Evaluation of Pile Load Tests for Use in Missouri LRFD Guidelines

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FINAL REPORT

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<tr><td colspan="4">16. Abstract
This research studied the use of pile load tests for the development of resistance factors in the LRFD design specifications currently adopted by the Missouri DOT. To this end, three (3) bridge sites were identified to conduct a pile load test in conjunction with the normal bridge contracting activities. This allowed a side by side comparisons with production piles and the impact to the design. Since most of the issues with pile driving and pile capacities are for friction piles that require dynamic testing, the pile load tests locations were selected in geologic regions with relatively deep firm ground or bedrock. In this way, an end bearing condition was avoided. Two sites were selected in the Southeastern Lowlands region and one in the Northern Glaciated Plains region. The pile load tests yielded high resolution data for both the load capacity and load transfer distributions. The different pile and soil types produced different degrees of pile setup and these conditions were examined with a series of restrikes, as necessary. Full displacement piles in overconsolidated clays had unexpected results and the highest restrike capacities compared to the other sites. All the static and dynamic pile load tests produced capacities higher than the ones estimated by the design engineers. The resistance factors back calculated based on the pile load test data generated higher resistance factors than the 0.65 suggested by AASHTO LRFD specifications. Additionally, pile load tests data from other sources in Missouri and the neighboring states were collected and compiled for future use by the Missouri DOT.</td></tr>
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1. Introduction

1.1. Project Goal

The overall goal of this project was to evaluate the effectiveness of pile load tests within the current efforts of calibration of resistance factors for driven piles used in state highway bridges in Missouri.

1.2. Research Objectives

- To develop research grade static pile load test data sets for driven piles on bridge sites of the Missouri highway system. The test piles were driven in accordance to industry and MoDOT standards including the special provisions for dynamic pile testing.
- To collect data of recent and available pile load tests in Missouri and neighboring states.

1.3. Scope of Work

This project completed five specific tasks in close collaboration with MoDOT and the Missouri S&T team. The tasks described below are also part of the project task order issued by MoDOT and it is associated with the project timeline.

Task **1 –** Literature Review and Data Collection – The topics related to this research were searched in the published literature. Additionally, neighboring states were contacted to explore the current practices. The investigators made an effort to obtain data from all sources available and willing to share data.

Task 2 – Design of Pile Load Test Program – The bridge site locations and pile types were selected according to the immediate needs by the MoDOT, as shown in the table below:

Job No.	Bridge No.	County & Route	Letting Date	Remarks:
J0P2239	A7956	Scott Co., Rte. 91	Mar. 2012	14” CIP with dynamic testing
J0P0959	A7669	Butler Co., Rte. 67	May 2012	14” or 16” CIP with dynamic testing
J2S0787	A7932	Livingston Co., Rte. V	Aug. 2012	20” and 14” (end bents) CIP with PDA testing

The design of a static pile load test for each of these bridge sites was completed in conjunction with the construction of the newly designed bridge.

Task 3 – Pile Installation and Load Test - The installation of all the driven piles was conducted by the same bridge contractor. For the production piles the dynamic testing and wave equation analysis results were the responsibility of the contractor, as part of the special provisions for the bridge construction. Additionally, the contractor was responsible for the rigging and construction of the load frame according to the plans and specs prepared in Task 2. Dynamic testing of the test piles was the responsibility of the subcontractor, Geotechnology, Inc. Missouri S&T performed the load test, instrumentation, and data collection. After the testing was complete, the contractor was responsible to breakdown the static pile load test setup, cutting piles, and grading according to plans and specifications.

Task 4 – Interpretation and Reporting – The results were interpreted as the data became available and complete and presented in the progress quarterly reports. This final report provides a summary of the Missouri S&T team activities on the project and detailed, technical resources for review and use by MoDOT geologists and engineers. This report also provides background details on the pile load tests (static and dynamic) and the data collected in Task 1 from other pile load tests in neighboring states.

Task 5 – MoDOT Review and Revisions **-** This task was a period for MoDOT to review the final draft report and provide feedback to the research team and conduct revisions, as necessary.

1.4. Organization of Report

This report is organized in six (6) sections. Following the technical documentation cover page, Section 1 Introduction includes the goals, objectives, and scope of work. Then, Section 2 presents some background information and literature review. A summary of the pile load test data collected from other studies is included in Section 3 and other sources referenced. Section 4 describes the pile load test program and the details of the three bridge sites selected for this study. The results and discussion are included in Section 5, followed by conclusions and recommendations in Section 6.

2. Background and Literature Review

2.1. Load and Resistance Factor Design

Load and Resistance Factor Design (LRFD) is an alternative design method that has progressively developed specifically for bridges since the mid-1980s. LRFD became 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 factor of safety (FS) as in Allowable Stress Design (ASD) because loads and resistances have considerably separated and unrelated sources of uncertainty (Becker, 1996). For instance, the 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 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 criterion is expressed by the following equation:

$$\Sigma(LF)Q_n \leq (RF)R_n \qquad (2.1)$$

where, LF = load factors,
Q_n = nominal loads,
RF = resistance factor, and
R_n = 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 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 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 that all new bridges initiated after October 1, 2007 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.

The current MoDOT Engineering Policy Guide (EPG) 751.36 for Driven Piles includes resistance factors to be used in the Load and Resistance Factor Design (LRFD). These resistance factors were adopted from the current AASHTO LRFD Bridge Manual (2010). The resistance factors at the national level tend to be more conservative and closely tied to the level of effort and engineering conducted during the installation of the foundation elements (static pile load and dynamic method tests).

Table 2.1 - AASHTO LRFD Guidelines for Resistance Factors

Condition or Resistance Determination Method	Resistance Factor (ϕ)
Using Pile Load Tests and PDA/CAPWAP (>2% tested)	0.80
Using Pile Load Tests only (one per site condition)	0.75
Using PDA/CAPWAP on All production piles	0.75
Using PDA/CAPWAP analysis (>2% tested)	0.65
Wave equation analysis (with hammer performance)	0.50
FHWA-Gates dynamic pile formula	0.40
ENR dynamic pile formula	0.10

A new series of resistance factors were previously developed by researchers from the University of Missouri (Rolla and Columbia campuses). For driven piles, the resistance factors were calibrated based on existing data from historical construction records of dynamic testing of piles.

That is, pile driver analyzer (PDA) data and Case Pile Wave Analysis Program (CAPWAP) software. No records of static pile load test data were available to evaluate the actual ultimate capacity of the piles. Therefore, the resistance factors obtained from these previous research efforts are strictly based on the assumption that the dynamic methods provide actual ultimate capacity values. Several neighboring states are also engaging in projects to build databases of pile load tests that can be used to calibrate the resistance factors for similar or common geologic regions. To that end, data that is currently available at other DOTs can also be useful, if it is of good quality.

2.2. Various States LRFD Implementation 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 to which state DOTs had implemented LRFD bridge foundation design. AbdelSalam et al. (2009) 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 et al. (2009).

2.2.1. 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 factors. 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:

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

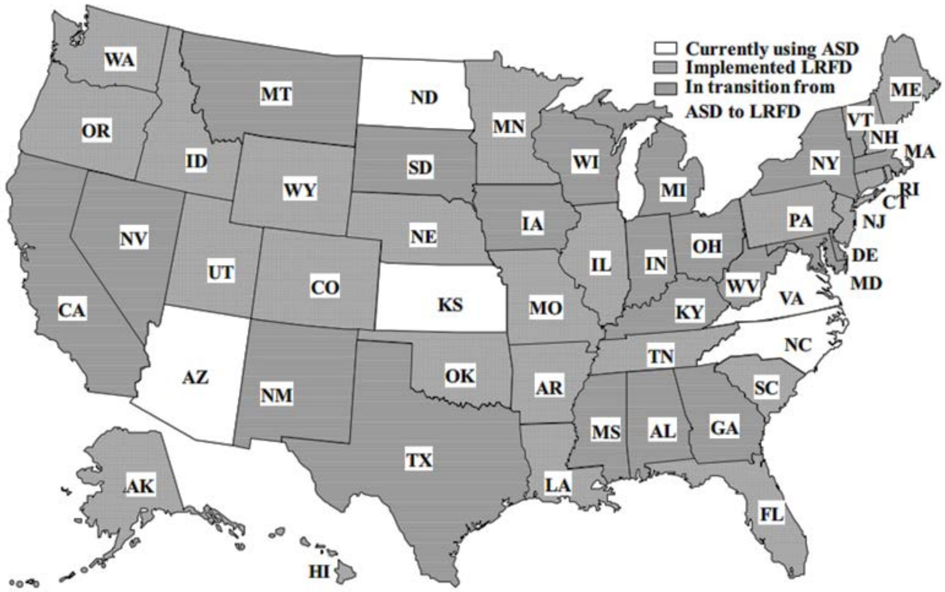


Figure 2.1 Extent of LRFD Implementation Following Oct. 1, 2007 Deadline (AbdelSalam et al. 2009)

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.

2.2.2. Illinois

The Illinois Department of Transportation (IDOT) used to estimate 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 for Transportation (ICT) 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.
ICT 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 only 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 on this study, including pile data, statistical analysis, and the development of resistance factors, refer to Long et al. (2009a).

2.2.3. 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 containing both adequate subsurface information and results from 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 data, and 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.

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).

2.2.4. Wisconsin

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 (Beginning of Restrike, 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).

2.2.5. Iowa

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 (from 1966 to 1989) on steel H-piles, timber, pipe, monotube, 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 for IowaDOT bridges. This study focused on creating an electronic database of the historical IowaDOT pile load test 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 effective calibration of regional LRFD resistance factors. For detailed information of both the methods evaluated and the results determined in this study, refer to AbdelSalam et al. (2009) and Roling et al. (2010).

2.3. Previous Studies in Missouri

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 initiated its first research project to optimize design from both an economic and safety point of view.

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 Driving 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 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 shown 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 previously calibrated set of resistance factors were thus, established under the strict assumption that dynamic testing methods provide the actual nominal resistance values.

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

Notes: Beta values (β) are reliability indices associated with a probability of failure. These β-values were agreed upon consensus with the MoDOT. For example, β=3.0 is associated with a probability of failure (P_f) of 0.1%.

2.4. Missouri DOT State of the Practice for Driven Piles

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.

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 over 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 appears in Section 751.36.3 of the EPG. A flow-chart of this process is shown in Figure 2.3.

2.4.1. 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.

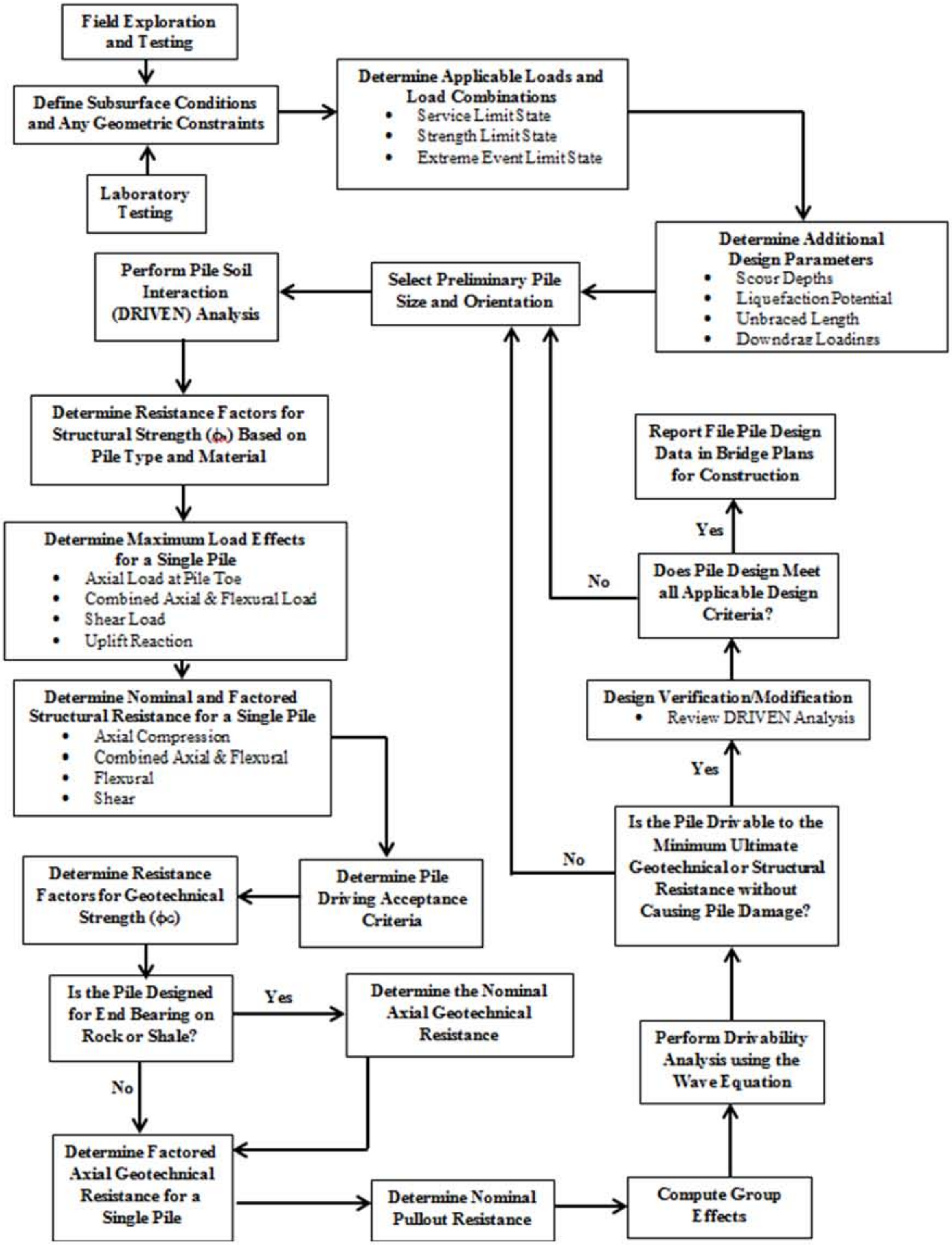


Figure 2.3 - Interpreted Flow Chart of MoDOT Pile Design Process (based on MoDOT, 2013)

2.4.2. Static Predictive Ultimate Capacity Methods

Once the preliminary pile type, size, and orientation have been determined, MoDOT uses the FHWA provided software DRIVEN as its primary analytical method to determine the ultimate capacity of the pile. When bedrock is located at great depths, DRIVEN is used to estimate both pile length and the pile resistance for friction piles. However, when bedrock is not located at great depth, DRIVEN is used only to estimate pile length in one of two situations:

- 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].)

- 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 in skin friction these materials provide.)

2.4.3. 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 2.3 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 2.3 - MoDOT Pile Structural Resistance Factors

Resistance Condition		Resistance Factors for Structural Strength (ϕ_S) per Pile Type	
		Steel Shell	H-Piles
Axial Resistance in Compression Subject to Damage Due to Severe Driving Conditions		0.6	0.5
Axial Resistance Compression Under Good Driving Conditions		0.7	0.6
Combined Axial and Flexural Resistance of Undamaged Piles	Axial	0.8	0.7
	Flexural	1.0	1.0

2.4.4. 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 2.4 lists the geotechnical resistance factors MoDOT adopted from AASHTO (2010) for each resistance condition.

