OKLAHOMA TRANSPORTATION CENTER

ECONOMIC ENHANCEMENT THROUGH INFRASTRUCTURE STEWARDSHIP

DEVELOPING COUNTY BRIDGE REPAIR AND RETROFIT TECHNIQUES

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16. ABSTRACT

Oklahoma rated first in the Nation in the percentage of bridges that are structurally deficient or functionally obsolete. According to Federal Highway Administration data, Oklahoma uses approximately 23,250 bridges maintained by state, County, City, and Tribal governments. The Oklahoma Department of Transportation (ODOT) estimated that it would cost $3.4 billion to replace these bridges.

The research team worked with engineers and bridge maintenance personnel from the ACCO, ODOT, and Oklahoma CED's to identify, evaluate, and develop two repair techniques for deteriorating off system bridges. The project focused on repairing decayed timber piles and corroded H-piles. To repair decayed timber piles, the decayed pile was removed from the pile cap to sound wood below grade and then replaced with steel members that are commonly available to bridge maintenance personnel. These included steel pipe and steel H-pile sections. The corroded H-piles were repaired by welding steel plates to the flanges of H-piles containing corroded webs. The repair techniques were evaluated in the field through load testing strain gauge instrumented bridges. The steel H-pile repair was also investigated under controlled laboratory testing. The repairs were found to adequately restore strength and stiffness and transfer loads from the super structure to the foundation. Straight forward design guidelines and design tables were developed for each of the repair techniques.

The recommended repairs are useful for extending the life of a bridge that contains decayed timber piles or steel H-piles with corroded webs. The repairs allow for adequate safety to be maintained while a deteriorating bridge is waiting to be replaced. This allows for county, tribal, and local government officials to prioritize and wisely spend their bridge and road maintenance resources.

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SI (METRIC) CONVERSION FACTORS

Approximate Conversions to SI Units					Approximate Conversions from SI Units				
Symbol	When you know	Multiply by	To Find	Symbol	Symbol	When you know	Multiply by	To Find	Symbol
		LENGTH					**LENGTH**		
in	inches	25.40	millimeters	mm	mm	millimeters	0.0394	inches	in
ft	feet	0.3048	meters	m	m	meters	3.281	feet	ft
yd	yards	0.9144	meters	m	m	meters	1.094	yards	yd
mi	miles	1.609	kilometers	km	km	kilometers	0.6214	miles	mi
		AREA					**AREA**		
in^2	square inches	645.2	square millimeters	mm	mm^2	square millimeters	0.00155	square inches	in^2
ft^2	square feet	0.0929	square meters	m^2	m^2	square meters	10.764	square feet	ft^2
yd^2	square yards	0.8361	square meters	m^2	m^2	square meters	1.196	square yards	yd^2
ac	acres	0.4047	hectares	ha	ha	hectares	2.471	acres	ac
mi^2	square miles	2.590	square kilometers	km^2	km^2	square kilometers	0.3861	square miles	mi^2
		VOLUME					**VOLUME**		
fl oz	fluid ounces	29.57	milliliters	mL	mL	milliliters	0.0338	fluid ounces	fl oz
gal	gallons	3.785	liters	L	L	liters	0.2642	gallons	gal
ft^3	cubic feet	0.0283	cubic meters	m^3	m^3	cubic meters	35.315	cubic feet	ft^3
yd^3	cubic yards	0.7645	cubic meters	m^3	m^3	cubic meters	1.308	cubic yards	yd^3
		MASS					**MASS**		
oz	ounces	28.35	grams	g	g	grams	0.0353	ounces	oz
lb	pounds	0.4536	kilograms	kg	kg	kilograms	2.205	pounds	lb
T	short tons (2000 lb)	0.907	megagrams	Mg	Mg	megagrams	1.1023	short tons (2000 lb)	T
		TEMPERATURE (exact)					**TEMPERATURE (exact)**		
°F	degrees Fahrenheit	(°F-32)/1.8	degrees Celsius	°C	°C	degrees Celsius	9/5+32	degrees Fahrenheit	°F
		FORCE and PRESSURE or STRESS					**FORCE and PRESSURE or STRESS**		
lbf	poundforce	4.448	Newtons	N	N	Newtons	0.2248	poundforce	lbf
lbf/in^2	poundforce per square inch	6.895	kilopascals	kPa	kPa	kilopascals	0.1450	poundforce per square inch	lbf/in^2

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Final Report

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1 EXECUTIVE SUMMARY

Oklahoma rated first in the Nation in the percentage of bridges that are structurally deficient or functionally obsolete. According to Federal Highway Administration data, Oklahoma uses approximately 23,250 bridges maintained by state, County, City, and Tribal governments. In 2003 32% of these bridges were identified as structurally deficient and another 6% were identified as functionally obsolete. The Oklahoma Department of Transportation (ODOT) estimated that it would cost $3.4 billion to replace these bridges. Obviously, any methods that can successfully extend the life of these bridges will be of great benefit to the safety of the users as well as the pocketbooks of the tax paying citizens.

Approximately 16,000 of the 23,250 (69%) bridges in the state are considered off-system bridges. As such ODOT does not construct or maintain these bridges. Therefore, the majority of bridges in the state are maintained by County, City, and Tribal governments. Unfortunately, these entities generally only have in-house personnel and expertise to barely keep up with basic bridge maintenance. Various counties have grouped together to form Circuit Engineering Districts which have provided for limited centralized planning and design. However, bridge repairs are commonly designed on-the-fly using salvaged materials or other left over materials that have been stock piled in storage yards.

The research team worked with engineers and bridge maintenance personnel from the ACCO, ODOT, and Oklahoma CED's to identify, evaluate, and develop two repair techniques for deteriorating off system bridges. The project focused on repairing decayed timber piles and corroded H-piles. To repair decayed timber piles, the decayed pile was removed from the pile cap to sound wood below grade and then replaced with steel members that are commonly available to bridge maintenance personnel. These included steel pipe and steel H-pile sections. The corroded H-piles were repaired by welding steel plates to the flanges of H-piles containing corroded webs. The repair techniques were evaluated in the field through load testing strain gauge instrumented bridges. The steel H-pile repair was also investigated under controlled laboratory testing. The repairs were found to adequately restore strength and stiffness and transfer loads from the super structure to the foundation. Straight forward design guidelines and design tables were developed for each of the repair techniques. The recommended repairs are useful for extending the life of a bridge that contains decayed timber piles or steel H-piles with corroded webs. The repairs allow for adequate safety to be maintained while a deteriorating bridge is waiting to be replaced. This allows for county, tribal, and local government officials to prioritize and wisely spend their bridge and road maintenance resources.

The repair techniques that were evaluated and developed during the project have been shared and implemented for county and local bridges within the state. The repair techniques have been passed on to county and state personnel that participated in the project. The repair techniques have also been presented to engineering students in the timber engineering course taught at Oklahoma State University. Technology transfer efforts continue in a variety of forms. The technology will further be transferred to county commissioners and county bridge personnel through the Association of County Commissioners of Oklahoma. The technology will be transferred to other local and tribal officials through the Center for Local Government Technology at OSU. The technology will also be continued to be shared with students in appropriate courses at OSU.

2 INTRODUCTION

2.1 Problem and Background

Oklahoma rated first in the Nation in the percentage of bridges that are structurally deficient or functionally obsolete. According to Federal Highway Administration data, Oklahoma uses approximately 23,250 bridges maintained by state, County, City, and Tribal governments. In 2003 32% of these bridges were identified as structurally deficient and another 6% were identified as functionally obsolete. The Oklahoma Department of Transportation (ODOT) estimated that it would cost $3.4 billion to replace these bridges. Obviously, any methods that can successfully extend the life of these bridges will be of great benefit to the safety of the users as well as the pocketbooks of the tax paying citizens.

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2.2 Objectives

The overall objective of the proposed project was developing repair and retrofit techniques for deteriorating or deficient bridges maintained by county, city, and tribal governments. These repair and retrofit techniques will be employed by engineers, maintenance crews, and government officials in charge of maintaining County, City, and Tribal bridges. Therefore, the project had three major objectives.

1. Develop and grow contacts with County, City, and Tribal officials and personnel involved with maintain bridges.
2. Develop repair and retrofit techniques for critical bridge problems.

3. Develop educational modules on the developed repair and retrofit techniques for inclusion in short courses for County, City, or Tribal personnel and university structural engineering design courses.

A team was put together to successfully accomplish the project objectives. This team was formally composed of Robert Emerson (PI), Tyler Ley (Co-PI), and two graduate research assistants, David Pretorius and Jared Sparks. ODOT and County personnel that participated in the project were extremely valuable to the project's success and were considered informal members of the research team. The PI and Co-PI will actively participated in all aspects of the project. Two graduate research assistants performed most of the analytical and experimental work in the development and evaluation of the bridge repair and retrofit techniques. The researchers worked with ACCO and CLGT personnel to adopt the design guidelines for the bridge repair and retrofit techniques into short courses for bridge maintenance personnel. The PI also adopted the developed repair and retrofit techniques into appropriate engineering courses.

2.3 Scope and Methodology

Bridge repair and retrofit techniques were evaluated and developed. These bridge repair and retrofit techniques were developed specifically for use by County, City, and Tribal engineers and bridge maintenance officials. As such the research team worked closely with county officials within the state.

