: interpreting site conditions, understanding soil behavior, accounting for construction effects, and more (Paikowsky, 2004). Because each of these uncertainties increases the level of risk associated with a project, various methods are available to improve reliability within a design, ensuring a required level of performance is met. Regardless of the design philosophy used, the fundamental requirement of all design criteria is that the resistance (or strength) of the system must be greater than the demands (or loads) on a system (Becker, 1996). In the United States, the geotechnical community has traditionally used the Allowable Stress Design (ASD) method to produce sufficient structural foundations. Over the past two decades, however, both the American Association of State Highway and Transportation Officials (AASHTO) as well as the Federal Highway Administration (FHWA) have developed a new specification based on the Load and Resistance Factor Design (LRFD) method to replace its previous ASD specification (DiMaggio et al., 1999). It is important to note the differences in terminology between the ASD and the LRFD methodologies. In ASD the term "ultimate capacity" was used to define a member's failure load. Conversely, in LRFD the term "nominal resistance" is used to define the failure load. In the following ASD section, the term ultimate capacity will be used because it is standard in the methodology. However, in the LRFD section and the remainder of this thesis term nominal resistance will be used to refer to the pile's failure load. The following sections describe the traditional method of ASD and the transition to the contemporary design method of LRFD.

2.4.1. Allowable Stress Design (ASD). Allowable Stress Design (ASD), also known as Working Stress Design (WSD), has been the principal design method of civil engineering since the early 1800s (Paikowsky, 2004). ASD reduces the estimated ultimate capacity ($Q_{ultimate}$) to be applied to the structure by a value known as a factor of safety (FS). To produce a conservative estimate of the member's resistance, or allowable capacity (Q_{allow}), ASD is expressed in equation-form as:

$$Q_{allow} = \frac{Q_{ultimate}}{FS} \tag{2.5}$$

Under ASD, the FS is a summary of the engineer's best estimate in the uncertainty associated in determining the actual structural loads, material strengths, potential failure modes, geotechnical strength parameters, and so forth (Becker, 1996). Traditionally, different magnitudes of FS have been used to reflect the different levels of control in foundation design and construction. Presumably, when more reliable methods are used to establish a higher level of control, a smaller FS can be used. This smaller FS, in turn, leads to a more economical design (Paikowsky, 2004). Table 2.1 reflects the minimum value of FS permitted by AASHTO (2004) for the ultimate axial geotechnical capacity of driven piles based on the level of construction control (Withiam, 2003).

The primary advantage of ASD is its simplicity. A number of weaknesses, however, have been cited with regard to its approach in designing driven piles. For example, "analyses varying in quality and/or quantity cannot be incorporated directly into reduction of the required FS for design" (Rahman et al., 2002). Essentially, more intensive subsurface exploration or laboratory testing programs do not necessarily result in the ability to use a smaller FS. Additionally, ASD also does not associate different degrees of uncertainty with both the estimated loads on the structure and its available resistance. As a result, different probabilities of failure may correspond to the same FS.

Table 2.1 Factor of Safety Based on Level of Construction Control (AASHTO, 2004)

Basis for Construction Control	Increasing Design/Construction Control				
Subsurface Exploration	✓	✓	✓	✓	✓
Static Calculation	✓	✓	✓	✓	✓
Dynamic Formula	✓				
Wave Equation		✓	✓	✓	✓
CAPWAP Analysis			✓		✓
Static Load Test				✓	✓
Factor of Safety (FS)	3.50	2.75	2.25	2.00	1.90

2.4.2. Load and Resistance Factor Design (LRFD). Load and Resistance Factor Design (LRFD) is an alternative design method that has been progressively developed specifically for bridges since the mid-1980s. LRFD was well established in design codes around the world for structural engineering, but was first adopted in North America by the American Concrete Institute (ACI) Code in 1953 (DiMaggio et al. 1999). The objective of LRFD is to produce engineering designs with consistent levels of reliability using procedures from probability theory to ensure a prescribed margin of safety (Paikowsky, 2004).

Under LRFD, the uncertainties in loading are assessed separately from the uncertainties in resistance through a series of partial factors. These factors are known as load factors and resistance factors. The use of separate factors is a more rational approach than the use of a single FS (as in ASD) because loads and resistances have considerably separate and unrelated sources of uncertainty (Becker, 1996). For instance, the nominal loads of a structure are significantly influenced by the uncertainty related to estimating their magnitude; their influence has little impact on the uncertainty associated with evaluating the subsurface conditions that are providing resistance. Therefore, through LRFD, the design is not "penalized" for any uncertainties that pertain primarily to either the nominal load or the resistance (as it is in ASD).

Load factors, (typically those greater than 1) are used to account for the inherent uncertainties in determining the magnitude of the structural loads (dead load, live load, wind load, and so forth). In contrast, resistance factors (usually those less than 1) are used to account for the uncertainty in individual resistance components (e.g., shaft resistance and end bearing) caused by such factors as soil behavior during different modes of failure, model specifications, and variations in soil conditions (Yoon, 2011). The LRFD criteria is expressed by the following equation:

$$\Sigma(LF)Q_n \leq (RF)R_n \tag{2.6}$$

where LF is the load factors,

Q_n is the nominal loads,

RF is the resistance factor, and

R_n is the nominal resistance.

By applying the load factors and resistance factors, the engineer is, in effect, over-estimating the structure's loads and underestimating the structure's strength. The primary advantage of LRFD is that it allows a more consistent, uniform level of safety. This, in turn, produces a more economical, repetitive design.

AASHTO published the first edition of LRFD bridge specifications in 1994. This new LRFD specification contained comprehensive design and construction guidance for both structural and geotechnical features. Initial use of the new specification, however, revealed showed that the approach used in LRFD for bridge superstructures (structural engineering design) was not fully compatible with the needs of bridge substructures (geotechnical engineering design). The primary disadvantage stems from the uncertainties in external loads being relatively small when compared with the uncertainties in strength-deformation behaviors of soils (DiMaggio et al., 1999). As a result, many geotechnical engineers reverted back to the ASD method of designing foundations they were accustomed to using in the past.

When structural engineers used the LRFD method to design a bridge's superstructure, engineers struggled when designing the substructure with ASD because the critical load conditions were defined differently for the two procedures (Goble, 1996). Implementing different design methods for superstructures and substructures not only created uneconomical designs but also decreased the reliability of the designs that were constructed.

To ensure consistency between design methods, AASHTO and the Federal Highway Administration (FHWA) together issued a policy memorandum requiring all new bridges initiated after October 1, 2007 to be designed using the LRFD approach (Densmore, 2000). Resistance factors included in the LRFD specifications were calibrated using the FHWA developed Deep Foundation Load Test Database (DFLTD). The DFLTD consists of load test data for 1307 deep foundations collected between the years of 1985 and 2003 from all over the world. Following the mandate, concern rose that the nationally developed resistance factors were overly conservative when applied to localized regions because of the variability in not only the geology but also the construction practices used to calibrate them. For this reason, AASHTO permitted state Departments of Transportation (DOTs) to develop their own resistance factors based on regional practices and geology to minimize the unnecessary conservatism built into a design.

2.5. VARIOUS STATES LRFD IMPLENTATION EFFORTS

Following the release of the first edition of LRFD Bridge Specifications (1994) multiple state DOTs, including Florida, Pennsylvania, and Washington, began aggressively developing plans to fully implement LRFD.

Following the imposed October 1, 2007 deadline, a number of surveys were conducted to determine the extent of LRFD state DOTs had implemented in bridge foundation design. AbdelSalam (2010) found that approximately 52% of the respondents were fully implementing LRFD, 33% were in a transition stage from ASD to LRFD, and the remaining 15% were still using ASD with FS between 2 and 2.5. Many of the states either implementing LRFD or in transition from ASD to LRFD initiated research programs to develop their own regionally calibrated LRFD resistance factors for foundation designs. Florida, Illinois, Louisiana, Wisconsin, and Iowa each published notable studies recommending LRFD resistance factors for driven pile foundations. The following sections will briefly summarize select efforts of multiple state DOTs to develop resistance factors for use within their respective states. Figure 2.1 illustrates the implementation status of each state as determined by AbdelSalam (2010).

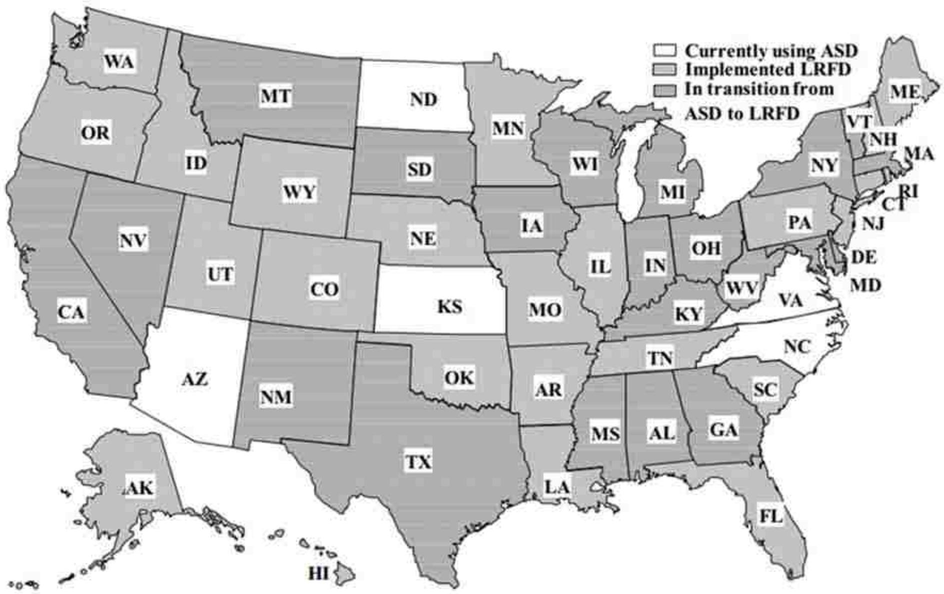


Figure 2.1 Extent of LRFD Implementation Following Oct. 1, 2007 Deadline (AbdelSalam, 2010)

2.5.1. Florida. The Florida Department of Transportation (FDOT) began training its engineers to incorporate LRFD after the original specification became available in 1994. Like most state DOTs, Florida recognized the over-conservatism built into the AASHTO recommended resistance factor. Resistance factors, however, were not included in AASHTO specifications for the common pile design software used by FDOT. Thus, FDOT was particularly interested in developing resistance factors based on the common geotechnical practices currently used in that state. In 1995, FDOT presented a plan to implement LRFD through the state's specifications by 1998. FDOT outlined the process to fully implement LRFD specifications in the following steps:

1. Convert all design documents to LRFD
2. Modify all software to reflect LRFD environments
3. Calibrate geotechnical resistance factors for Florida foundations.

Both FDOT and the University of Florida (UF) used a series of pile load test databases progressively developed at UF since 1989 to calibrate geotechnical resistance factors for use in the state of Florida. The UF pile load test database for driven piles, entitled PILEUF, included data collected from over 72 different sites and more than 180 different tests (both End-of Drive and Beginning of Restrike) conducted across Florida (McVay, 2000).

FDOT recently initiated several research efforts focused on calibrating resistance factors for new foundations types. FDOT plans to continuously adjust and refine the calibrated resistance factors as more data becomes available. McVay et al. (2000) presented detailed information on this study, including pile data, statistical analysis, and the development of resistance factors.

2.5.2. Illinois. Previously, the Illinois Department of Transportation (IDOT) estimated pile lengths using static analysis methods. The final pile length, however, was determined with a dynamic formula that was based on the pile driving resistance as determined in the field (Long et al, 2009a). Using separate methods to establish the design and acceptance criteria often resulted in a significant difference between the estimated lengths and actual pile lengths installed. For this reason, the Illinois Center of Transportation (ICOT) performed a study to evaluate IDOTs methods for predicting pile resistance and length. The objective of this research was to define the abilities of each predictive method, provide improvement if possible, and develop a calibrated series of resistance factors for the most reliable methods to be used in IDOT's LRFD specifications.

ICOT developed and analyzed three separate databases of driven pile data to quantify the agreement between evaluated methods (Long et al, 2009). These databases included the International Database (a composite database of pile data used in several different studies), the Comprehensive Database (a database of 26 static pile load test records), and the IDOT Database (a database of piles only driven by IDOT). The analysis was used to not only identify but also correct the most accurate predicative methods for predicting pile resistance, including: combinations of static methods and dynamic formulas, pile type, and soil type. Findings from this study resulted in a series of LRFD resistance factors developed for the most reliable predicative methods. For detailed information of this study, including pile data, statistical analysis, and the development of resistance factors, refer to Long et al. (2009a).

2.5.3. Louisiana. The Louisiana Department of Transportation and Development (LADOTD) began considering the use of LRFD specifications in 1995 but did not fully implement the method until 2005 (Yoon et al, 2008). Initially, LADOTD began using LRFD on select local projects by applying the national resistance factors suggested by AASHTO. As the familiarity and confidence in using LRFD increased, both LADOTD and the Louisiana Transportation Research Center (LTRC) initiated a research effort to calibrate regional geotechnical resistance factors for driven piles. This effort consisted of an extensive search of historical pile load test records collected within Louisiana. The search itself was limited to the installation records of containing both adequate subsurface information and a static load test performed to failure. The results of the search yielded 42 pile load tests that met these criteria. The soil boring information, pile driving logs, dynamic testing and analysis, static load test results were organized into a driven pile database. Using the collected data, LADOTD developed a series of resistance factors for various static and dynamic methods to be used within Louisiana. The resulting LADOTD resistance factors were 25 to 60 percent greater than the AASHTO recommended resistance factors, with an equivalent factor of safety at approximately 2.6 for the static methods analyzed.

As a result of their research program, LADOTD has currently initiated a major effort to not only write a geotechnical design manual but also rewrite the 2006 Louisiana Standard Specification for Roads and Bridges. In the future, LADOTD intends to continue improving their LRFD design and calibration for various methods and tests. They also hope to improve the state's code to account for the new methods of contracting, construction, and ownership needed to properly implement LRFD. For detailed information, including the various static methods considered, statistical characterization performed, and LRFD resistance factors developed, refer to Yoon et al. (2008).

2.5.4. Wisconsin. In the past, the Wisconsin Department of Transportation (WisDOT) often drove piling in the field based on the Engineering News (EN) dynamic formula. The Federal Highway Administration (FHWA), however, has encouraged state

DOTs to migrate away from the EN Formula and toward a more accurate dynamic formula known as the FHWA-modified Gates formula (Long et al., 2009b). As a result, the University of Illinois initiated a study through the Wisconsin Highway Research Program to assess the use of both the Gates formula and other dynamic formulas in WisDOT practice.

Several datasets were collected and organized into two databases to provide a quantitative comparison of the predictive methods. The first database contained data from several smaller load test databases collected from various locations across the United States. The dataset collected for the nationwide database was limited to historical installation records of h-piles, pipe piles, and metal shell piles. It included static pile load test data and provided sufficient information to predict pile resistance using various dynamic formulae (if dynamic analysis was not already provided). A total of 156 records were compiled within this database.

The second database was created from the installation records of 316 piles driven exclusively by WisDOT. In some cases, CAPWAP (BOR) predictions were available. Very few records, however, included static pile load test data. At a minimum, each installation record included in this database was required to include the appropriate data needed to estimate the nominal resistance from simplistic dynamic formulas.

These program findings resulted in a new series of resistance factors for three commonly used WisDOT dynamic formulas. These new factors exceeded the values provided in the AASHTO (2010) specification by between 20 and 50 percent. For detailed information of this study, including the pile datasets, statistical analyses, and resulting resistance factors, refer to Long et al. (2009b).

2.5.5. Iowa. Historically, the Iowa Department of Transportation (IowaDOT) has aggressively collected static pile load test data. According to Roling et al. (2011), this data includes information from 264 pile static load tests conducted over a 24 year period (between 1966 and 1989) on steel H-piles, timber, pipe, monotone, and concrete piles. In 2005 IowaDOT and Iowa State University conducted a joint research project directed at the development of LRFD procedures for driven piles in IowaDOT bridges. This study focused on creating an electronic database of the historical IowaDOT pile load tests data to allow for the calibration of LRFD regional resistance factors.

The electronic database PIle-LOad Tests (PILOT) was developed using Microsoft Access™ to organize the available IowaDOT static load tests records. Currently, PILOT contains 274 records of static pile load tests, varying in pile type and geological conditions, performed in Iowa. Researchers at Iowa State University surveyed both different state DOTs and Iowa county engineers to identify the most common, well-performing dynamic pile driving formulas. They then calibrated geotechnical resistance factors according to their response using the information available in PILOT. In all cases, the new series of calibrated resistance factors either equaled or exceeded the resistance factors recommended in the AASHTO (2010) specifications.

This compilation of available data into an electronic database allows IowaDOT designers and researchers the opportunity to access not only the quality but also the quantity of data needed for the accurate, effective calibration of regional LRFD resistance factors. For detailed information of both the methods evaluated and the determined results in this study, refer to AbdelSalam et al. (2008) and Roling et al. (2011).

2.6. MISSOURI LRFD IMPLEMENTATION EFFORTS

MoDOT adopted the national resistance factors found in the AASHTO LRFD Bridge Design Specifications Manual (2007) to design bridge foundations according to the FHWA mandate imposed in 2007. These specifications allow state DOTs to develop resistance factors based on their own regional practices and geology. To take advantage of this provision, MoDOT initialed its first research project to optimize design from both an economic and safety point of view.

2.6.1. Former Research Projects. In 2008, researchers from both Missouri University of Science and Technology (Missouri S&T) and the University of Missouri (Columbia) began the first MoDOT supported research program to develop a series of regional resistance factors for use within the state. These researchers used existing data from historical construction records on dynamic pile testing (i.e., Pile Driver Analyzer [PDA] and CAse Pile Wave Analysis Program [CAPWAP] software) to develop a new set of resistance factors for the static methods used by MoDOT. These factors were to be based on the various geologic regions within Missouri. Following the project's completion in 2010, the newly calibrated set of resistance factors suggested that the AASHTO recommended resistance factors should be increased. The resulting resistance factors are given in Table 2.2 (Kebede, 2010).

These results do suggest the AASHTO recommended resistance factors for static methods are overly conservative for use in Missouri. Static pile load test data was not used, however, to evaluate the actual nominal resistance. This newly calibrated set of resistance factors were thus established under the strict assumption that dynamic testing methods provide the actual nominal resistance values.

For this reason, a subsequent research effort was initiated to locate historical pile load test data from MoDOT's records and not only establish a database for adjusting the newly developed resistance factors but also calibrate new resistance factors for other prediction methods. As this project progressed, the majority of the data located was PDA and/or CAPWAP results of dynamic testing, with a limited number of records containing PLT data.

Particularly, the PLT data that was available was not representative of MoDOT's current methods and pile types used in practice. Furthermore, the dynamic testing data did not include any corresponding results from other predictive methods performed for the test piles. Therefore, a comparison between predicted resistances and measured pile resistance from dynamic testing could not be performed (Cravens, 2011). As a result, researchers could not establish a database for the calibration of resistance factors.

Subsequently, a questionnaire was distributed to neighboring state DOTs through a questionnaire to better understand their practices and locate available pile load test data for use in calibration. Although different states have different geologies, these neighboring states have somewhat similar geologic conditions. Thus data obtained from the surrounding states could be matched to the appropriate geologic regions in Missouri according to similar soil and rock formations. Although PLT data would not be directly related to MoDOT's local practices, the calibration of resistance factors based on surrounding state's PLT data would be at least more representative of Missouri's local conditions than the resistance factors provided by AASHTO (Cravens, 2011).

Table 2.2 Suggested Geotechnical Resistance Factors (adapted from Kebede, 2010)

Geological Region	Pile Type	Design Method	Resistance Factor Total		
			β = 2.33	β = 2.5	β = 3.0
Southeastern Lowland	Steel Pipe	Nordlund	0.55	0.53	0.45
		Meyerhof	0.43	0.40	0.33
		Beta	0.57	0.54	0.47
	H-Pile	Nordlund	0.71	0.69	0.61
		Meyerhof	0.58	0.55	0.45
		Beta	0.75	0.72	0.63
Glacial Plains	Steel Pipe	Nordlund	0.65	0.62	0.65
		Meyerhof	0.63	0.60	0.53
		Beta	0.68	0.66	0.58
	H-Pile	Nordlund	0.53	0.50	0.43
		Meyerhof	0.50	0.47	0.40
		Beta	0.77	0.66	0.56

The request for information included:

- common pile types used in practice
- common predictive methods used in practice
- pile installation procedures
- PLT data including:
  - installation procedures
  - results including measured loads and displacements
  - pile driving records,
  - subsurface conditions with laboratory testing
  - bridge plans with pile foundation plans and design capacities,
  - end-of-drive (EOD) and beginning-of-restrike (BOR) data associated with PLTs
  - PDA and/or CAPWAP dynamic testing data associated with PLTs

The results of the effort are summarized in Table 2.3. Table 2.3 reveals that responses to the questionnaire yielded few results, with only 4 of 8 states providing a response and only one state (Tennessee) providing PLT data. Although seven PLT records were received from Tennessee, 6 were not loaded to failure and only proof tested to 200% of the design load. As a result, the actual nominal resistance of the piles was not determined, and the records were not useful for input into the Missouri database.

2.6.2. Current Research Project. Although MoDOT has performed PLTs in the past, these PLTs were not implemented with research objectives in mind and are not commonly implemented into current practice. For MoDOT to benefit from the advantages LRFD offers, research grade PLT data based on MoDOT's current practices needs to be developed.

To address this need, MoDOT issued a two-phase research program entitled "Evaluation of Pile Load Tests for use in Missouri LRFD Guidelines." The initial phase (Phase I) consists of conducting a series of pile load tests at three construction bridge sites along the Missouri highway system within specific geologic regions. The nominal resistance of the test pile from each test is to be determined through both dynamic and static load test methods. Furthering the previous effort to collect both recent and available PLT data from Missouri's neighboring states will also be included as part of this initial phase. A potential future phase (Phase II) will use the data sets collected in Phase I, additional PLT in other geologic regions in Missouri, and any available PLT data in neighboring states to calibrate a series of the resistance factors for use in the Missouri LRFD guidelines. The remainder of this document will discuss only the activities completed as part of Phase I.

Table 2.3 Results of Neighboring State Questionnaires (adapted from Cravens, 2011)

Dashes (-) – no direct yes or no

Neighboring State	Response	LRFD Resistance Factors	Common Pile Type				Common Predictive Method				Perform PLT in Their State	Provided SLT Data
			H-Pile	Concrete	CIP	Timber	Static Method	Dynamic Formula	WEAP	Dynamic Testing		
Arkansas	YES	AASHTO Recommended	YES	YES	YES	NO	-	ENR	YES	PDA CAPWAP	NO	NO
Oklahoma	NO										YES	
Kansas	NO										NO	
Nebraska	YES	AASHTO Recommended	YES	NO	YES	NO	DRIVEN	ENR	-	PDA CAPWAP	NO	NO
Iowa	NO										YES	
Illinois	YES	-	-	-	-	-	-	-	-	-	YES	NO
Kentucky	NO										YES	
Tennessee	YES	AASHTO Recommended	YES	YES	YES	NO	-	NO	NO	NO	YES	YES

3. MISSOURI'S STATE OF PRACTICE

3.1. BACKGROUND

In the past, MoDOT reduced the estimated ultimate capacity of piles by a prescribed factor of safety (FS) to obtain the allowable loads of the structure for design. Although this approach was straightforward and coincided well with ASD methodologies, the resultant design loads often led to conservative values. In 2007, MoDOT adopted the national resistance factors from the AASHTO LRFD Bridge Design Specifications Manual (2007) to design bridge foundations within the state. The following sections will discuss both MoDOT's current state-of-practice and the various geologic conditions found in Missouri.