Table 2.4 - 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)

2.4.5. 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.

2.4.6. 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 2.5. Substitute manufacturers of PDA software and hardware are permitted by special provision.

Table 2.5 - 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

notes: (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 the 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 2.6 illustrates the minimum restrike durations typically used by MoDOT.

Table 2.6 - 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.

2.4.7. 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 a hammer, but also present a proposal of the PLT procedures and arrangement following ASTM D 1143 (2007). These proposals are reviewed by MoDOT and approved accordingly.

2.4.8. Missouri Geology

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. These four geologic 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 2.4 illustrates the general delineation of these geologic regions. This project focused only on two of these regions, **Southeast Lowlands** and **North Glaciated Plains**. In these regions, most of the bridges on deep foundations rely primarily on side friction (friction piles). Specific description of the geologic region at each of the load test sites conducted for this research project are discussed in Section 4 of this report.

Figure 2.4 - Missouri's Geologic Regions (Saville, 1962)

3. Load Test Data Collection Efforts

3.1. Previous Efforts

Since the initial project in 2008, the Principal Investigator (PI) has collected dynamic and static pile load test results. 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, the researchers distributed a survey questionnaire to neighboring state DOTs 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). 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) PDA data associated with PLTs
  - CAPWAP dynamic testing data associated with PLTs

A summary of the survey results is included in Table 3.1. Table 3.1 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.

Table 3.1 - Results of Neighboring State Questionnaires (adapted from Cravens, 2011)

State	Response	LRFD Resistance Factors	Common Pile Type				Common Predictive Method				Performs PLT in their state	Provided PLT Data
			H-Pile	Concrete	CIP	Timber	Static Method	Dynamic Formula	WEAP	Dynamic Testing		
AR	YES	AASHTO Recommended	YES	YES	YES	NO	--	ENR	YES	PDA CAPWAP	NO	NO
OK	NO										YES	
KS	NO										NO	
NE	YES	AASHTO Recommended	YES	NO	YES	NO	DRIVEN	ENR	--	PDA CAPWAP	NO	NO
IA	NO										YES	
IL	YES	--	--	--	--	--	--	--	--	--	YES	NO
KY	NO										YES	
TN	YES	AASHTO Recommended	YES	YES	YES	NO	--	NO	NO	NO	YES	YES

notes: (--) where dashes appear there was an ambiguous response, not clear yes/no

3.2. Current Status

MoDOT has performed PLTs in the past; however, these PLTs were not implemented with comprehensive research objectives or for use in calibrating resistance factors. Therefore, they 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 needed 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 was 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 section will discuss only the activities completed as part of Phase I.

3.3. Available Data Sets

Several of the data sets generated from the efforts summarized in this section 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.

3.3.1. FHWA Deep Foundations Load Test Database

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 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). In 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 E.

3.3.1.1 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 the 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^{TM} XP edition operating system and the DFLTD data file was formatted in Microsoft Access^{TM} 2007. The user should be mindful that select features of the DFLTD may not function properly due to incompatibilities between newer editions of Windows^{TM} and Microsoft Office^{TM}. For this effort, the DFLTD was installed and fully functional on computers with Windows^{TM} XP and Microsoft Access^{TM} 2007.

3.3.1.2 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 3.1 illustrates the "User Query" screen in the DFLTD.

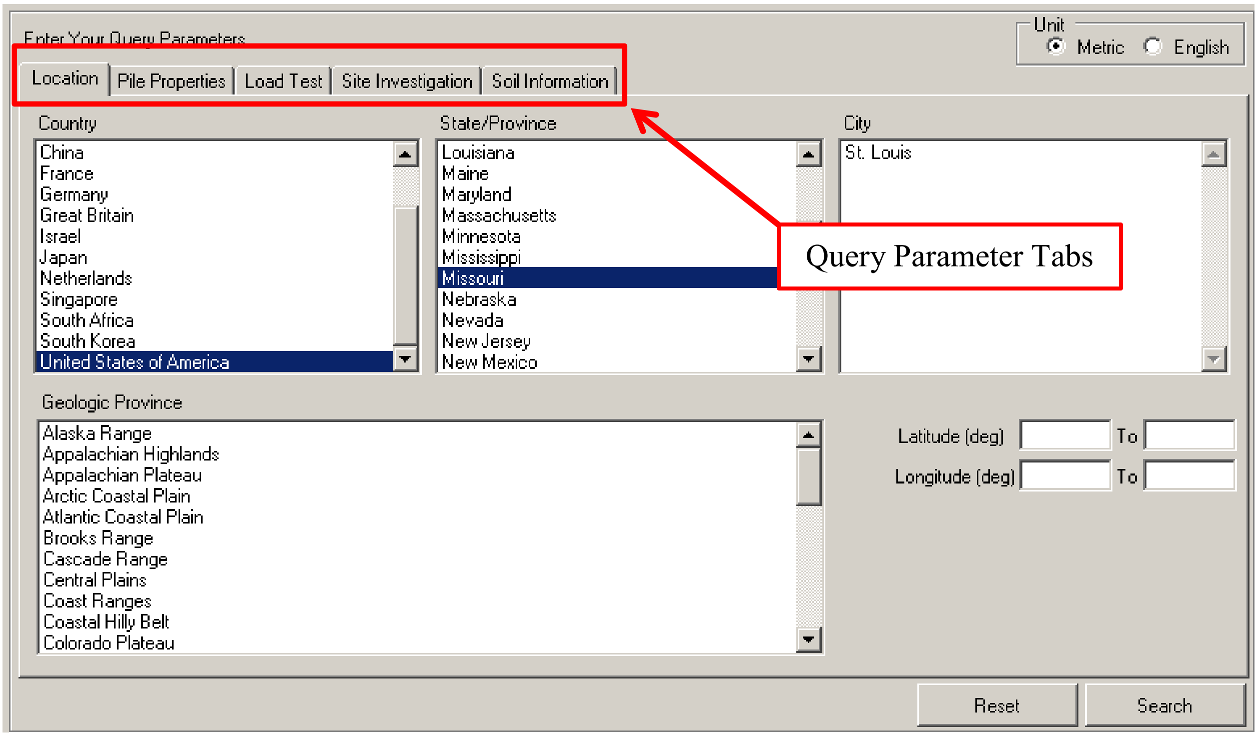


Figure 3.1 - DFLTD User Query Window

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 3.2 shows the distribution of the tests performed in Missouri and Missouri's neighboring states.

Table 3.2 - Distribution of DFLTD PLT Records from Missouri and 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 is 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.

3.3.2. Iowa State's PILOT Database

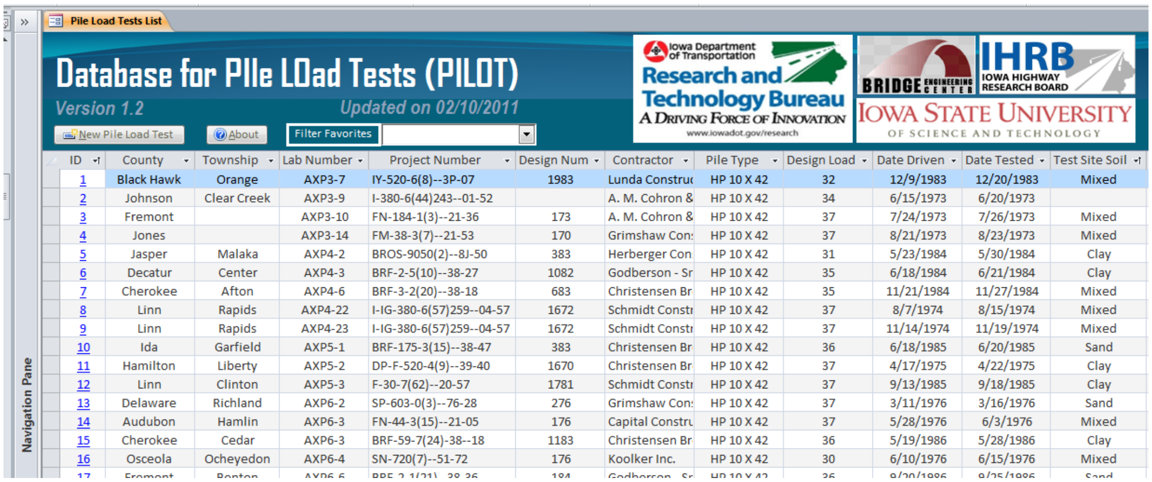
The PILOT database was developed with the specific objective of establishing both LRFD resistance factors and reliable construction control methods (i.e., development of new pile driving formulas) for driven piles. The database contains data from 264 pile static load tests conducted over a 24 year period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

3.3.2.1 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^{TM} 2007 and was last updated in February 2011. This version is included in Appendix F.

3.3.2.2 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 3.2.


Pile Load Tests List

Database for PIle LOad Tests (PILOT)

Version 1.2

Updated on 02/10/2011

New Pile Load Test

About

Filter Favorites

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 3.2 - PILOT's Display Form

The “Display Form” serves as the navigation page of PILOT and it is 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 3.3 shows the location subforms included in each PLTRF.

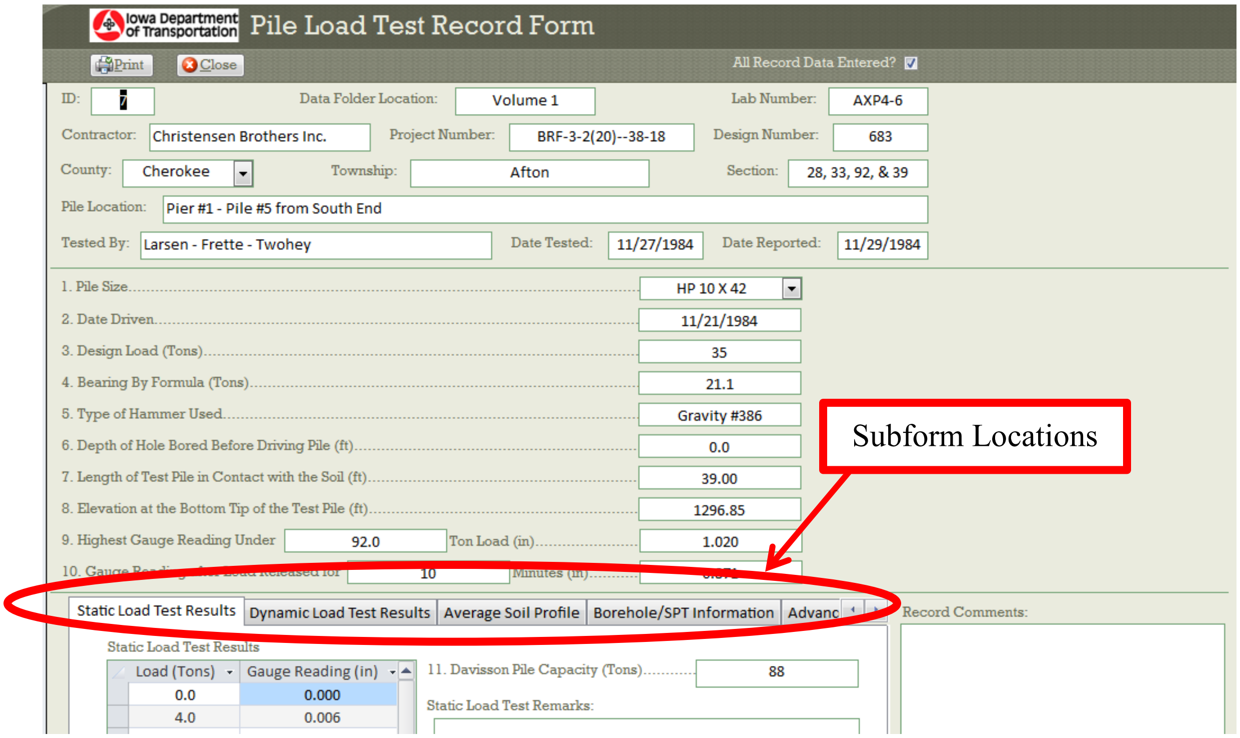


Figure 3.3 - Location PLTRF Subforms

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^{TM}. 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, it also 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.

3.3.3. Missouri Previous Efforts

Section 3.1 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.

3.3.4. 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 to be included and existing records can be removed without affecting 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 limited Missouri PLT database was created. The Access™ database containing the records of the PLTs performed in Phase I of this research project is included in Appendix F.

4. Load Test Program

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.

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 the 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 fastened 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.

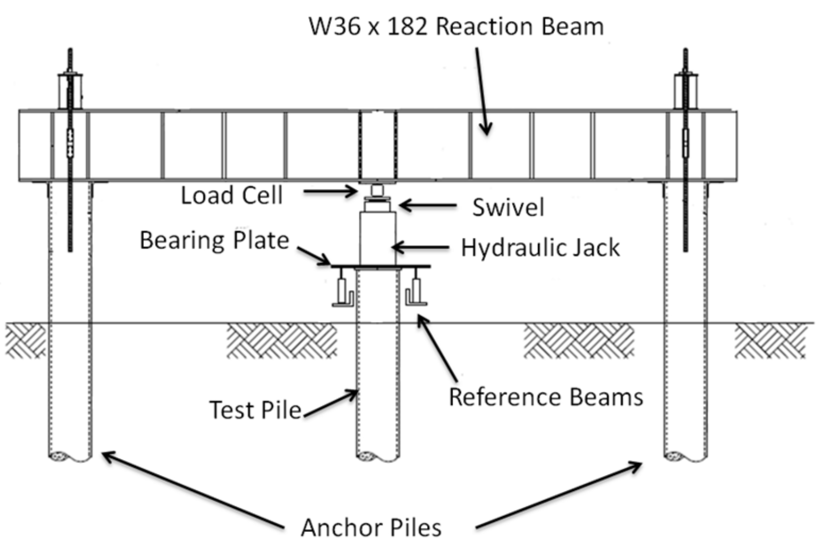


Figure 4.1 - Diagram of the Pile Load Test Components (Not to Scale)

4.3. Supporting Instrumentation

In conjunction with the applied load measurements of displacement at the pile head were also collected. These measurements are required for all pile load tests. Measurement of changes in strain along the test pile were also collected to determine the distribution of load transfer with depth and 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 was then be recorded by a data acquisition system (DAS). Section 4.4 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 (which 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 mount onto 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

Placement of the reference beams was 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.

4.3.3. Incremental Strain

Each of the test piles were instrumented with five to 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.