The project started with collaboration between the OSU research team and county officials concerned with bridge repair. Common as well as difficult bridge repair problems were identified and reviewed. Decisions were made on which of these bridge problems have the biggest repair/retrofit needs to be addressed. Commonly available repair materials and repair equipment and personnel were identified and reviewed. Bridge repair and retrofit techniques were then developed for the critical bridge problems using commonly available materials, equipment, and personnel. The developed bridge repair/retrofit techniques were investigated both analytically and experimentally. Straightforward design guidelines were written for each successfully developed repair/retrofit technique.

2.4 Technology Transfer

The evaluated and developed bridge repair/retrofit techniques have been shared with the county officials that were interested in the results of this project. Further transfer of the technology will be accomplished through partnerships with the Oklahoma Transportation Center, the Association of County Commissioners of Oklahoma, the Oklahoma Department of Transportation, and the Center for Local Government Technology at OSU.

2.5 Report Order

The main body of the report is divided into two sections. Section three covers the design of steel splice repair for decaying timber piles on Oklahoma county bridges. Section four covers county bridge repair of steel H-piles with corrosion damage through the cross section.

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3 DESIGN OF STEEL SPLICE REPAIR FOR DECAYING TIMBER PILES ON OKLAHOMA COUNTY BRIDGES

3.1 INTRODUCTION

Throughout the United States, bridges serve as an extremely important part of our infrastructure. As time passes and bridges age, they inevitably deteriorate and require maintenance. Unfortunately, all across the country the resources are not made available to maintain the structural reliability of bridges, both on and off of State Highway systems. Oklahoma is not exempt from this issue. In fact, “Oklahoma rated first in the Nation in the percentage of bridges that are structurally deficient or functionally obsolete” [1]. In 2003, the cost of replacing all of these bridges was estimated to be $3.4 billion by the Oklahoma Department of Transportation (ODOT) [1]. Obviously, this creates situation where repairs to extend the service life of bridges are a necessity. While ODOT is responsible for many of these bridges and their maintenance, 69% are off-system bridges [1]. This means they are looked after by County, City, and Tribal governments, who generally have even less resources and expertise in bridge maintenance. These entities use what they can to keep the functionality and structural capability of their bridges up to code. It was estimated that only 15 off-system bridges are replaced each year. Again, it is reiterated that repairs to keep bridges in service are a necessity.

Bridges superstructures are constructed from steel, concrete, or timber. The typical timber pile that needs replacement has undergone around fifty years of exposure to the elements [2]. Bridge inspections determine which piles are in the worst shape, and which ones need to be repaired or replaced. Typically, the inspector uses a sound test to estimate the amount of section loss in a decaying timer pile. An experienced inspector can use this sound test effectively to determine the state of decay in a timber structure [3]. Different agents of wood deterioration include moisture, oxygen, temperature, bacteria, fungi and insects. These are in addition to physical problems that might damage a pile such as impacts or every day wear and tear [3].

3.2 OBJECTIVE

The purpose of this project is to provide County, City, and Tribal engineers with a simple, inexpensive bridge repair that utilizes readily available material and manpower. Specifically, this thesis describes a “splice” that replaces old, rotten timber piles with steel members (pipes or H-Piles). This repair is simple, cost effective, and reliable. The repair, which has been installed previously before this project on a few bridges across Oklahoma, was field tested and analyzed for the project. The final results of the project consist of installation instructions and simple design tables that have been calculated using the latest codes. These design tables and installation instructions standardize the repair, insuring any future use of the repair will be a safe, useful improvement on the bridges superstructure.

3.3 REVIEW OF LITERATURE

3.3.1 Information from County and State Meetings

In order to identify reoccurring structural issues in county bridges, meetings were organized with both ODOT and the Association of County Commissioners of Oklahoma (ACCO). In a meeting with Walter Peters, P.E., an assistant bridge engineer at ODOT, numerous topics regarding bridge functionality and inspection were discussed. Although most of the information gathered did not apply to the focus of the project, a few details were relevant. The substructure, rather than the deck, of a bridge is thought to be more important to the bridges rating during an inspection. This means if given the option, the piles, beams, pile caps, and abutments are highest priorities when choosing components to repair. Mr. Peters was able to mention a few common issues ODOT experienced with their bridges, but none were of the structural nature. He also referred us to a number of other sources for more information.

A second meeting was organized with The ACCO in Oklahoma City. Here, Randy Robinson, P.E., Donny Head, and Jimmy Watson described common issues and the current methods used to repair county bridges. The poor condition and lack of resources was reinforced throughout the meeting, as make-shift repairs were discussed. According to ACCO, 600 county bridges are functionally obsolete and 4300 are structurally deficient. It was mentioned that timber piling used in bridges that were built as long ago as the 1930's were deteriorating all over the state.

This was especially true in conditions that provided excessive moisture, such as creeks or rivers. Donny Head explained that timber bridge components should "last" about fifty years before they need to be replaced or repaired. This figure was estimated by Mr. Head's observations in the field throughout his career and is highly variable depending on the conditions of each bridge. While it is evident that these timber bridges will need to be replaced altogether, the resources are simply not there to achieve every bridge at this time. ACCOK claims that if the deck and beams are in relatively good condition, that the piles should be repaired in order to extend the service life of the bridge for at least a few years.

One specific repair was mentioned in the meeting that had been used around the state previously. This repair involved removing decayed timber piles from under the bridge and splicing metal shapes, either pipe or H-Pile, under the bridge to replace the timber. In the field, this repair has been considered when 50% section loss of a pile has been estimated. It has been observed by ACCOK that the timber piles do not decay or rot below grade. Because of this, the steel member can be connected to the existing timber pile that extends into the ground, thus resulting in a pile repair that does not require driving a new pile into the soil. Avoiding pile driving in the repair provides numerous benefits. The connection of the steel member to the existing pile below ground consisted of a "sleeve" of metal pipe that capped off the top of the timber pile. This cap is welded onto the new steel pile, which extends all the way to the pile cap. The repair is displayed below.

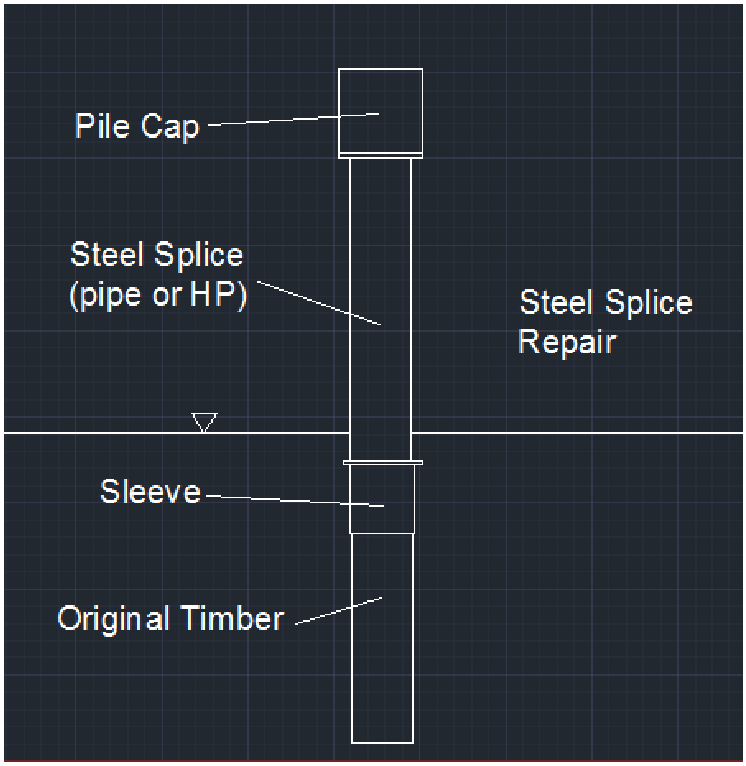


Figure 1. Steel Splice Repair

This repair method provided the perfect opportunity for the project. It was an idea that ACCOK came up with that is inexpensive and easy to install. However, ACCOK has not tested the repair's effectiveness of transferring load from the bridge deck to the soil. There is also no design standard for the repair.

3.3.2 Other Repair Procedures

In order to design the best possible repair, different timber piling repairs were researched. The following are some of the more common or feasible repairs available. Epoxy injection: Epoxy injection is repair technique that is used in both timber and concrete structures. Epoxy itself is a mixture of two agents that combine to form a hard, durable material. This mixture is used to fill voids in timber or concrete before it hardens, and restores the structural integrity of the member. Epoxy can provide additional strength as well as an increase of protection from the elements. There are different types of epoxy mixes, each varying for the type of application needed. It can either be applied to the surface of a pile, or injected deep into the timber. In the case of epoxy injection, a closed system is required, such as a fiberglass sleeve wrapped around the timber, to prevent epoxy from escaping the pile. For this type of application, Type A-2 Epoxy should be used [3]. According to Dr. Riding of Kansas State during his lecture of a concrete repair class, epoxy injection can be expensive and difficult to use. While it does not require continued maintenance, special training is needed to correctly install an epoxy injection repair,

C-Channel Jacket: Suggestions from Roe Enchayan, P.E., Nebraska Department of Roads, were presented at the Midwest AASHTO Bridge Preservation Conference in 2010. One of his suggestions involves two C-Channel shapes forming a jacket around a damaged timber pile, as shown in figures 2 and 3 These shapes are held together by a series of bolts that run through the pile. When tightened, the C-Channel's compress the pile around the longitudinal axis, strengthening any axial loads applied to the timber [4]. This repair is inexpensive, but is not as effective as a complete pile replacement.