3.2. MODOT's STATE OF PRACTICE

The standard specifications and practices followed by MoDOT are compiled in their publically available Engineering Policy Guide (EPG) (2013). Category 700 of the EPG outlines the standard specifications for bridges constructed in Missouri. Category 751 summarizes MoDOT's LRFD Bridge Design Guidelines. From the EPG, "Once the need for a bridge has been identified a team [of engineers] is established to develop the scope of the project, submit a bridge survey, and begin the preliminary design" (MoDOT, 2013).

Of the nearly 10,000 bridges encompassed within Missouri's state highway system, driven piles are the most commonly used foundation systems (MoDOT, 2013). MoDOT's design procedure for driven piles is outlined in Section 751.36.3 of the EPG. A flow chart of this process is interpreted in Figure 3.1.

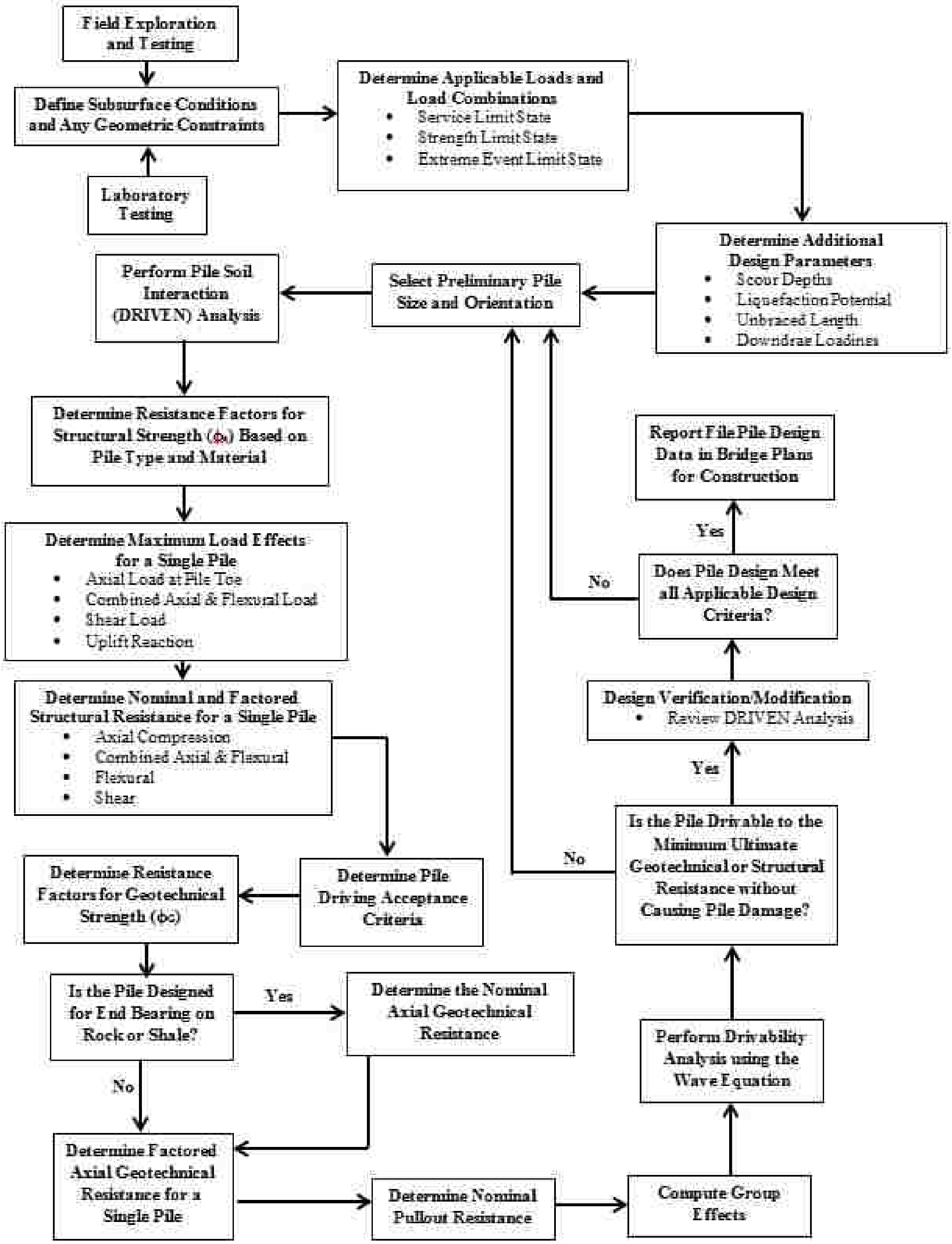


Figure 3.1 Interpreted Flow Chart of MoDOT Pile Design Process (based on MoDOT, 2013)

3.2.1. Pile Types. MoDOT typically uses both structural steel H-section piles and cast-in-place (CIP) concrete piles. H-section piles are the most widely used pile type in the state of Missouri. Typical section sizes include HP10x42, HP12x53, and HP14x73 (MoDOT, 2013). If difficult driving conditions are expected pile shoes (also referred to as points) are usually specified for reinforcement. When CIP piles are specified, typical pile sizes include 14- and 16-inch diameter steel shells with wall thicknesses (a minimum) of 0.25 and 0.375 inches, respectively.

Bridges in Missouri may contain varying pile sizes or types from bent to bent. MoDOT, however, requires that the same size and type be used for the same bent. In general, MoDOT uses H-section piles as end-bearing piles that will be driven to bedrock; they use CIP piles as friction piles when the bedrock is located at great depths.

3.2.2. Static Methods. Once the preliminary pile type, size, and orientation has been determined, MoDOT uses the FHWA provided software DRIVEN as its primary analytical method for design. When bedrock is located at great depths, DRIVEN is always used to estimate both pile length and the pile resistance for friction piles. However, when end-bearing piles are to be used, DRIVEN is used only to estimate pile length in one of two situations:

1. When depths to bedrock exceed 45 feet. (MoDOT typically always uses end-bearing piles when the depth to bedrock is equal to or less than 45 ft. [Cravens 2011].)
2. When the subsurface above bedrock depths contain glacial till or similar layers. (DRIVEN is used to determine if pile resistance can be reached at a higher elevation due the increase is skin friction these materials provide.)

3.2.3. Pile Structural Resistance Factors. The MoDOT EPG (2013) presents structural resistance factors (for the selected pile type) based on the expected driving conditions at a site. Table 3.1 summarizes the resistance factor for pile structural strength as presented in the MoDOT EPG (2013). Note that MoDOT indicates that the use of pile point reinforcement is necessary for severe driving conditions, whereas it is not for good driving conditions; the inclusion or absence of reinforcement tips has been considered in the specified resistance factor for each condition.

Table 3.1 MoDOT Pile Structural Resistance Factors

<table>
<tr><th colspan="2" rowspan="2">Resistance Condition</th><th colspan="2">Resistance Factors for Structural Strength (ϕ_S) per Pile Type</th></tr>
<tr><th>Steel Shell</th><th>H-Piles</th></tr>
<tr><td colspan="2">Axial Resistance in Compression Subject to Damage Due to Severe Driving Conditions</td><td>0.6</td><td>0.5</td></tr>
<tr><td colspan="2">Axial Resistance Compression Under Good Driving Conditions</td><td>0.7</td><td>0.6</td></tr>
<tr><td rowspan="2">Combined Axial and Flexural Resistance of Undamaged Piles</td><td>Axial</td><td>0.8</td><td>0.7</td></tr>
<tr><td>Flexural</td><td>1.0</td><td>1.0</td></tr>
</table>

3.2.4. Geotechnical Resistance Factors. In the EPG (2013), MoDOT specifies the use of the FHWA-Modified Gates Equation to calculate the nominal axial resistance of a pile for design (unless another method is specified in the contracts). The resistance factor used to compute the factored geotechnical resistance is determined from the pile driving acceptance criteria used during construction. Table 3.2 lists the geotechnical resistance factors MoDOT adopted from AASHTO (2010) for each resistance condition.

Table 3.2 MoDOT Geotechnical Resistance Factors

Resistance Condition	Resistance Factors for Geotechnical Strength (ϕ_G)
FHWA Modified Gates Formula	0.40
Dynamic Testing on 1 to 10% of Production Piles	0.65
Other Methods	Refer to AASHTO (2010)

3.2.5. Special Provisions. Special provisions are included within a project's contract documents to define work/procedures that are not specifically covered in MoDOT's standard specifications. These special provisions are also used to either supplement or modify items within the standard specifications when unique items are not adequately explained on the construction plans or in the EPG. MoDOT commonly includes the specific requirements and procedures for both dynamic pile testing and static pile load tests in special provision documents provided to the contractor. The following sections will describe these items, in general, as they would be outlined in special provisions documents.

3.2.5.1 Dynamic testing. MoDOT requires the contractor to conduct High-Strain Dynamic Testing of piles in accordance with ASTM D 4945 (ASTM, 2008). The products approved by MoDOT for use in the various requirements of dynamic pile testing are listed in Table 3.3.

Table 3.3 MoDOT Approved Manufacturers and Products for Dynamic Pile Testing

Component	Product[a]
Pile Driving Modeling – Wave Equation Software	GRL WEAP
Pile Driving Monitoring – Hardware and Software	Pile Driving Analyzer Model PAK
Pile Driving Analysis – Signal Matching Software	CAPWAP

a. Each product listed is manufactured by Pile Dynamics, Inc.

Prior to construction, the contractor (typically an independent consultant hired by the primary contractor) must perform a wave equation analysis (using GRLWEAP) to define the performance for the proposed driving system pile, hammer, and cushion within the anticipated subsurface conditions. During pile driving, the consultant must use the PDA to not only monitor but also process the data while in field. MoDOT requires that piles be driven until both the specified tip elevation and the nominal pile resistance are reached unless the monitoring indicates additional driving will cause damage to the pile (MoDOT, 2013). CAPWAP signal matching is required for each pile tested at the end of driving (EOD) to determine the distribution of resistance from end bearing and skin friction. MoDOT requires restrike tests to be performed after initial EOD on select projects. As a default, a value of 7 days is used. However, this value is adjusted in accordance with AASHTO LRFD Bridge Construction Specification (2010) based on the subsurface materials at a site. Table 3.4 illustrates the minimum restrike durations typically used by MoDOT.

Table 3.4 Minimum Restrike Durations Based on Subsurface Materials (AASHTO, 2010)

Soil Type	Time Delay Until Restrike
Clean Sands	1 Day
Silty Sands	2 Days
Sandy Silts	3-5 Days
Silts and Clays	7-14 Days*
Shales	7 Days

*Longer delay times may be required

During the beginning of restrike (BOR), the pile must be instrumented and monitored in the same manner as it was at EOD. MoDOT requires dynamic testing be performed on a minimum of one production pile for each bent of the proposed structure.

3.2.5.2 Static Pile Load Test (PLT). MoDOT typically specifies that PLTs should be performed only on structures that have an unusually large number of piles. In this case, the primary purpose of load testing is to check the effectiveness of the dynamic pile driving formula or calibrate the pile hammer with the selected dynamic pile formula (MoDOT, 2013). In general, when a PLT is specified, the contractor is required to not only select but also present a proposal of the PLT procedures and arrangement following ASTM D 1143 (2007) for use. This selection, however, must be approved by MoDOT. Once both have been accepted, special provisions regarding the load increments, application intervals, maximum load, failure criteria, and so forth. are established by MoDOT.

3.3. GEOLOGY IN MISSOURI

MoDOT's construction practices vary depending on the geologic region of the bridge site. For this reason, the following sections will describe the various geologic regions in Missouri. Specific details of each of the tests performed in Phase I are discussed in their respective Subsurface Conditions sections in Section 5.

Missouri can be divided roughly into four regions. These four regions, characterized by soil type, topography, and geologic features, include the Ozark Highlands, the Western Plains, the North Glaciated Plains, and the Southeast Lowlands. Figure 3.2 illustrates the general delineation of these geologic regions.

Figure 3.2 Missouri's Geologic Regions (Saville, 1962)

3.3.1. The Ozark Highlands. The Ozark Highlands (or simply Ozarks) cover, primarily, the central portion of Missouri south of the Missouri River, with the exception of the flatlands in the west and the Bootheel section in the southeast. The Ozarks, one of the less populated areas of the state, is characterized by rough topography, thick forests, and meandering streams. Karst topography (i.e., caves and sinkholes) is found more often in the Ozarks than in any other region in the state. The bedrock in this region consists of Ordovician, Cambrian, and Pennsylvanian age dolomites interbedded with layers of sandstone (Saville, 1962). Some of the most common formations in the Ozarks include the Roubidoux sandstone and Jefferson City dolomite formations (Hayes, 1961). These formations are usually located at shallow depths and are often exposed. Decomposition of the bedrock materials produces predominantly chert residual soils. Some portions of the Ozarks containing larger quantities of sandstone decompose to modify the residual soils. The modified residual soils form some characteristic sandy soils, but these areas are restricted at most. Other isolated areas within the region encompass high plastic red clay consistent with liquid limits near 100.

3.3.2. The Western Plains. The Western Plains region of Missouri is relatively the most level part of the state. This geologic region includes the portion of the state below the Missouri River and east of the Kansas state line. The bedrock consists of Mississippian aged sedimentary formations, such as the Osagean Series and Meramecian Series, and Pennsylvanian aged cherty limestones with shale materials from both the Missourian and Desmoinesian Series (Hayes, 1961). These formations are generally located at shallows depths. Karst topography is a common feature in the Western Plains region as well. Decomposition of the Mississippian bedrock materials provides, primarily, silty to gravelly loam residual soils. Soils formed from the Pennsylvanian aged constituents are usually are higher in clay content.

3.3.3. The Glaciated Plains. The Glaciated Plains region of Missouri extends north of the Missouri River to the Iowa state line. This area was covered by glacial ice during both Nebraskan and Kansan ages of glaciation (Hayes, 1961). The bedrock in this area contains formations similar to that of the unglaciated Western Plains. Much of the Glaciated Plains bedrock, however, is located at great depths (Saville, 1962). A thick heterogeneous mixture of glacial till (e.g., sand, clay, rocks, and boulders) was deposited as the glaciers moved. As a result, these glacial deposits are heavily over-consolidated, varying greatly in both composition and particle size. In general, the glacial till soils can be described as very dark gray to yellow (depending on the level of oxidation) silty clay that contains localized collections of cobbles and boulders (Hayes, 1961). Sand lenses are also common throughout the till soils.

3.3.4. The Southeast Lowlands. The Southeast Lowlands region occupies, primarily, the Bootheel area of the state. Delineated by the Ozark Highlands region to the west, this area consists of relatively flat topography. The bedrock, located at great depths, is, primarily, dolomite and sandstone of Quaternary, Tertiary, and Cretaceous formations (Saville, 1962). The soils across this region are comprised, mostly, of alluvial deposits. More specifically, they consist of a mixture of either clay or silt underlain by thick deposits of sand with varying amounts of gravel.

4. PILE LOAD TEST PROGRAM METHODS

4.1. INTRODUCTION

The pile load test program was designed to evaluate the actual nominal resistance of a driven pile. Both the test equipment and the instrumentation were thus selected according to this principle. The following sections provide a summary of the load applying system, instrumentation, data acquisition system, loading procedure, and data reduction procedures of the pile load test program. More specific details regarding the aspects of each load test are discussed in Section 5.

4.2. TEST EQUIPMENT

The primary aspects of the pile load test equipment consist of:

- Load application arrangement
- Instruments used to measure the applied load, the resulting pile head displacements, and the strains within the pile.

The following sections will discuss these items separately.

4.2.1. Load Frame Design. Both a steel reaction load frame and a hydraulic jack were used to apply an axial compressive load to the test pile. The reaction frame used in each PLT was designed as part of a collaborative effort between the MoDOT structural bridge engineer of each project and the Missouri S&T researchers. The load frame used in each PLT was consistent with the description provided in ASTM D1143, Section 6.3 for an anchored reaction frame. This frame consisted of four anchor piles spaced laterally no less than 8 pile diameters from the test pile. The reaction frame was designed for 1.5 times the maximum anticipated resistance of the test pile.

The anticipated resistance of the test pile varied from site to site. For convenience, the piles for the load frame were designed to use same pile types specified for the production piles of the actual structure. The reaction frame’s final design was included in the bridge plans that were provided to the contractor. The design used in each PLT is included in the select bridge plans that are provided in Appendix A.

4.2.2. Load Frame Construction. Load frame construction began with the installation of reaction anchor piles. As a result, any influences the installation of these anchor piles may have had on the subsurface were captured in the data collected when the test pile was installed. Next, a W36x182 reaction beam was placed on top of the anchor piles. This beam was made secure by placing cross-beam members on top of the reaction beam and then connecting those members to the reaction piles with a series of threaded dywidag bars, thin bearing plates, and steel nuts. Once these connections were established, the entire frame was rigid and secured.

4.2.3. Load Application and Measurement. With the load frame constructed, a one-inch thick steel bearing plate was welded to the head of the pile. This plate allowed the applied load to be evenly distributed over the entire cross-sectional area of the test pile. A 400 kip hydraulic jack was placed (centrally) on top of the bearing plate. A steel swivel was then placed on top of the jack to eliminate eccentric loading that would occur as the result of any misalignment incorporated in the reaction frame after construction; a calibrated 500 kip load cell was placed on the swivel.

The additional space between the top of the load cell and the bottom of the reaction beam was filled with steel plates, ensuring the hydraulic jack provided sufficient travel for the anticipated displacements/deflections (e.g., settlement of the pile, deflection of the reaction beam, and elongation of the connection anchoring devices). The load was applied through the hydraulic jack using a manual hand pump; it was electronically measured with the calibrated load cell. Figure 4.1 illustrates the various components of the load frame, labeled for clarification.

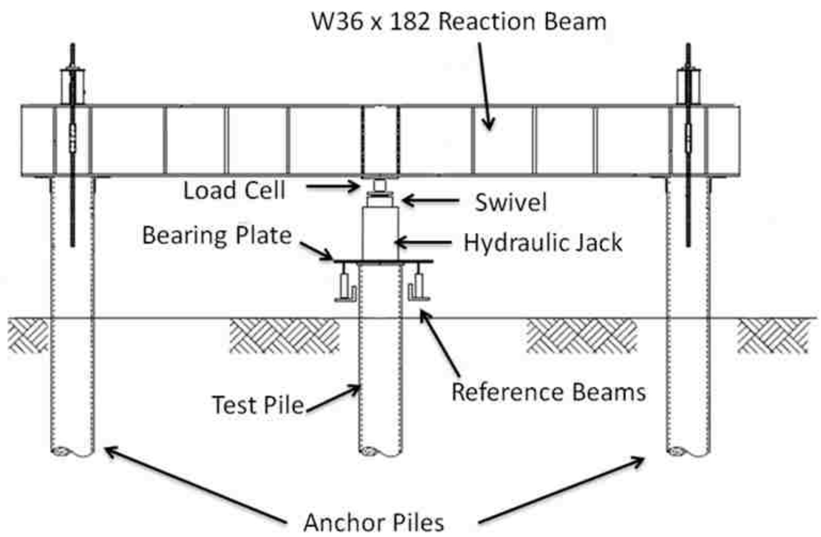


Figure 4.1 Diagram of the Pile Load Test Components (Not to Scale)

4.3. SUPPORTING INSTRUMENTATION

In conjunction with the applied load, both measurements of displacement at the pile head and changes in strain along the test pile were collected. These measurements are required for all pile load tests. Incremental strain measurements used to determine the distribution of load transfer with depth, however, are typically viewed as optional (Prakash, 1990).

Various instruments were incorporated into the PLT program to measure the applied load, axial movement of the pile head, and incremental strain measurements along the pile length. The following sections discuss the instrumentation used to measure these conditions.

4.3.1. Applied Load. The applied load was measured with a 400 kip load cell. Prior to use in the field, this load cell was calibrated with an MTS System test frame located at the Missouri S&T high-bay laboratory. Its use allowed the force applied to the test pile (by the hydraulic jack) to be converted into an electronic signal. This electronic signal could then be recorded by a data acquisition system (DAS). Section 4.3 provides an explanation of the DAS used in this project.

4.3.2. Pile Head Displacement. Two linear variable differential transformers (LVDT) were used to record the pile's displacement during loading. LVDTs are a common type of electromechanical transducer that can convert the linear motion of an object (in which it is coupled to) into a corresponding electrical charge. The LVDTs used during each test have the capabilities to measure displacements as small as thousandths of an inch and as large as 4 inches. They were mounted to two independently supported reference beams, using a series of magnets and connecting hardware, as shown in Figure 4.2.

Figure 4.2 Orientation of LVDT When Mounted to the Reference Beam

The reference beams were placed such that one was located on each side of the test pile and perpendicular to the reaction beam. The concrete blocks used to support the reference beams were located approximately 8 feet away from the test pile to ensure that settlement of the pile did not influence displacement readings of the LVDTs. Figure 4.3 shows the orientation of the reference beams with respect to the load frame.

Figure 4.3 Orientation of Reference Beams With Respect to Load Frame

4.3.3. Incremental Strain. Each of the test piles were instrumented with between five and six vibrating wire strain gages (VWSG) during installation. These gages were located such that one was near the pile head and one was near the pile toe. The remaining gages were spaced in equal intervals either throughout the rest of the pile length or near locations of anticipated change in stratigraphy. VWSGs were used for this project for their durability during installation. Additionally, the wire length of VWSGs does not influence the gage's signal response. These gages were used to obtain strain measurements along the length of the pile. The measurements themselves can later be converted into load readings during the data reduction. The ensuing load readings were used to determine how much of the pile's load was carried separately through both shaft resistance and tip resistance. The VWSG model used in each PLT was specifically dependent on the pile type tested.

4.3.3.1 Concrete embeddable VWSGs. Geokon Model 4200, concrete embeddable VWSGs were used in the PLTs that contained cast-in-place (CIP) test piles. These gages were tied at various locations along a steel centralizing bar that was lowered into the test pile before concrete placement. Figure 4.4a shows a CIP test pile as it is being instrumented with concrete embeddable VWSGs. These VWSGs were used in the A7956 PLT. A complete description of installation procedures is included in Section 5.1.5.

4.3.3.2 Weldable VWSGs. Geokon Model 4000, weldable VWSGs were used to instrument the H-section test pile of the A7669 PLT. These gages were welded along the pile's web and covered with a steel section for protection during installation. A complete description of the weldable VWSG installation process is provided in Section 5.2.5. "A7669 Test Pile Instrumentation." Figure 4.4b shows an H-section test pile being instrumented with weldable VWSGs.

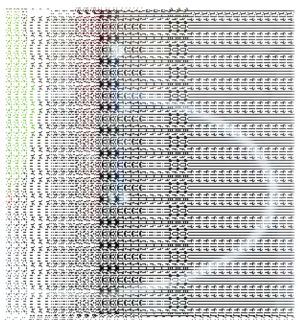
A)  B) 

Figure 4.4 The VWSGs Used to Measure Load Transfer Distribution. A) Concrete Embeddable (Geokon Model 4200) VWSG Installed in CIP Test Piles. B) Weldable (Geokon Model 4000) VWSG Installed on H-Section Test Pile.

4.3.4. Redundant Instrumentation. As previously mentioned, measurements of the applied load and the pile head displacement are required measurements of all pile load tests. Each of the instruments discussed thus far is an electronic device. Thus, these measurements were recorded with the electronic data acquisition system discussed in Section 4.3. In the event that any of the electronic components malfunctioned, a supplementary measuring system was established to double-check the data collected. The components of this system included both a mechanical dial gage and a calibrated pressure gage. The mechanical dial gage was mounted on the reference beams, similar to the LVDTs, to measure the pile's displacement. The pressure gage was located within the hydraulic lines (between the pump and the hydraulic jack). In the event the electronic system lost power, the applied load can be calculated from the pressure gage readings, and the corresponding displacement from the mechanical dial gage could be read.