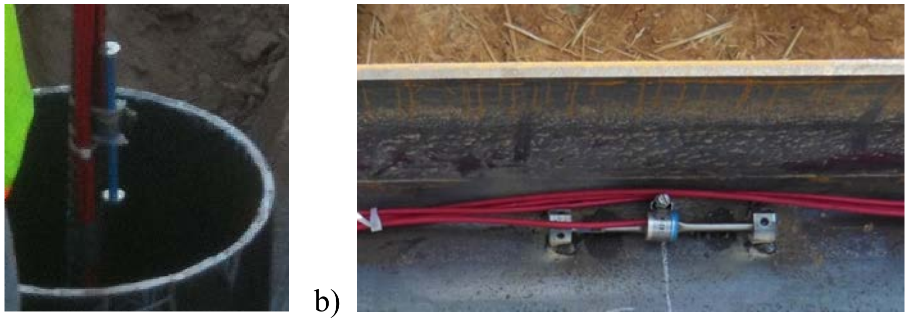
Figure 4.3 - Orientation of reference beams with respect to load frame

4.3.3.1 Concrete Embedded VWSGs

Geokon Model 4200, concrete embedded 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. A description of the installation procedures for each bridge site is included in Section 5.

4.3.3.2 Weldable VWSGs

Geokon Model 4000, weldable VWSGs were used to instrument the steel section of an H-pile. These gages were welded along the pile’s web and covered with a steel section for protection during installation. The description of the weldable VWSG installation process is provided in Section 5. 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, the measurements were recorded with the electronic data acquisition system discussed in Section 4.4. In the event that any of the electronic components malfunctioned, a supplementary measuring system was installed 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 mounted on the reference beams were similar to the LVDTs and measured the pile head 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 needs 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 DAS

The base platform for this data acquisition system was selected to be the Compact RIO platform by National Instruments (NI) based on the previously described load test requirements. Once this basic platform was designed, the individual system components were selected according to the anticipated types and quantity of instrumentation 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. 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 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 laptop/PC software (NI’s LabVIEW graphical programming tool) controls the data acquisition box. 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 System 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).

With the data collected, the user specifies through the laptop interface, whether the data is saved 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 provide redundancy in the event a component malfunctioned (Kershaw, 2011).

A series of additional components added to the data acquisition box make the system easier to use in the field. An AC to DC power converter allows the system to use 120 to 240 volt supplies from either typical outlets or generators (Kershaw, 2011). Power conditioners also add to the system to produce a constant power flow to the controller. A channel board mounted on the carrying case's lid holds 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 matches 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, portable by a single person. Figure 4.5 is a photograph of the completed data acquisition system.

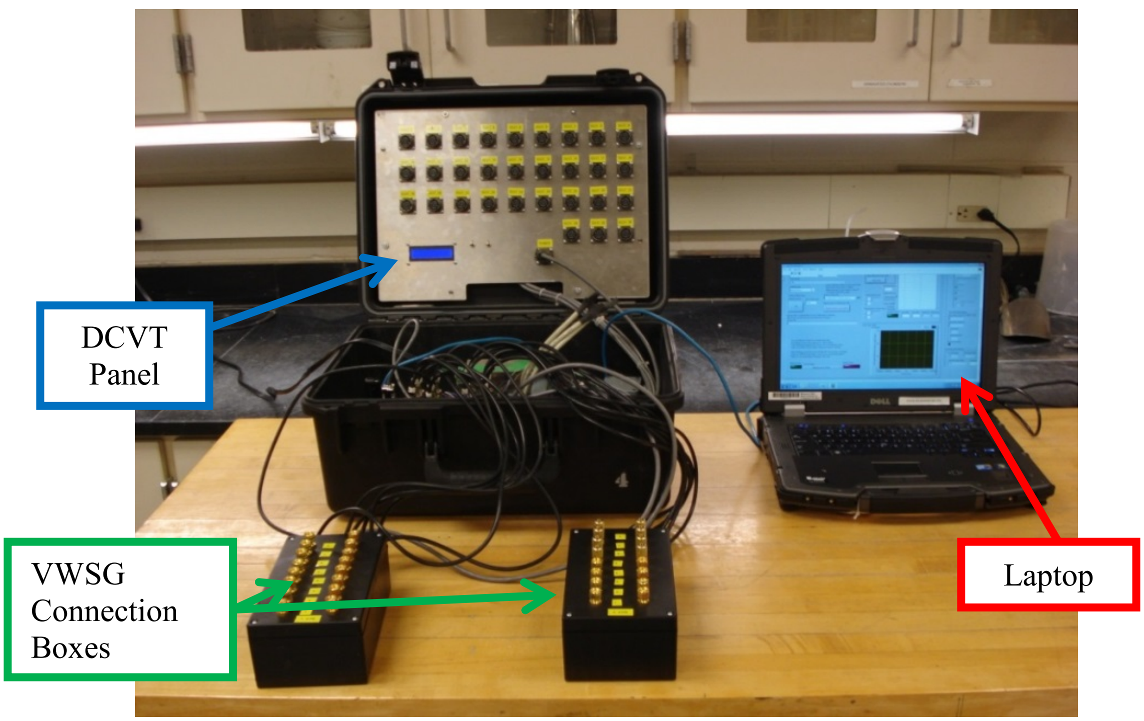


Figure 4.5 - Data Acquisition System Peripherals

4.5. Dynamic Monitoring Procedure

Geotechnology, Inc. (of St. Louis, Missouri) conducted dynamic monitoring as each test pile was driven into the soil. Prior to testing, two strain gauges and two accelerometers were mounted two pile diameters below the pile head. 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 indicates the location within the data acquisition system where the instruments connect 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

The data (i.e., readings from the load cell, LVDTs, and VWSGs) were recorded in the data acquisition system during the actual load test. Missouri S&T field personnel recorded the mechanical analog instrumentation (pressure and mechanical dial gages) readings manually.

In general, application of load followed 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 simply a single 200% cycle. Conducting the loading procedure in this manner allowed the pile's behavior to be recorded at different magnitudes of loading. Additionally, this procedure ensured data quality checks. The load increased on increments of 12.5% design load by the hand-pump until the digital readout connected to the load cell verified the corresponding applied load. Each load increment held constant for approximately 5 to 10 minutes depending on the pile's ability to sustain the current load increment. The next load increment was applied in a similar manner after the holding period elapsed. The test pile was incrementally unloaded after the maximum cycle load. 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 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:

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

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:

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

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.

4.8. Description of Pile Load Test Sites

The site locations of each pile load test (PLT) were MoDOT's most immediate needs. To that end, MoDOT identified three bridge projects along the Missouri highway system to be let on 2012 or thereafter. Due to the range of the subsurface conditions within Missouri, each test site was located in a different geologic region within the state. Figure 4.6 below shows the locations of each test with respect to Missouri's geologic regions discussed herein.

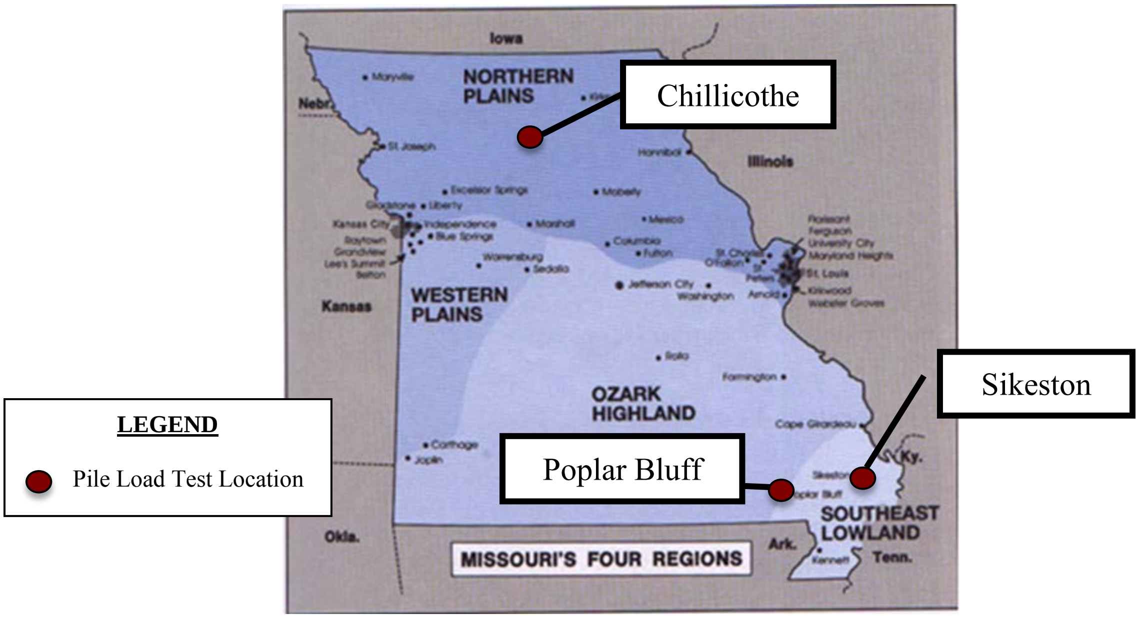


Figure 4.6 - Static Pile Load Testing Locations

4.8.1. Bridge Site 1: Sikeston, Scott County

The first pile load test associated with the MoDOT A7956 bridge replacement is 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 4.7 shows the approximate location of the construction site. (Latitude/Longitude: 37°02'18.93"N/89°40'40.98"W).

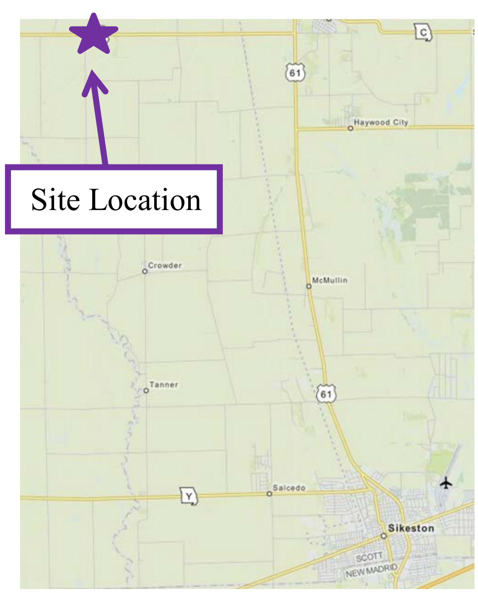


Figure 4.7 - A7956 Site Location Map (Google Maps, 2013)

4.8.1.1 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 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 4.8 shows a construction drawing of the proposed structure and select bridge plans are included in Appendix A.

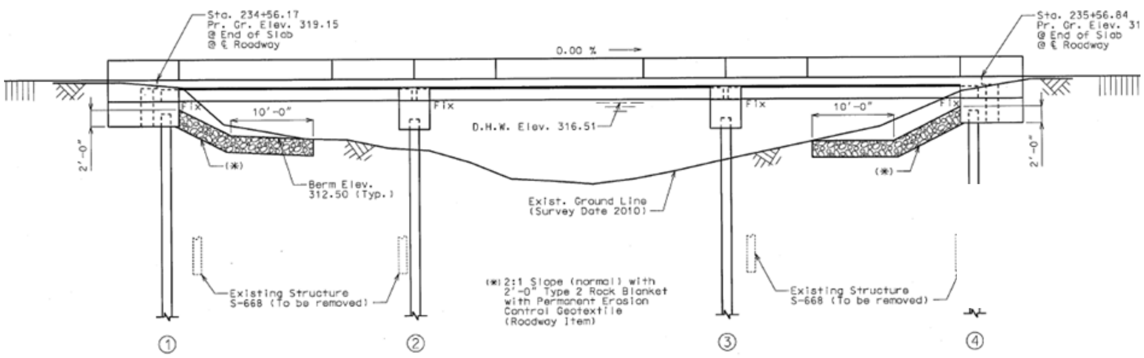


Figure 4.8 - MoDOT Illustration of the Proposed Structure (MoDOT, 2013)

The new foundation system included 14-inch cast-in-place (CIP) piles in each bent, 50 to 60 feet in length. Other superstructure components consisted of prestressed concrete box girder spans and precast prestressed concrete panels supported on concrete abutments. Table 4.4 shows the foundation data for the proposed structure.

Table 4.4 - A7956 Foundation Data (adapted from MoDOT Plans, included in appendix)

	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

4.8.1.2 Subsurface Conditions

The subsurface characterization was performed by MoDOT prior to the initiation of this research 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.

4.8.1.2.1 Geology

The site's geology was consistent with the 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.

4.8.1.2.2 Soil and groundwater

The subsurface soil conditions consist 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 extends to depths of about 4 feet. Below the lean clay, brown, medium dense, 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 4.9 shows the subsurface profile used in the WEAP analysis. It should be noted, that the sand was separated into two layers based on SPT N-values, solely in an attempt to refine the static analyses performed.

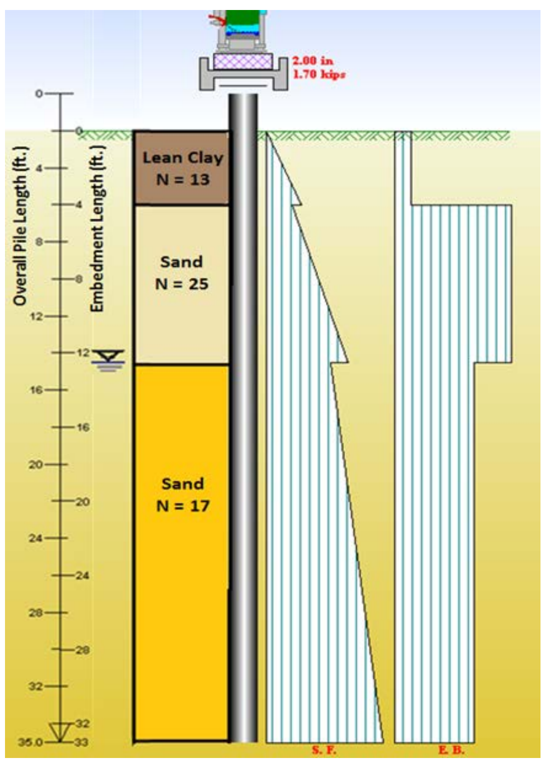


Figure 4.9 - A7956 Soil Profile along the Test Pile

4.8.1.3 Static and Wave Equation Analyses

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.

4.8.1.3.1 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 an energy 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 335 kips.

4.8.1.3.2 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 layers outlined in the description. 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 on 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, typically this is compared to 90% of the yield strength (F_y) of steel. 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 is shown in Table 4.5.

Table 4.5 - A7956 WEAP Analysis Results for Gain/Loss Ratios at the Shaft and Toe
(a) 0.8/1.0 and (b) 1.0/1.0

Gain/Loss 1 at Shaft and Toe 0.800 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
6.0	115.7	2.5	113.2	15.1	24.731	-0.138	6.24	18.2
12.0	122.2	9.0	113.2	16.6	25.312	-0.126	6.35	17.9
18.0	93.6	17.1	76.6	11.8	23.495	-0.183	5.78	18.3
24.0	103.2	26.6	76.6	13.7	24.272	0.000	6.00	17.9
30.0	114.4	37.9	76.6	16.0	25.243	0.000	6.31	17.7
32.9	120.6	44.0	76.6	17.4	25.605	0.000	6.43	17.5
33.0	120.7	44.1	76.6	17.4	25.626	0.000	6.44	17.5

a) Total Continuous Driving Time 9.00 minutes; Total Number of Blows 440

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
6.0	116.3	3.1	113.2	15.3	24.805	-0.173	6.26	18.1
12.0	124.5	11.3	113.2	17.1	25.550	-0.062	6.39	17.8
18.0	97.9	21.3	76.6	12.6	23.977	-0.146	5.88	18.2
24.0	109.8	33.3	76.6	14.9	25.089	0.000	6.20	17.9
30.0	123.9	47.3	76.6	18.1	25.964	0.000	6.49	17.4
32.9	131.6	55.0	76.6	19.9	26.371	0.000	6.64	17.2
33.0	131.7	55.2	76.6	20.0	26.375	0.000	6.63	17.1

b) Total Continuous Driving Time 10.00 minutes; Total Number of Blows 471

4.8.1.4 Anchor Pile and Test Pile Installation

The reaction frame and test piles at the A7956 site were installed by CBCC on June 26, 2012. 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 driven piles used a Delmag D19-32 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 had an embedment depth of 30 feet, resulting in a stick-up height of five feet to construct the rest of the frame. Figure 4.10 shows the reaction piles and the test pile being installed.