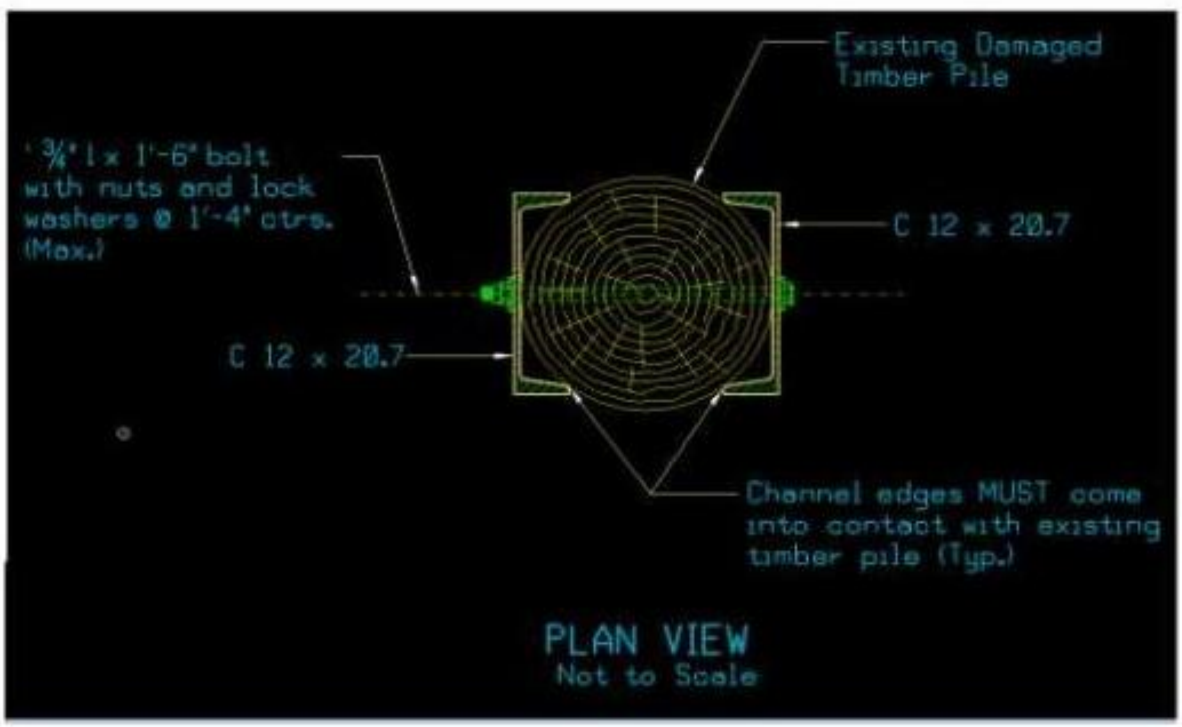


Figure 2. Plan View of C-Channel Jacket

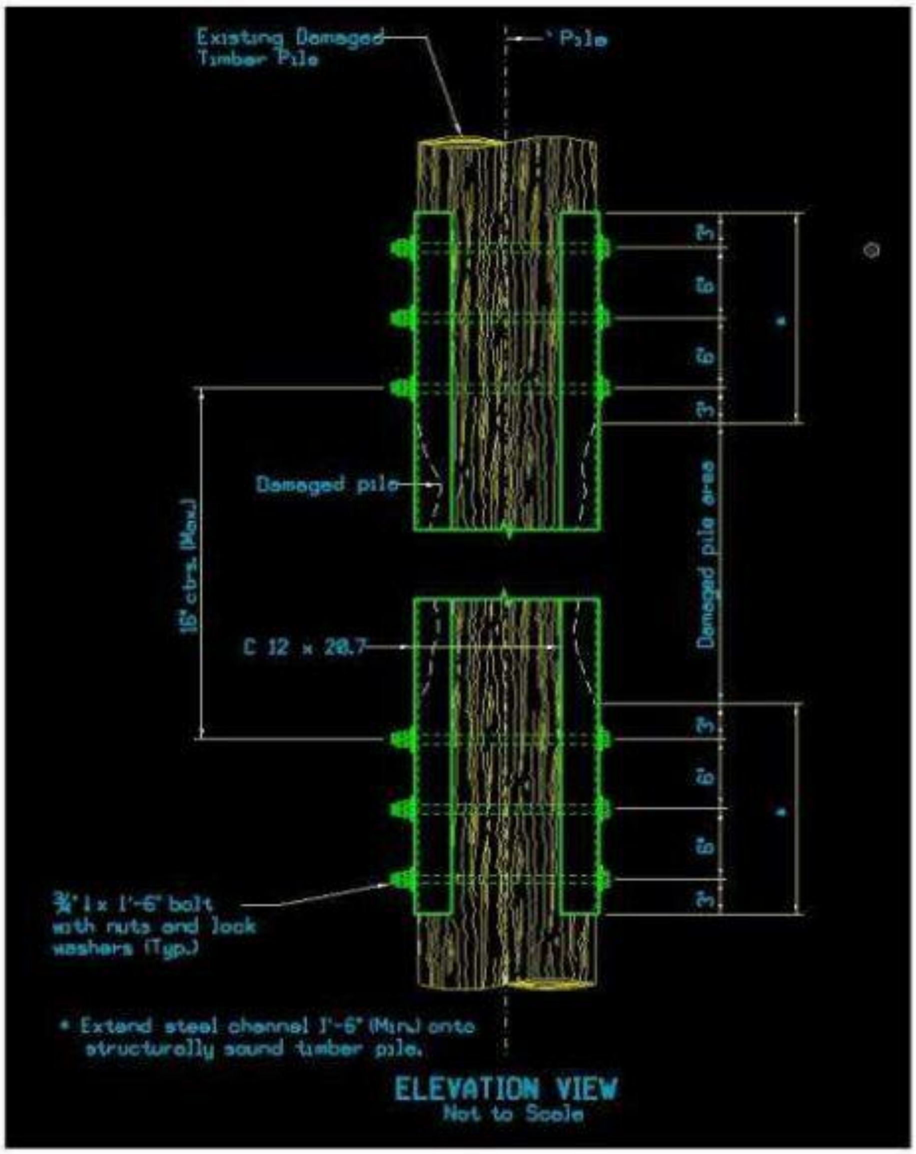


Figure 3. Elevation View of C-Channel Jacket

Concrete Casing: Another repair suggested by Enchayan is a concrete casing, illustrated in figure 4. Similar to the C-Channel design, a cast-in-place concrete form is poured around the outside of the damaged timber pile. This repair protects the damaged timber from the elements and adds strength to the pile. However, forming and pouring concrete can be an expensive, labor intensive process.

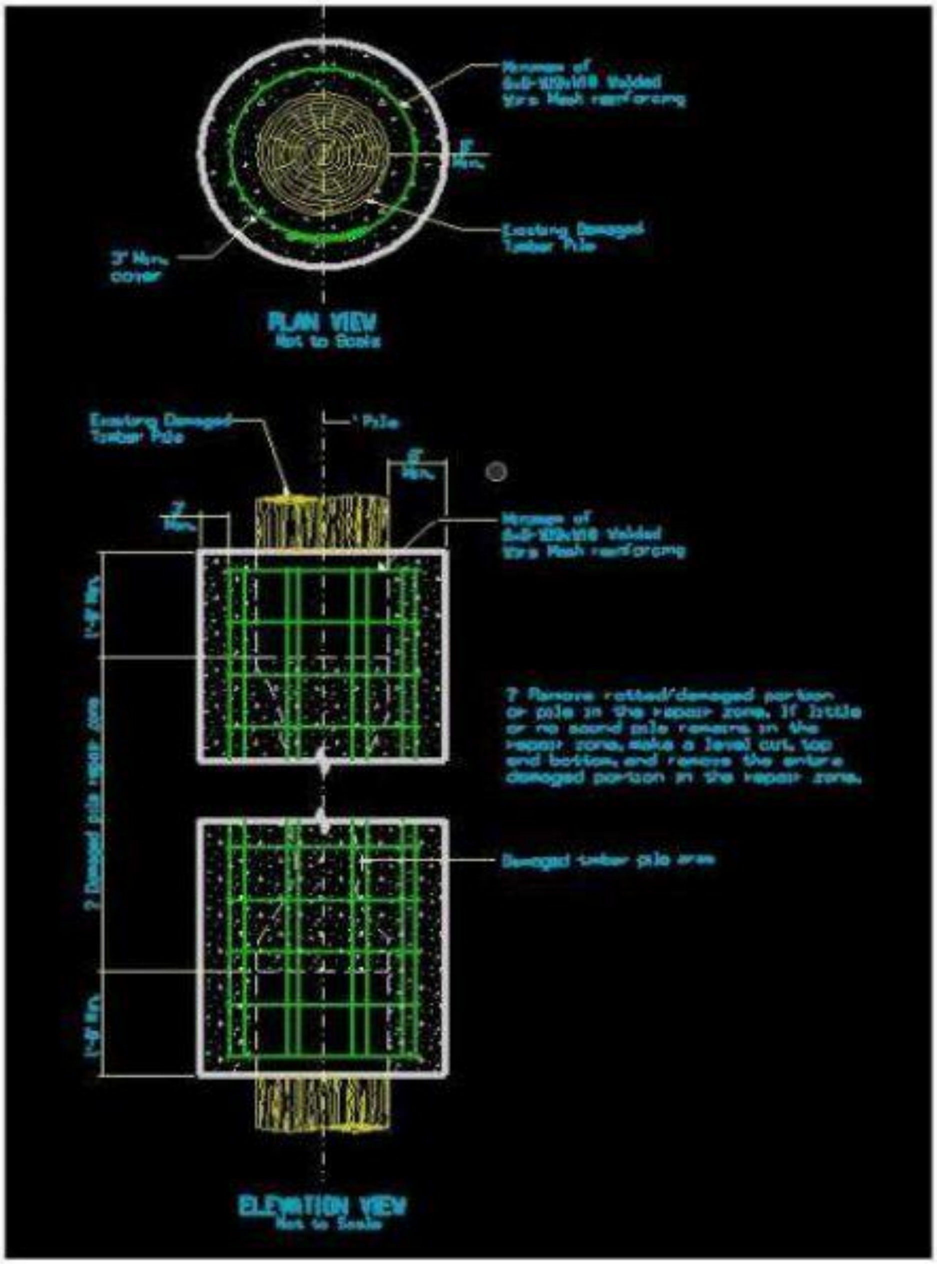


Figure 4.1 Elevation View of Concrete Jacket

Additional Piles: Another simple solution to timber pile decay is to install additional piles as shown in figure 5. This repair requires a large amount of material and labor, especially due to the need for the new piles to be driven. This solution provides an alternate load path, which may be necessary depending on the condition of the existing timber piles.