4.4. DATA ACQUISITION SYSTEM

A data acquisition system provides an automated means of efficiently reading and recording data from installed instrumentation. Due to the variety of specialized instruments used within this project, implementing the use of such a system provided the advantage of being able to read and record data from all of the devices simultaneously. The data acquisition system used in this project resembled the system designed and built by Brian Swift, an electrical engineer for the Missouri S&T Civil Engineering Department, for a previous project (Kershaw, 2011). The following paragraphs discuss both the system requirements and components of the completed system used during this project.

4.4.1. System Requirements. The system's primary requirement was to be able to read and record data from several different instruments simultaneously. This capability allowed data to be obtained and stored in a far more efficient manner than a pen-and-paper method. It also reduced the possibility of human-error in the readings. The system needed to be portable. Because most of the sites within this project did not allow for vehicular access to the testing location, one person need to be able to carry the system. Due to the likelihood of electricity being unavailable at most test locations, the data acquisition system needed to supply its own power. Finally, the system needed to be user-friendly. (Kershaw, 2011)

4.4.2. Description of the Completed System. With the system requirements of the data acquisition system established, Swift completed both the electronic and the computer software design and began constructing the system (Kershaw, 2011). Based on the previous requirements, the CompactRIO platform, manufactured by National Instruments (NI), was selected as the basic platform in this data acquisition system. Once this basic platform was designed, the individual system components were selected according to the anticipated types and quantity of instrumentation being used. The basic components of the system included the controller, the chassis, device modules, software, housing, and peripherals.

The controller operates the data acquisition system. It has an internal CPU that can run software, execute commands from the software (i.e., turning devices on and off) log data received from the devices, and complete a basic processing of data (Kershaw, 2011). One of NI's high-performance, programmable controllers (the cRIO-9022) was selected for use within the system (National Instruments, 2010). In addition to connections between the chassis and the power source, the cRIO-9022 contained two Ethernet ports, one serial port, and one USB port. These ports provided additional connections for other devices (Kershaw, 2011). The USB port served as a backup for data storage in the event the controller itself malfunctioned unexpectedly.

The 8-slot, reconfigurable, embedded chassis (NI cRIO-9116) served as the housing that connected the proceeding modules to the controller. The device modules were instrument-specific cartridges that slid into the chassis. The specific cartridges selected were dependent on both the type and quantity of instrumentation being used. As previously mentioned, the data acquisition system for this load testing program was required to read vibrating wire strain gages (VWSG), LVDTs, and a load cell. Therefore following capabilities were compiled into the 8-slot chassis:

- 16 VWSG (6 slots),
- 4 load cells (1 slot), and
- 31 linear displacement devices (1 slot).

Note that each VWSG cartridge could accommodate four vibrating wire devices. However, for every pair of VWSG cartridges (8 devices) another cartridge was required to provide the excitation signal for the gages (Kershaw, 2011). Refer to Table 4.1 for the specific components used in the data acquisition box .

The data acquisition box was controlled by a laptop containing software developed from NI’s LabVIEW graphical programming tool. The user was able to monitor all instruments simultaneously, in real-time, by coupling the laptop to the controller using an Ethernet cable.

Table 4.1 Data Acquisition Components

Model Number	Image of Device	Device Description
NI 9022		Operates the data acquisition system
NI 9116		Houses the device modules
NI 9237 NI 9205		Controls the inputs and outputs of the peripherals connected to the 10-pin DCVT panel.
NI 9234 NI 9474		Controls the excitation and output of the VWSGs

The user interface (designed from the LabVIEW graphical tool) was designed for maximum flexibility. This flexibility supported a number of various functions including:

- Turn devices on and off,
- Begin and end data recording,
- Modify individual device's gage factors, and
- View data in real-time (numerically or graphically) (Kershaw, 2011).

Once the data was collected, the user specified through the laptop interface, whether the data was to be stored within the controller's hard drive, on the laptop's hard drive, or on a USB device connected to the system's controller. Multiple data storage locations were built into the system to provide redundancy in the event a component malfunctioned (Kershaw, 2011).

A series of additional components was added to the data acquisition box to make the system easier to use in the field. An AC to DC power converter was added so that the system could use 120 to 240 volt supplies from either typical outlets or generators (Kershaw, 2011). Power conditioners were also added to the system to produce a constant power flow to the controller. A channel board was added to the carrying case's lid to hold a series of female, 10-pin connectors for the linear displacement devices. (These connectors are a standard connection for many of the instruments used within the Missouri S&T Civil Engineering Department.) Each 10-pin connector was labeled to a corresponding channel visible within the user interface. This coordination allows the user to monitor the response of each individual instrument by selecting the designated channel. Finally, two peripheral connection boxes were constructed to simplify the connection of the VWSGs.

With all of these components installed, the entire system weighed approximately 15 pounds and could thus be carried easily by a single person. Figure 4.5 is a photograph of the completed data acquisition system. The individual peripherals are labeled.

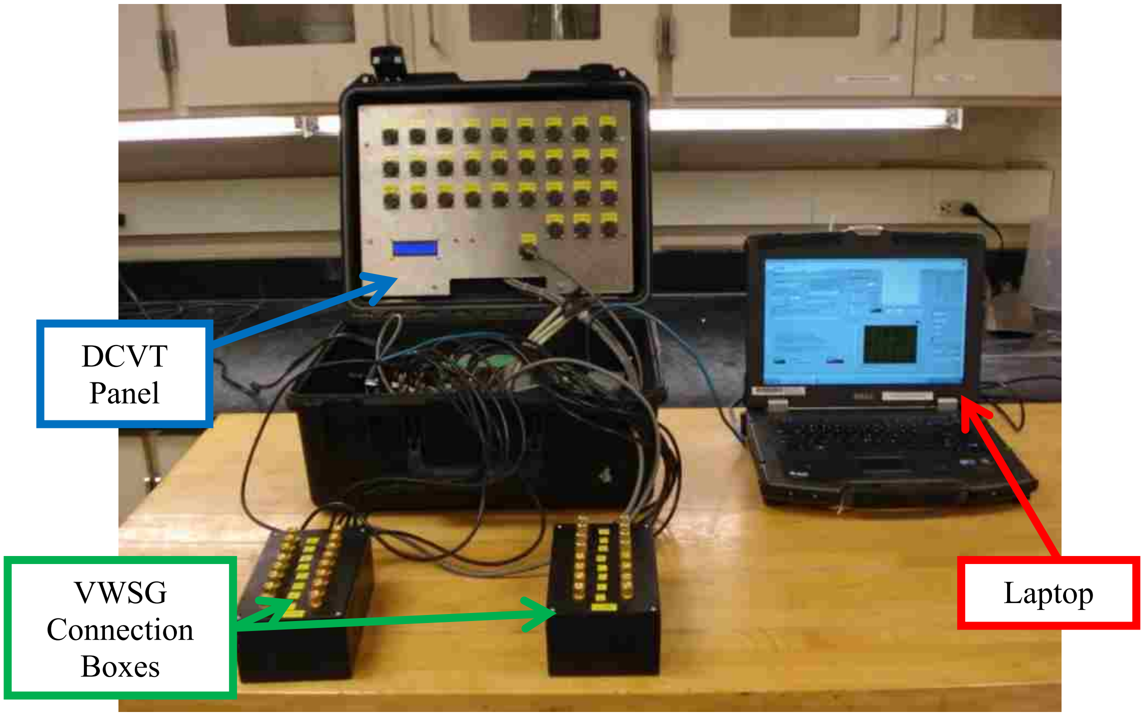


Figure 4.5 Data Acquisition System Peripherals

4.5. DYNAMIC MONITORING PROCEDURE

Prior to testing, two strain gauges and two accelerometers were mounted two pile diameters below the pile head. Geotechnology, Inc. (of St. Louis, Missouri) conducted dynamic monitoring as each test pile was installed. During the installation process, a driving record of the blows required to penetrate the pile each foot was completed. During testing, dynamic measurements of both strain and acceleration were recorded with a Pile Driving Analyzer (PDA) Model PAX (manufactured by Pile Dynamics, Inc). The PDA uses these measurements to calculate the transferred energy, the stresses (both compression and tension) induced in the pile, and the mobilized bearing resistance (with the maximum Case Method equations). The recorded force and velocity curves were viewed in real-time to evaluate pile integrity, data quality, and estimated resistance. Representative blows from the data collected by the PDA at the initial end-of-drive (EOD) and near the beginning-of-restrike (BOR) were analyzed with the Case Pile Wave Analysis Program (CAPWAP) signal matching software. Results from the dynamic monitoring conducted at each site are summarized in their respective "Dynamic Monitoring Results" in Section 5.

4.6. STATIC PILE LOAD TEST PROCEDURE

Table 4.2 displays the location within the data acquisition system where the instruments were connected to before testing.

Table 4.2 Instrument Connection Locations Within the DAS

Instrument	Locations Within DAS
LVDT	10-pin connectors on the case's lid
Load Cell	10-pin connector on the case's lid
Vibrating Wire Strain Gages	Peripheral custom connection boxes

During the actual tests, electronic measurements (i.e., readings from the load cell, LVDTs, and VWSGs) were continuously recorded and digitally stored by the data acquisition system; readings from the redundant instrumentation (the pressure gage and the mechanical dial gage) were recorded manually by Missouri S&T field personnel.

In general, loading was applied following the quick-maintained load test method (ASTM D 1143). The method, however, was modified to include three loading cycles consisting of 50%, 100%, and 200% of the allowable design load, instead of the simply a single 200% cycle. Conducting the loading procedure in this manner allowed for the pile's behavior to be monitored at different magnitudes of loading. It also helped ensure a quality dataset was obtained. When testing began, the load was added in increments of 12.5% by manually pumping the hand-pump until the digital readout connected to the load cell verified the corresponding applied load. Loads were held constant for approximately 5 to 10 minutes; the time held was dependent on the pile's ability to sustain the current load. After the holding period elapsed, the next loading increment was applied in a similar manner. Once the maximum cycle load was reached, the test pile was incrementally unloaded. Monitoring during the unloading portion of the cycle allowed for any rebound of the pile to be observed.

Subsequent cycles followed a similar procedure; these cycles varied only in magnitude of the loading increment and the holding time. The third cycle was loaded until the pile reached a plunge of approximately 1.5 - 2.0 inches.

4.7. DATA REDUCTION

The following is an overview of how the data was managed once it was obtained from the data acquisition system. As previously discussed, the data acquisition system simultaneously recorded data from the load cell, LVDTs, and vibrating wire strain gages. The data was then recorded as an .lvm (LabVIEW Measurement) file within the controller's hard drive, the laptop's hard drive, or the removable USB flash drive. Once located, the .lvm file can be opened and manipulated in Microsoft Office EXCEL™. In the file, the data recorded from each instrument was located in adjacent columns labeled with the respective channel number to which each instrument was coupled.

Both the load cell and the LVDTs were calibrated with the data acquisition system prior to testing (i.e., the voltage produced by each instrument is standardized to reflect the equivalent load (kips) and displacement (inches) measurements from the load cell and LVDTs, respectively, when received by the data acquisition system). As a result, the data from these instruments was available for immediate use. However, the output from the vibrating wire strain gages required some reduction before the desired parameters could be obtained from the readings.

VWSGs are designed to measure the strain between two points. This design is based on the theory that the frequency of a vibrating wire changes as the tension in the wire either increases or decreases. When the ends of these gages are secured, the encased wire connecting the two ends is plucked, and the resulting frequency is transmitted through the instrument cable to the data acquisition system. The data acquisition box then converts the frequency reading (currently in Hertz) to a microstrain reading based on the theoretical conversion:

$$\mu\varepsilon = G(\Delta f^2 * 10^{-3}) \qquad (4.1)$$

where $\mu\varepsilon$ is the microstrain,

G is the Gage Factor (see Table 4.3), and

Δf is the change in the wire's vibration frequency.

To determine the load transfer distribution during loading, the apparent changes in the microstrain that developed along the length of the pile as the applied load increased needed to be calculated. The equation used to calculate the apparent change in strain was:

$$\Delta\mu\varepsilon_{apparent} = B(\mu\varepsilon_i - \mu\varepsilon_0) \qquad (4.2)$$

where $\mu\varepsilon_i$ is the microstrain reading at any point in time

$\mu\varepsilon_0$ is the initial microstrain reading

B is the Batch factor per gage type (see Table 4.3).

It is important to note that because of the manner in which the VWSGs were constructed, the vibrating wire was shortened slightly causing the microstrain reading to be inflated. Therefore, to determine the actual apparent change in microstrain, a manufacturer-supplied batch factor for each gage type (see Table 4.3) was added to calculations to remove this effect and thus determine the apparent change in strain.

Table 4.3 Geokon VWSG Calibration Factors

Model	**4200**	**4000**
Theoretical Gage Factor	3.304	4.062
Typical Batch Factor	0.97 to 0.98	0.96

The apparent change in microstrain was then used to compute the load (P) in the test pile:

$$P = E * \Delta\mu\varepsilon_{apparent} * A \tag{4.3}$$

where E is the elastic modulus of the pile and
A is the cross-sectional area of the pile.

For test piles consisting of more than one material (e.g., concrete and steel shell of a CIP pile) transformed sections were used to calculate the cross-sectional area (A) of the pile. More specifically the concrete was transformed to an equivalent area of steel by multiplying the concrete area by the ratio of the elastic modulus of steel to the elastic modulus of concrete. It should be noted that the alternative of transforming the area of steel to an equivalent area of concrete would have yielded similar results. The transformed areas were calculated following:

$$A_{trans} = A_{shell} + A_{centerbar} + \frac{A_{concrete}}{\eta} \tag{4.4}$$

where η is equal to $\frac{E_{steel}}{E_{concrete}}$,
A_{shell} is the cross-sectional area of the steel shell,
$A_{centerbar}$ is the cross-sectional area of the steel center bar,
$A_{concrete}$ is the cross-sectional area of the concrete.

For test piles consisting of one material (e.g., steel, H-section piles) transformed sections were not required to calculate the cross-sectional area (A) of the pile.

5. RESULTS OF PILE LOAD TESTS

5.1. TESTING SITES

The site location of each pile load test (PLT) was selected based on MoDOT's most immediate needs by MoDOT. To that end, MoDOT identified three bridge projects along the Missouri highway system to be initiated in 2012. Due to the range of the subsurface conditions within Missouri, each test site was located in a different geologic region within the state. Figure 5.1 below shows the locations of each test with respect to Missouri's geologic regions discussed in Section 4. Although three PLTs were performed during Phase I of this project, the analysis of the PLT performed in Chillicothe was not completed at the time of this writing. Therefore only the results of the two PLTs performed in the southeast portion of Missouri (Sikeston and Poplar Bluff) are reported in this thesis. The following sections will summarize the results Sikeston and Poplar Bluff PLTs.

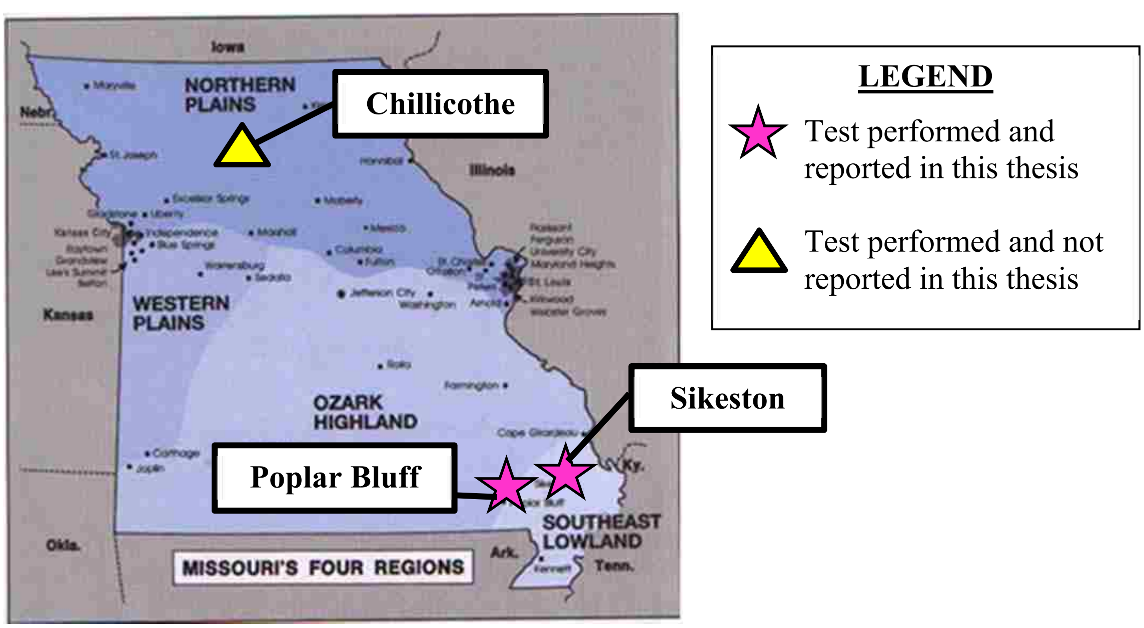


Figure 5.1 Static Pile Load Testing Locations

5.2. SIKESTON, MISSOURI

The first pile load test was conducted at the MoDOT A7956 bridge replacement site located approximately 12 miles north of Sikeston, Missouri, on State Hwy. 91. More specifically, the site was located 3 miles west of the intersection of Hwy. 61 and Hwy. 91 in Morley, Missouri. Figure 5.2 shows the approximate location of the construction site. (Latitude/Longitude: 37°02’18.93”N/89°40’40.98”W).

Figure 5.2 A7956 Site Location Map (Google Maps, 2013)

5.2.1. Site and Project Description. The existing structure consisted of a three span steel bridge crossing an irrigation drainage ditch and was completely demolished for the bridge replacement. The superstructure of the bridge included steel girders supported by driven H-pile foundations and timber abutments. The site was relatively flat, sloping slightly to the southwest. The site was contained by agricultural fields on all four sides and overhead utilities were located along the northern shoulder of the roadway throughout the length of the construction site. The testing location was positioned approximately 50 feet to the southwest of Bent 1 (within the MoDOT right-of-way). This particular location provided the closest available location to a characterized bent that would not conflict with regular construction activities and existing utilities. The contractor for the project was Chester Bross Construction Company (CBCC) of Hannibal, Missouri.

` The proposed structure was designed to support east-bound and west-bound traffic and consist of two lanes and three spans. Figure 5.3 shows a construction drawing of the proposed structure and select bridge plans are included in Appendix A.

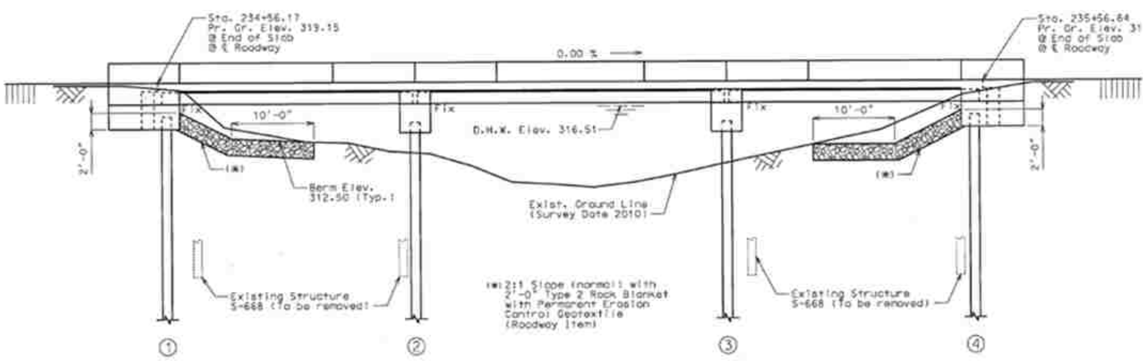


Figure 5.3 MoDOT Illustration of the Proposed Structure (MoDOT, 2013)

The new foundation system included 14-inch cast-in-place (CIP) piles in each bent, 50 to 60 feet in length. Other substructure components consisted of prestressed concrete box girder spans and precast prestressed concrete panels supported on concrete abutments. The foundation data of the proposed structure are shown in Table 5.1.

Table 5.1 A7956 Foundation Data (adapted from MoDOT Plans, 2013)

	Bent No.	**1**	**2**	**3**	**4**
Driven Pile	Pile Type and Size:	14” CIP	14” CIP	14” CIP	14” CIP
	Number:	5	6	6	5
	Approx. Length (ft):	50	60	60	50
	Minimum Nominal Axial Compressive Resistance (kip)	157	181	181	157

5.2.2. Subsurface Conditions. The subsurface Characterization was performed by MoDOT prior to the initiation of the project. Two borings, designated H-11-16 and H-11-17 were drilled in the proximity of Bent 1 and Bent 4, respectively. Approximate ground surface elevations at the boring locations were 317.7 and 317.8 feet, respectively.

5.2.2.1 Geology. The site’s geology was consistent with description of the Southeast Region previously discussed in Section 3. Since the project site was located in the Southeast Lowlands region of Missouri and bedrock was not encountered during the subsurface characterization, it was assumed that bedrock was located at great depths.

5.2.2.2 Soil and groundwater. The subsurface soil conditions consisted of low plasticity lean clay (CL) and poorly graded sand (SP). Based on the boring information provided, the upper soil layer was a brown, lean clay that extended to depths of about 4 feet. Below the lean clay, medium dense, brown, fine to coarse sand was encountered to the borings’ termination depths of about 100 feet. Groundwater was observed at a depth of approximately 13.0 feet below the surface during drilling. Figure 5.4 shows the subsurface profile used in the WEAP analysis. It should be noted that the sand was separated into two layers solely in an attempt to refine the static analyses performed based on SPT N-values.

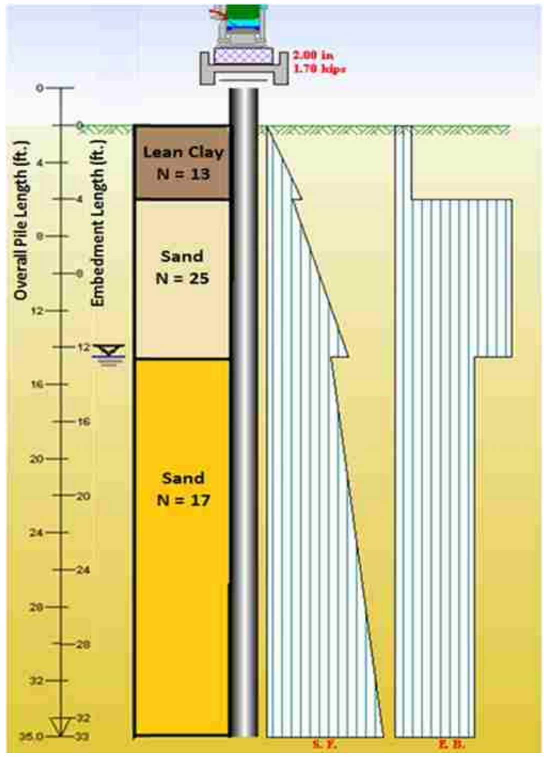


Figure 5.4 A7956 Soil Profile along the Test Pile

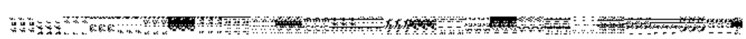
5.2.3. Static and Wave Equation Analyses and Results. Static and Wave Equation analyses were performed using the data collected from the subsurface characterization (prior to conducting the dynamic and static loading test at the site) to determine the nominal resistance of the test pile. These evaluations were performed to ensure the load frame and equipment used by Missouri S&T provided sufficient capacity to fail the test pile. The test pile in both analyses was assumed to be 35 feet in length (33 feet in the ground with 2-foot-stickup). The A7956 Static and Wave Equation analyses are included in Appendix B and Appendix C, respectively.

5.2.3.1 Static analysis. The Meyerhof (1976) SPT method was used to estimate the resistance contributed by the side friction and end-bearing of the test pile. This method was based on a correlation corrected (N_{60}) average standard penetration test values for a given soil layer. For the 33-foot-long pile tested, Meyerhof's method predicted a nominal resistance of of 335 kips.