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 to the 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 VWSG instrumentation cables to pass through to the DAS box.

Figure 4.10 - A7956 Test Pile Installation

4.8.1.5 Dynamic Testing

Following 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 next section of this report.

4.8.1.6 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 4.11 shows the center bar being lowered into the test pile.

Figure 4.11 - Installation of the Center Bar and VWSGs

Concrete was placed within the test pile to complete its construction. To avoid damage of the VWSGs the concrete was placed from the bottom of the pile upwards by the use of a tremie pipe. A series of 4 inch PVC pipes were used to avoid the gages. The slump of the concrete was increased by adding water to allow the concrete to flow more easily through the PVC tremie 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 4.12 illustrates the concrete placement process.

Figure 4.12 - Process of test pile concrete placement. (a) Centerbar lowered into test pile, (b) PVC tremie lowered around VWSGs, (c) Begin concrete placement, (d) PVC tremme removed and shortened with sawzall, (e) PVC tremie re-lowered into test pile, (f) Resume concrete placement, and (g) Concrete placement finished.

4.8.1.7 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 described previously in this section. The A7956 load test setup and reaction frame are shown in Figure 4.13.

Figure 4.13 - Completed A7956 Pile Load Test Set-up

4.8.2. Bridge Site 2: Poplar Bluff, Butler County

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 4.14 shows the approximate location of the construction site (Latitude/Longitude: 36°41'36.19"N/90°28'46.72"W.)

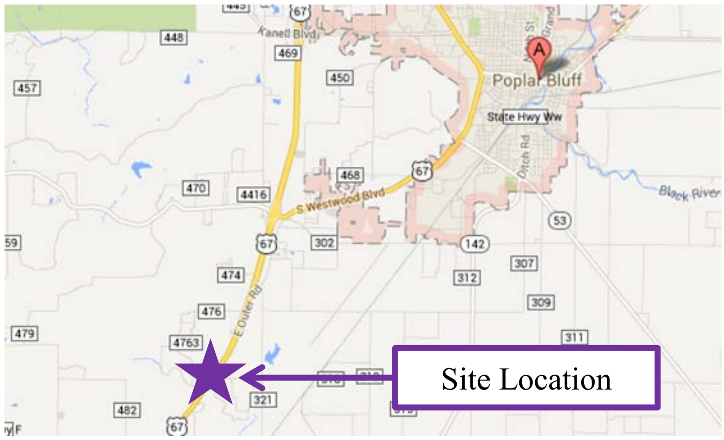


Figure 4.14 - A7669 Site Location Map (Google Maps, 2013)

4.8.2.1 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 Cane Creek Overflow. Figure 4.15 shows a construction drawing of the proposed structure.

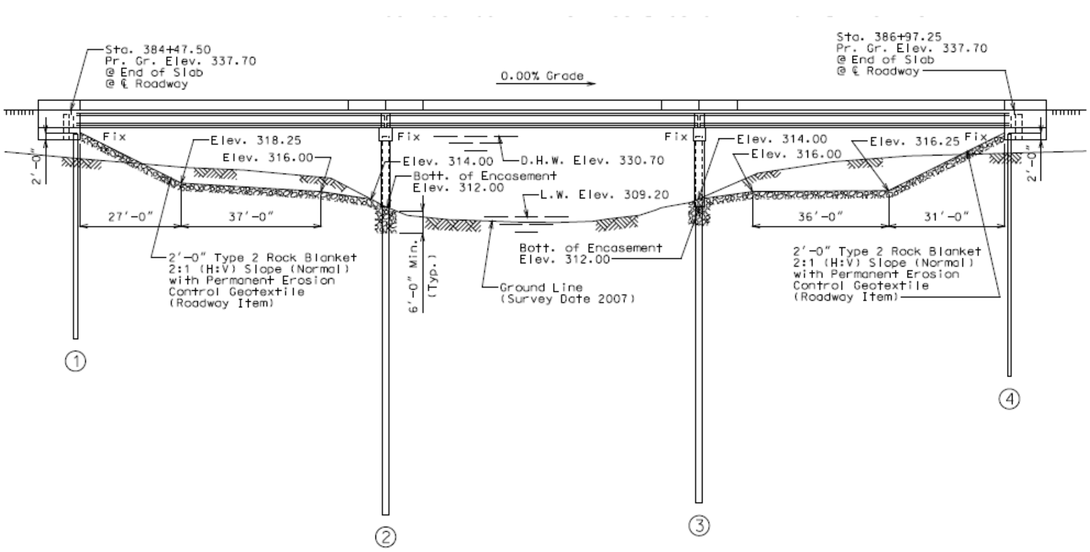


Figure 4.15 - MoDOT Illustration of A7669 Proposed Structure (MoDOT, 2013)

The foundation system included 14x73 steel H-section piles at the outer abutment bents and 20-inch CIP piles in the intermediate bents. Table 4.6 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.

Table 4.6 - A7669 Foundation Data (adapted from MoDOT Plans, included in appendix)

	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

4.8.2.2 Subsurface Conditions

The subsurface characterization was performed by 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.

4.8.2.2.1 *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 the description of the Southeast Lowlands region discussed in Section 2. 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.

4.8.2.2.2 *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 was 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 4.16 shows the subsurface conditions modeled for the WEAP analysis.

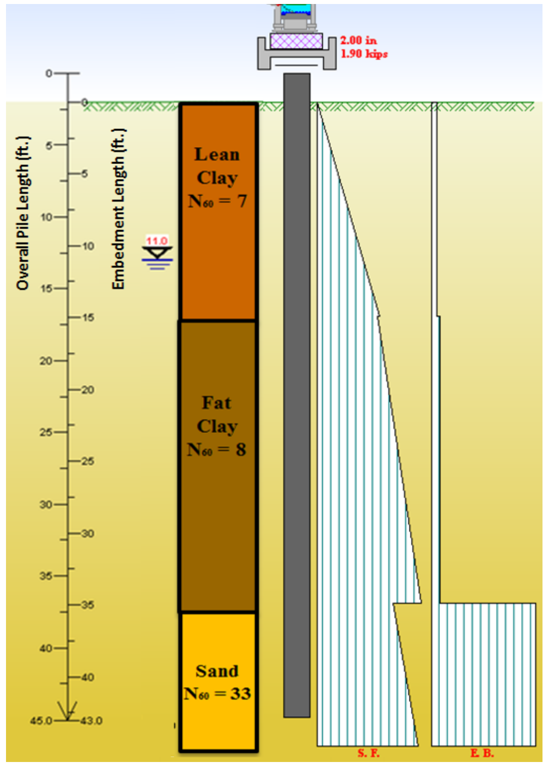


Figure 4.16 - A7669 Soil Profile Along Test Pile

4.8.2.3 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. 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.

4.8.2.3.1 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.

4.8.2.3.2 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 layers outlined in Figure 4.16. 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 233.4 to 255.7 kips depending on 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, typically this is compared to 90% of the yield strength (F_y) of steel. 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 4.7.

Table 4.7 - A7669 WEAP Analysis Results for Gain/Loss Ratios at Shaft and Toe (a) 0.8/1.0 and (b) 1.0/1.0

Gain/Loss 1 at Shaft and Toe 0.800 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
6.0	9.8	2.0	7.8	1.3	6.162	0.000	3.34	21.5
12.0	16.0	8.2	7.8	1.7	10.226	-0.542	3.74	23.6
16.5	25.9	15.0	10.9	2.5	13.632	-1.845	4.14	22.7
21.1	34.7	23.8	10.9	3.6	16.012	-2.388	4.54	21.5
21.8	36.0	25.1	10.9	3.7	16.293	-2.411	4.59	21.3
22.4	37.3	26.4	10.9	3.9	16.608	-2.474	4.64	21.1
27.0	49.0	38.1	10.9	5.5	18.591	-2.001	5.02	20.0
29.9	57.1	46.2	10.9	6.7	19.377	-1.418	5.20	19.2
32.7	66.0	55.1	10.9	8.0	20.279	-1.287	5.45	18.7
37.3	214.4	70.1	144.3	41.2	25.903	-0.167	7.34	16.6
38.8	218.8	74.5	144.3	43.1	26.005	-0.139	7.39	16.6
40.2	223.4	79.1	144.3	45.2	26.104	-0.114	7.44	16.5
43.0	233.4	89.1	144.3	49.8	26.329	-0.068	7.54	16.5

(a) Total Continuous Driving Time 10.00 minutes; Total Number of Blows 472

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
6.0	10.3	2.6	7.8	1.3	6.276	-0.124	3.42	22.0
12.0	18.0	10.2	7.8	1.8	11.145	-0.881	3.82	23.7
16.5	29.6	18.7	10.9	2.9	14.813	-2.215	4.34	22.2
21.1	40.6	29.7	10.9	4.3	17.191	-2.367	4.76	20.8
21.8	42.2	31.3	10.9	4.6	17.505	-2.305	4.81	20.6
22.4	43.9	33.0	10.9	4.8	17.808	-2.244	4.87	20.5
27.0	58.5	47.6	10.9	6.9	19.564	-1.243	5.24	19.1
29.9	68.6	57.7	10.9	8.5	20.565	-0.988	5.51	18.6
32.7	79.8	68.9	10.9	10.2	21.493	-0.672	5.77	18.1
37.3	232.0	87.7	144.3	47.8	26.451	-0.183	7.53	16.6
38.8	237.5	93.2	144.3	50.7	26.578	-0.160	7.58	16.5
40.2	243.2	98.9	144.3	53.4	26.700	-0.137	7.64	16.5
43.0	255.7	111.4	144.3	59.4	26.928	-0.075	7.74	16.4

(b) Total Continuous Driving Time 12.00 minutes; Total Number of Blows 559

4.8.2.4 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 4.17 shows the installation of the test pile.

Figure 4.17 - 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.

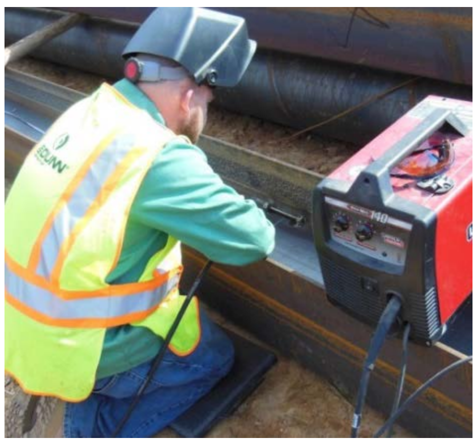
4.8.2.5 Dynamic Testing

Following 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 the following Section 5.

4.8.2.6 Test Pile Instrumentation

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 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 4.18 illustrates the instrumentation process of the HP 14x73 test pile.

a)  b) 

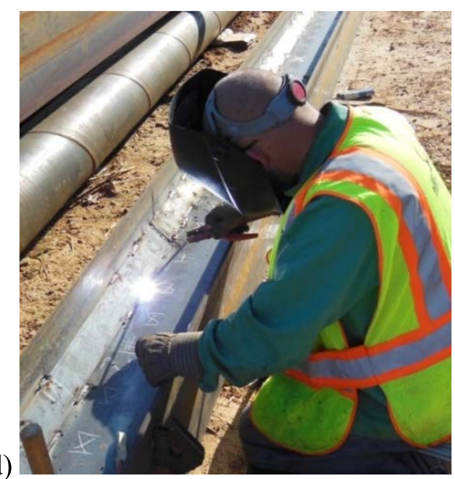
b)  d) 

Figure 4.18 - H-Pile Instrumentation Process
(a) Welding VWSG Mounts, (b) Installing VWSGs,
(c) Securing Gage Wires with All-Purpose Caulk, and (d) Welding Steel Strap Over Gages.

4.8.2.7 Static Load Testing

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 previously in this section. The A7669 load test setup and reaction frame are shown in Figure 4.19.

Figure 4.19 - Completed A7669 Pile Load Test Set-up

4.8.3. Bridge Site 3: Chillicothe, Livingston County

The third pile load test was conducted at the MoDOT A7932 bridge site located approximately half a mile east of Missouri Hwy. 65 along Polk Street. The bridge connects Highway V with the town of Chillicothe as an overpass for the railroad tracks. The site topography consisted of flat grassy fields with some occasional trees and brush. The pile load test location was located approximately 90 feet to the south of Bent 3 within the MoDOT right-of-way. Figure 4.20 shows the approximate location of the construction site (Latitude/Longitude: 39°47'48.99" N / 93°32'35.37" W). The test pile location was at the same level as the railroad tracks at El. 785 ft.

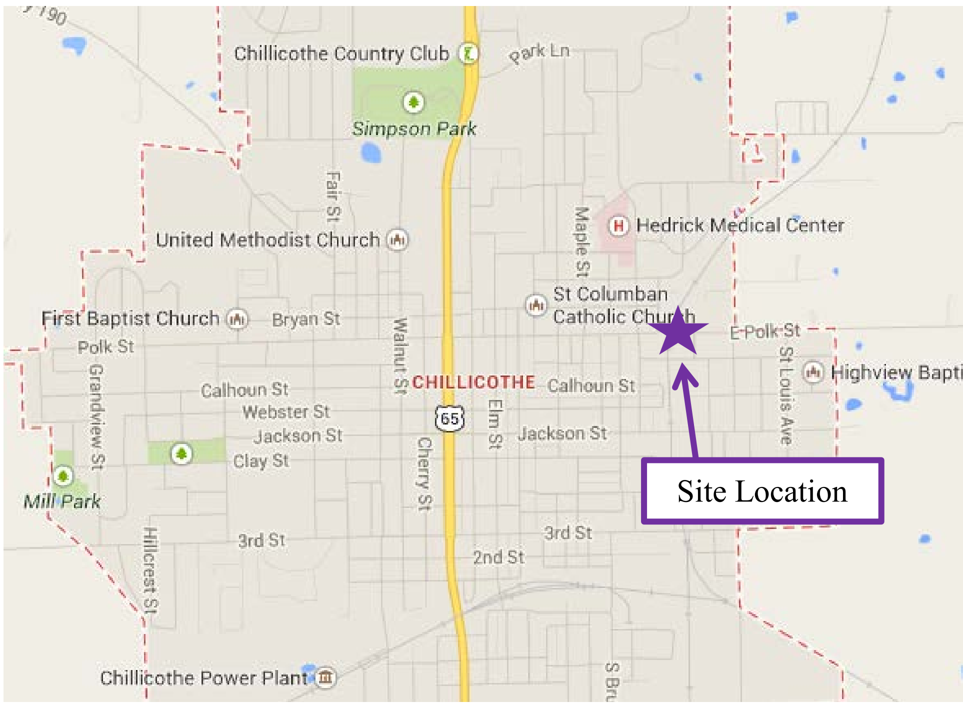


Figure 4.20 - A7932 Site Location Map (Google Maps, 2013)

4.8.3.1 Site and Project Description

The new structure was part of a highway improvement project to replace the railroad overpass bridge south of the new bridge locations. It is a two-lane, three-span bridge to support traffic coming from Highway V and crossing the railroad tracks into the town of Chillicothe. Figure 4.21 shows a construction drawing of the proposed structure.