Figure 5. Example of Additional Piles as Repair

Splice: The Splice repair described in Enchayan's presentation is very similar to the repair described in this thesis. A few minor details make Enchayan's more complex and therefore more expensive. As shows in figures 6 through 10, the splice is assembled in two sections, the top and bottom. After the rotten timber is cut out and removed, both pieces of the splice assembly are moved into place. The top unit of the splice acts as a mechanism to ensure the entire replacement assembly will take load from the pile cap. By raising the top of the unit with a jack, as shown in figure 10, stress is transferred from the pile cap to the splice to the existing timber pile. The differences between this splice repair and the one designed from this project are in the size of the member and the mechanism through which the steel will take the load. This project specifies a pipe size or H-Pile shape for an individual timber pile, dependent on its length and diameter. By designing for a specific case, the steel shape can be optimized and the cost will be minimized. In addition, this projects splice repair will rely on a jack which is separate from the replacement unit to relieve and then reapply stresses from the pile cap. The process is similar, however the fabrication and design of Enchayan's top unit would be more expensive and more time consuming than the procedure that will be followed in the new repair.

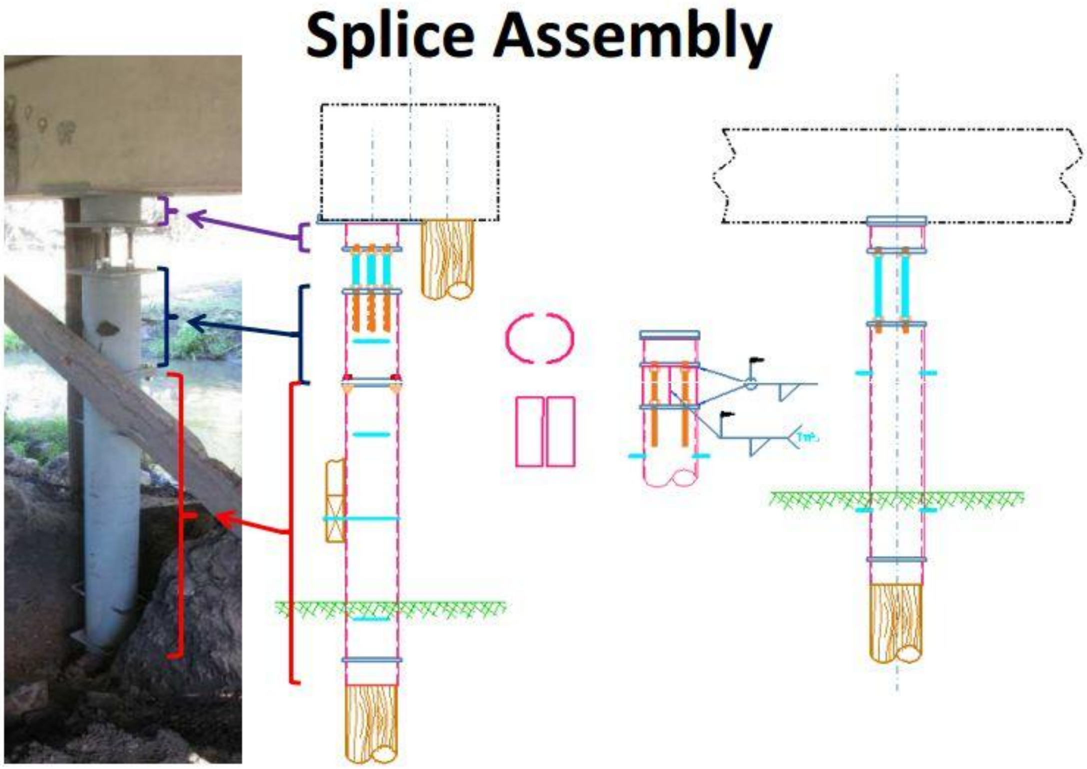


Figure 6. Splice Assembly

Figure 7.2 Splice Assembly

Figure 7. Installing the Splice

Figure 9. Installed Splice

Figure 10. Raising the Jack

3.4 METHODOLOGY

The project can be split into three areas, each of which needed to be investigated thoroughly. These three areas are field testing, the repair design, and the repair installation.

3.4.1 FIELD TESTING METHODOLOGY

Ongoing communications with Donnie Head presented the opportunity to visit a bridge that had utilized the splice repair. By testing the performance of the repair and surrounding piles, the effectiveness of the repair could be determined. As stated before, ACCOK had no information or data supporting the ability of the splice repair to effectively transfer load from the bridge to the existing, buried pile. By using bridge testing technology and simple strengths of materials theory, the amount of load passing through the replacement pile could be collected.

BDI Testing

Bridge Diagnostics Inc. produces a variety of testing systems to evaluate the performance of a bridge. The BDI product used for this project was the Structural Testing System II (STS-II). The STS-II combines sensors and software during a loading event to measure the strain of up to 40 different locations on a testing specimen. By measuring the strain of a member, we can calculate the load passing through it. The STS-II system consists of 40 strain sensors that can be attached to different components of a bridge. These strain sensors, shown attached to timber in figure 11, are designed to be attached to steel, timber, or concrete bridge parts. To attach to timber, a screw is drilled into each end of the sensor and into the wood. Again, this is illustrated in figure 11.

Figure 11.3 STS-II Strain Transducer Installed in 4X4

To attach to steel, small steel tabs are bolted to each end of the strain transducer. These tabs are shown in figure 12. The tabs are then glued to the bridge using Loctite Prism 410 Black Toughened adhesive and Loctite Tak Pac 7452 accelerator.

Figure 12. Tabs and Nuts for STS-II Assembly

If the location of the sensor is on the edge of a member such as a flange, C-clamps can be used to hold both ends of the sensor instead of the tabs, as shown in figure 13.

Figure 13. Strain Transducers Installed on H-Pipe

To attach the sensors to concrete, either screws or the adhesive can be used. For the field testing purposes, both timber attachments and steel attachments were utilized. Each one of the strain transducers is plugged into a box, shown in figure 14. These boxes are linked together, and plug into the STS-II Power Supply interface. The power supply interface is hooked into a laptop, which runs the STS-II software program that will record the change in strain from each transducer over a given time period. The STS-II Power Supply and laptop are shown in figure 15.

Figure 14. STS-II Boxes and Wires

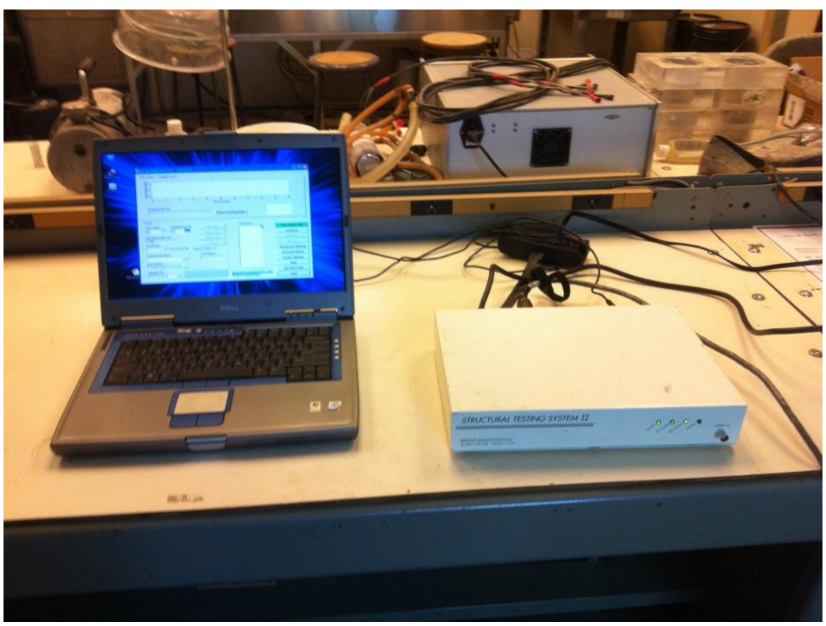


Figure 15. STS-II Power Supply and Laptop

The STS-II measures strain at multiple locations on one or multiple components of a bridge. To use this data and discover how much load that component is withstanding, the following equations and theories are used:

$$\varepsilon = \frac{\delta}{L_o} \tag{2-1}$$

Where ε is the strain, δ is the change in length of the specimen, and L_o is the original gauge length of the specimen. The STS-II system's output in the strain at a sensor, measured in microstrain. The average strain for each member is calculated. For example, if four transducers are attached to a pile, the average strain of those four sensors is calculated and assumed for that pile. Once the strain is calculated, Hooke's Law can be used to determine the stress at that specific location:

$$\sigma = E\varepsilon \tag{2-2}$$

Where σ is the stress, E is the modulus of elasticity, and ε is the strain. The application of Hooke's Law is limited by restraints regarding the amount of loading and deformation. However, in the field test, the piles will not be loaded beyond yield strength; therefore Hooke's Law can be applied. The stress calculated in the piles is the average stress throughout the cross section of the member.