5.2.3.2 Wave equation analysis. A wave equation analysis was completed using the GRLWEAP software program. A drivability analysis based on SPT N-Values was completed by averaging the N_{60}-values reported by MoDOT for each of the soil layer outlined in the description above in Section 5.1.2.1. Two separate analyses were performed by adjusting the resistance gain/loss factors along the shaft and toe to 0.8 and 1.0 and 1.0 and 1.0, respectively. The WEAP analysis estimated the nominal resistance of the test pile (using the N-value static model) to be within the range of and 121.7 to 131.7 kips depending to the gain/loss factors used. The results of these analyses indicate the estimated maximum stresses induced by the Delmag 19-32 pile hammer would not compromise the structural integrity of the pile and the resulting set per blows would meet the minimum field energy requirements necessary for driving the test pile. The drivability output for each set of gain/loss factors are shown in Table 5.2.

Table 5.2 A7956 WEAP Analysis Results for Gain/Loss Ratios at the Shaft and Toe of (A)0.8/1.0 and (B) 1.0/1.0

A)



B)

5.2.4. Anchor Pile & Test Pile Installation. The reaction frame and test piles at the A7956 site were installed on June 26, 2012 by CBCC. The reaction piles and test pile were 35 ft. long, 14 inch, closed-end steel pipe piles with a 3/8 inch wall thickness. All of the piles were installed using a Delmag D19-32 pile driving hammer. The special provisions and installation equipment were consistent with the materials and installation techniques used in the construction of the new structure and provided in Appendix A.

Prior to driving the first reaction pile, the location of the PLT was leveled using an excavator. The locations of the reaction piles were measured and staked to ensure the frame was constructed to the required specifications. Each reaction pile was then driven to a depth of 30 feet, resulting in a stick-up height of five feet to construct the rest of the frame. Figure 5.5 shows the reaction piles being installed.

Figure 5.5 A7956 Reaction Pile Installation

The test pile was installed last to limit the influence of the reaction piles during driving. Prior to the installation of the test pile, an excavator was used to remove 2.5 feet of soil in the proposed location of the test pile to ensure driving began on natural soils. The test pile obtained the nominal resistance based on the PDA Case Method analysis at a depth of 25 feet and driving ceased. Due sandy subsurface it was concluded the effects of pile set-up (or relaxation) would be minimal. However, a restrike was completed within 2 hours of the initial end-of-drive for verification, resulting in an additional 0.5 feet pile set in 19 blows. A stick up height of three feet was marked on the test pile and the remaining portion was cut off. The final embedment length of the pile was 28 feet. A small hole was also cut in the sidewall of the pile for the instrumentation cables to pass through to the DAQ box.

5.2.5. Dynamic Testing. Following to the special provisions in the MoDOT contracts, dynamic testing was conducted during the installation of the test pile by Craig Kaibel, P.E. of Geotechnology, Inc. A general description of the dynamic testing process is outlined in Section 4.5 and the results from the analysis are summarized in Section 5.2.8.1.

5.2.6. Dynamic Testing Results. The analysis of the dynamic data was performed by Craig Kaibel, P.E. using Case Pile Wave Analysis Program (CAPWAP) signal matching software. A summary of the CAPWAP estimated ultimate axial capacities are summarized in Table 5.3.

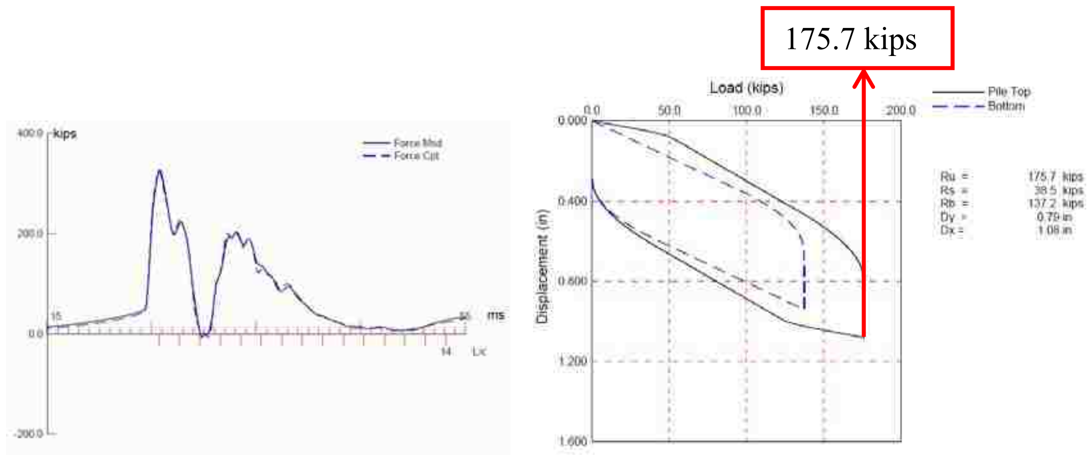
Table 5.3 Summary of CAPWAP Estimated Nominal Resistance for the A7956 test pile (adapted from the A7956 Geotechnology Report)

Test Type	Nominal Resistance (kips)		
	Total	Shaft	Tip
End-of-Drive (EOD)	175.7	38.5	137.2
Restrike (BOR)	184.1	38.4	145.7

Figure 5.6 shows the wave matching analyses and the estimated load-settlement curves from the CAPWAP analyses. From Table 5.3 and Figure 5.6, the total resistance increased approximately 5% (8.4 kips) between the EOD and BOR. The increase was attributed primarily through an increase in tip resistance. More details on the dynamic

analysis of the test pile are included in the Geotechnology report dated July 6, 2012 is included in Appendix C.

A)



B)

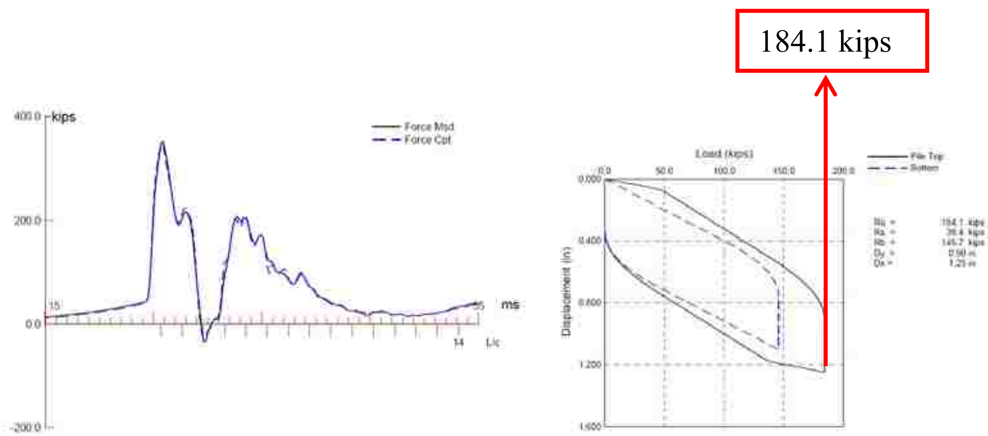


Figure 5.6 A7956 CAPWAP Wave Match and Load-Displacement Curve for (A) EOD and (B) BOR (adapted from the A7956 Geotechnology Report)

5.2.7. Test Pile Instrumentation. Five concrete embedded (Geokon Model 4200) VWSGs were used to instrument the test pile after driving for the pile load test. The gages were mounted on a center bar established by coupling a series of #9, 75 ksi dywidag bars together such that they would extend the length of the test pile. The gages were located at 4.0’, 10.0’, 15.5’, 21.5’, and 27.0’ from the top of the pile and referred to as VWSG 1-5, respectively. Each gage was equipped with a pre-specified length of wire and once attached to the center bar, each gage's wire was stretched the length of the center bar and secured using zip-ties. Each gage's wire was labeled with its' corresponding number to ensure they were connected sequentially to the data acquisition system. A series of centralizers were also mounted on the center bar. The centralizers were constructed from scrap pieces of #4 rebar, bent into a diamond shape approximately 16 inches wide (diagonally). The centralizers were equally spaced along the center bar using wire. Mounting the centralizers such that one end was secure and the other was left free allowed for the tightest possible fit within the pile.

When the bar is lowered into the test pile, the centralizers ensure the bar is centered, thus locating the mounted gages down the center of a test pile as well. Once the center bar was lowered into the pile the excess gage wires were threaded through the hole cut in the side wall of the pile. Figure 5.7 shows the center bar being lowered into the test pile.

Figure 5.7 Installation of the Center Bar and VWSGs

Concrete was placed within the test pile to complete its' construction. To avoid damage of the VWSGs during concrete placement, the was placed from the bottom of the pile upwards. Since no tremme pipe was available onsite, a series of 4 inch PVC pipes were used to place the concrete without damaging the gages. By avoiding the centralizers and gages, this long tube was first lowered all the way to the bottom of the pile and concrete was then guided directly from the concrete truck's shoot into an 8 inch PVC funnel that rest on top of the 4 inch pipe. The slump of the concrete was increased by adding water to allow the concrete to flow more easily through the PVC tremme and the resultant slump of the mix was measured at 4.5 inches by MoDOT personnel. A handheld concrete vibrator was used as well to remove block-ups that occurred in the restricted throat of the 4 inch tube. Figure 5.8 illustrates the concrete placement process. The construction events (placing the reaction beam and connecting the threaded bars) that took place between the instrumenting the test pile and the actual initiation of the static load test followed the general outline presented in Section 4.

A) B)

Figure 5.8 Process of Test Pile Concrete Placement. (A) Centerbar lowered into Test Pile. (B) PVC Tremme Lowered Around VWSGs.


C)


D)


E)


F)


G)

Figure 5.8 (cont.) Process of Test Pile Concrete Placement. (C) Begin Concrete Placement. (D) PVC Tremme Removed and Shortened with Sawzall. (E) PVC Tremme Re-lowered into Test Pile. (F) Resume Concrete Placement. (G) Concrete Placement Finished.

5.2.8. Static Load Test. The static load test at the A7956 bridge site began on July 3, 2012. However, testing ceased after the second loading cycle due to a structural deficiency in the reaction beam. The test was delayed until August 8, 2012 allowing for a replacement beam to be constructed for the test's completion. The testing methods completed at the A7956 site followed the Quick ML Test methods and general testing procedure provided in Sections 2.3.4.1.2. and 4.6, respectively. The A7956 load test setup and reaction frame are shown in Figure 5.9.

Figure 5.9 Completed A7956 Pile Load Test Set-up

5.2.9. Static Load Test Results. The test pile was incrementally loaded until failure following the loading schedule presented in Table 5.4. The data collected from the static load test was reduced following the data reduction methods presented in Section 4. The values used to perform the data reduction are shown in Table 5.5.

Table 5.4 A7956 Load Test Schedule

Job No.:	JOP2239
Design:	A7956
Date:	8/7/2012
Est. Nom. Resistnace:	200 kips
Design Load:	100 kips
Factor of Safety:	2.0

Load Cycle	Applied Load	
	(% DL)	(kips)
Zero Values	Jack	0.3
Seating	AL	0.3
Cycle 1 (100 kips)	12.5	25
	25.0	50
	37.5	75
	50.0	100
	37.5	75
	25.0	50
	12.5	25
Unload	AL	0.3
Cycle 2 (200 kips)	12.5	25
	25.0	50
	37.5	75
	50.0	100
	62.5	125
	75.0	150
	62.5	125
	0.0	0

Load Cycle	Applied Load	
	(% DL)	(kips)
Seating	AL	0.3
Cycle 3 (Plunge)	12.5	25
	25.0	50
	50.0	100
	62.5	125
	75.0	150
	87.5	175
	92.5	185
	97.5	195
	102.5	205
	105.0	210
	107.5	215
	110.0	220
	112.5	225
	115.0	230

DL - Design Load

AL - Alignment Load

Table 5.5 Parameters Used in A7956 Data Reduction

Parameter	Value
Steel Modulus of Elasticity, E_{steel}	29,000 ksi
Steel Area of Pile, A_{pile}	16.05 in^2
Steel Area of Center Bar, $A_{centerbar}$	0.994 in^2
Concrete Modulus of Elasticity, $E_{concrete}$	3685 ksi
Concrete Area of Pile, $A_{concrete}$	136.89 in^2
Transformed Area, A_{trans}	34.44 in^2

The load cell and LVDT data from all three cycles were used to plot axial load versus axial displacement at the pile head, as shown in Figure 5.10. During the unloading portions of cycle 1 and 2, it was observed that the pile rebounded slightly from the maximum displacement measured in each corresponding cycle. Displacement of the pile began to occur more rapidly once the applied load increased above 195 kips, however once the load cell reading reached 210 kips, the pile began to plunge. The data obtained from the A7956 static load test and corresponding results are included in Appendix D.

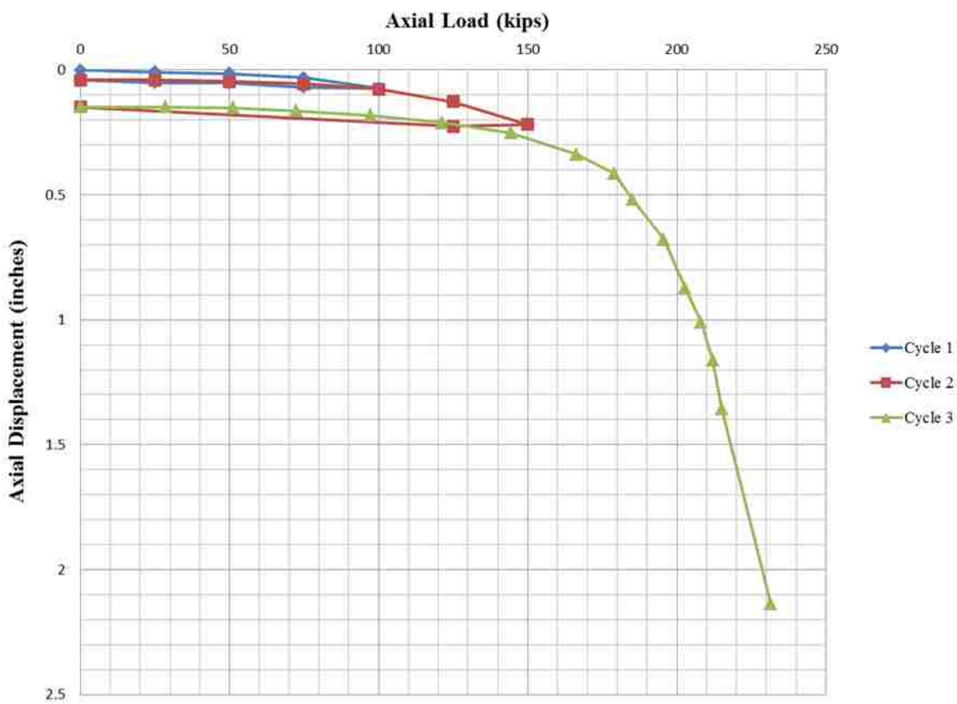


Figure 5.10 A7956 Static Load Test Results

5.2.9.1.1 <u>Nominal resistance.</u> A series of methods (as described in Section 2) were used to interpret the failure load from the load-displacement curve. The resulting plot of each method is illustrated in Figures 5.11-5.15. A summary of the nominal resistances interpreted from each method are presented in Table 5.6. Note that only the curve of cycle 3 is used in the interpretation for each method.

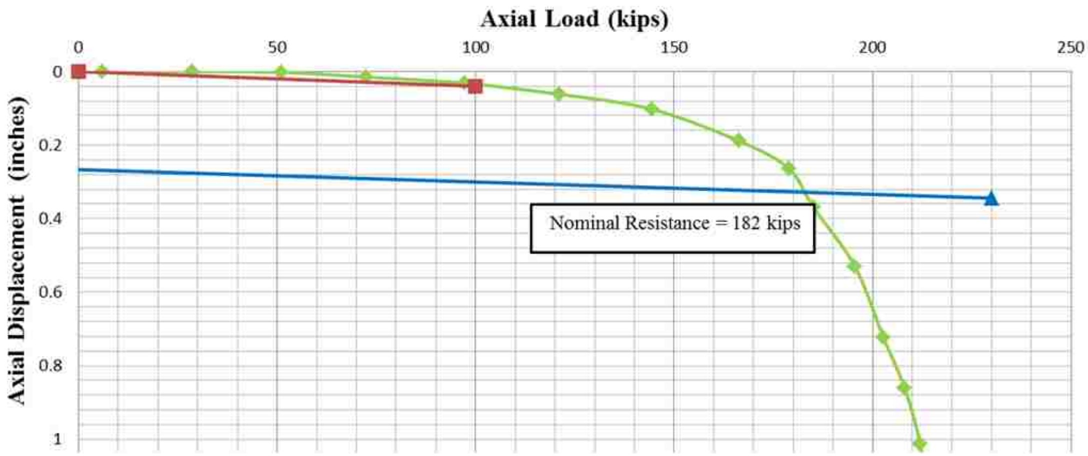


Figure 5.11 Interpretation of A7956 Nom. Resistance Using the Davisson (1972) Method

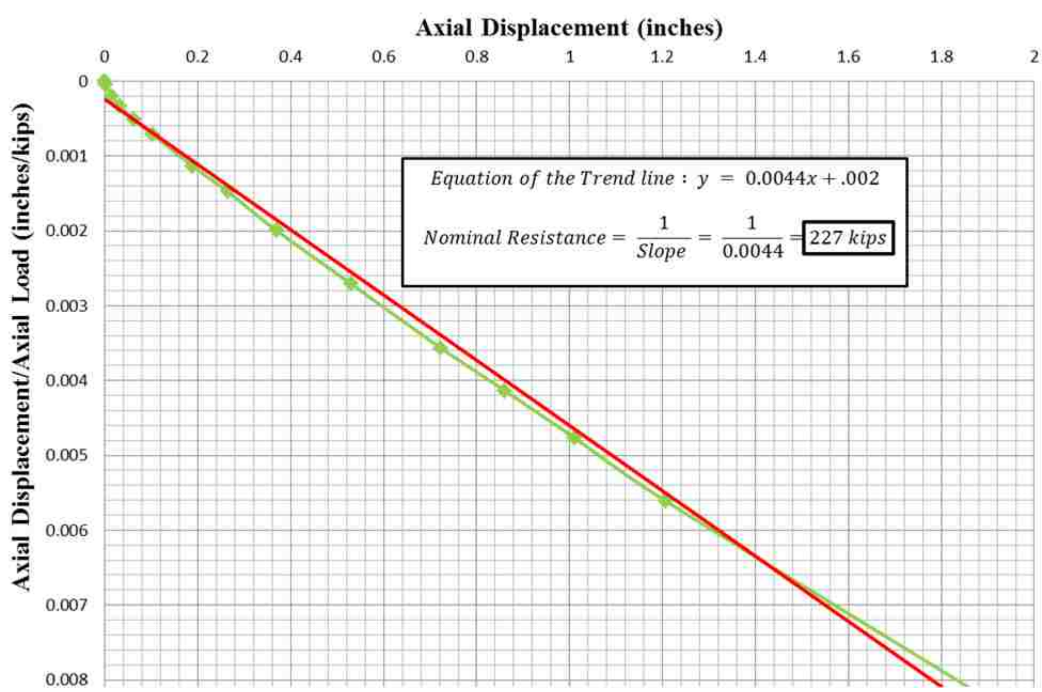


Figure 5.12 Interpretation of A7956 Nom. Resistance Using the Chin (1970) Method

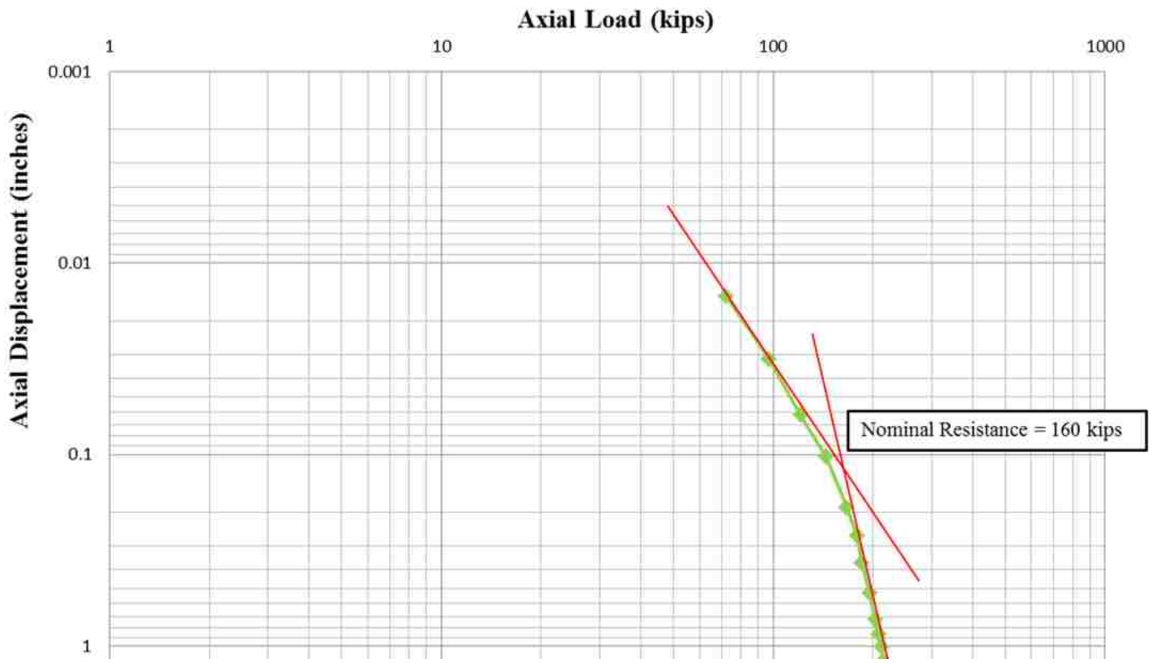


Figure 5.13 Interpretation of A7956 Nom. Resistance Using the De Beer (1968) Method

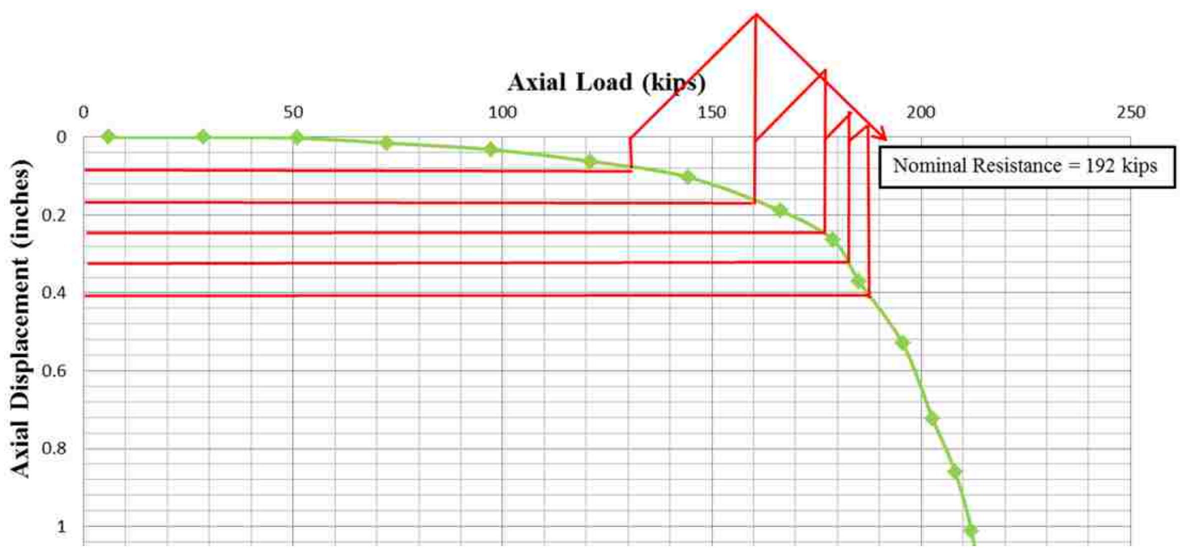


Figure 5.14 Interpretation of A7956 Nom. Resistance Using the Mazerkiewicz (1980) Method

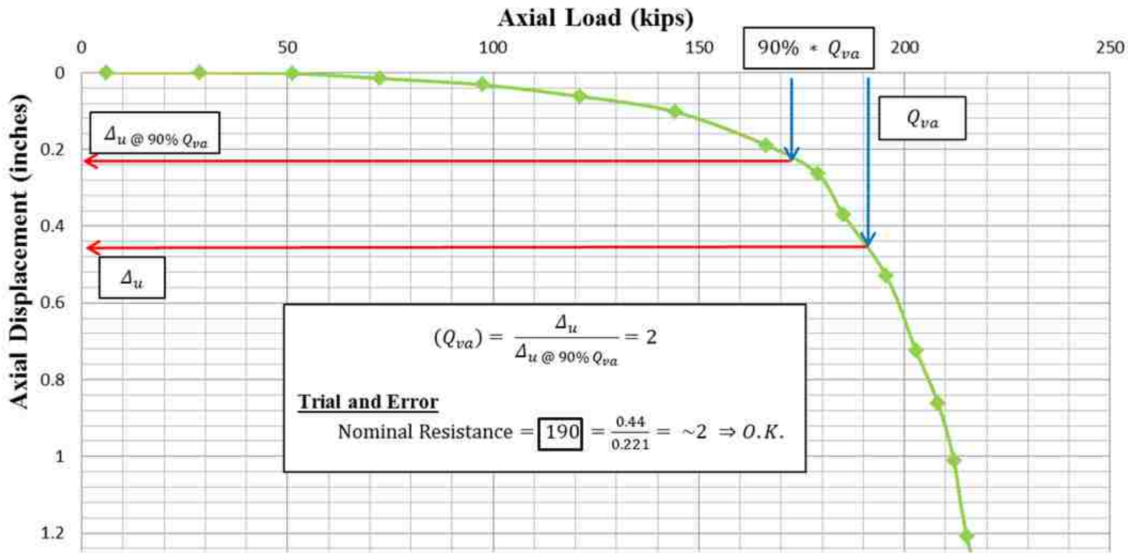


Figure 5.15 Interpretation of A7956 Nom. Resistance Using the Brinch Hansen 90% (1963) Method

Table 5.6 Summary of Interpreted A7956 Nominal Resistances

A7956 Static Load Test Nominal Resistance Summary	
Method	**Nominal Resistance (kips)**
Davisson (1972)	182
Chin (1970)	227
De Beer (1968)	145
Mazurkiewicz (1980)	192
Brinch Hansen 90% Criteria (1963)	190
Minimum Value	145
Maximum Value	227
Average Value	187

The static load test results showed a close agreement with the estimated dynamic load test resistance resulting in a difference of 1%, as shown is Table 5.7. It's important to note that the AASHTO LRFD Specification (2010) specifies the use of Davisson's (1972) method (for piles 24 in. in diameter or less) to interpret the ultimate resistance from a QM static load test. Therefore, the nominal resistance interpreted using this method was reported for comparison.