The foundation system included a total of forty 14-in CIP steel pipe piles at the outer and intermediate bents. Table 4.6 summarizes the foundation data for each bent of the new structure. The superstructure consisted of pre-stressed concrete beam girder spans and precast pre-stressed concrete panels. The contractor for the project was APAC-Missouri of Clinton, Missouri.

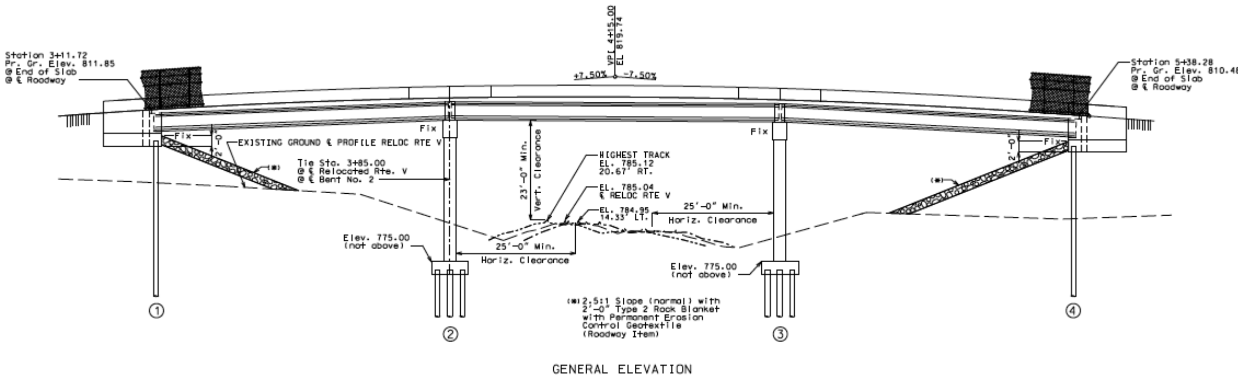


Figure 4.21 - MoDOT Illustration of A7932 Proposed Structure (MoDOT, 2013)

Table 4.6 - A7932 Foundation Data (adapted from MoDOT Plans, included in appendix)

	Bent No.	1	2	3	4
Driven Pile	Pile Type and Size:	14" CIP	14" CIP	14" CIP	14" CIP
	Number:	7	18	18	7
	Approx. Length (ft):	55	48	48	50
	Minimum Nominal Axial Compressive Resistance (kip)	220	232	232	220

4.8.3.2 Subsurface Conditions

The subsurface characterization was performed by MoDOT prior to the initiation of the project. Two borings, designated V-11-05 and V-11-04 were drilled for the end bents only (one and four), respectively. Approximate ground surface elevations at the boring locations were 793.7 and 784.9 feet, respectively.

4.8.3.2.1 Geology

Chillicothe's geology is consistent with the description of the Northern Glaciated Plains region discussed in Section 2. As anticipated bedrock was not reached during the subsurface explorations and the entire profile consisted of stiff to very stiff overconsolidated clays. These conditions extended to the borings' termination depths of about 102 feet.

4.8.3.2.2 Soil and groundwater

The existing soils observed consisted of low plasticity lean clay (CL) separated by a 5-ft layer of sand (SP). Based on the results of the boring information provided, the borings initially encountered gray, lean clay that extended to depths of about 15 feet. Below the lean clay, the sand layer was encountered, but it was not present at all boreholes. Groundwater was observed at approximately 30.0 feet below the surface during drilling. Figure 4.22 shows the subsurface conditions modeled for the WEAP analysis.

4.8.3.3 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 50 feet in length. The A7932 Static and Wave Equation analyses are included in Appendix B and Appendix C, respectively.

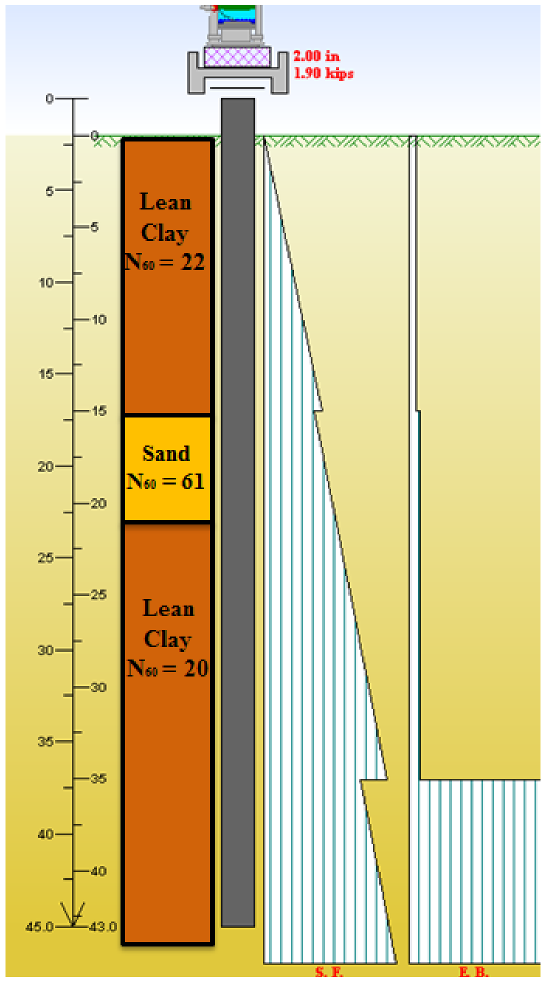


Figure 4.22 - A7932 Soil Profile Along the Test Pile

4.8.3.3.1 *Static analysis*

The Alpha and Beta methods were used to estimate the available resistance of the test pile. For the 48-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.

4.8.3.3.2 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 layers outlined in Figure 4.20. 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 172.3 to 207.3 kips depending on 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, typically this is compared to 90% of the yield strength (F_y) of steel. 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 4.7.

Table 4.7 - A7932 WEAP Analysis Results for Gain/Loss Ratios at Shaft and Toe
(a) 0.8/1.0 and (b) 1.0/1.0

Gain/Loss 1 at Shaft and Toe 0.800 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
6.0	36.6	4.1	32.5	3.5	15.794	0.000	4.50	21.7
12.0	48.2	15.6	32.5	5.1	18.366	0.000	4.91	20.5
18.0	63.2	30.6	32.5	7.3	21.360	-0.294	5.39	19.3
24.0	80.4	47.9	32.5	10.1	23.479	-0.437	5.81	18.3
30.0	100.0	67.4	32.5	13.7	25.250	-0.376	6.26	17.6
32.9	110.4	77.8	32.5	15.6	26.066	-0.190	6.51	17.3
33.0	110.6	78.1	32.5	15.6	26.100	-0.180	6.52	17.4
48.0	172.3	139.8	32.5	32.8	28.638	-0.061	7.43	15.9

Total Continuous Driving Time 13.00 minutes; Total Number of Blows 604

(a)

Gain/Loss 2 at Shaft and Toe 1.000 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
6.0	37.6	5.1	32.5	3.6	16.029	0.000	4.54	21.6
12.0	52.1	19.5	32.5	5.6	19.137	0.000	5.04	20.1
18.0	70.8	38.3	32.5	8.5	22.451	-0.507	5.55	18.8
24.0	92.4	59.8	32.5	12.3	24.780	-0.534	6.09	17.8
30.0	116.8	84.3	32.5	16.8	26.676	-0.053	6.62	17.2
32.9	129.8	97.3	32.5	19.6	27.381	0.000	6.85	16.9
33.0	130.1	97.6	32.5	19.6	27.412	0.000	6.86	17.0
48.0	207.3	174.7	32.5	40.7	29.866	0.000	7.80	15.9

Total Continuous Driving Time 16.00 minutes; Total Number of Blows 736

(b)

4.8.3.4 Anchor Pile & Test Pile Installation

The reaction frame and test piles at the A7932 site were installed on September 19, 2013, by APAC. The pile driving hammer used during the installation consisted of a Delmag D19-42 open ended diesel hammer. The reaction piles were 50 ft. long, 14 inch closed-ended, steel pipe piles with a 3/8 inch wall thickness with a 5-ft stickup to build the frame. 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 50-ft reaction piles were driven to a depth of 45 feet, resulting in a stick-up height of five feet. The test pile (14-in CIP) 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 about 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 4.23 shows the completed installation of the test pile.

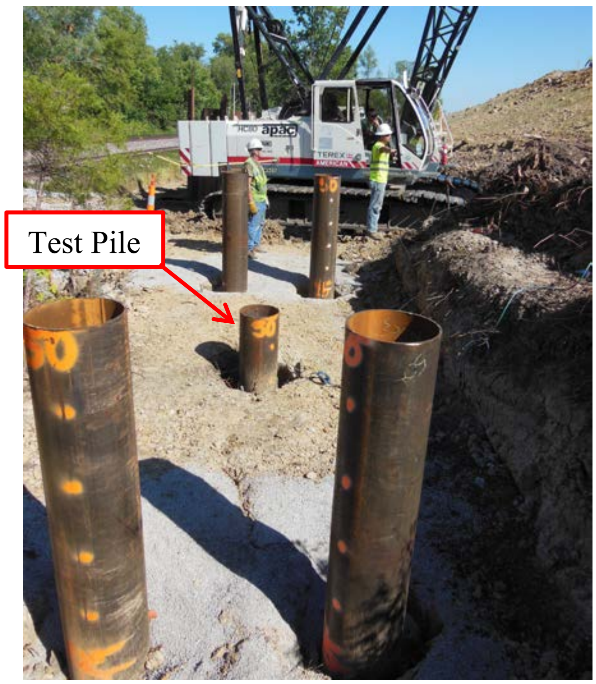


Figure 4.23 - A7932 Test Pile Installation

The test pile for the PLT was installed to an approximate elevation of 737 ft. resulting in an embedment length of 48 ft. Providing a 2 ft. stick-up height, the final length of the test pile was 50 ft. Since the soil conditions were primarily clay, a restrike was scheduled 7 days later to observe the effects of pile setup.

4.8.3.5 Dynamic Testing

Following the special provisions in the MoDOT contracts, dynamic testing was performed during the installation of the A7932 test pile on September 19, 2013 by Joseph Cravens of Geotechnology, Inc. The dynamic testing events followed the description outlined in Section 4.5 and the results from this analysis are summarized in the following Section 5.

<u>Note:</u> The installation of instrumentation and test setup was very similar to Bridge Site #1 that also used 14-in CIP driven piles.

5. Test Pile Results and Discussion

Presented in this section are the results of the three (3) bridge sites included in this research project. The description of each bridge site, subsurface conditions, and pile load test installation and methods are included in Section 4. This section will present the results of the dynamic testing and the static pile load tests.

5.1. Bridge Site 1 (A7956): Sikeston, Scott County

5.1.1. Dynamic Testing Results

The analysis of the dynamic data was performed by Craig Kaibel, P.E. using the Case Pile Wave Analysis Program (CAPWAP) signal matching software. A summary of the CAPWAP estimated ultimate axial capacities are summarized in Table 5.1.

Table 5.1- 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
Beginning-of-Restrike (BOR)	184.1	38.4	145.7

Figure 5.1 shows the wave matching analyses and the estimated load-settlement curves from the CAPWAP analyses. From Table 5.1 and Figure 5.1, 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 included in Appendix C.

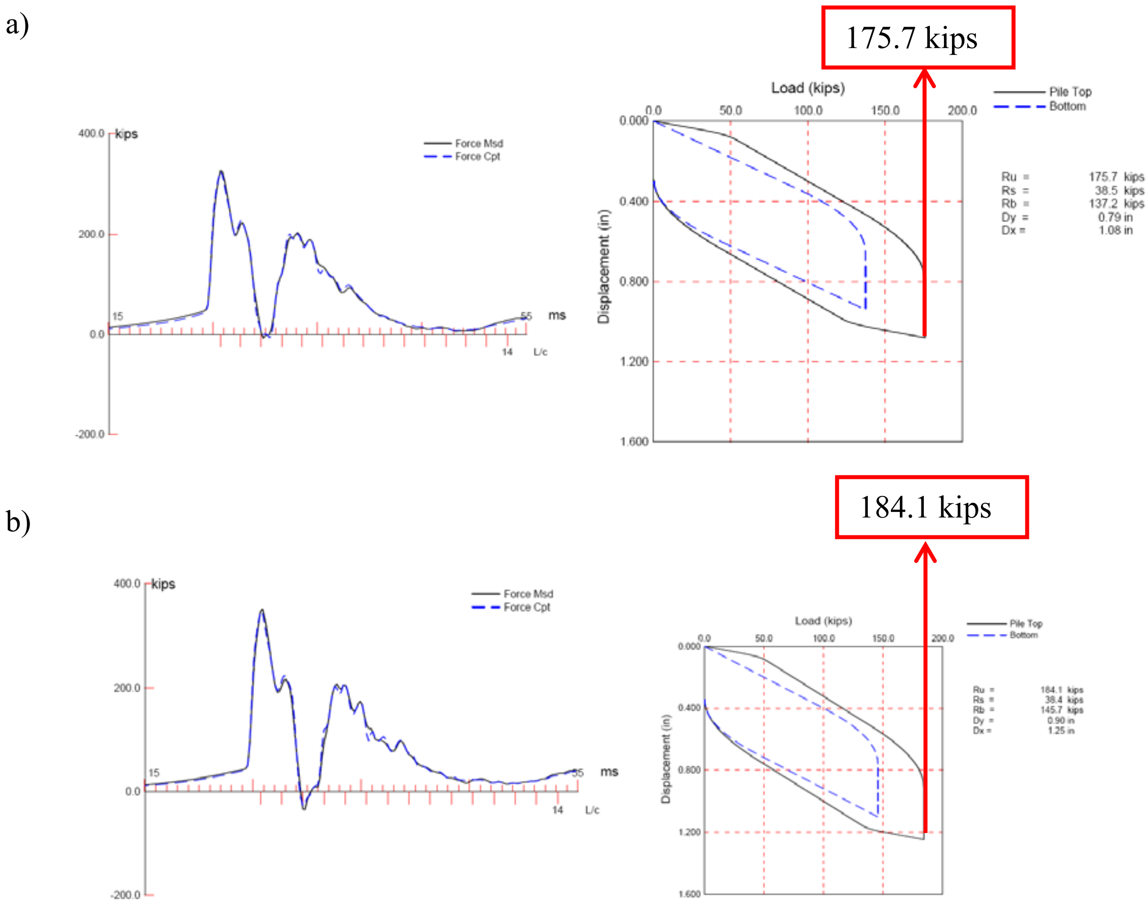


Figure 5.1 - A7956 CAPWAP Wave Match and Load-Displacement Curve for (a) EOD and (b) BOR (adapted from the A7956 Geotechnology Report)

5.1.2. Static Load Test Results

The test pile was incrementally loaded until failure following the loading schedule presented in Table 5.2. The data collected from the static load test followed the data reduction methods presented in Section 4. The values used to perform the data reduction are shown in Table 5.3.