Once the stress through a member is calculated, the force that member is undergoing can be calculating using the following equation:

$$\sigma = \frac{P}{A}$$

Or

$$P = \sigma A \tag{2-3}$$

Where P is the internal resultant normal force, A is the cross-sectional area of the pile, and σ is the average stress at any point on the cross sectional area (All Eqns from Mechanics of Materials).

In conclusion, the strain of the replacement pile can be measured and the load that the pile is "taking" can be calculated. This can be compared to the surrounding piles and the theoretical load the replacement pile should be undergoing.

The STS-II components were initially connected and tested in a controlled lab environment. Four strain transducers were used to test a scrap piece of 4X4. The purpose of this test was to familiarize the graduate students with the STS-II configuration and software. The test specimen was placed in a compression machine, as shown in figure 16 and loaded. The strains were recorded throughout the loading process in the STS-II software program.

Figure 16.4 Testing the STS-II and Sensors in a Compression Machine

Using equations 3-1 through 3-3, the calculated load was compared to the actual load applied to the specimen. Again, the purpose of this lab testing was to familiarize the graduate students with the STS-II system, not to test the accuracy of the sensors. The sensors averaged an 18.5% error in load calculation. It was assumed that this error in the calculation was due to the poor condition of the wood sample, which made the modulus of elasticity difficult to estimate. Furthermore, the sample was not evenly distributing the load throughout the entire cross sectional area because the ends were not exactly perpendicular to the compression machine's loading surfaces. The results of this test were not considered important or relevant.

A separate lab test was later conducted, ensuring the performance of every component of the STS-II system. Unlike the previous lab test, which only used four transducers, this second test required the use of all 40 sensors. Each transducer was plugged into the STS-II system and tested, as shown in figure 17.

Figure 17. Testing All (40) Strain Sensors

The results of this simple test concluded that every sensor worked and could be balanced out to provide an accurate strain value during testing.

Field testing took place on a county bridge (NBI No: 14300; Local ID: 355) located approximately seven miles north of Medford, Oklahoma in Grant County. This three-span bridge carried a two-lane, asphalt road (N2960) over a small creek. It was built in 1959 with timber piles, timber pile caps, timber beams, and timber abutments. The bridge, whose load limit is posted at 5 tons, has span lengths of 16 feet, 15 feet, and 16 feet. There are five piles under each bent cap, spaced from 59” to 77” apart, center to center. The pile configuration is illustrated in figures 18 and 19.

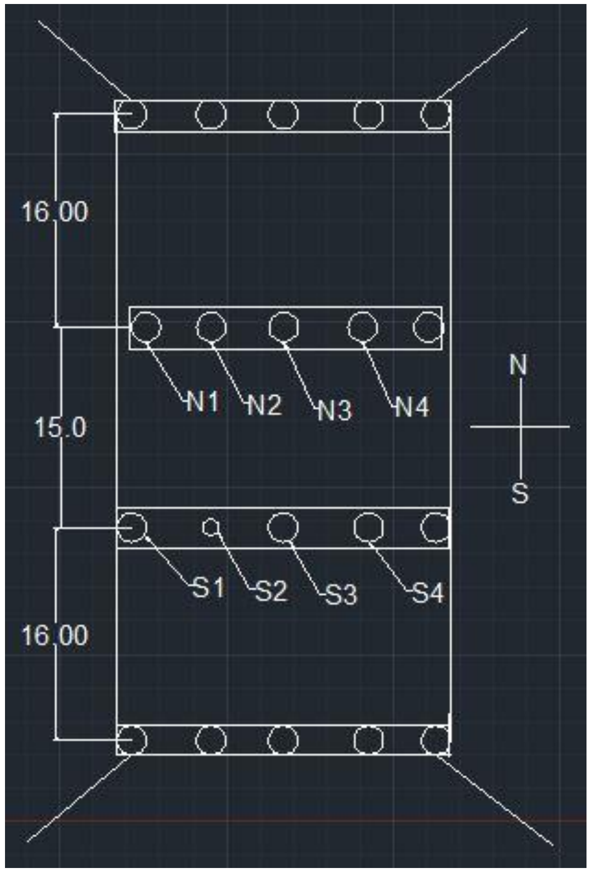


Figure 18. Plan View of Grant County Bridge Piles

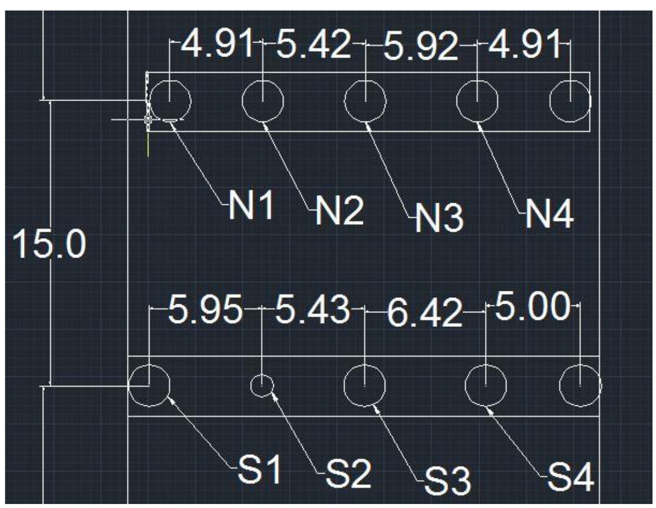


Figure 19. Plan View of Grant County Bridge Piles (with dimensions)

The dimensions of the piles range from 11.7 inches to 14.2 inches, not including the replaced steel pipe. They also ranged from 71 inches to 93.5 inches in height. The pile properties are summarized in table 1.

Table 1 Pile Properties

Pile	Height (in.)	Diameter (in)
N1	71.00	14.16
N2	69.00	11.70
N3	72.00	12.02
N4	69.00	13.01
S1	89.00	11.94
S2	83.00	7.63
S3	84.50	11.94
S4	93.50	12.77

The timber beams are 15 inches by 4 inches and are spaced at 18 inches, center to center. Some are showing signs of damage such as cracking or decay. One of these beams has been replaced with a steel W-Shape, as shown in figure 20.

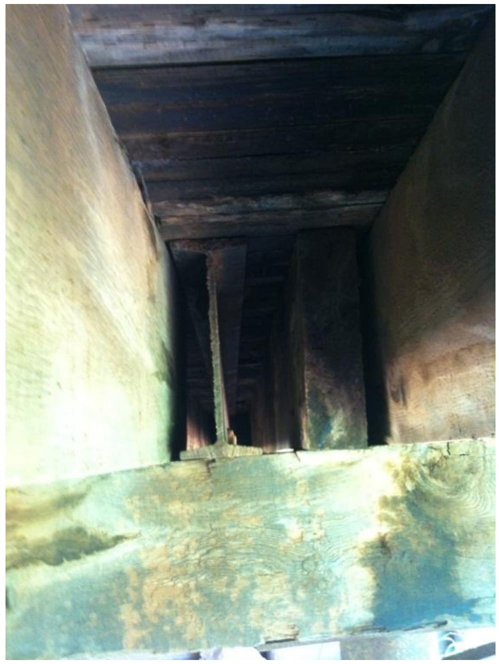


Figure 20. W-Shape Acting as a Replacement Timber Beam

The Pile caps are 12 inches by 12 inches, and seem to be in satisfactory condition. The bridge was considered structurally deficient after its last inspection in May of 2011. Throughout the duration of the field test, which took approximately three hours, there were numerous trucks that drove across the bridge that were well beyond the posted load limit of 5 tons.

The splice repair was installed on the south bent cap, designated as S2 (shown in figures 18 and 19). According to past inspections the replaced timber pile had undergone more than 50% section loss and severe splitting. The replacement pipe is illustrated in figure 21.

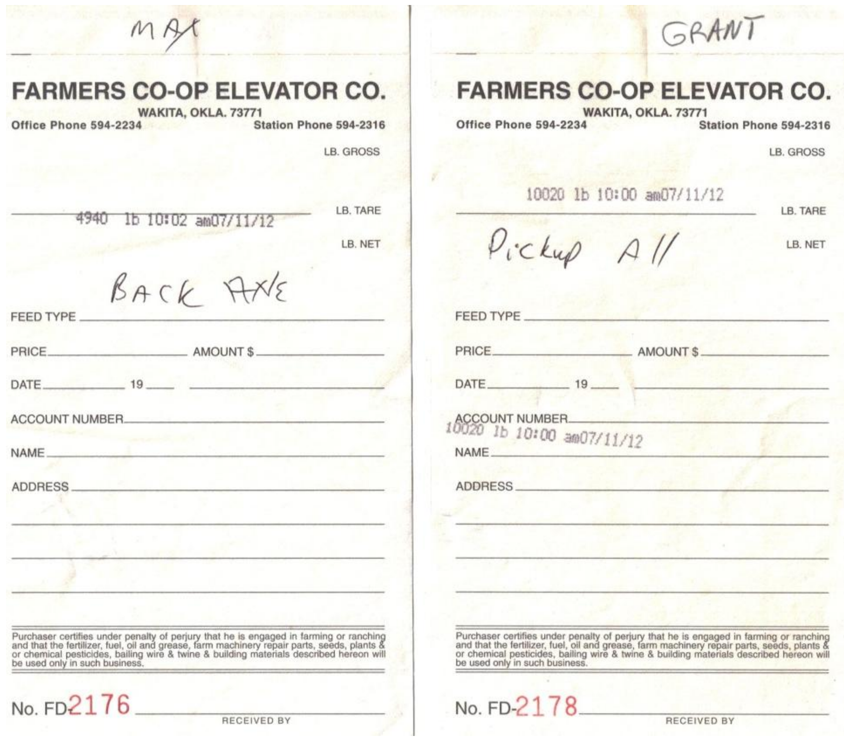
Figure 21. Installed Steel Splice

The pipe which replaced the timber pile was 7 5/8 inches in outside diameter and had a wall thickness of 0.450 inches. The grade of the steel is unknown, but was assumed to be grade 50. The pipes connection to the bent cap is shown in figure 22.