Table 5.7 Comparison of A7956 Nominal Resistance Results

Bridge (geologic region)	Nominal Resistance (kips)			Difference (%)
	Static Load Test	Dynamic Testing		
		EOD	BOR	
A7956 Sikeston, MO (SE Lowlands)	182.0* (145-227)	164.6	184.1	± 1 %

*Davisson's 1972 method reported, in parenthesis the range of all methods

5.2.9.1.2 <u>Load transfer distribution</u>. Figure 5.16 illustrates the load-transfer plot corresponding to each applied load increment during the static load test. At failure, the shaft and tip resistance was 104 kips and 78 kips, respectively, concluding approximately 57% of the pile's nominal resistance was contributed by the shaft resistance and 43% was contributed by end bearing. A schematic of the approximate location of the VWSGs with respect to the test pile and subsurface conditions is also provided in the Figure.

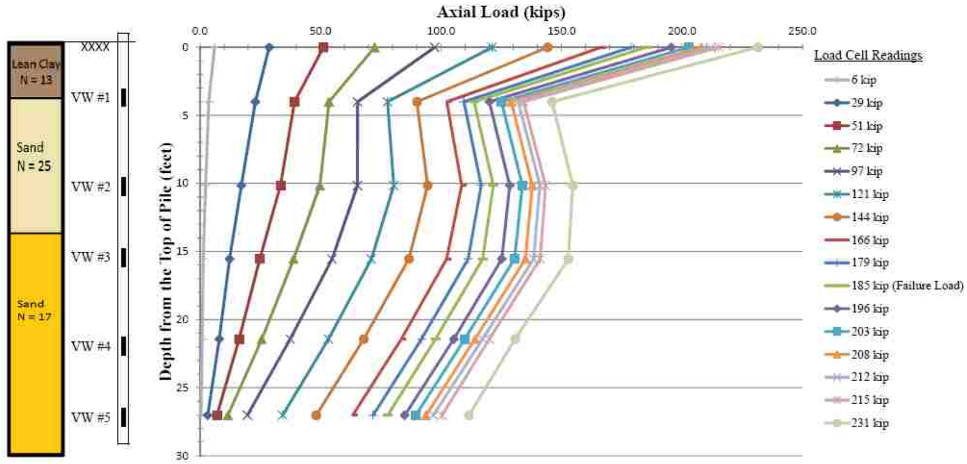


Figure 5.16 A7956 Load Transfer Plot

5.3. POPLAR BLUFF, MISSOURI

The second pile load test was conducted at the MoDOT A7669 bridge site located approximately 8 miles south of Poplar Bluff, Missouri on Hwy. 67. The site topography consisted of heavily wooded, rolling hills. The testing location was located approximately 50 feet to the northwest of Bent 1 within the MoDOT right-of-way. Figure 5.17 shows the approximate location of the construction site (Latitude/Longitude: 36°41’36.19”N/90°28’46.72”W.)

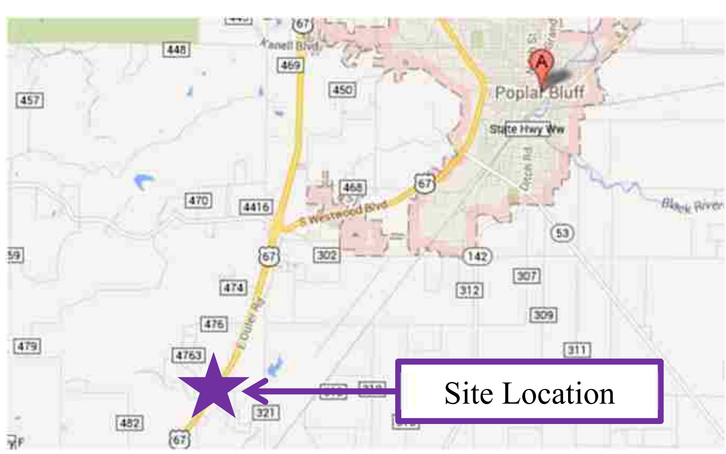


Figure 5.17 A7669 Site Location Map (Google Maps, 2013)

5.3.1. Site and Project Description. The new structure was part of a highway expansion project which included a new two-lane, three-span bridge to support south-bound traffic crossing the Crane Creek Overflow. Figure 5.18 shows a construction drawing of the proposed structure.

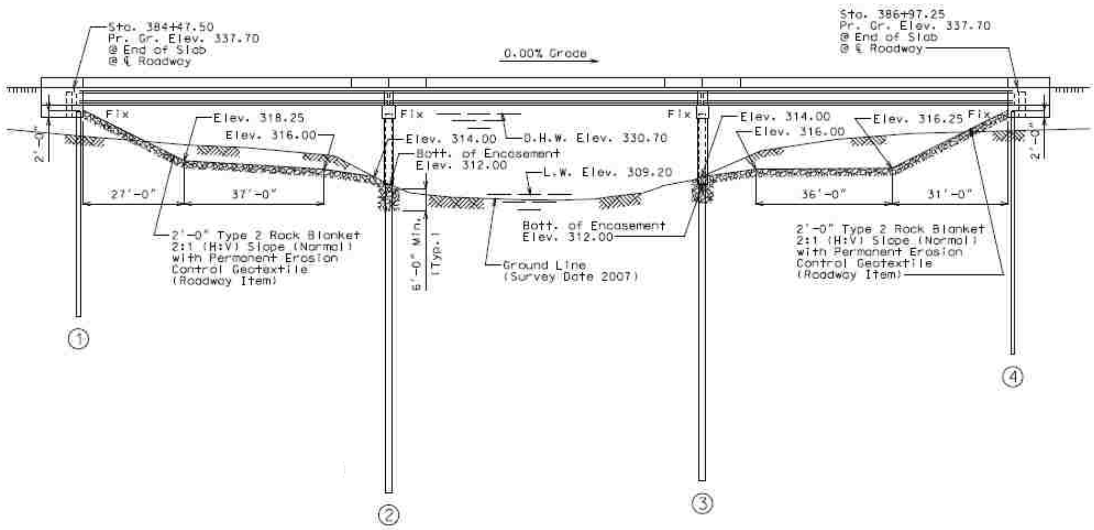


Figure 5.18 MoDOT Illustration of A7669 Proposed Structure (MoDOT, 2013)

The foundation system included 14x73 steel H-section piles at the outer abutment bents and 20 inch CIP piles in the intermediate bents. Table 5.8 summarizes the foundation data for each bent of the new structure. The superstructure consisted of prestressed concrete box girder spans and precast prestressed concrete panels. The contractor for the project was Robertson Contractors, Inc. (RCI) of Poplar Bluff, Missouri.

5.3.2. Subsurface Conditions. The subsurface characterization was performed MoDOT prior to the initiation of the project. Four borings, designated A-10-29, O-10-113, O-10-114, and A-10-30 were drilled for Bents one through four, respectively. Approximate ground surface elevations at the boring locations were 323.5, 317.6, 318.1, and 327.1 feet, respectively.

Table 5.8 A7669 Foundation Data (adapted from MoDOT Plans, 2013)

	Bent No.	**1**	**2**	**3**	**4**
Driven Pile	Pile Type and Size:	HP 14x73	20” CIP	20” CIP	HP 14x73
	Number:	12	9	9	12
	Approx. Length (ft):	53	96	97	55
	Minimum Nominal Axial Compressive Resistance (kip)	168	387	387	168

5.3.2.1 Geology. Poplar Bluff lies on an escarpment which separates the Ozark region from the Southeast Lowlands to the east. The site’s geology was consistent with description of the Southeast Lowlands region discussed in Section 3. However, the site contained thicker clay deposits than the A7956 site, which was also located in the Southeast Lowlands. Highly weathered, thinly bedded dolomite was encountered below the sand layers and extended to the borings’ termination depths of 107.5 feet.

5.3.2.2 Soil and groundwater. The existing soils observed consisted of low plasticity lean clay (CL), high plasticity fat clay (CH), and poorly graded sand (SP). Based on the results of the boring information provided, the borings initially encountered brown, lean clay that extended to depths of about 15 feet. Below the lean clay, gray fat clay with varying amounts of sand were encountered to a depth of about 38.0 feet. Below the fat clay, medium dense, brown, fine to medium sand with varying amounts of clay were encountered to depths of about 84.6 feet. Groundwater was observed at approximately 11.0 feet below the surface during drilling. Figure 5.19 shows the subsurface conditions modeled for the WEAP analysis.

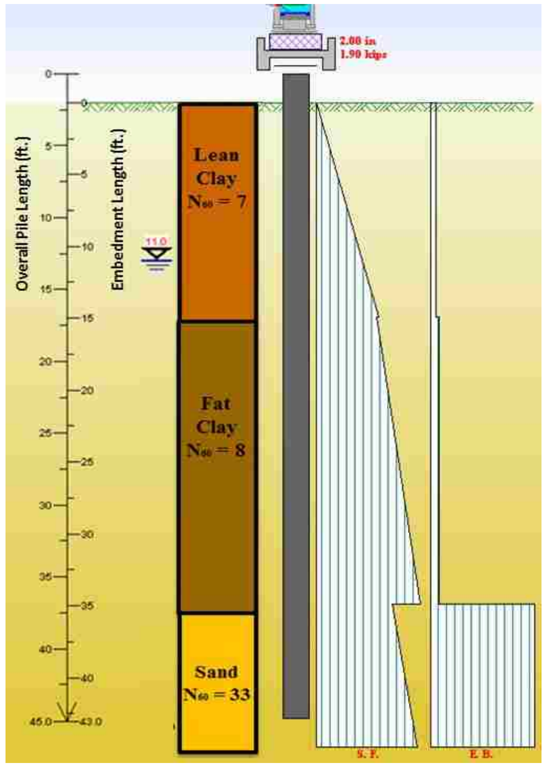


Figure 5.19 A7669 Soil Profile Along Test Pile

5.3.3. Static and Wave Equation Analyses and Results. Static and Wave Equation analyses were performed using the data collected from the subsurface characterization (prior to conducting the dynamic and static loading test at the site) to determine the nominal resistance of the test pile. The test pile in both analyses was assumed to be 45 feet in length (43-foot-embedded with 2-foot-stickup). The A7669 Static and Wave Equation analyses are included in Appendix B and Appendix C, respectively.

5.3.3.1 Static analysis. The Alpha and Beta methods were used to estimate the available resistance of the test pile. For the 45-foot-long pile tested, these methods predicted a nominal resistance of 287.7 kips. Although static methods have a tendency to over-predict the actual nominal resistance, the estimated value was still below the actual capacity of the reaction frame.

5.3.3.2 Wave equation analysis. A wave equation analysis was completed using GRLWEAP software program. A drivability analysis based on SPT N-Values was completed by averaging the N-values reported by MoDOT for each of the soil layer outlined in the description above in Section 5.3.2.1. Two separate analyses were performed by adjusting the resistance gain/loss factors along the shaft and toe from 0.8 and 1.0 and 1.0 and 1.0, respectively. The WEAP analysis estimated the nominal resistance of the test pile (using the N-value static model) to be within the range of and 233.4 to 255.7 kips depending to the gain/loss factors used. The results of these analyses indicate the estimated maximum stresses induced by the Delmag 19-42 pile hammer would not compromise the structural integrity of the pile and the resulting set per blows would meet the minimum field energy requirements necessary for driving the test pile. The drivability output for each set of gain/loss factors are shown in Table 5.9.

Table 5.9 A7669 WEAP Analysis Results for Gain/Loss Ratios at Shaft and Toe of (A) 0.8/1.0 and (B) 1.0/1.0

A)

B)

5.3.4. Anchor Pile & Test Pile Installation. The reaction frame and test piles at the A7669 site were installed on October 22, 2012, by RCI. The pile driving hammer used during the installation consisted of a Delmag D19-42. The reaction piles were 55 ft. long, 14 inch closed-ended, steel pipe piles with a 3/8 inch wall thickness. The test pile and pile driving hammer were consistent with the materials and installation techniques used in the adjacent bent of the actual structure.

A bulldozer was used to level the area around the testing location. The locations of the reaction piles were measured and staked before each reaction pile was installed. The reaction piles were driven to a depth of 50 feet, resulting in a stick-up height of five feet. The test pile (HP 14x73) was installed after the reaction piles to limit the influence of the reaction piles during driving. Preceding the installation of the test pile, a backhoe was used to remove 2.0 feet of soil in the proposed location of the test pile to ensure driving began on natural soils and to facilitate instrumentation installation at the pile head. Figure 5.20 shows the installation of the test pile.

Figure 5.20 A7669 Test Pile Installation

The test pile for the PLT was installed to an approximate elevation of 271 ft. resulting in an embedment length of 43 ft. Providing a 2 ft. stick-up height, the final length of the test pile was 45 ft. Since the soil conditions were primarily clay, a restrike was scheduled 7 days later to observe the effects of pile setup.

5.3.5. Dynamic Testing. Following to the special provisions in the MoDOT contracts, dynamic testing was performed during the installation of the A7669 test pile on October 22, 2012 by Craig Kaibel, P.E. of Geotechnology Inc.. The dynamic testing events followed the description outlined in Section 4.5 and the results from this analysis are summarized in Section 5.3.8.1.

5.3.6. Dynamic Testing Results. A summary of the nominal resistances (EOD and BOR) estimated by CAPWAP are summarized in Table 5.10. Figure 5.21 shows the wave matching analyses and the estimated load-settlement curves from the CAPWAP analyses.

Table 5.10 Nominal Resistances Estimated From the A7669 CAPWAP Analysis (adapted from the A7956 Geotechnology Report)

Test Type	Nominal Resistance (kips)		
	Total	Shaft	Tip
End-of-Drive (EOD)	88.2	76.9	11.3
Restrike (BOR)	223.6	151.9	71.7

As Table 5.10 and Figure 5.21 show, the total resistance increased approximately 154% (135.4 kips) from EOD to BOR. The increase was attributed primarily through an increase in shaft resistance. More details on the dynamic analysis of the test pile are included in the Geotechnology report dated November 14, 2012, included in Appendix C.

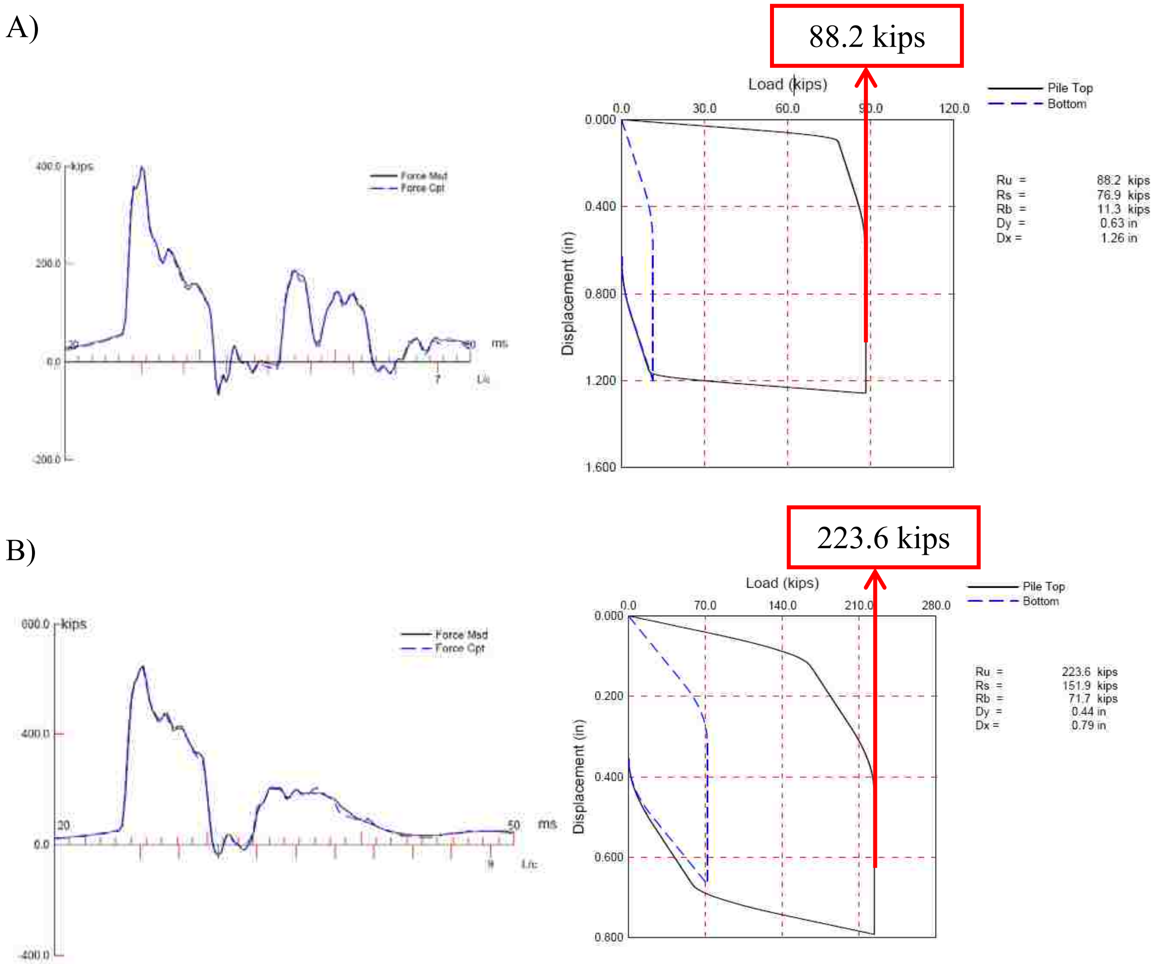


Figure 5.21 A7669 CAPWAP Wave Match and Load-Displacement Curve for (A) EOD (B) BOR (adapted from the A7956 Geotechnology Report)

5.3.7. Test Pile Instrumentation Installation. Since the test pile at the A7669 site was an H-pile, special consideration was given to effectively instrument the pile. Five weldable (Geokon Model 4000) VWSGs were used to instrument the test pile before installation. The strain gages, labeled VWSG #1 through VWSG #5 successively from the pile head downward, were located at 7', 16', 25' 34', and 43', respectively. It is important to note that VWSG #3 was damaged during the installation of the test pile and yielded no useable measurements.

The VWSGs were installed the day prior to driving the test pile. The first step included welding the gage's mounts to into the pile's web at predetermined intervals along the length of the pile. A pre-cut piece of steel, equal in diameter and length of an actual gage, was used as a substitute when the mounts were welded, to avoid damage to the actual gages. Nozzle Gel was spread on the precut piece of steel to keep slag from sticking to it during installation. The use of Nozzel Gel allowed the piece of steel to be easily removed once the welding was completed. Once each set of gage mounts were installed, the actual gages were installed and their wires was stretched the length of the pile. Since the wires of VWSGs are known for being susceptible to damage during installation, their movements had to be restricted. All-purpose caulk was applied around the wires to keep them from bouncing during the installation of the test pile. After the gages and their wires were secured, a four inch wide (0.25 inch thick) piece of steel strap was spot welded over the top of all the components to protect them during driving. Figure 5.22 illustrates the instrumentation process of the HP 14x73 test pile.

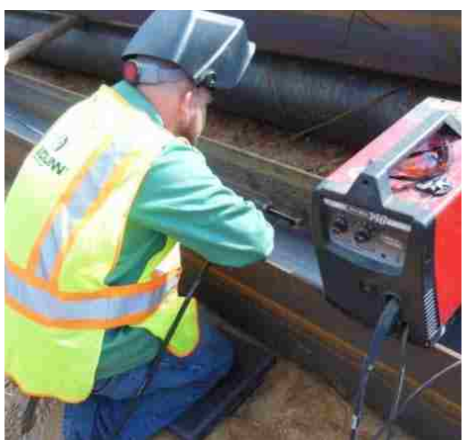
A)  B) 

Figure 5.22 H-Pile Instrumentation Process. (A) Welding VWSG Mounts (B) Installing VWSGs.

C)  D) 

Figure 5.22 (cont) H-Pile Instrumentation Process. (C) Securing Gage Wires with All-Purpose Caulk (D) Welding Steel Strap Over Gages.

5.3.8. Static Load Test. The static load test at the A7669 site began on October 31, 2012. The testing methods at the A7669 site followed the Quick ML Test methods and general testing procedure described in Sections 2.3.4.1.2. and 4.6, respectively. The A7669 load test setup and reaction frame are shown in Figure 5.23.

Figure 5.23 Completed A7669 Pile Load Test Set-up

5.3.9. Static load test results. The test pile was axially loaded following the loading schedule presented in Table 5.11. The data collected from the static load test was reduced following the data reduction methods presented in Section 4. Because the test pile only consisted of steel, the use of a transformed area was not required. The modulus of elasticity and pile area used in the data reduction are shown in Table 5.12.