The load cell and LVDT data from all three cycles resulted in axial load versus axial displacement plots at the pile head, as shown in Figure 5.2. It was observed that during the unloading portions of cycle 1 and 2 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.

Table 5.2 - A7956 Load Test Schedule

Job No.:	J0P2239
Design:	A7956
Date:	8/7/2012
Est. Nom. Resistance:	200 kips
Design Load:	100 kips
Factor of Safety:	2.0

Load Cycle	**Applied Load (% DL)**	**Applied Load (kips)**	**Load Cycle**	**Applied Load (% DL)**	**Applied Load (kips)**
Zero Values	Jack	0.3	Seating	AL	0.3
Seating	AL	0.3		12.5	25
Cycle 1 (100 kips)	12.5	25	Cycle 3 (Plunge)	25.0	50
	25.0	50		50.0	100
	37.5	75		62.5	125
	50.0	100		75.0	150
	37.5	75		87.5	175
	25.0	50		92.5	185
	12.5	25		97.5	195
Unload	AL	0.3		102.5	205
Cycle 2 (200 kips)	12.5	25		105.0	210
	25.0	50		107.5	215
	37.5	75		110.0	220
	50.0	100		112.5	225
	62.5	125		115.0	230
	75.0	150			
	62.5	125			
	0.0	0			

DL - Design Load
AL - Alignment Load

Table 5.3 - 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

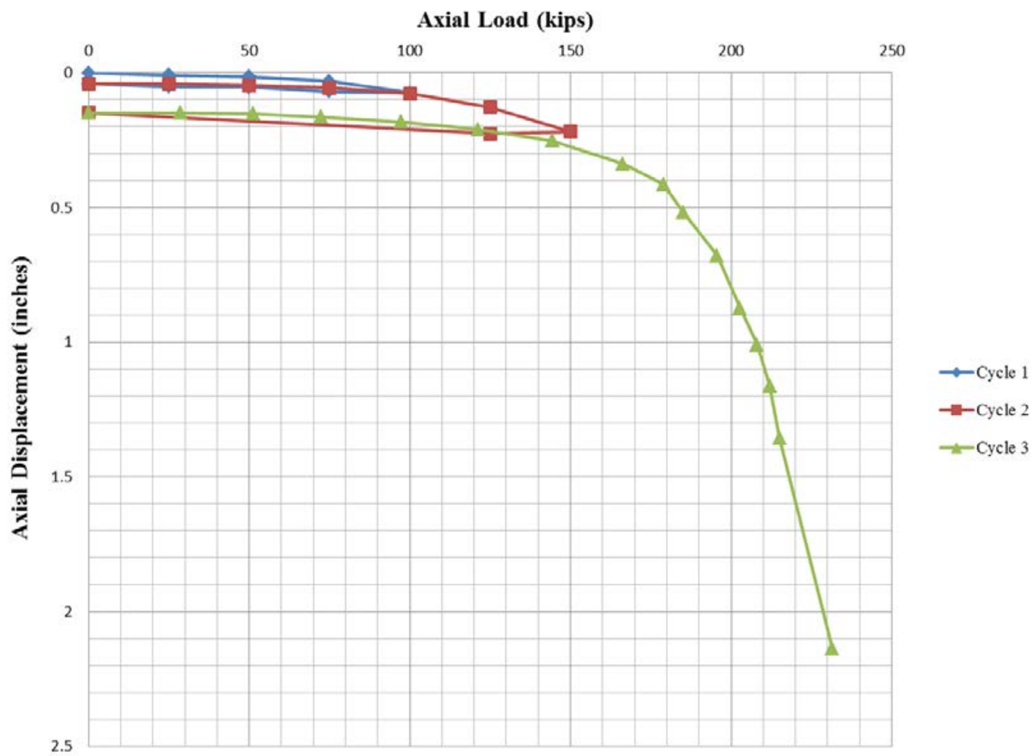


Figure 5.2 - A7956 Static Load Test Results

5.1.3. Nominal Resistance

Several methods to interpret the failure load from the load-displacement curve are available and described Stuckmeyer (2013). A summary of the nominal resistances interpreted from each method are shown in Table 5.4. Note that only the curve of cycle 3 is used in the interpretation for each method.

Table 5.4 - 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 5.5. 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 is preferred to compare with the other resistances.

Table 5.5 - 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

5.1.4. Load Transfer Distribution

Figure 5.3 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 Figure 5.3.

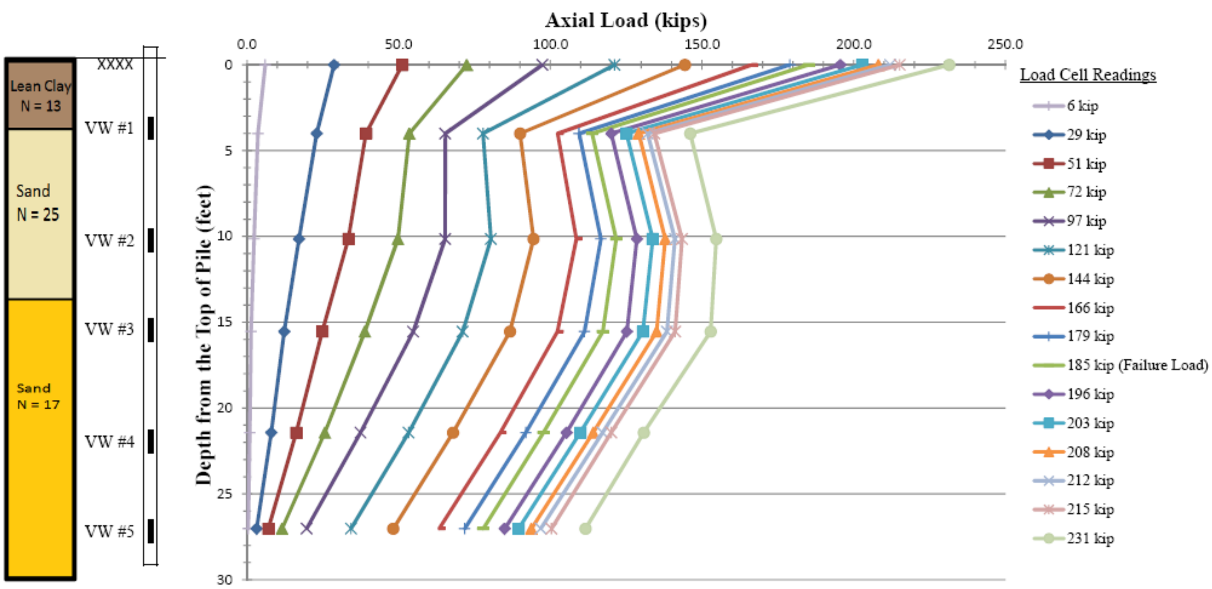


Figure 5.3 - A7956 Load Transfer Plot

5.2. Bridge Site 2 (A7669): Poplar Bluff, Butler County

5.2.1. Dynamic Testing Results

A summary of the nominal resistances for the end-of-drive (EOD) and beginning-of-restrike (BOR) estimated by CAPWAP are summarized in Table 5.6. Figure 5.4 shows the wave matching analyses and the estimated load-settlement curves from the CAPWAP analyses.

Table 5.6 - Nominal Resistances Estimated From the A7669 CAPWAP Analysis (adapted from the A7669 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 5.6 and Figure 5.4 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.

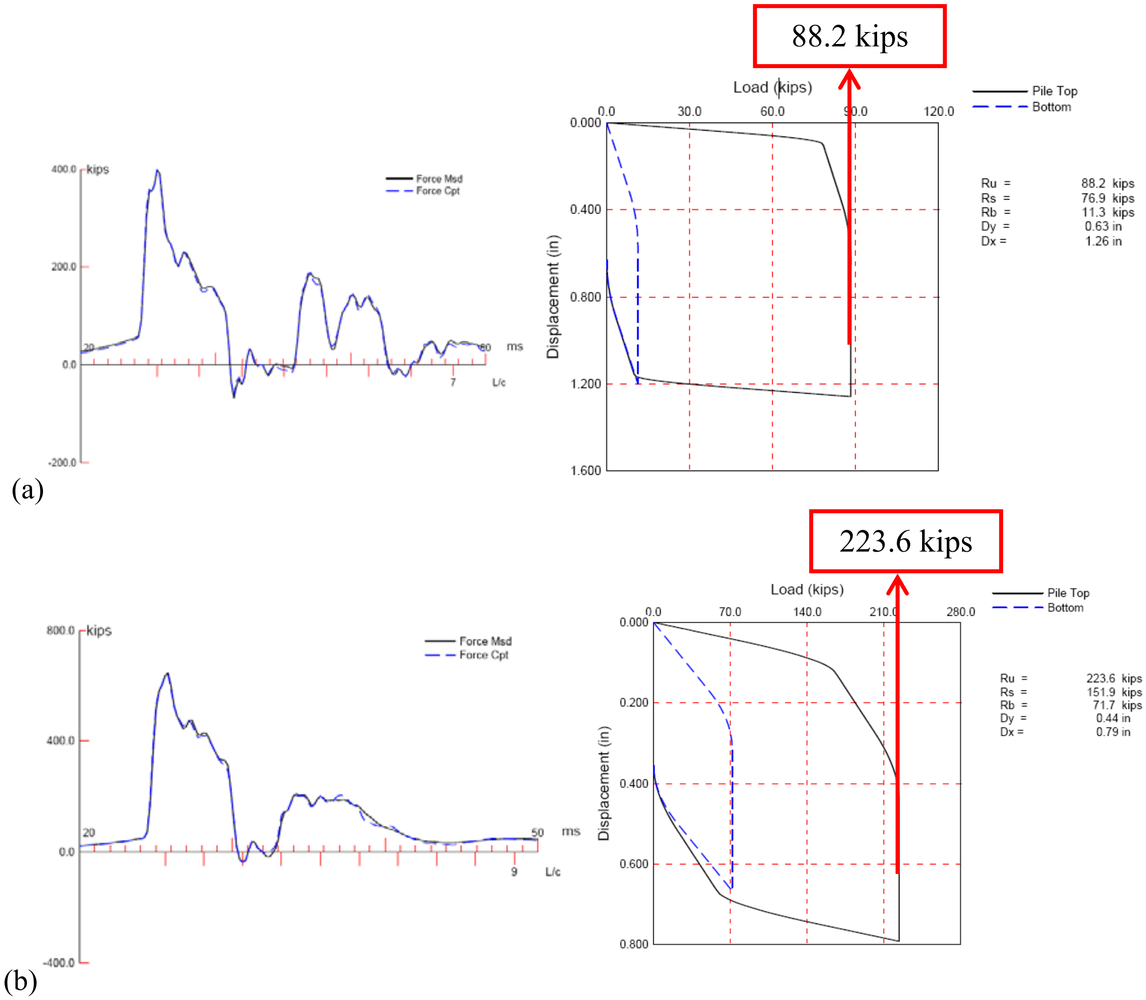


Figure 5.4 - A7669 CAPWAP Wave Match and Load-Displacement Curve for (a) EOD, and (b) BOR (adapted from the A7669 Geotechnology Report)

5.2.2. Static Load Test Results

The test pile was axially loaded following the loading schedule presented in Table 5.7. 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 5.8.

Table 5.7 - A7669 Loading Schedule

Job No.: J0P0959
Design: A7669
Date: 31-Oct-2012
Est. Nom. Resistance: 200 kips
Design Load: 168 kips
Factor of Safety: 2.0

Load Cycle	**Applied Load (% DL)**	**(kips)**	**Load Cycle**	**Applied Load (% DL)**	**(kips)**
Zero Values	Jack	0.3	Seating	AL	0.3
Seating	AL	0.3		25.0	50
	12.5	25		50.0	100
	25.0	50		75.0	150
Cycle 1 (100 kips)	37.5	75		100.0	200
	50.0	100		105.0	210
	37.5	75		110.0	220
	25.0	50		112.5	225
	12.5	25		115.0	230
Unload	AL	0.3	Cycle 3 (Plunge)	117.5	235
	25.0	50		120.0	240
	50.0	100		122.5	245
	75.0	150		125.0	250
Cycle 2 (200 kips)	100.0	200		127.5	255
	75.0	150		130.0	260
	50.0	100		132.5	265
	25.0	50		135.0	270
	0.0	0		137.5	275

DL - Design Load
AL - Alignment Load

Table 5.8 - 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 5.5. 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 from the A7669 static load test and the corresponding reduced results are included in Appendix D.

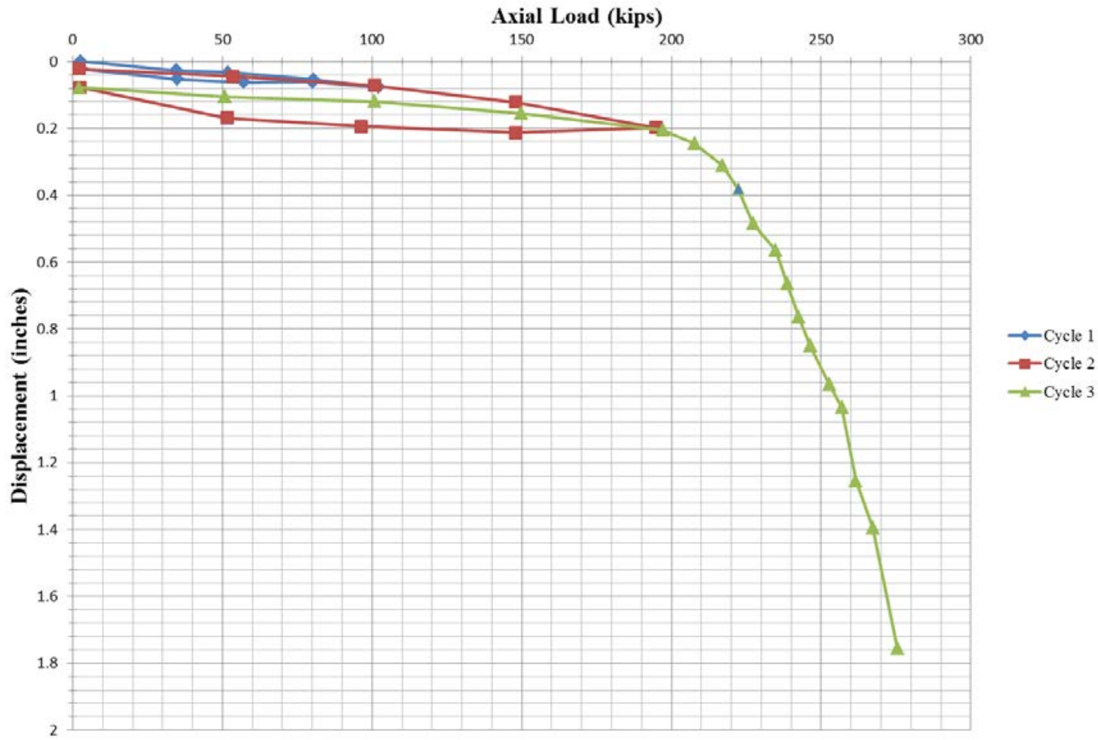


Figure 5.5 - A7669 Static Load Test Results

5.2.3. Nominal Resistance

The same series of methods (mentioned earlier in this section) were used to interpret the failure load from the applied load-axial displacement curve. The ultimate capacities interpreted from each method are shown in Table 5.9. It is important to note that only the curve of the failure cycle (Cycle 3) is used in the interpretation for each method.