Figure 22. Top Connection

As illustrated, the pipe is welded to a plate, which is bolted to the bent cap. The bottom of the pipe is welded to the sleeve, underground.

The performance of the bridge and repaired pile were recorded with the STS-II system. The strain of eight piles was recorded as a 5 ton truck drove across the bridge. Max Hess, the county commissioner of Grant County, was courteous enough to supply a 2008 Ford F-250 Extended Cab for the test. The truck had a wheelbase of 6'-6" wide and 12'-2.5" long. The trucks front axle weighed 5,060 pounds (2.52 tons) and the back axle weighed 4940 pounds (2.47 tons). The total weight of the truck was 10,020 pounds (5.01 tons). These weights were recorded at the Farmers Co-Op elevator Co. in Wakita Oklahoma. The weigh slips are illustrated in figures 23 and 24.



MAX

FARMERS CO-OP ELEVATOR CO.
WAKITA, OKLA. 73771
Office Phone 594-2234 Station Phone 594-2316

LB. GROSS

4940 lb 10:02 am07/11/12 LB. TARE

LB. NET

BACK AXLE

FEED TYPE

PRICE AMOUNT $

DATE 19

ACCOUNT NUMBER

NAME

ADDRESS

Purchaser certifies under penalty of perjury that he is engaged in farming or ranching and that the fertilizer, fuel, oil and grease, farm machinery repair parts, seeds, plants & or chemical pesticides, bailing wire & twine & building materials described hereon will be used only in such business.

No. FD2176 RECEIVED BY

GRANT

FARMERS CO-OP ELEVATOR CO.
WAKITA, OKLA. 73771
Office Phone 594-2234 Station Phone 594-2316

LB. GROSS

10020 lb 10:00 am07/11/12

LB. TARE

Pickup All LB. NET

FEED TYPE

PRICE AMOUNT $

DATE 19

ACCOUNT NUMBER
10020 lb 10:00 am07/11/12

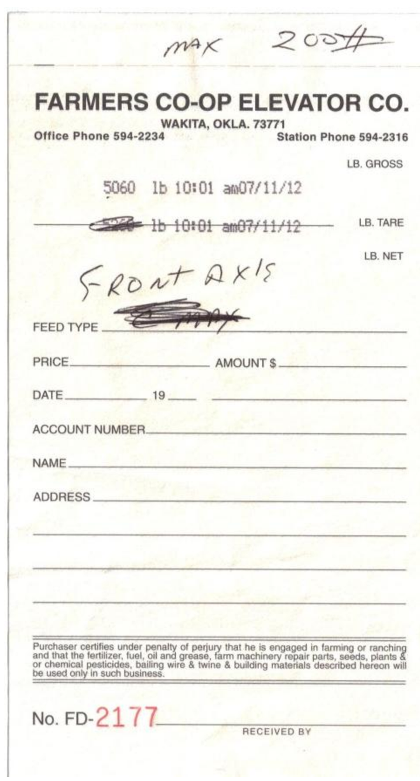
NAME

ADDRESS

Purchaser certifies under penalty of perjury that he is engaged in farming or ranching and that the fertilizer, fuel, oil and grease, farm machinery repair parts, seeds, plants & or chemical pesticides, bailing wire & twine & building materials described hereon will be used only in such business.

No. FD-2178 RECEIVED BY

Figure 23. Truck Weight Slip (1)



MAX 200#

FARMERS CO-OP ELEVATOR CO.
WAKITA, OKLA. 73771
Office Phone 594-2234 Station Phone 594-2316

LB. GROSS

5060 lb 10:01 am07/11/12

~~5060 lb 10:01 am07/11/12~~ LB. TARE

LB. NET

FRONT AXLS

FEED TYPE ~~MAX~~

PRICE AMOUNT $

DATE 19

ACCOUNT NUMBER

NAME

ADDRESS

Purchaser certifies under penalty of perjury that he is engaged in farming or ranching and that the fertilizer, fuel, oil and grease, farm machinery repair parts, seeds, plants & or chemical pesticides, bailing wire & twine & building materials described hereon will be used only in such business.

No. FD-2177 RECEIVED BY

Figure 24. Truck Weight Slip (2)

An illustration of the dimensions and distribution of load is provided in figures 25 and 26.

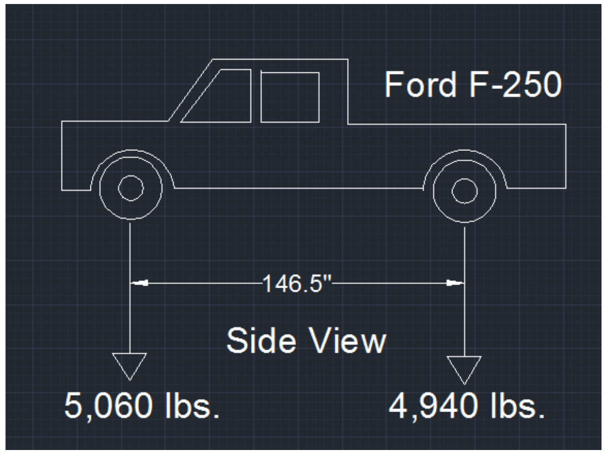


Figure 25. Side View of Ford F-250

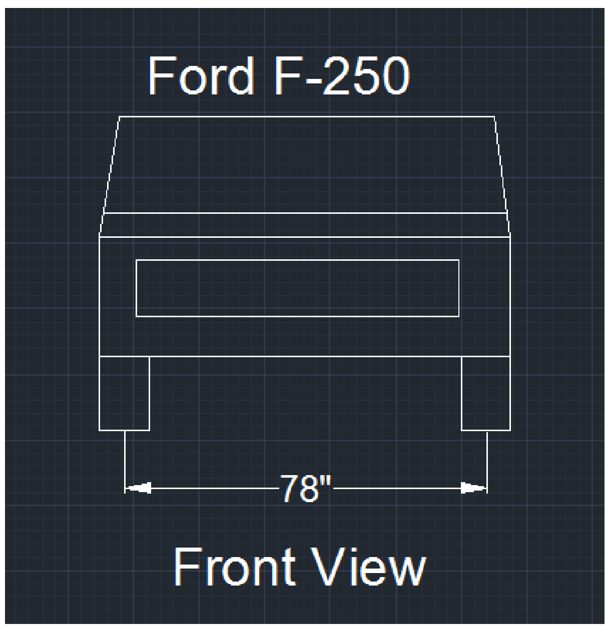


Figure 26. Front View of Ford F-250

The truck was driven along the bridge as close to the West edge as possible. By driving the truck in that lane, the loads throughout the bridge would be concentrated on the repaired pile and the piles surrounding it.

The strain transducers were applied to eight different piles. As illustrated in figure 18, these piles have been designated N1, N2, N3, N4, S1, S2, S3, and S4. These piles were chosen because they would theoretically be taking the most load when the load was driving across the bridge. 4 sensors were put on piles N1, N2, N3, S1, and S3. 2 sensors were installed on N4 and S4. 6 sensors were installed on S2. This distribution of sensors was chosen based on the importance of accuracy involved with different piles and the number of sensors available. N4 and S4, although important, were not as high of a priority to calculate an accurate strain value as S2, which had 6 sensors installed. The sensors were installed onto the timber piles with roofing screws on opposite sides of the pile. This is illustrated in figure 27, a picture of pile N1.

Figure 27. Pile N1 and Attached Strain Sensors

The other two sensors cannon be seen in the frame. Every “line” of sensors on each pile was the same height. The location of the line of sensors for each pile is described in table 2.

Table 2 Sensor Locations on Grant County Bridge Piles

Pile	Distance from Pile Cap to Sensor (in)
N1	25.00
N2	24.00
N3	22.50
N4	23.00
S1	34.00
S3	34.50
S4	31.75

For pile S2, the repaired pile, six sensors were installed. All were attached using the steel tabs and adhesive method. Four sensors were installed at mid-height of the pile (34 inches from the pile cap), and two were attached towards the top (3 inches from the pile cap). This was done to ensure a high level of accuracy when calculating the average strain of the repaired pile. The selection of these eight piles provided symmetry when comparing the performance of each pile cap.