Table 5.11 A7669 Loading Schedule

Job No.:	JOP0959
Design:	A7669
Date:	31-Oct
Est. Nom. Resistance:	200 kips
Design Load:	168 kips
Factor of Safety:	2.0

Load Cycle	Applied Load	
	(% DL)	(kips)
Zero Values	Jack	0.3
Seating	AL	0.3
Cycle 1 (100 kips)	12.5	25
	25.0	50
	37.5	75
	50.0	100
	37.5	75
	25.0	50
	12.5	25
Unload	AL	0.3
Cycle 2 (200 kips)	25.0	50
	50.0	100
	75.0	150
	100.0	200
	75.0	150
	50.0	100
	25.0	50
	0.0	0

Load Cycle	Applied Load	
	(% DL)	(kips)
Seating	AL	0.3
Cycle 3 (Plunge)	25.0	50
	50.0	100
	75.0	150
	100.0	200
	105.0	210
	110.0	220
	112.5	225
	115.0	230
	117.5	235
	120.0	240
	122.5	245
	125.0	250
	127.5	255
	130.0	260
	132.5	265
	135.0	270
	137.5	275

DL - Design Load
AL - Alignment Load

Table 5.12 Parameters Used in the A7669 Data Reduction

Parameter	Value
Steel Modulus of Elasticity, E_{steel}	29,000 ksi
Steel Area of Pile, A_{pile}	21.5 in^2

The load cell and LVDT data from all three cycles were used to plot axial load versus axial displacement at the pile head, as shown in Figure 5.24. During the unloading portions of Cycle 1 and 2, the pile rebounded slightly from the maximum displacement measured in each corresponding cycle. Although very little displacement occurred in the first two cycles, displacement began to occur more rapidly once the applied load was increased above 200 kips. When the load cell reading reached 260 kips, the pile began to plunge. The raw data obtained collected form the A7669 static load test and the corresponding reduced results are included in Appendix D.

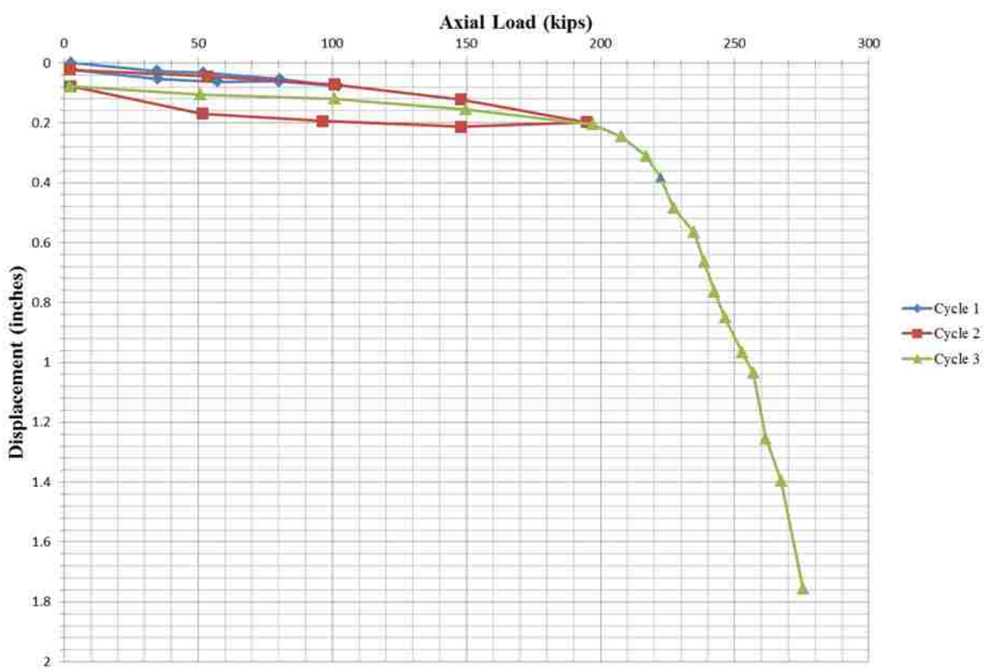


Figure 5.24 A7669 Static Load Test Results

5.3.9.1.1 <u>Nominal resistance</u>. The same series of methods (displayed earlier in this Section) were used to interpret the failure load from the applied load-axial displacement curve. The resulting plot of each method is expressed in Figures 5.25-5.29. The ultimate capacities interpreted from each method are presented in Table 5.13. It is important to note that only the curve of the failure cycle (Cycle 3) is used in the interpretation for each method.

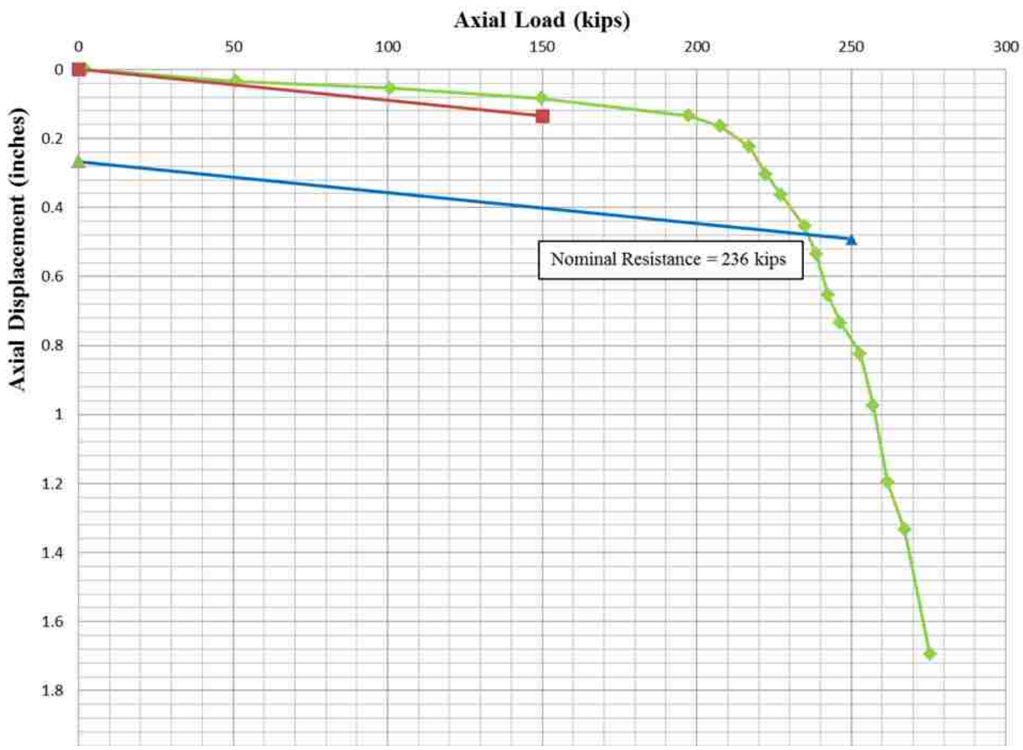


Figure 5.25 Interpretation of A7669 Nom. Resistance Using the Davisson (1972) Method

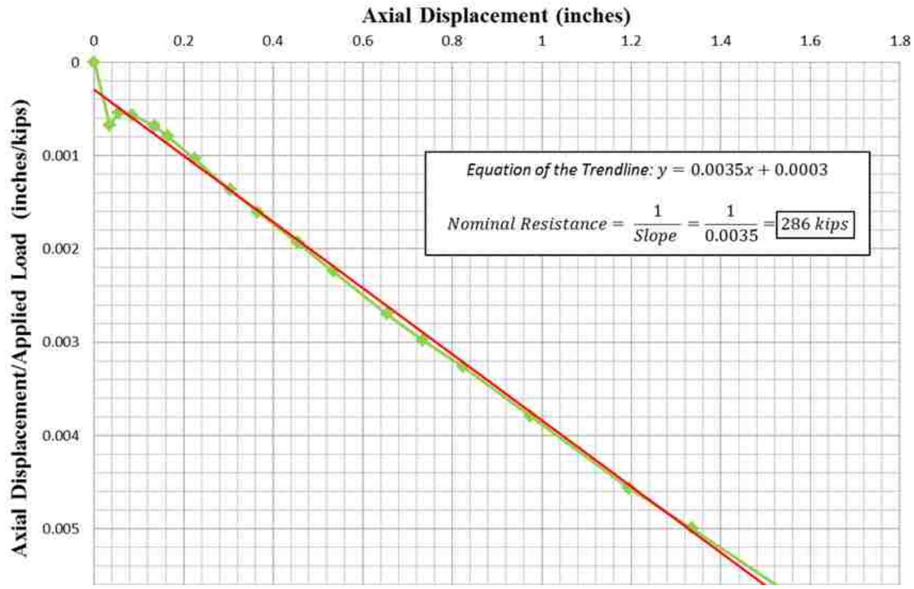


Figure 5.26 Interpretation of A7669 Nom. Resistance Using the Chin (1970) Method

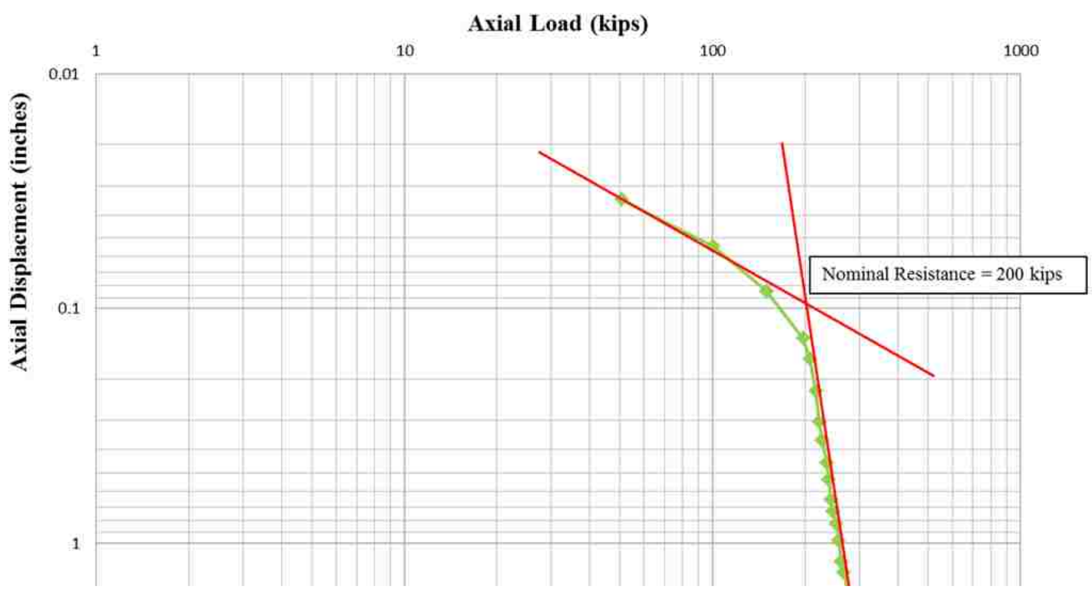


Figure 5.27 Interpretation of A7669 Nom. Resistance Using the De Beer (1968) Method

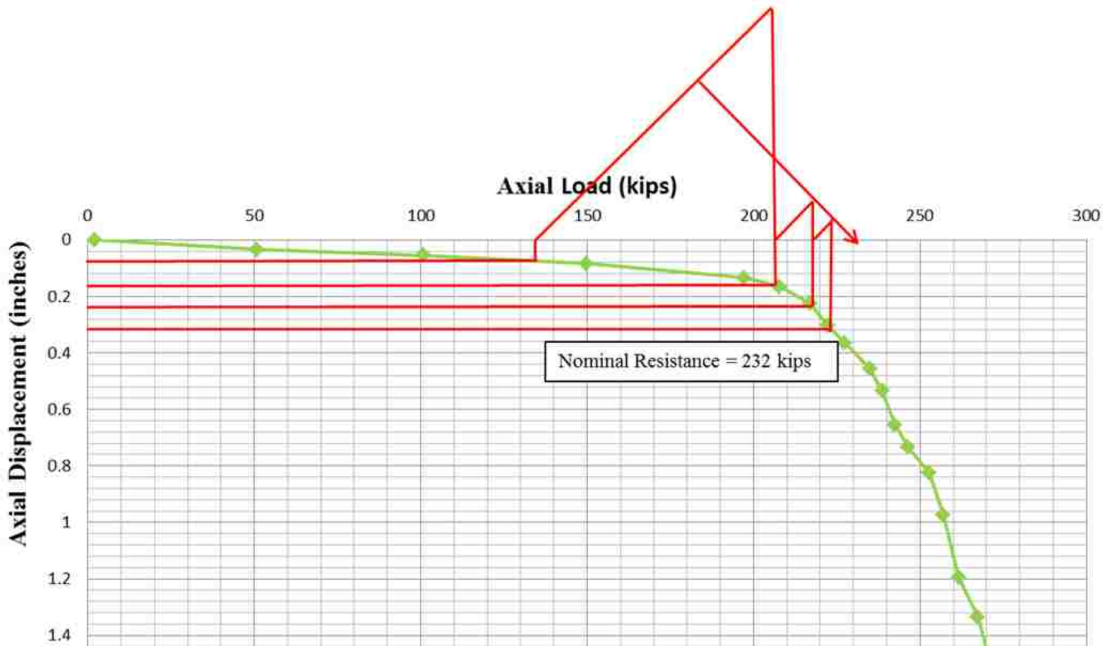


Figure 5.28 Interpretation of A7669 Nom. Resistance Using the Mazurkiewicz (1980) Method

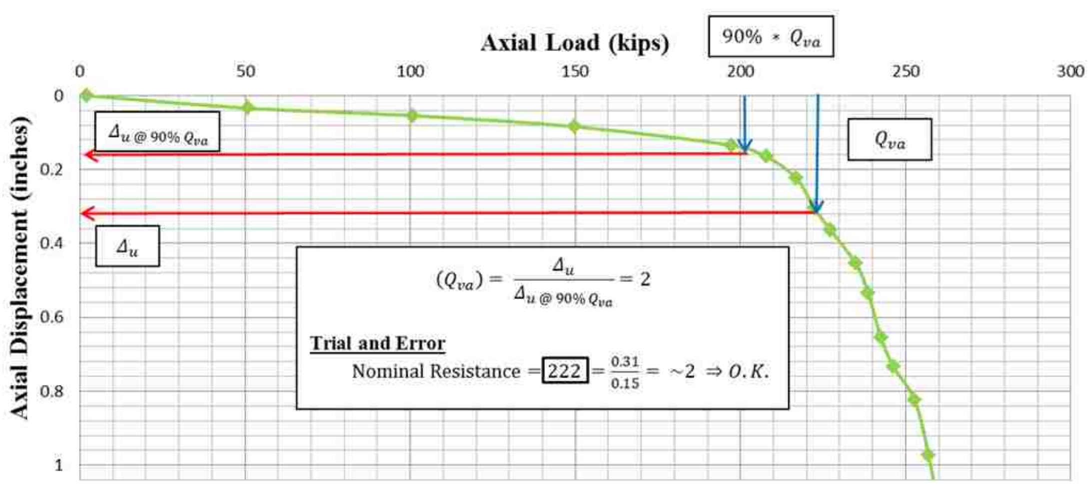


Figure 5.29 Interpretation of A7669 Nom. Resistance Using the Brinch Hansen 90% (1963) Method

Table 5.13 Summary of Interpreted A7669 Nominal Resistance

A7669 Static Load Test Nominal Resistance Summary	
Method	**Nominal Resistance (kips)**
Davisson (1972)	236
Chin (1970)	286
De Beer (1968)	200
Mazurkiewicz (1980)	232
Brinch Hansen 90% Criteria (1963)	222
Minimum Value	200
Maximum Value	286
Average Value	236

The difference in the nominal resistance measured by the static load test and the nominal resistance estimated at BOR by the dynamic test is about 5%, as shown is Table 5.14. As state in Section 5.2.9.1.1., because the AASHTO LRFD Specification (2010) specifies the use of Davisson's (1972) method (for piles 24 in. in diameter or less) to interpret the ultimate resistance from a QM static load test, the nominal resistance interpreted using Davisson's method was reported for comparison.

Table 5.14 Comparison of A7669 Pile Nominal Resistance Results

Bridge (geologic region)	Nominal Resistance (kips)			Difference (%)
	Static Load Test	Dynamic Testing		
		EOD	BOR	
A7669 Poplar Bluff, MO (SE Lowlands)	236.0* (200-286)	82.2	223.6	± 5 %

*Davisson's 1972 method reported, in parenthesis the range of all methods

5.3.9.1.2 <u>Load transfer distribution</u>. Figure 5.30 illustrates the load-transfer distribution corresponding to each applied load increment from the A7669 static load test. At failure, the shaft and tip resistance was 172 kips and 64 kips, respectively, concluding approximately 73% of the pile's nominal resistance was contributed by the shaft resistance and 27% was contributed by end bearing. A schematic of the approximate location of the VWSGs with respect to the test pile and subsurface conditions is also provided in the Figure.

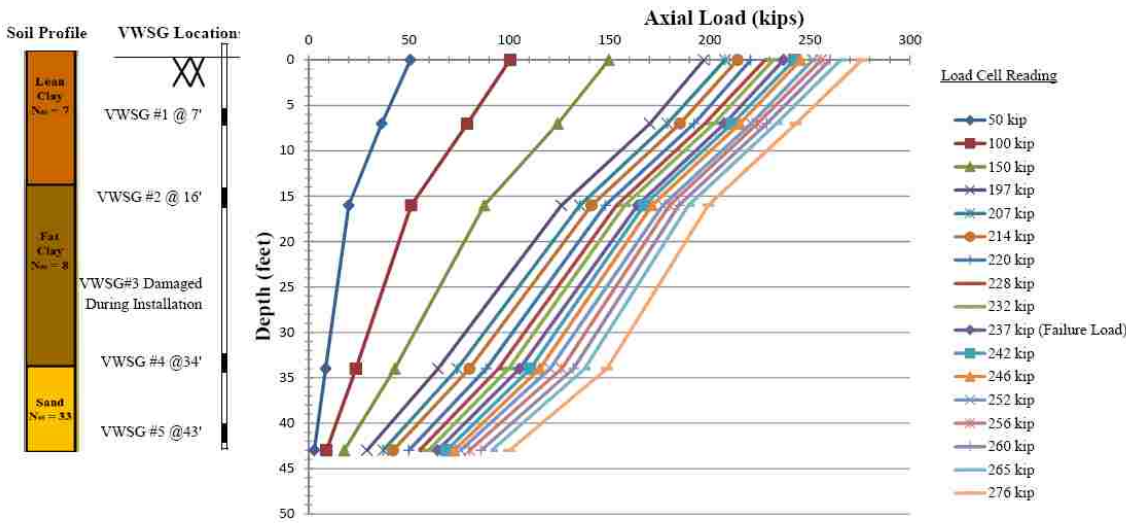


Figure 5.30 A7669 Load Transfer Plot

6. SUMMARY AND DISCUSSION OF PILE LOAD TEST RESULTS

6.1. INTRODUCTION

This section presents a summary and discussion of the results from the two full scale pile load tests completed as part of Phase I of this research project.

6.2. PILE LOAD TEST – DYNAMIC AND STATIC

6.2.1. Dynamic Load Tests. As mentioned in Section 5, representative hammer blows from the data collected at the EOD and near BOR of each test pile were subsequently analyzed using CAPWAP signal matching software. Table 6.1 summarizes the dynamic testing results of each test pile. Although the nominal resistance increased from EOD to BOR at each test site, the nominal resistance measured near BOR at the A7669 test site was far more significant.

Table 6.1 Nominal Resistance Estimated From the CAPWAP Analyses

Bridge (geologic region)	Test Type	Nominal Resistance (kips)			Pile Set-up
		Total	Shaft	Tip	
A7956 Sikeston, MO (SE Lowlands)	End-of Drive	175.7	38.5	137.2	4.7%
	Restrike	184.1	38.4	145.7	
A7669 Poplar Bluff, MO (SE Lowlands)	End-of-Drive	88.2	76.9	11.3	153.5%
	Restrike	223.6	151.9	71.7	

As a pile is driven, the soil against the test pile is sheared and remolded. This combination generates an increase in the porewater pressure of the soil. As the porewater pressure increases, the soil's effective stress is reduced, thus decreasing the strength of the soil. Over time the excess porewater pressure dissipates, increasing the soil's effective stress, which results in an associated increase in the strength of the soil. This mechanism is referred to as "pile setup" (AASHTO, 2010).

The hydraulic conductivity of cohesionless soils allows for the excess porewater pressure to dissipate relatively quickly. Therefore, the changes in nominal resistance from EOD to near BOR are typically subtle, as seen in the dynamic results from the A7956 site. Conversely, the hydraulic conductivity of cohesive soils cause the excess porewater pressure to dissipate far more slowly. In some clays, setup may continue to develop over a period of weeks and even months (AASHTO, 2013). The test pile installed in clay soils at the A7669 site displayed a significant increase in the nominal resistance estimated from EOD to near BOR. This site illustrates the effects of pile setup in the clay deposits.

In practice, a restrike test is usually performed several days after EOD to assess the effects of pile setup. At bridge sites were pile setup is predicted to be significant, piles that do not reach their nominal resistance at EOD can be left undisturbed to allow the excess porewater pressures to dissipate. The restrike results are then used to validate if the pile reached design nominal resistance at BOR.

The practical significance of pile setup was highlighted at the Poplar Bluff (A7669) site. The A7669 Job Special Provisions (JSP) state, "Monitoring of pile driving shall begin when pile driving begins. Unless monitoring indicates that additional driving will damage the pile, pile driving and monitoring shall continue until both the specified tip elevation and the specified pile resistance are reached." At EOD the contractor's consultant [Foundations Testing and Consulting, LLC (FTC)] determined the design resistance of the production piles was not met at the specified tip elevation. In MoDOT practice if a pile does not reach the design resistance at EOD, the contractor has the ability to:

- Alter the contract amount and time and continue driving until the pile reaches its design resistance or
- Wait and restrike the pile to see if the design resistance is obtained through pile setup (T. Fennessey, personal communication, November 21, 2013).

Because it's the contractor's responsibly to produce a foundation consistent with the design, their decision amounts to which option is more economically viable. In other words, does the cost of waiting to resume the construction activities until after the restrike outweigh the cost of installing additional piling?

At the A7669 site, the contractor elected to continue driving. As a result, each production pile was extended an additional 30 to 55 ft. and driven to bedrock where the design resistance was met at EOD (instead of allowing time for the pile to setup).

During the A7669 PLT, the test pile was installed to the specified embedment depth (Approximate El. 271 ft.) in the design. At EOD, the test pile was estimated to have an nominal resistance of 88.2 kip as shown is Table 7.1. The resistance estimated at EOD was approximately half (about 52 percent) of the design resistance (168 kips) of the pile. In accordance with the JSP, a restrike was performed 7 days after EOD. After the 7-day period, the pile restrike estimated a nominal resistance of 223 kips. From EOD to near BOR the nominal resistance of the pile increased approximately 153% and exceeded the design resistance by approximately 55 kips (about 33 percent). These results illustrate the importance of observing pile setup on clay deposits and confirm that the additional pile lengths installed by the contractor were not necessary.

6.2.2. Static Load Test – Nominal Resistance. The nominal resistance of each test pile was interpreted from the load-displacement curve using several methods, as shown in the Static Load Test Results sections of Section 5. Because AASHTO (2010) specifies the use of Davisson's (1972) method to interpret the nominal resistance from a QM static load test, the nominal resistance interpreted using this method was reported for comparison. In each PLT, nominal resistance interpreted using Davisson's (1972) method exceeded the specified (design) nominal resistance of the production piles in the structure's corresponding bent.

The capacities that compare well with the static pile load tests are close only at the BOR. Given that the test piles were tested days after the pile was driven to allow for the construction of the reaction frame, these results suggest the delay provided sufficient time for the excess porewater pressures to dissipate. As a result, the effects of pile setup observed at the BOR were also captured in the static pile load test. The difference determined from the static and dynamic tests of each site are shown in Table 6.2.