Table 5.9 - 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 5.8. As stated in Section 5.2.9.1.1 of the AASHTO LRFD Specification (2010), the use of Davisson's (1972) method (for piles 24 in. in diameter or less) is recommended to interpret the ultimate resistance from a QM static load test, the nominal resistance interpreted using Davisson's method was reported for comparison.

Table 5.8 - 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

5.2.4. Load Transfer Distribution.

Figure 5.6 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 Figure 5.6.

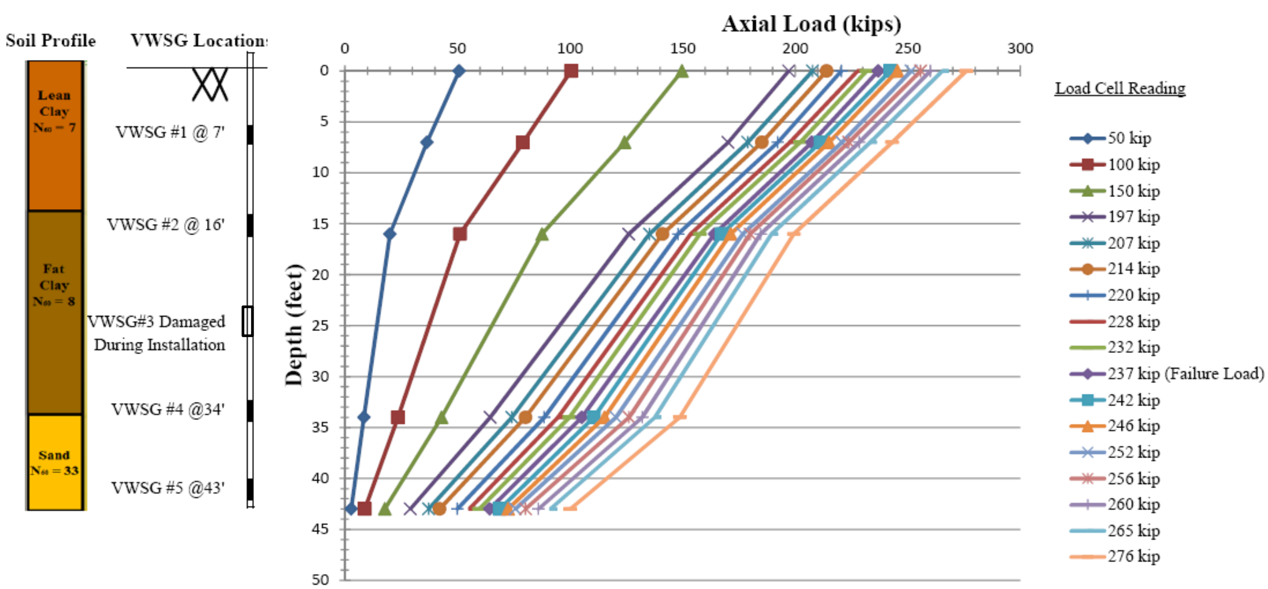


Figure 5.6 - A7669 Load Transfer Plot

5.3. Bridge Site 3 (A7932): Chillicothe, Livingston County

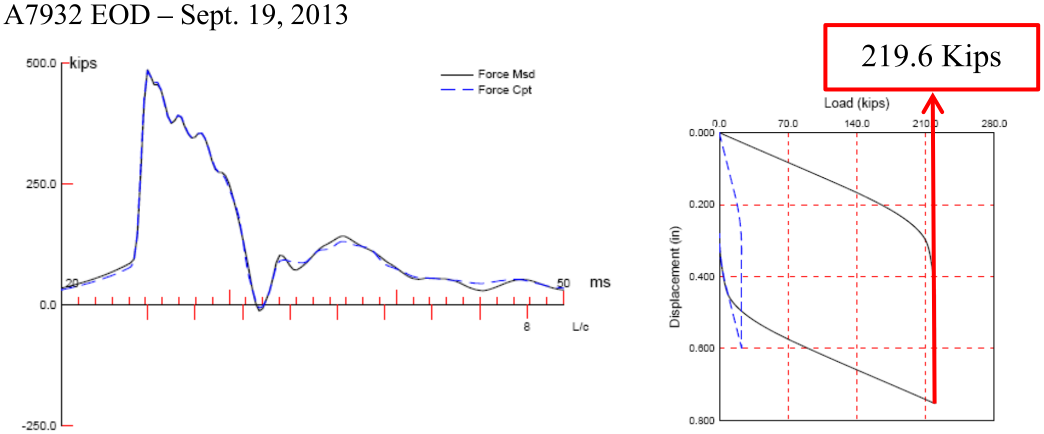
5.3.1. Dynamic Testing Results

A summary of the nominal resistances for the end-of-drive (EOD) and beginning-of-restrike (BOR) estimated by CAPWAP are summarized in Table 5.9. An additional restrike CAPWAP test was ordered from the research team given the inconsistent results obtained in the static load test. Note that an additional restrike is reported in Table 5.9 one at 7-day and the other at 35-day from the end of drive date. The same hammer and practices where used for all driving conditions within the 35-day period. Such variability in the results surprised all the research investigators and a number of reasons for the wide discrepancy are discussed further in this section. Figure 5.7 shows the wave matching analyses and the estimated load-settlement curves from the CAPWAP analyses.

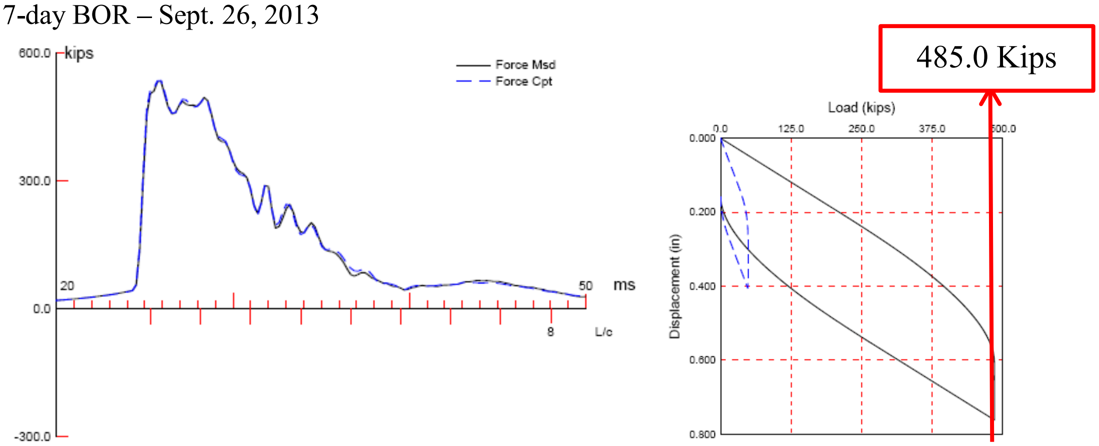
Table 5.9 - Nominal Resistances Estimated From the A7932 CAPWAP Analysis
(Adapted from the A7932 Geotechnology Report)

Test Type	Nominal Resistance (kips)		
	Total	Shaft	Tip
End-of-Drive (EOD, Sept. 19)	219.6	197.0	22.6
Restrike 1 (7-day BOR, Sept. 26)	485.0	436.5	48.5
Restrike 2 (35-day BOR, Oct. 24)	376.1	356.1	20.0

(a) A7932 EOD – Sept. 19, 2013



(b) A7932, 7-day BOR – Sept. 26, 2013



(c) A7932, 35-day BOR – Oct. 24, 2013

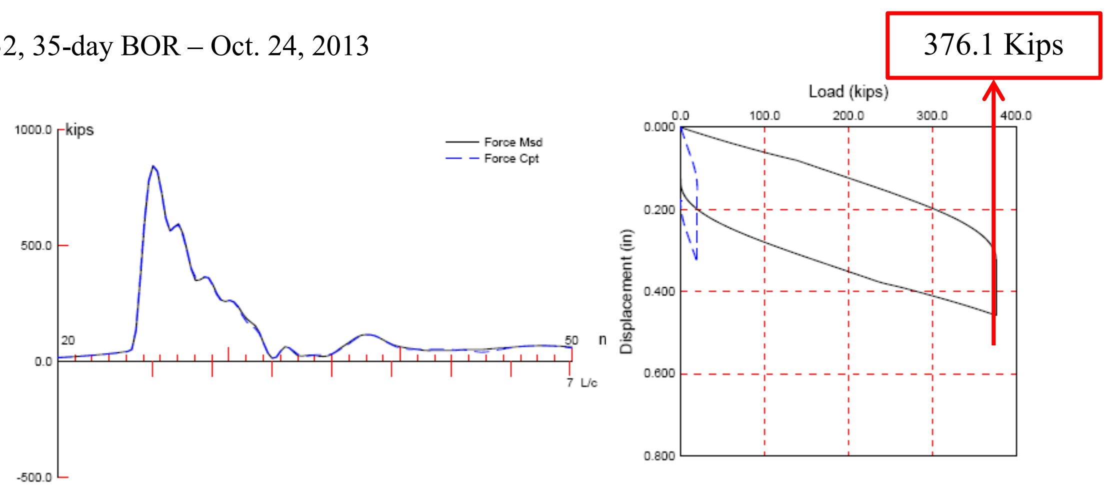


Figure 5.7 - A7932 CAPWAP Wave Match and Load-Displacement Curve
(a) EOD, (b) BOR, 7-day, and (c) BOR, 35-day (extra)
(adapted from the A7932 Geotechnology Report)

5.3.2. Static Load Test Results

The test pile was axially loaded following the loading schedule presented in Table 5.10. The data collected from the static load test was reduced following the data reduction methods presented in Section 4. This test pile required the use of a transformed area like the one used for the Scott County Bridge A7956 test pile, which was also a 14-inch CIP. The modulus of elasticity and pile area used in the data reduction are shown in Table 5.11.

Table 5.10 - A7932 Load Test Schedule

Job No.:	J2S0787
Design:	A7932
Date:	Oct. 4, 2013
Est. Nom. Resistance:	485 kips
Design Load:	232 kips
Factor of Safety:	2.0

Load Cycle	Applied Load (% DL)	Applied Load (kips)
Zero Values	Jack	0.3
Seating	AL	0.3
Cycle 1 (200 kips)	12.5	25
	25.0	50
	50.0	100
	75.0	150
	100.0	200
	75.0	150
	50.0	100
	25.0	50
	12.5	25
Unload	AL	0.3
Cycle 2 (300 kips)	25.0	50
	50.0	100
	75.0	150
	100.0	200
	125.0	250
	150.0	300
	125.0	250
	100.0	200
	75.0	150
	50.0	100
	25.0	50
	0.0	0

Table 5.11 - Parameters Used in A7932 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, $E_{concrete}$	3,685 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 load cycles were used to plot axial load versus axial displacement at the pile head, as shown in Figure 5.8. 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 reached 263 kips. When the load cell reading reached 263 kips, the pile was not able to sustain the load and residual load dropped to about 230 kips and eventually to 220 kips. This appears to be a post-peak softening condition unique of soils that shear in dense conditions such as what is encountered in overconsolidated soils. After the residual capacity was reached at about one inch of displacement, the pile was not able to develop any more capacity above this value of 230 kips. The raw data obtained from the A7932 static load test and the corresponding reduced results are included in Appendix D.

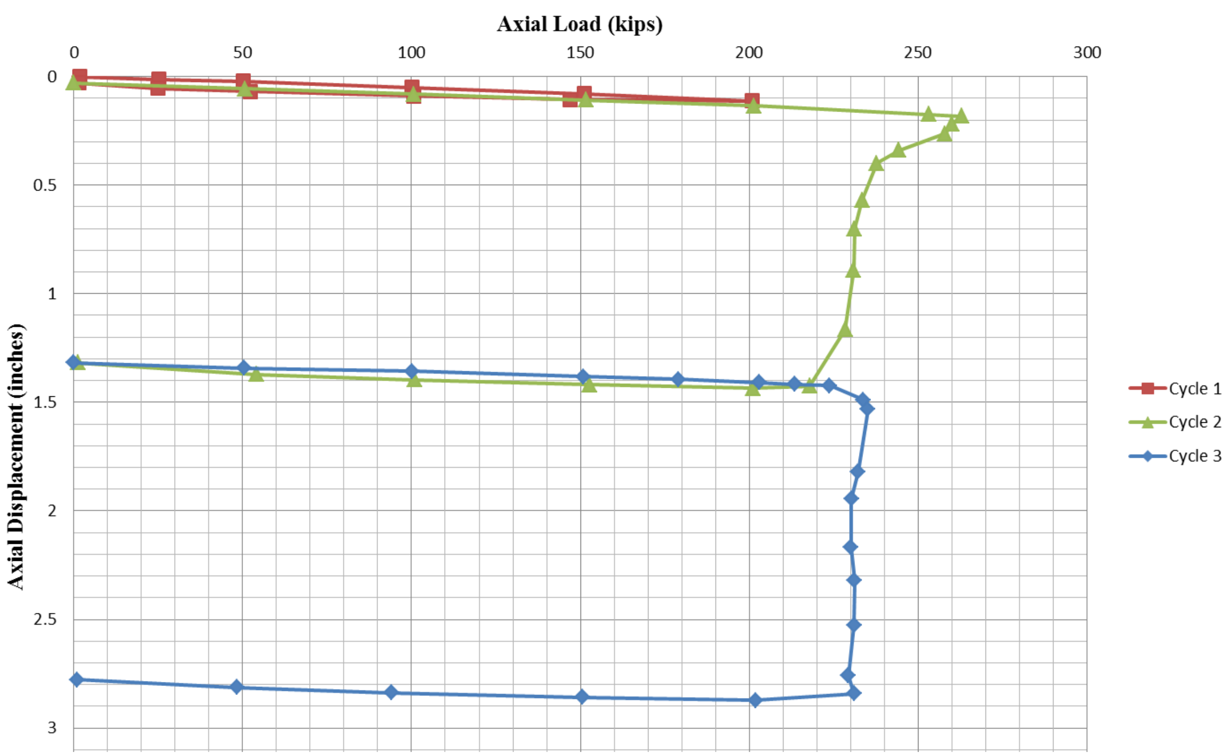


Figure 5.8 - A7932 Static Load Test Results (All cycles performed Oct. 4, 2013)

5.3.3. Nominal Resistance

The same series of methods (mentioned earlier in this section) were used to interpret the failure load from the applied load-axial displacement curve. The ultimate capacities interpreted from each method are summarized in Table 5.12. It is important to note that only the curve of the failure cycle (Cycle 3) is used in the interpretation for each method.