In order to accurately measure the strain of each pile with a known load being applied to it, the truck stopped 6 times during its drive across the bridge. The truck stopped with its front wheels directly over the south bent (1), centered over the south bent (2), back wheels directly over the south bent (3), front wheels directly over the north bent (4),

centered over the north bent (5), and with its back wheels directly over the north bent (6). At each of these stops, a “click” was recorded in the STS-II software. These clicks provided a way to observe the trucks location throughout the duration of the test. For example, the third click in the software represented the third stop on the bridge.
Given the weight and dimensions of the truck and the dimensions of the bridge, structural analysis was used to determine the theoretical load applied to each bent cap at each stop. The known axle loads and their positions were used to calculate the resultant force on each bent cap (Front axle weight=5.060 kip, Rear Axle Weight=4.940 kip). These results are expressed in table3. The following figures illustrate the different positions of the truck along the length of the bridge.

Table 3 Theoretical Loads on North and South Bent Cap

Truck Position	South Load (kips)	North Load (kips)
1	6.23	0
2	6.05	2.06
3	5.88	4.12
4	4.02	5.98
5	2.02	6.04
6	0	6.14

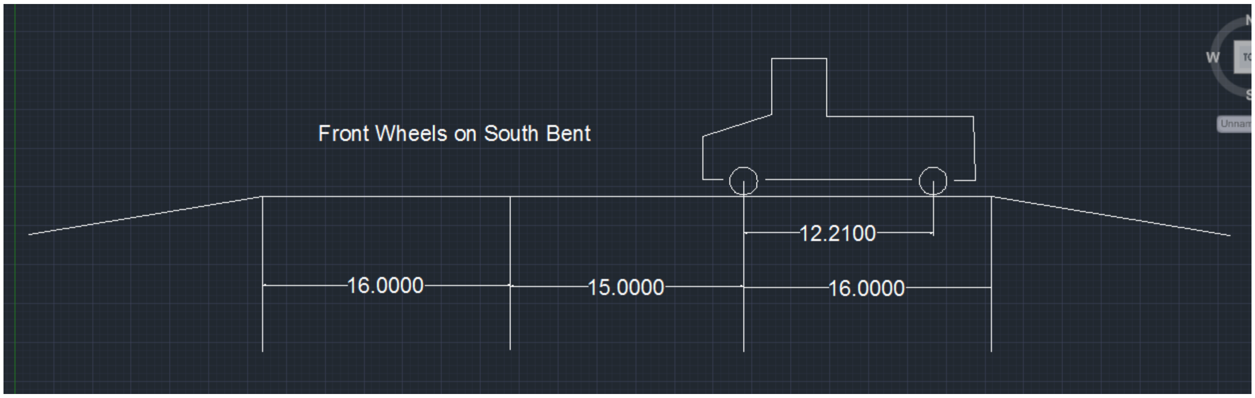


Figure 28. Truck Position "Front Wheels on South Bent"

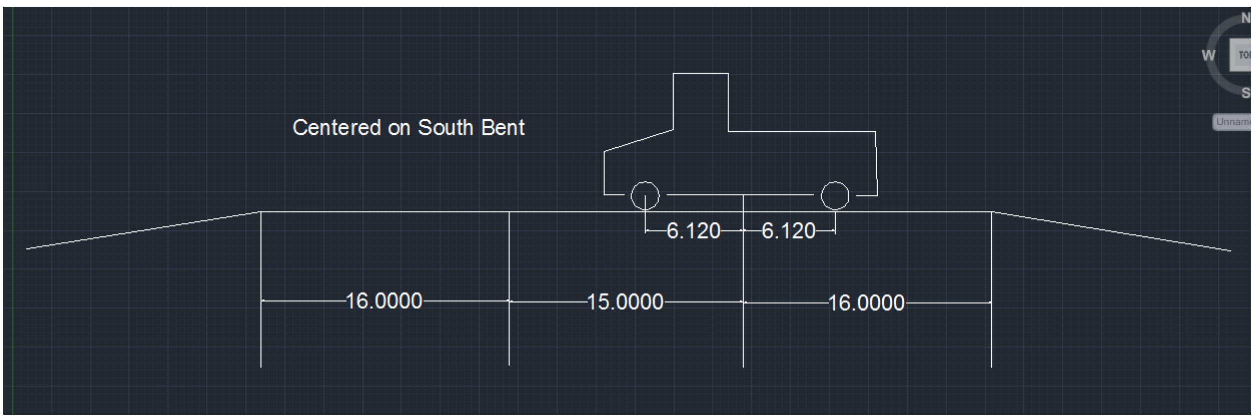


Figure 29. Truck Position "Centered on South Bent"

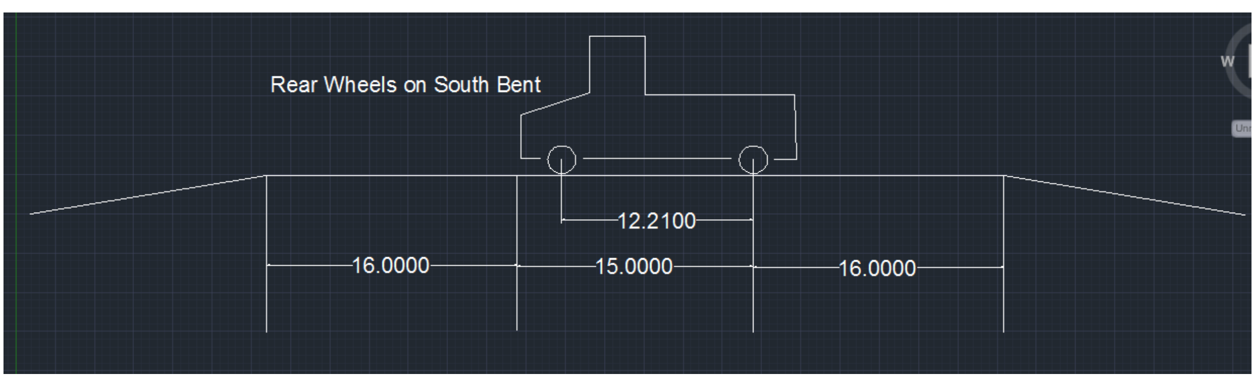


Figure 30. Truck Position "Rear Wheels on South Bent"

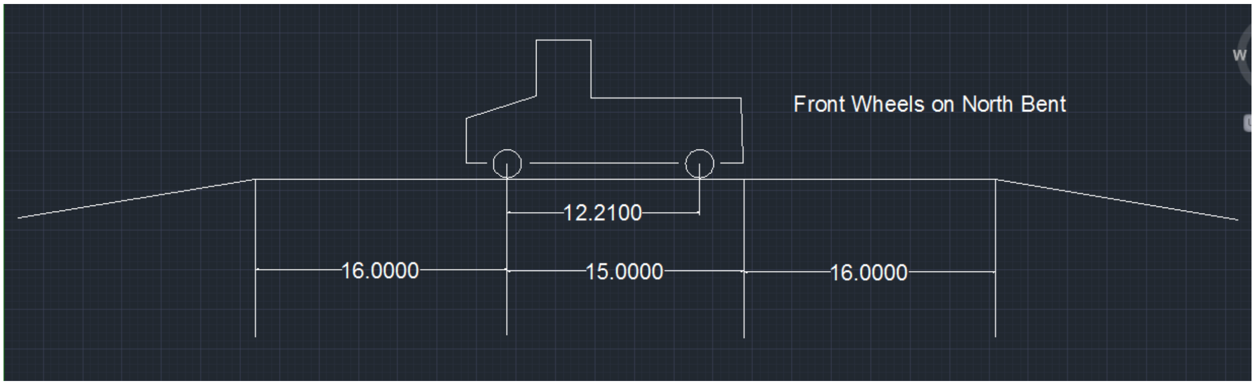


Figure 31. Truck Position "Front Wheels on North Bent"

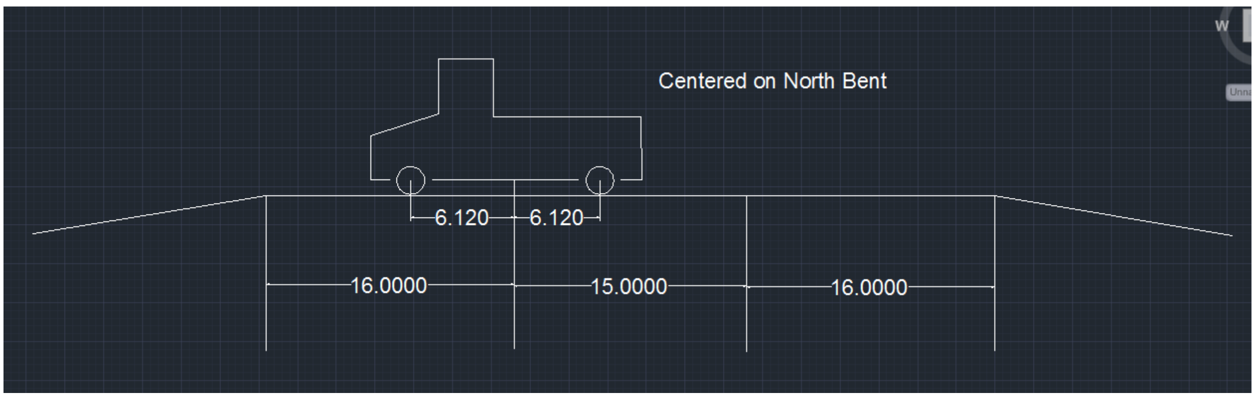


Figure 32. Truck Position "Centered on North Bent"