Table 6.2 Summary of Static and Dynamic Load Test Results

Bridge (geologic region)	Nominal Resistance (kips)			Difference (%)
	Static Load Test	Dynamic Testing		
		EOD	BOR	
A7956 Sikeston, MO (SE Lowlands)	182.0	164.6	184.1	± 1 %
A7669 Poplar Bluff, MO (SE Lowlands)	236.0	82.2	223.6	± 5 %

6.2.3. Static Load Test – Load Transfer Distribution. The results of the measured load transfer distribution of the CIP test pile at the Sikeston (A7956) site did not compare well to the estimated load transfer distribution results of CAPWAP wave matching analysis. During the first loading increments of the A7956 load transfer distribution plot (Figure 5.16) the load at the pile head was linearly transferred further down the pile length as expected. However, as additional load increments were applied, there was a significant decrease between the load measured at load cell and the load measured at VWSG #1. The low VWSG measurements could be explained by the considerable differences in elastic properties of the steel shell and backfilled concrete where the VWSGs are located. Although a bearing plate was used to distribute the applied load evenly across the test pile's cross section, if a small void existed between the bearing plate and the top of the concrete, the majority of the applied load would be transferred through the metal shell of the pile instead of the concrete. As a result, the VWSGs would only measure a portion of the entire magnitude of the strain.

It's anticipated that the interface between the steel shell and the concrete backfill could also be disrupting the strain from being fully transferred to the concrete. During the construction of the CIP test pile the concrete was not placed under pressure. Therefore, the only means for the concrete to create a solid contact with the test pile would be from its own dead weight. As a result, the lower gauges would be under more dead weight and possibly gain a greater contact between them and the steel shell (the load transfer does behave as expect from VWSG #3 through VWSG #5). However, without additional weight pushing down on the concrete around the VWSG #1 and VWSG #2, the interphase between the concrete and steel around these gauges may not be as strong. As the strain travels down the pile this weak interface would disrupt the full magnitude (of strain) from reaching the location of VWSG #1 and VWSG #2.

Overall the measured load transfer distribution from the A7669 PLT compared relatively well to the estimated load transfer distribution results of the CAPWAP wave matching analysis. Unlike the CIP test pile used at the A7956 site, the A7669 test pile was a steel H-pile. The A7669 load transfer plot (Figure 5.22) demonstrates that the applied load at the pile head was transferred relatively linearly with depth. The consistency between both the measured distributions and the estimated distributions may be due to the test pile consisting of only one material. In contrast to a CIP pile, there is no potential for strain losses to occur between different materials.

A comparison of the load-transfer results from the static and dynamic tests of each site are shown in Table 6.3.

Table 6.3 Load Transfer Distribution Results

Bridge (geologic region)	Test Type	Nominal Resistance (kips)		
		Total	Shaft	Tip
A7956 Sikeston, MO (SE Lowlands)	CAPWAP	184.1 (100%)	38.4 (21%)	145.7 (79%)
	PLT VWSG Data	182.0 (100%)	100.0 (55%)	82.0 (45%)
A7669 Poplar Bluff, MO (SE Lowlands)	CAPWAP	223.6 (100%)	151.9 (68%)	71.7 (32%)
	PLT VWSG Data	236.0 (100%)	188.0 (80%)	48.0 (20%)

It is important to note that the variation in the measured versus estimated load transfer distribution values from the CAPWAP analysis may also be a result of:

- The results of the CAPWAP analysis are an estimate of the actual nominal resistance (since high-strain dynamic testing indirectly predicts resistance), and
- The results of the CAPWAP analysis are dependent on the engineers judgment decisions made with performing the analysis. Because these decisions are based on knowledge and experience, they will differ person to person; thus the results of a specific CAPWAP analysis will differ as well.

6.3. CALCULATION OF RESISTANCE FACTORS

As stated in Section 1.2, MoDOT adopted the resistance factors from the AASHTO LRFD Bridge Design Specifications (2010) for designing bridge pile foundations in Missouri. Considering the variability in soil conditions and construction practices at the national level, the resistance factors recommended by AASHTO tend to be conservative when applied to localized regions (Roling et al., 2011). Given the data that had been collected during this research project, a back-analysis was performed to determine the actual resistance factors of the A7956 and A7669 sites based on the nominal resistances measured from each PLT. The following illustrates an example of the calculations using the results from the A7956 PLT. As shown in Equation 2.6 of Section 2, the LRFD criteria is expressed by the following equation:

$$\Sigma(LF)Q_n \leq (RF)R_n$$

where LF is the load factors,

Q_n is the nominal loads,

RF is the resistance factor, and

R_n is the nominal resistance.

For design, MoDOT sets the Maximum Factored Load $[\Sigma(LF)Q_n]$ equal to the Minimum Nominal Resistance $[(RF)R_n]$. From the A7956 structural design, the

Maximum Factored Load [$\Sigma(LF)Q_n$] per pile was 102 kips (Joseph Alderson, personal contact, November 21, 2013). To obtain the Nominal Resistance (R_n), the Maximum Factored Load [$\Sigma(LF)Q_n$] is divided by the resistance factor (RF). A resistance factor (RF) of 0.65 was used at the A7956 site since dynamic testing was used during installation. It's important to note that the $\Sigma(LF)Q_n$ is defined as the maximum load the pile must carry regardless of the resistance factor used, thus this value [$\Sigma(LF)Q_n$] is a constant. Knowing these parameters, the Minimum Nominal Resistance (used for the design) of each pile was calculated as follows:

$$R_{n\,(design)} = \frac{\Sigma(LF)\ Q\_n}{RF_{design}} = \frac{102\ kips}{0.65} = 157\ kips \tag{6.1}$$

However, the results of the static load test measured the $R_{n\,(measured)}$= 182 kips. Knowing the $\Sigma(LF)Q_n$ is a constant in the design, when the $R_{n\,(measured)} \geq R_{n\,(design)}$, the true resistance factor of the subsurface is greater than the one used in the design. As a result, linear interpolation can be used determine the measured resistance factor following:

$$\frac{R_{n\,(design)}}{RF_{design}} = \frac{R_{n\,(measured)}}{RF_{measured}} \Rightarrow \frac{157\ kips}{0.65} = \frac{182\ kips}{RF_{measured}}$$

Solving for $RF_{measured}$:

$$RF_{measured} = \frac{(182 * 0.65)}{157} = 0.75 \tag{6.2}$$

By substituting the $RF_{measured}$ into the fundamental LRFD equation, the additional Maximum Factored Load that the pile can effectively support can be calculated.

To summarize, the measured resistance was greater than the resistance used in the design. As a result, the uncertainty in the piles ability to resist the applied load is reduced. Therefore, the additional resistance of the test pile can be used to calculate the actual resistance factor of the site. The actual resistance factor at the A7669 site was calculated in the same manner. The calculated resistance factors are shown in Table 6.4.

Table 6.4 Calculated Resistance Factors

Bridge (geologic region)	Calculated Resistance Factor
A7956 Sikeston, MO (SE Lowlands)	0.75
A7669 Poplar Bluff, MO (SE Lowlands)	0.91

The calculated resistance factors at the A7956 and A7669 sites illustrate the test piles could support an additional 16% and 40% increase in the Maximum Factored Load of each design, respectively (at their current pile lengths). Although these results are site-specific, they suggest the AASHTO resistance factors used during pile design were conservative when applied to these regions. Based on these findings, the pile lengths or pile sizes could have been reduced and still met the reliability levels incorporated into the AASHTO LRFD criteria.

7. COMPILATION OF PILE LOAD TEST DATA

7.1. INTRODUCTION

Collecting data from static pile load tests allows for the pile's measured resistance from the load test to be compared with the pile's estimated resistance determined from various predictive methods analyzed during design. The comparison between these values can be used by:

- Designers to conduct more accurate and economical geotechnical design for foundations in their projects, and
- Researchers to develop more reliable and economical geotechnical design methods for foundation's (Abu-Hejleh, 2013).

Section 2.5 summarized a number of state DOT efforts to calibrate new resistance factors using PLT data within their respective states. Several researchers compiled the PLT data into electronic databases to increase the efficiency of the analysis procedures needed to effectively calibrate LRFD resistance factors. The following sections identify some of the factors that contribute to the overall design of a PLT database for LRFD.

7.2. PLT DATABASE CONSIDERATIONS

7.2.1. Comprehensive Data. Database design is largely driven by the requirements or needs of the user. Typically, the data requirements increase as the complexity of user's intensions increase. In any case, the database must be comprehensive enough to provide a distinct purpose and meet the user's objectives. The data requirements of a PLT database are developed by systematically identifying and prioritizing the extent of data needed to calibrate LRFD resistance factors for driven piles. In general, these data requirements are obtained from three portions of a PLT record: general, design, and testing. Figure 7.1 illustrates an example of the data requirements needed to calibrate LRFD resistance factors for driven piles.

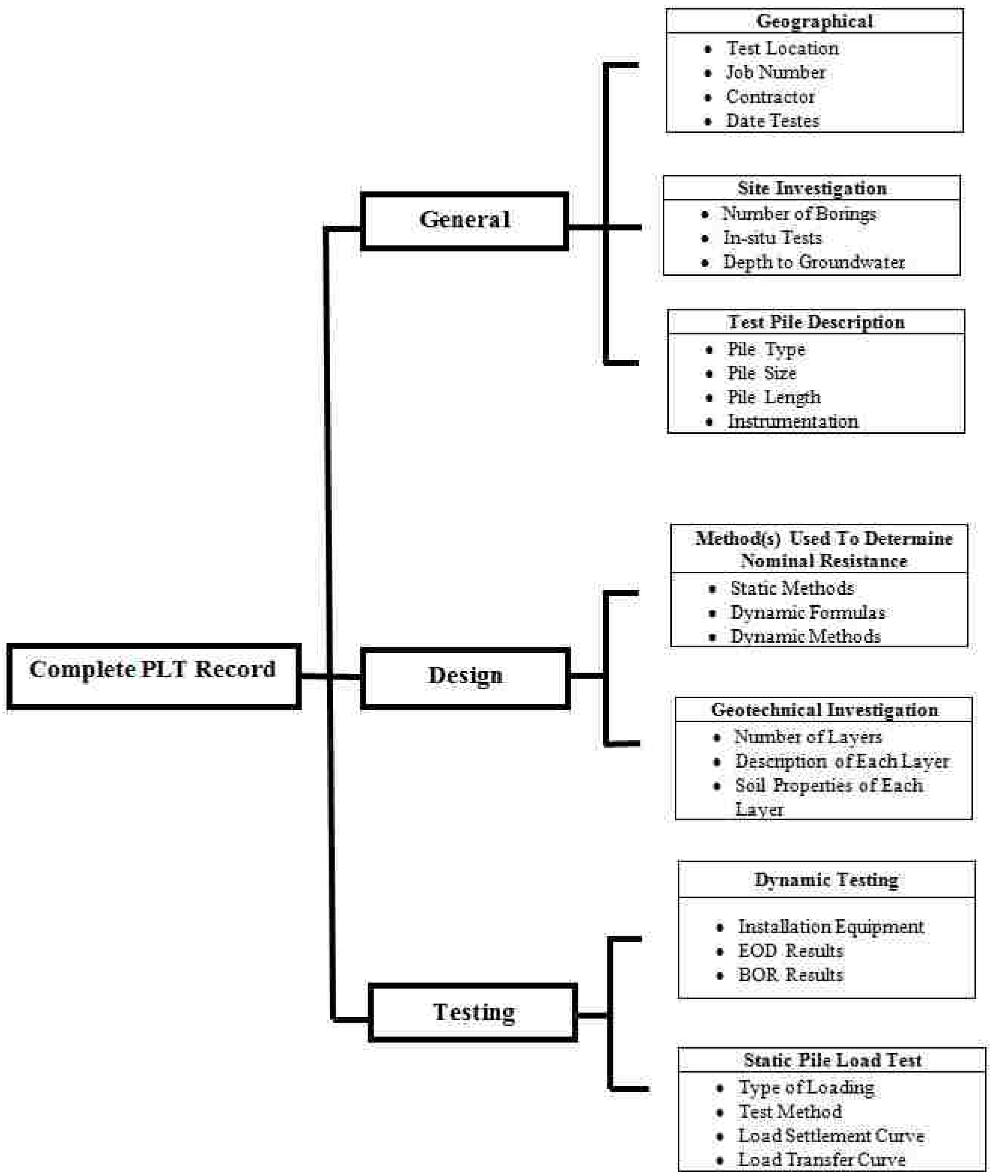


Figure 7.1 Data Requirements of a PLT Record

The following sections will briefly describe, in general, the data requirements for each portion of a PLT record displayed above in Figure 7.1.

7.2.1.1 General. The General portion of the record includes the metadata of each PLT record. Metadata refers to a set of data that describes or gives information about other data (National Information Standards Organization, 2004). In other words, metadata are typically values/parameters that describe or quantify the actual testing records. In a database, metadata provides the user information to locate or identify the corresponding PLT data record. Geographical metadata requirements include the when, where, and by who portion of the PLT record. Individual test information (i.e., job number, date, and so forth) becomes increasing important if a large number of tests were performed in a localized region. The data requirements of the test pile pertain to the type pile, construction method, and instrumentation details. These along with other properties like, length, diameter, and so forth are self-explanatory, but are critical of the PLT record. Metadata regarding the subsurface investigation provides information about the type and frequency of in-situ tests performed. Detailed subsurface investigation data also provides the user insight to the construction control at the test site. Comprehensive metadata provides the user the ability to locate records which are most fitting to their analysis. Figure 7.2 shows an example of the metadata requirements in the General portion of a PLT record.

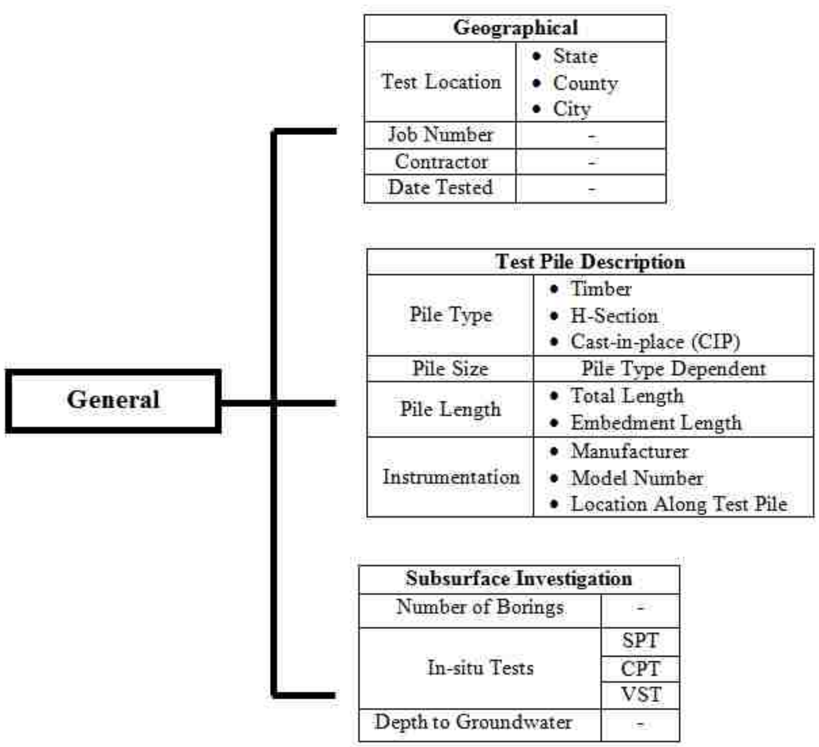


Figure 7.2 Example General Data Requirements of a PLT

7.2.1.2 Design. The data requirements of the pile design process are included in the Design portion of the PLT record. These requirements include:

- The measured parameters from subsurface investigation to define the soil conditions and determine the soil resistance near the test pile
- The estimated nominal resistance of the test pile (from one or more of the various analytical methods) based on the available soil resistance.

In general, to determine the nominal resistance of a pile using one (or more than one) of the common predictive methods (i.e., static methods, dynamic formulae, and dynamic methods) conventional subsurface information is required. These parameters may include, but are not limited to, the number of soil layers, a standardized description of each layer, and the available geotechnical properties of each layer. The resulting nominal resistance (predicted from one or more of these methods) is required for comparison with measured resistances obtained from the static load test to calibrate LRFD resistance factors. An example of the Design Information data requirements are shown in Figure 7.3.

Design

Geotechnical Analysis		
Generalized Soil Profile (i.e., Sand site, Clay site, Mixed Site)	Number of Layers	-
	Description of Each Layer	Depth USCS Classification ASTM Classification Average Insitu Test Results (N_{60}, q_u)
	Soil Properties of Each Layer	Unit Weight ϕ/s_u Lab Test Results

Method(s) Used To Determine Nominal Resistance		
Static Methods	• Alpha Method • Beta Method • Meyerhof • Norlund • Other	Resulting Nominal Resistance
Dynamic Formula	• ENR • Gates Formula • Modified Gates Formula	Resulting Nominal Resistance
Dynamic Methods	GRL WEAP	Resulting Nominal Resistance

Figure 7.3 Example Design Data Requirements of a PLT

7.2.1.3 Testing. The Testing portion of the PLT record includes the results of dynamic and static load tests. Comprehensive dynamic testing provides data containing:

- Description of the pile driving methods
- Description of installation equipment used for driving
- Predicted nominal resistance obtained at EOD and BOR (if available) from PDA and CAPWAP analyses.

The results from the dynamic load test are included in the Testing portion of the PLT record because they can be used (with a higher degree of reliability than analytical methods) to predict the nominal resistance of a pile when static load test results are not available.

The data requirements from the static load test include:

- Description of the test method and orientation of the applied load
- The nominal resistance interpreted from the load-settlement curve using one (or more) of the available methods
- The load transfer distribution (if available from instrumentation)

The measured resistance from the static pile load test will be compared with the pile's estimated resistance determined from various predictive methods analyzed during design. The comparison between these values is the basis for calibrating LRFD resistance factors. Figure 6.4 shows an example of the data requirements in the Testing Information portion of a PLT record.

7.2.2. Data Quality. Data quality is the perception or assessment of data's fitness to serve its purpose in a given context (Sivathanu, 2005). In a database, data quality refers to the accuracy and reliability of the stored data, providing the user assurance that the data displayed in the database represents a valid version (i.e., free of input errors) of its original form. A system to establish data quality is typically initiated during the design phase of a database through the use of standard procedures or guidelines for data input.

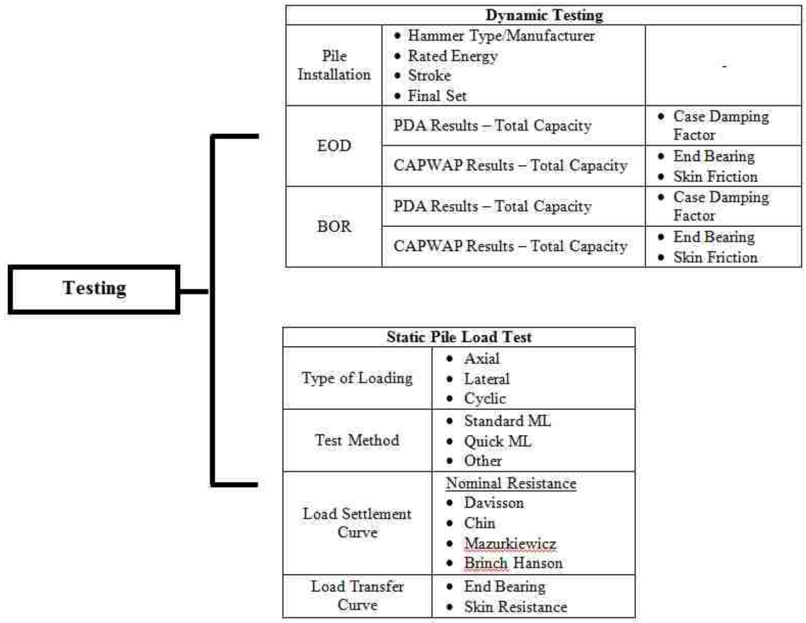


Figure 7.4 Example Test Data Requirements of a PLT Record

For example, according to the Abu-Hejleh (2013) the data for the DFLTD were manually organized in a series of paper input forms designed to reflect the database's tables. The data were then checked for validity, correctness, and manual data entry errors before they were added to the database. Once entered into the database from these forms, the data was reviewed again for input-errors (Abu-Hejleh, 2013). The data input processes in both the DFLTD and PILOT databases are strictly controlled by only providing access to designated individuals. Developing a series of input guidelines and regulating administrative access limits the databases vulnerability to inconsistencies and enhances the quality and integrity of the stored data.

7.2.3. Database Queries. Queries are the primary tool for retrieving information from the structured format of a database ("Query", 2011). The ability to form effective queries is one of the keys to developing a quality database. Database queries allow the user to ask questions to the database or use a filter to separate only the records that contain certain criteria of interest. Queries can be relatively broad or highly selective. However, the more restrictions the user implies through a query, the more the selective the results become. For example, the user wants to calculate LRFD resistance factors of static methods, for driven H-piles, in Missouri sandy soils. The user could begin by querying the database for PLT records performed in Missouri. From this basic query, a specific set of records is separated from the total information available in the database. Although this basic search separates the data records as the user intended, it has only separated records in the database using a single criterion. As a result, in a comprehensive database, these query results might be too broad to be efficiently evaluated for calibrating LRFD resistance factors. The ability to formulate an additional search and further refine the query results for records containing the specific attributes:

- H-piles,
- Driven in sandy soils, and
- Designed using static methods.

Having this ability provides the user with a data set that may better serve their initial requirements in a much more practical and efficient way.

7.3. AVAILABLE DATA SETS

Several of the data sets generated from the efforts summarized in Section 2.5 have been made available to the engineering community for future use. As MoDOT considers developing their own electronic PLT database to calibrate regional resistance factors for pile foundations in the future, the qualities and capabilities of the available data sets should be evaluated for inclusion. The following sections will describe data sets (from these projects and previous efforts in Missouri) that have been compiled to assist the effort to calibrate LRFD resistance factors.

7.3.1. FHWA Deep Foundations Load Test Database. As discussed in Section 2.4.2., the Deep Foundation Load Test Database (DFLTD) was used to calibrate the current national resistance factors provided by AASHTO. In 2003, the FHWA had to suspend the effort to continue developing and sustaining the DFLTD it due to unavailable funds and resources. In 2012, the FHWA evaluated the DFLTD in its current version (last updated in 2003) to see how the best value of the previous work could be realized with the available resources (Abu-Hejleh, 2013). During the course of this writing (October 2013) the FHWA distributed the current version of the DFLTD and its user's manual to all interested users. The DFLTD database and its user's manual are included in Appendix F.

7.3.1.1 Installation. To install the DFLTD, the user must locate the DFLTD V1.0 software included in Appendix F and follow the prompts to complete the installation. Once installed, the user can access the database through the DFLTD shortcut key automatically placed on computers desktop. (The database can also be accessed through the application file in program's folder).

When the FHWA's efforts were suspended in 2003, the current version of the DFLTD was used with the Windows™ XP edition operating system and the DFLTD data file was formatted in Microsoft Access™ 2007. The user should be mindful that select features of the DFLTD may not function properly due to incompatibilities between newer editions of Windows™ and Microsoft Office™. At the time of this writing, the DFLTD was installed and fully functional on computers with Windows™ XP and Microsoft Access™ 2007.

7.3.1.2 Overview. When the DFLTD is opened, the main screen presents a file menu and a horizontal toolbar containing four action buttons. These buttons allow the user to perform correlations, determine frequency distributions, determine statistics, and perform queries on the data records. The appended user's manual provides a detailed explanation of each toolbar feature.

The most significant feature of the DFLTD is its capability to create multiple-item queries. In the DFLTD each PLT record contains comprehensive details regarding:

- Location,
- Pile Properties,
- Load Tests,
- Site Investigation, and
- Soil Information.