Table 5.12 - Summary of Interpreted A7932 Nominal Resistances

A7932 Static Load Test Nominal Resistance Summary	
Method	**Nominal Resistance (kips)**
Davisson (1972)	*233*
Chin (1970)	238
De Beer (1968)	240
Mazurkiewicz (1980)	240
Brinch Hansen 90% Criteria (1963)	234
Minimum Value	233
Maximum Value	240
Average Value	***237***

The difference in the nominal resistance measured by the static load test and the nominal resistance estimated at BOR by the dynamic test was a surprise to all involved on this project. In fact, an additional re-strike was requested to get another data point to find an explanation to the unusual behavior of the pile. As stated before in the methods section, the AASHTO LRFD Specification (2010) specifies the use of Davisson's (1972) method (for piles 24-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 in Table 5.13.

Table 5.13 - Comparison of A7932 Nominal Resistance Results

Bridge (geologic region)	Nominal Resistance (kips)				Difference (%)
	Static Load Test (10/4/13)	Dynamic Testing			
		EOD (9/19/13)	7-Day BOR (9/24/13)	35-Day BOR (10/24/2013)	
A7932 Chillicothe, MO (Northern Plains)	233.0* (233-240)	219.6	485.0	376.1	±72%

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

Figure 5.9 illustrates the load-transfer distribution corresponding to each applied load increment from the A7932 static load test. At failure, the shaft and tip resistance was 212 kips and 21 kips, respectively, concluding approximately 91% of the pile's nominal resistance was contributed by the shaft resistance and 9% 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 Figure 5.9.

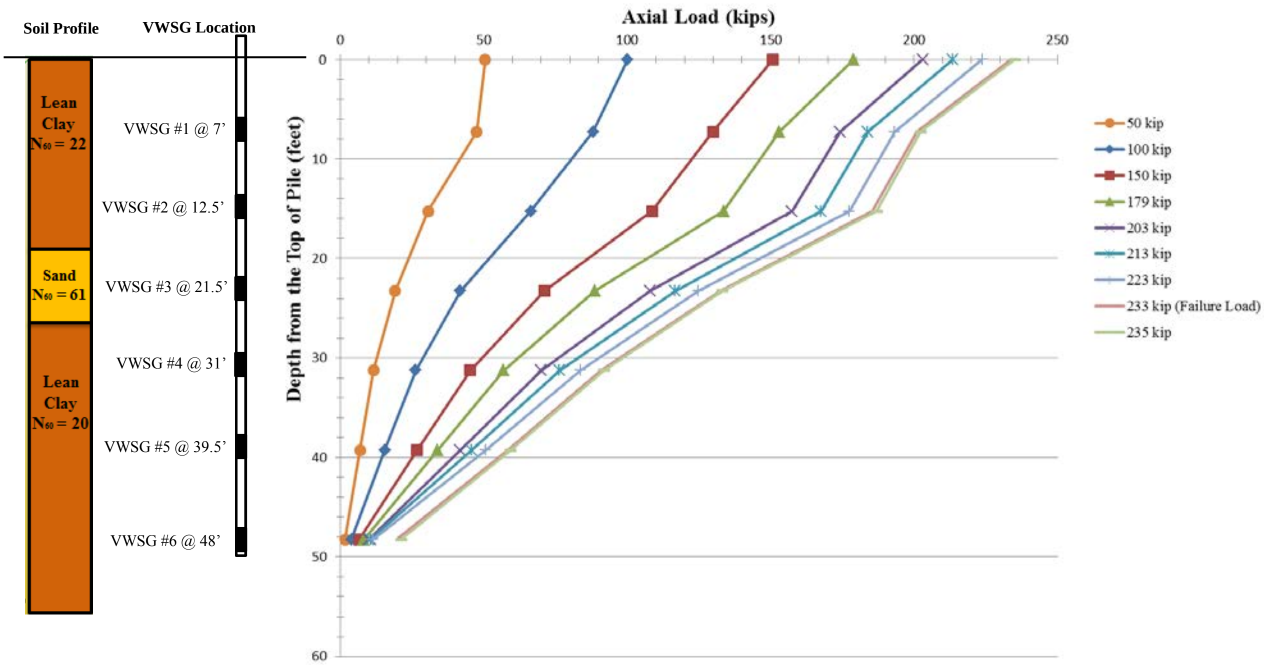


Figure 5.9 - A7932 Load Transfer Plot

5.4. Discussion of Results

5.4.1. Dynamic Load Tests

Representative blows from the data collected at the EOD and near BOR of each test pile were subsequently analyzed using CAPWAP signal matching software. Table 5.14 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 5.14 - Nominal Resistance Estimated From the CAPWAP Analyses

Bridge site and pile type *(geologic region)*	Test Type	Nominal Resistance (kips)			Pile Set-up
		Total	Shaft	Tip	
A7956 - Sikeston, MO Full displacement, Sand *(SE Lowlands)*	End-of Drive	175.7	38.5	137.2	4.7%
	Restrike	184.1	38.4	145.7	
A7669 - Poplar Bluff, MO Low displacement, Clay *(SE Lowlands)*	End-of-Drive	88.2	76.9	11.3	153.5%
	Restrike	223.6	151.9	71.7	
A7932 - Chillicothe, MO Full displacement, Clay *(N. Glaciated Plains)*	End-of-Drive	219.6	197.0	22.6	118.6% *(average)*
	Restrike 1	485.0	436.5	48.5	
	Restrike 2	475.2	441.1	31.1	

When a pile is driven, the soil against the test pile is displaced, sheared, and remolded. This combination generates an increase in the porewater pressure of the soil. Full displacement piles (e.g., closed end pipe piles) tend to generate more porewater pressure, especially if this is in combination with a soil profile that is fine-grained or low permeability. As the porewater pressure increases, the soil's effective stress is reduced, thus decreasing the strength of the soil in the short term. 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). When the opposite happens, or the pile capacity reduces with time, this is referred to as "pile relaxation".

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 Scott Co. site. This bridge site used a full displacement CIP into the sand deposit. Conversely, the hydraulic conductivity of cohesive soils causes the excess porewater pressure to dissipate much 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 Butler Co. 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. There was much more pile setup at the A7669 clay site, even though the pile was a low displacement H-pile.

In practice, a restrike test is usually performed several days after EOD to assess the effects of pile setup. At bridge sites where pile setup is anticipated 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 confirm that 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 that 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).

Since it is the contractor's responsibly to produce a foundation consistent with the design, their decision amounts to which option is more economically viable for the contractor. MoDOT pays for all the pile driven in the ground, even if it is more than what was shown in the plans. In other words, does the cost of waiting to resume the construction activities until after the restrike outweigh the cost of installing additional piling? Time often is a critical element. Continuing driving completes work and can free up personnel and equipment. Waiting may delay critical path, construction completion, and costs extra for non-productive crews, and equipment as well as additional PDA testing.

At the A7669 site, the contractor elected to continue driving to a deeper elevation by adding steel pile sections. 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 (Approx. tip El. 271 ft.) in the design. At EOD, the test pile was estimated to have a nominal resistance of 88.2 kip as shown is Table 5.14. 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 on the test pile at 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%). 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.

What was much more surprising was the third bridge site A7932 in Livingston Co. The 14-in CIP full displacement pile was driven into an overconsolidated clay with a relative shallow groundwater table. The combination of a full displacement pile and the cohesive low permeability soils would anticipate a higher porewater pressure development. However, the overconsolidated condition of the clay may have contributed to a relative less pile setup of only 118.6%.

5.4.2. 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 (5.1.2, 5.2.2, and 5.3.2.) of this report. Since the AASHTO (2010) specifies the use of Davisson's (1972) method to interpret the nominal resistance from a QM static load test, the nominal resistance was estimated using this method. 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 difference determined from the static and dynamic tests of each site are shown in Table 5.15. The capacities that compare well with the static pile load tests are close only at the BOR for the sites that had normally consolidated soils. Given that the test piles were tested about 7-days after the pile was driven to allow for the construction of the reaction frame, these results suggest the delay provided sufficient time for dissipation of excess porewater pressure and pile setup. As a result, the effects of pile setup observed at the BOR were also captured in the static pile load test. However, the capacities that did not compare well were for the overconsolidated soil conditions in the N. Glaciated Plains. When the full displacement CIP pile was driven into the clay soils the BOR resulted in the opposite response, the EOD capacity was much more comparable to the PLT than the BOR.

Table 5.15 - Summary of Static and Dynamic Load Test Results

Bridge Site (geologic region)	Nominal Resistance (kips)			Percent Difference Static to Dynamic (%)
	Static Pile Load Test	**Dynamic** Testing		
		EOD	BOR	
A7956 – Scott County (SE Lowlands)	182.0	164.6	184.1	- 9.0 + 1.0
A7669 – Butler County (SE Lowlands)	236.0	82.2	223.6	- 65.2 - 5.0
A7932 – Livingston County (N. Glaciated Plains)	233.0	219.6	480.1	- 5.6 + 106.0

5.4.3. 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 5.3) 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 the 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, a small void could be present between the bearing plate and the top of the concrete. If this is the case, 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.

For both the A7956 and 7932 (Livingston Co.) bridge sites 14-in CIP pile were tested. It is anticipated that the interface between the steel pipe 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 pressure and possibly gain a greater contact between them and the steel shell (the load transfer does behave as expected from VWSG #3 thru VWSG #5). However, for VWSG #1 and VWSG #2, the interface between the concrete and steel around these gauges may not be as strong. As the load 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 (Butler Co.) 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 and the strain gages were welded directly to the pile. The A7669 load transfer plot (Figure 5.9) demonstrates that the applied load at the pile head was transferred relatively linear 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 between the different materials.

A comparison of the load-transfer results from the static and dynamic tests of each site are shown in Table 5.16. Note that the results compared for the A7932 (Livingston Co.) test site was based on the EOD dynamic test results, since the BOR capacities experienced significant over-capacity due to the setup in the overconsolidated soils of the N. Glaciated Plains.

Table 5.16 Load Transfer Distribution Results

Bridge (geologic region)	Test Type	Nominal Resistance (kips)		
		Total	Shaft	Tip
A7956 – Scott County (SE Lowlands)	CAPWAP (BOR)	184.1	38.4 (21%)	145.7 (79%)
	PLT VWSG Data	182.0	100.0 (55%)	82.0 (45%)
A7669 – Butler County (SE Lowlands)	CAPWAP (BOR)	223.6	151.9 (68%)	71.7 (32%)
	PLT VWSG Data	236.0	188.0 (80%)	48.0 (20%)
A7932 –Livingston County (N. Glaciated Plains)	CAPWAP (EOD)	219.6	197.0 (90%)	22.6 (10%)
	PLT VWSG Data	233.0	212 (91%)	21 (9%)

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 from person to person; thus the results of a specific CAPWAP analysis will differ as well.

5.5. Calculation of Resistance Factors

As stated in Section 2.1, MoDOT adopted the resistance factors from the AASHTO LRFD Bridge Design Specifications (2010) to design 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 bridge 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 2.1 of Section 2, the LRFD criterion 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:

$$R_{n\,(design)} = \frac{\Sigma(LF)\ Q\_n}{RF_{design}} = \frac{102\ kips}{0.65} = 157\ kips \qquad (5.1)$$

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 to 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 $RF_{measured}$:

$$RF_{measured} = \frac{(182 * 0.65)}{157} = 0.75 \qquad (5.2)$$

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 5.17.

Table 5.17 Calculated Resistance Factors based on Static Load Tests

Bridge (geologic region)	Calculated Resistance Factor
A7956 Sikeston (SE Lowlands)	0.75
A7669 Poplar Bluff (SE Lowlands)	0.91
A7932 Chillicothe (N. Glaciated Plains)	0.69 (1.0 using BOR)

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). For the A7932 site the increase was minimal based on the static load tests, but the surprising results in setup could result in a resistance factor of 1.0. 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 meet the reliability levels incorporated into the AASHTO LRFD criteria.

6. Conclusions and Recommendations

6.1. 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 3. 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 for two to the three sites.

- Davisson’s (1972) method is proven to be the most common method for interpretation of 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.

6.2. 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 by MoDOT:

- 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 a positive agreement with the CAPWAP results at BOR for two of the three sites. Additional PLT data sets need to be established to observe if this trend continues.

- 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.

- 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.

- A standardized pile driving record needs to be kept during the installation of all piles (production and test) on MoDOT projects. The information in this document should fully describe the project, location of the pile with respect to the structure, pile length, and blow-count per foot during installation. This is similar to the worksheets included in the form available in MoDOT EPG (http://epg.modot.mo.gov/files/9/93/702_Pile_Driving_Worksheet.pdf). 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. The pile driving records should be made part of the "as built" plans of record. These may become useful when MoDOT would like to reuse the existing foundations for modification or expansion of the bridge. The pile driving records where not being kept for the projects described herein.

- 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.

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APPENDICES

Included with this report 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

MODOT BRIDGE PLANS AND SPECIAL PROVISIONS ON CD-ROM

A.1 INTRODUCTION

This appendix 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
MoDOT Bridge A7932 Bridge Plans.pdf	Adobe PDF
MoDOT Bridge A7932 Special Provisions.pdf	Adobe PDF

APPENDIX B

STATIC ANALYSIS RESULTS ON CD-ROM

B.1 INTRODUCTION

This appendix 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
MoDOT Bridge A7932 Static Analysis.xlsx	Microsoft Excel 2010

APPENDIX C

WEAP ANALYSES AND DYNAMIC TESTING REPORTS ON CD-ROM

C.1 INTRODUCTION

This appendix 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
MoDOT Bridge A7932 FTC WEAP Analysis Report.pdf	Adobe PDF
MoDOT Bridge A7932 Geotechnology Dynamic Testing Report.pdf	Adobe PDF
MoDOT Bridge A7932 Geotechnology Addendum.pdf	Adobe PDF
MoDOT Bridge A7932 MS&T WEAP Analysis.gww	GRL WEAP 2010

APPENDIX D

STATIC LOAD TEST DATA AND RESULTS ON CD-ROM

D.1 INTRODUCTION

This appendix 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 Results.xlsx	Microsoft Excel 2010
MoDOT Bridge A7956 PLT Results.xlsx	Microsoft Excel 2010
MoDOT Bridge A7932 PLT Results.xlsx	Microsoft Excel 2010

APPENDIX E

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

E.1 INTRODUCTION

This appendix 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 **LO**ad **T**est (PILOT) database.accdb	Microsoft Access 2010
Previous MS&T Pile Load Tests Data.xlsx	Microsoft Excel 2010