Clicking the "User Query" button at the top of the Main Screen, the user can select parameters from five categorized tabs to query. Figure 7.5 illustrates the "User Query" screen in the DFLTD.

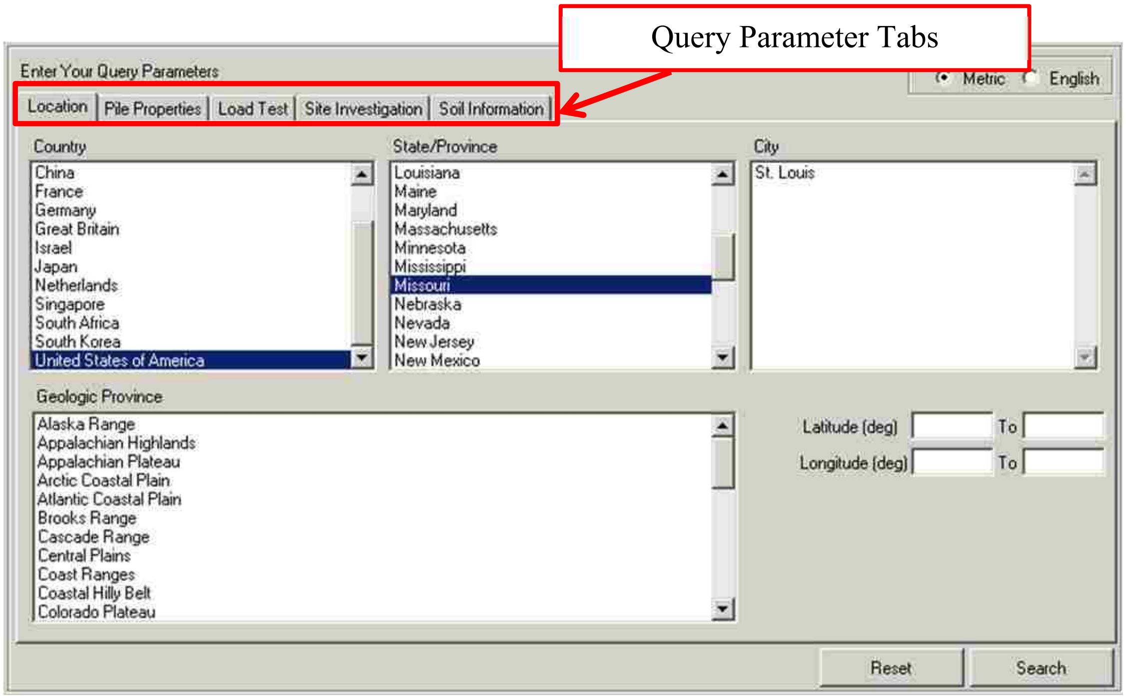


Figure 7.5 DFLTD User Query Window

To locate records which contain specific criteria the user can build a query to include (or exclude) only the parameters of interest. This type of query structure system is more valuable to users that need to locate very specific data. Once the query is performed, the results can then be downloaded into a .csv format file and imported into a spreadsheet for further analysis.

Before distributing the DFLTD to all interested users, the FHWA identified some of the recognized limitations of the DFLTD. Several of the most significant limitations presented by Abu-Hejleh (2013) include:

- In its current version, the DFLTD cannot be updated, expanded, or modified to include new information.
- Due to the storage and data-speed limitation during the initial development, the DFLTD only contains raw load test data. Supplementary text information and figures (i.e., construction plans, borehole logs) from the project were not stored.

- Descriptions of the procedures used during the subsurface investigation, construction of test foundations, and load testing are limited. In general, only the data requirements of PLT records are available.
- Information on the location of the groundwater table is not provided.

Although the DFLTD contains 1307 load test records, only the records collected from tests performed on driven piles in Missouri or Missouri's neighboring states are significant to this project. As a result, a query was performed to locate the records that match these criteria. The query results included two tests performed in Missouri and 17 performed in Missouri's neighboring states. These records contain valuable data and the ability to be immediately used for calibrating LRFD resistance factors in Missouri. Table 7.1 shows the distribution of the tests performed in Missouri and Missouri's neighboring states.

Table 7.1 Distribution of DFLTD PLT records from Missouri and Missouri's Neighboring States

Location	Number of Available PLT Records
Arkansas	1
Illinois	2
Iowa	4
Kansas	0
Kentucky	0
Missouri	2
Nebraska	4
Oklahoma	5
Tennessee	1

Despite its limitations, the DFLTD is the oldest developed database for load tests on deep foundations and still considered among the most comprehensive (Abu-Hejleh, 2013). Once the procedure and parameters needed to calibrate LRFD resistance factors in Missouri have been established in a future phase, the DFLTD will contribute several data sets to the effort.

7.3.2. Iowa State's PILOT Database. As discussed in Section 2.5.5., the PILOT database was developed with the specific objective of establishing both LRFD resistance factors and reliable construction control methods (i.e., development of new pile driving formulas) for driven piles. The database contains data from 264 pile static load tests conducted over a 24 year period (between 1966 and 1989) on steel H-piles, timber, pipe, monotone, and concrete piles driven in Iowa.

7.3.2.1 Installation. The most recent version of PILOT is publically available from Iowa State University's website ("Development of LRFD...", 2011). To download PILOT, the user must complete the PILOT Request Form on the webpage. Upon completion of the form, an electronic link to the database will be provided to the user through an email. The current version of the PILOT database was formatted in Microsoft Acesss™ 2007 and was last updated in February 2011. This version is included in Appendix F.

7.3.2.2 Overview. PILOT's user-friendly structure consists of two forms, the Display Form and the Pile Load Test Record Form (PLTRF). The Display Form is shown in Figure 7.6.

ID	County	Township	Lab Number	Project Number	Design Num	Contractor	Pile Type	Design Load	Date Driven	Date Tested	Test Site Soil
1	Black Hawk	Orange	AXP3-7	IY-520-6(8)--3P-07	1983	Lunda Construc	HP 10 X 42	32	12/9/1983	12/20/1983	Mixed
2	Johnson	Clear Creek	AXP3-9	I-380-6(44)243--01-52		A. M. Cohron &	HP 10 X 42	34	6/15/1973	6/20/1973	
3	Fremont		AXP3-10	FN-184-1(3)--21-36	173	A. M. Cohron &	HP 10 X 42	37	7/24/1973	7/26/1973	Mixed
4	Jones		AXP3-14	FM-38-3(7)--21-53	170	Grimshaw Con:	HP 10 X 42	37	8/21/1973	8/23/1973	Mixed
5	Jasper	Malaka	AXP4-2	BROS-9050(2)--8J-50	383	Herberger Con	HP 10 X 42	31	5/23/1984	5/30/1984	Clay
6	Decatur	Center	AXP4-3	BRF-2-5(10)--38-27	1082	Godberson - Sr	HP 10 X 42	35	6/18/1984	6/21/1984	Clay
7	Cherokee	Afton	AXP4-6	BRF-3-2(20)--38-18	683	Christensen Br	HP 10 X 42	35	11/21/1984	11/27/1984	Mixed
8	Linn	Rapids	AXP4-22	I-IG-380-6(57)259--04-57	1672	Schmidt Constr	HP 10 X 42	37	8/7/1974	8/15/1974	Mixed
9	Linn	Rapids	AXP4-23	I-IG-380-6(57)259--04-57	1672	Schmidt Constr	HP 10 X 42	37	11/14/1974	11/19/1974	Mixed
10	Ida	Garfield	AXP5-1	BRF-175-3(15)--38-47	383	Christensen Br	HP 10 X 42	36	6/18/1985	6/20/1985	Sand
11	Hamilton	Liberty	AXP5-2	DP-F-520-4(9)--39-40	1670	Christensen Br	HP 10 X 42	37	4/17/1975	4/22/1975	Clay
12	Linn	Clinton	AXP5-3	F-30-7(62)--20-57	1781	Schmidt Constr	HP 10 X 42	37	9/13/1985	9/18/1985	Clay
13	Delaware	Richland	AXP6-2	SP-603-0(3)--76-28	276	Grimshaw Con:	HP 10 X 42	37	3/11/1976	3/16/1976	Sand
14	Audubon	Hamlin	AXP6-3	FN-44-3(15)--21-05	176	Capital Constru	HP 10 X 42	37	5/28/1976	6/3/1976	Mixed
15	Cherokee	Cedar	AXP6-3	BRF-59-7(24)-38--18	1183	Christensen Br	HP 10 X 42	36	5/19/1986	5/28/1986	Clay
16	Osceola	Ocheyedon	AXP6-4	SN-720(7)--51-72	176	Koolker Inc.	HP 10 X 42	30	6/10/1976	6/15/1976	Mixed

Figure 7.6 PILOT's Display Form

The "Display Form" serves as the navigation page of PILOT and it's displayed immediately when the database is opened. This form allows the user to:

- View of all of the available PLT records,
- Create a new PLT record,
- Access additional details about the PILOT Database, and
- Apply preset queries to the data records.

By clicking the ID number of an individual test located on the Display Form, the test's PLTRF opens. The PLTRF in PILOT is a template that allows the user to input and organize the data of a specific PLT. In addition to the general information data fields included in the upper portion of the PLTRF, a series of nine tabbed subforms are included to organize the specific aspects of the record. For a detailed description of the database fields included in the PLTRF, refer to Roling et al. (2011). Figure 7.7 shows the location subforms included in each PLTRF.

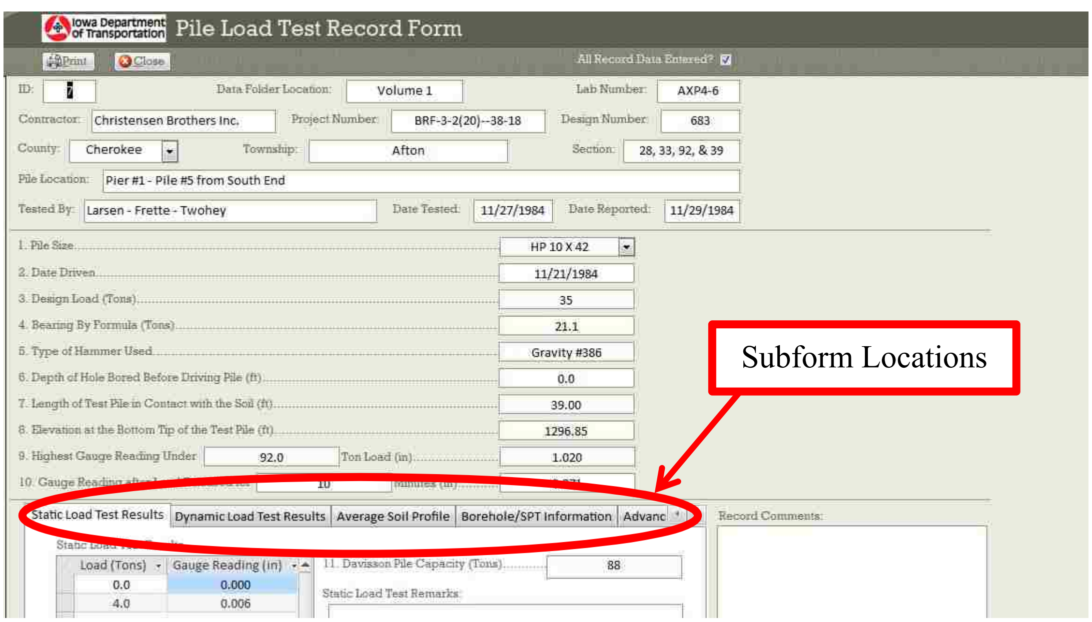


Figure 7.7 Location PLTRF Subforms

The most beneficial aspect of PILOT (not included in the DFLTD) is PILOT's capabilities to add, delete, and modify new and existing PLT records. To add a record the user can click the "New Pile Load Test" quick button on the "Display Form" and a blank PLTRF will appear for the user to populate. Conversely, PLT records can be deleted using the basic functions of Access™. Unlike the DFLTD, the data included in PILOT are unlocked. In other words, the user can modify existing records. Although this function allows the records to be updated if additional information becomes available, has the potential for the user to make unintended changes to existing data.

The most significant limitation of PILOT is its query system. Although the data in PILOT can be filtered by applying one of the 18 preset queries available on the Display Form, the user is limited to using one of the available preset queries and cannot build a query to meet their specific needs. In general, the preset queries search the database using one or two criteria (i.e., Steel H-piles in Sand, Usable-Static Wood Piles). If the user wants to locate records with additional criteria, they would be required to apply the closest preset query and manually eliminate the individual records that do not include the additional criteria. In a database containing hundreds of records, this process would not only be inefficient, but also impractical.

Although all of the PLTs in the PILOT database were performed in Iowa, these records are, at a minimum, more representative of Missouri's northern subsurface conditions than what was used to develop the resistance factors provided by AASHTO. The PILOT database will contribute several data sets to Missouri's effort once the procedure and parameters needed to calibrate LRFD resistance factors in Missouri have been established.

7.3.3. Missouri Previous Efforts. Section 2.6.1 summarized previous research efforts initiated to locate historical PLT data from MoDOT's records. However, only 10 records of pile load tests were available from MoDOT. According to Cravens (2011), "The PLT data collected was not well documented and the pile types that were tested were not representative of MoDOT's current pile used in practice." The available data from these records was organized in a Microsoft Excel™ spreadsheet which is included in Appendix F.

Each record contains information regarding general information, pile properties, pile driving equipment, and the resulting load-settlement curve of the PLT. There are, however, some recognized limitations in the data records that may prohibit their potential use in calculating LRFD resistance factors. Some of the recognized limitations include:

- Eight of the ten records were not tested to failure, resulting in load-settlement curves which do not reach a failure load (nominal resistance).
- Each record contains a generalized description of the surface soil and the toe bearing soil of the test pile. However, a complete description of subsurface and the data collected from in-situ tests performed during the site investigation are not reported.
- Each record contains the test pile's design resistance, but the methods used to determine the design resistance are not reported.

Based on the above limitations, it is unclear whether this set of data records contains the parameters needed to calibrate LRFD resistance factors. The data set will need to be reevaluated once the procedure and parameters needed to calibrate LRFD resistance factors in Missouri have been established in a future phase.

7.3.4. Current Research Project. All of the available information relating to the PLTs performed in Phase I of this research project have been organized and stored in the framework of the PILOT database. The add/delete records capabilities in PILOT allow for additional records can be included and existing records can be removed without effecting the structure of the database (performs the same way as PILOT). Using this availability, the Iowa-collected data was removed and the Missouri-collected data was used to populate the database until a Missouri PLT database is created. The Access™ database containing the records of the PLTs performed in Phase I of this research project is included in Appendix F.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1. CONCLUSIONS

The resistance factors included in the AASHTO LRFD specifications were developed from a collection of static pile load test data from around the U.S. For MoDOT to benefit from the advantages LRFD offers, research grade PLT data needs to be developed based on MoDOT's current practices.

The approach and methods of this research were conducted in an effort to achieve the appropriate levels of reliability for driven pile foundations in Missouri. The main objective of this research was to develop a research grade static pile load test data set from three construction bridge sites along the Missouri highway system within specific geologic regions. An effort to collect recent and available PLT data from Missouri's neighboring states was also conducted as part of this research and reported in Section 6 herein. Based on the results of the aforementioned tasks, some basic conclusions can be made:

- The pile load tests conducted so far have confirmed the nominal resistances predicted by the Dynamic Pile Testing (PDA/CAPWAP) at BOR.
- Davisson's (1972) method is proven to be the most common method for interpretation the nominal resistance from the static load-settlement curve. The ultimate capacities interpreted using Davisson's method compare well with the capacities obtained from the dynamic load test at BOR.
- Pile set-up after driving is a significant factor to consider in determining the need for a restrike. The additional resistance available following pile setup can have a substantial effect on the nominal resistance determined using dynamic methods. If in doubt, restrike.
- When BOR capacities are measured using dynamic methods they can be used with confidence for the calibration of resistance factors with respective pile types and geologic units.
- The AASHTO resistance factors are conservative when applied to Missouri soils. MoDOT will be unable to benefit from the advantages encompassed in LRFD

design until new LRFD resistance factors are calibrated based on the geology and construction practices used in Missouri.

- The appended data sets of available PLT data (from previous projects in Missouri and Missouri's neighboring states) contain additional valuable information for calibrating resistance factors for Missouri.

8.2. RECOMMENDATIONS

The results of this research indicate that improvements in MoDOT's practice for designing driven piles are essential to benefit from the advantages encompassed in LRFD design. The following items provide recommendations to be implemented as this project moves forward.

1. Additional research grade static pile load tests should be performed at ongoing construction bridge projects along the Missouri Highway System to increase the reliability and validity of the current data sets collected in Missouri. Further, the results of the PLTs performed as part of this study showed close agreement with the CAPWAP results at BOR. Additional PLT data sets need to be established to observe if this trend continues.

2. Pile setup is a significant factor in piles driven into clay deposits. Incorporating the effects of pile setup into design would provide the ability to reduce pile lengths and pile sizes that may not otherwise be considered.

3. The current language in the standard JSP should be adjusted to ensure the effects of pile setup are observed. MoDOT's current practice allows the contractor to continue driving when the minimum nominal resistance of a pile is not met at the minimum tip elevation and restrike testing is not included as a bid item. This methodology negates the importance of the restrike and often times results in unnecessary quantities of piling installed.

4. A standardized pile driving record needs to be kept during the installation of all piles (production and test) on MoDOT projects. The contents of this document needs to fully describe the project, location of the pile with respect to the structure, and blow-count per foot during installation of the test pile. Although data collected in a pile driving record are simple, they can be used to generally evaluate the consistency in the subsurface in the location of the piles.

5. The data sets that have been compiled from this project and others (i.e., DFLTD, PILOT, previous Missouri efforts) should be organized into a central database. Creating a database will be the most effective way to view and use the data that have been collected in an effort to calibrate regional LRFD resistance factors in Missouri.

APPENDIX A.

MODOT BRIDGE PLANS AND SPECIAL PROVISIONS ON CD-ROM

A.1 INTRODUCTION

Included with this thesis is a CD-ROM, which contains MoDOT bridge plans, MoDOT special provisions, static analysis results, GRL WEAP analysis results, dynamic testing reports (produced by Geotechnology, Inc.), unreduced static pile load data, and static pile load test results associated with each of the load tests performed during Phase I of this research project. A series of files containing pile load test data from other research projects are also included on the CD-ROM. Appendix A contains both the MoDOT bridge plans and the MoDOT special provisions associated with each of the load tests performed during Phase I of this research project. An outline of the contents of Appendix A on the CD-ROM is as follows.

A.2 CONTENTS

File Name	File Type
MoDOT Bridge A7669 Bridge Plans.pdf	Adobe PDF
MoDOT Bridge A7669 Special Provisions.pdf	Adobe PDF
MoDOT Bridge A7956 Bridge Plans.pdf	Adobe PDF
MoDOT Bridge A7956 Special Provisions.pdf	Adobe PDF

APPENDIX B.

STATIC ANALYSIS RESULTS ON CD-ROM

B.1 INTRODUCTION

Included with this thesis is a CD-ROM, which contains MoDOT bridge plans, MoDOT special provisions, static analysis results, GRL WEAP analysis results, dynamic testing reports (produced by Geotechnology, Inc.), unreduced static pile load data, and static pile load test results associated with each of the load tests performed during Phase I of this research project. A series of files containing pile load test data from other research projects are also included on the CD-ROM. Appendix B contains the static analysis results associated with each of the load tests performed during Phase I of this research project. An outline of the contents of Appendix B on the CD-ROM is as follows.

B.2 CONTENTS

File Name	File Type
MoDOT Bridge A7669 Static Analysis.xlsx	Microsoft Excel 2010
MoDOT Bridge A7956 Static Analysis.xlsx	Microsoft Excel 2010

APPENDIX C.

WEAP ANALYSES AND DYNAMIC TESTING REPORTS ON CD-ROM

C.1 INTRODUCTION

Included with this thesis is a CD-ROM, which contains MoDOT bridge plans, MoDOT special provisions, static analysis results, GRL WEAP analysis results, dynamic testing reports (produced by Geotechnology, Inc.), unreduced static pile load data, and static pile load test results associated with each of the load tests performed during Phase I of this research project. A series of files containing pile load test data from other research projects are also included on the CD-ROM. Appendix C contains the GRL WEAP analysis reports [produced by the Foundation Testing and Consulting, LLC (FTC)] and the dynamic testing reports (produced by Geotechnology, Inc.) associated with each of the load tests performed during Phase I of this research project. The GRL WEAP analyses (performed by the author) associated with each load test are included as well. An outline of the contents of Appendix C on the CD-ROM is as follows.

C.2 CONTENTS

File Name	File Type
MoDOT Bridge A7669 FTC WEAP Analysis Report.pdf	Adobe PDF
MoDOT Bridge A7669 Geotechnology Dynamic Testing Report.pdf	Adobe PDF
MoDOT Bridge A7669 MS&T WEAP Analysis.gww	GRL WEAP 2010
MoDOT Bridge A7956 FTC WEAP Analysis Report.pdf	Adobe PDF
MoDOT Bridge A7956 Geotechnology Dynamic Testing Report.pdf	Adobe PDF
MoDOT Bridge A7956 MS&T WEAP Analysis.gww	GRL WEAP 2010

APPENDIX D.

STATIC LOAD TEST DATA AND RESULTS ON CD-ROM

D.1 INTRODUCTION

Included with this thesis is a CD-ROM, which contains MoDOT bridge plans, MoDOT special provisions, static analysis results, GRL WEAP analysis results, dynamic testing reports (produced by Geotechnology, Inc.), unreduced static pile load data, and static pile load test results associated with each of the load tests performed during Phase I of this research project. A series of files containing pile load test data from other research projects are also included on the CD-ROM. Appendix D contains the unreduced static pile load test data and the static pile load test results associated with each of the load tests performed during Phase I of this research project. An outline of the contents of Appendix D on the CD-ROM is as follows.

D.2 CONTENTS

File Name	File Type
MoDOT Bridge A7669 PLT Unreduced Data Cycle 1.xlsx	Microsoft Excel 2010
MoDOT Bridge A7669 PLT Unreduced Data Cycle 2.xlsx	Microsoft Excel 2010
MoDOT Bridge A7669 PLT Unreduced Data Cycle 3.xlsx	Microsoft Excel 2010
MoDOT Bridge A7669 PLT Results.xlsx	Microsoft Excel 2010
MoDOT Bridge A7956 PLT Unreduced Data Cycle 1.xlsx	Microsoft Excel 2010
MoDOT Bridge A7956 PLT Unreduced Data Cycle 2.xlsx	Microsoft Excel 2010
MoDOT Bridge A7956 PLT Unreduced Data Cycle 3.xlsx	Microsoft Excel 2010
MoDOT Bridge A7956 PLT Results.xlsx	Microsoft Excel 2010

APPENDIX E.

PILE LOAD TEST DATA FROM OTHER RESEARCH PROJECTS ON CD-ROM

E.1 INTRODUCTION

Included with this thesis is a CD-ROM, which contains MoDOT bridge plans, MoDOT special provisions, static analysis results, GRL WEAP analysis results, dynamic testing reports (produced by Geotechnology, Inc.), unreduced static pile load data, and static pile load test results associated with each of the load tests performed during Phase I of this research project. A series of files containing pile load test data from other research projects are also included on the CD-ROM. Appendix E contains a series of pile load test data sets retrieved from other research projects. An outline of the contents of Appendix E on the CD-ROM is as follows.

E.2 CONTENTS

File Name	File Type
Deep Foundations Load Test Database (DFLTD) Application.exe	XML Configuration Software
DFLTD User's Manual.pdf	Adobe PDF
PIlot LOad Test (PILOT) database.accdb	Microsoft Access 2010
Previous MS&T Pile Load Tests Data.xlsx	Microsoft Excel 2010

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VITA

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