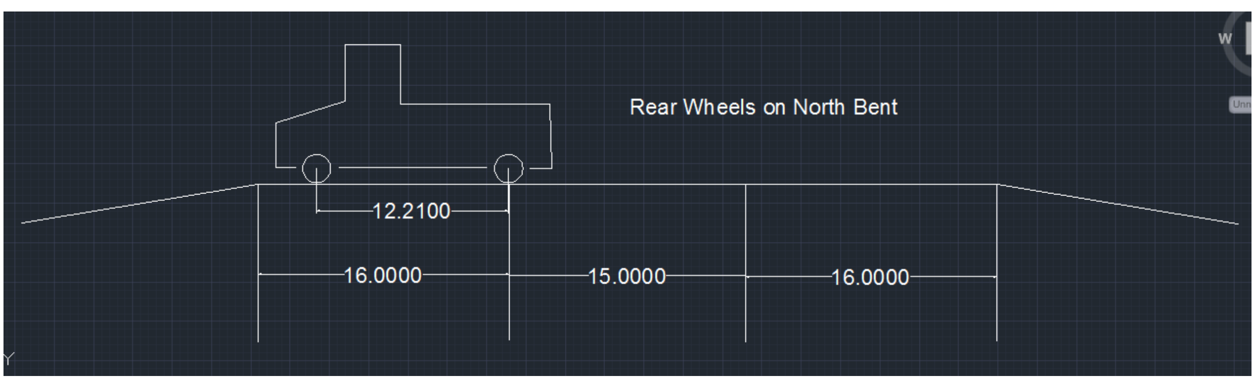


Figure 33. Truck Position "Rear Wheels on North Bent"

Both the North and South bent caps were modeled using RISA 3D. With a 100 Kip load distributed at the width of the truck tires, the axial forces on the South Bent are illustrated in figure 34. The pile cap was modeled to be a 12”X12” Southern Pine continuous beam. All dimensions are plotted as found in the field.

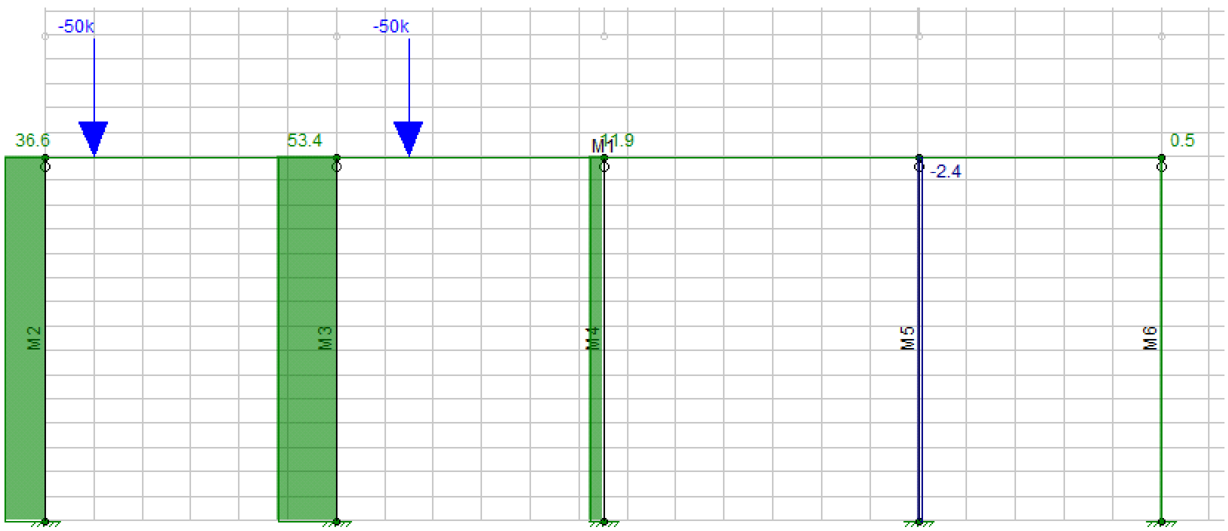


Figure 34. RISA3D Model of Load Distribution on South Bent Cap

Note that M3, the second member from the outside on the bent, should take about 53 Kip, or 53% of the axle weight. As a timber pile, this member should strain accordingly. For the North Bent, the spacing between the piles has changed by small amounts. Again, modeled in RISA 3D, the loads in kips taken by each pile is equivalent to the percentage of the load on the bent that the pile will be undergoing. The North bend is illustrated in figure 35.

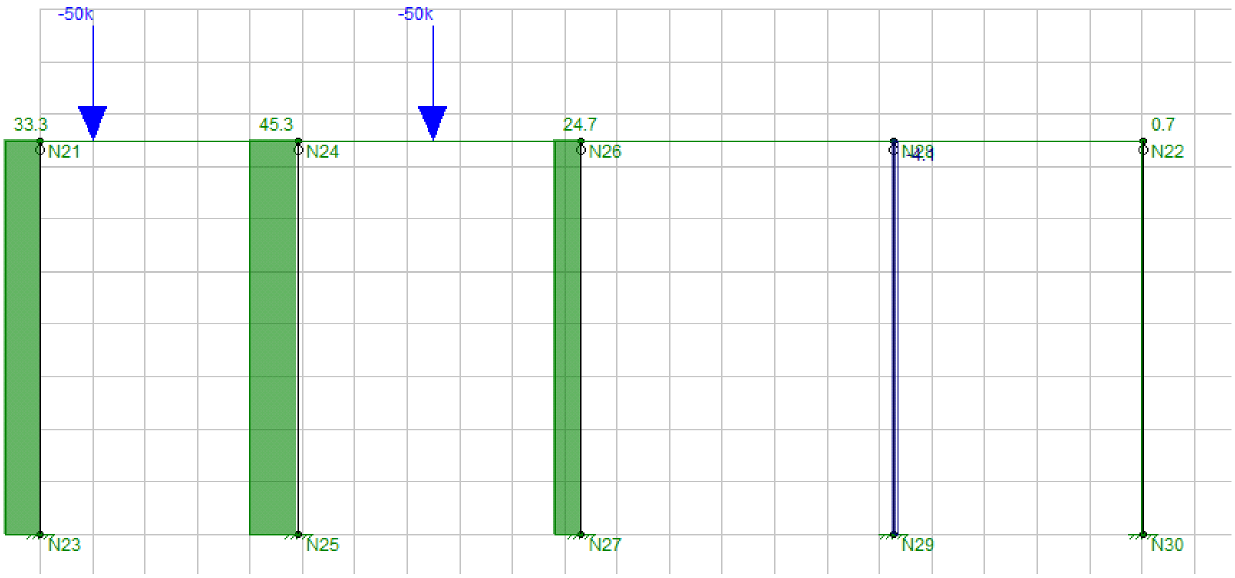


Figure 35. RISA3D Model of Load Distribution on North Bent Cap

The theoretical loads from the structural analysis can be combined with the RISA 3D modeling to calculate a theoretical load on each pile for each truck stop. These loads are illustrated in table 4 for the south bent and table 5 for the north bent.

Table 4 Theoretical Loads per Pile on the South Bent

Truck Position	Pile	Theoretical Load (pounds)
Front Wheels on South Bent	N1	0
	N2	0
	N3	0
	N4	0
	S1	2280
	S2	3326
	S3	741
	S4	-150
Centered Over South Bent	N1	656
	N2	933
	N3	509
	N4	-85
	S1	2214
	S2	3239
	S3	719
	S4	-145
Back Wheels on South Bent	N1	1371
	N2	1866
	N3	1078
	N4	-169
	S1	2152
	S2	3139
	S3	700
	S4	-141

Table 5 Theoretical Loads per Pile on the South Bent

Truck Position	Pile	Theoretical Load (pounds)
Front Wheels on North Bent	N1	1991
	N2	2709
	N3	1477
	N4	-245
	S1	1471
	S2	2147
	S3	478
	S4	-97
Centered Over North Bent	N1	2011
	N2	2736
	N3	1492
	N4	-247
	S1	739
	S2	1078
	S3	240
	S4	-49
Back Wheels on North Bent	N1	2044
	N2	2781
	N3	1517
	N4	-252
	S1	0
	S2	0
	S3	0
	S4	0

3.4.2 REPAIR DESIGN METHODOLOGY

The final products of the project are simple design tables and installation procedures of the splice repair. The design tables will make it easy to choose a replacement shape that will be able to safely take the load from the bridge deck to the old pile underground. Donny Head and Max Hess have listed a few common pipe and H-Pile shapes that the counties typically have access to. The pipe shapes are listed in table 6.

Table 6 Available County Pipe Sizes

Available County Pipe		
Pipe	Outside Diameter (in.)	Wall Thickness (in.)
D=7 5/8 t=0.450	7 5/8	0.450
D=7 5/8 t=0.500	7 5/8	0.500
D=9 t=0.450	9	0.450
D=9 t=0.500	9	0.500

In addition to these pipes, the counties routinely have access to H-Pile shapes HP8X36 and HP10X42. One design table will use only county pipe sizes. Another design table will utilize HP shapes that the county has access to. A third design table will use standard pipe found in the AISC Steel Manual. These pipes consist of pipe 3 X-strong, pipe 4 X-strong, pipe 5 X-strong, pipe 6 X-strong, pipe 8 X-strong, pipe 4 std., pipe 5 std., pipe 6 std., pipe 8 std., and pipe 10 std.

The final design specifies the details in all welds, including types of weld and sizes of weld. It also includes any details needed pertaining to both connections of the splice. In addition to the derivation and explanation of these details, diagrams will be provided to assist in the installation process.

The design of the replacement steel section will be dependent on the location of the pile being replaced. This is a result of higher lateral loads being placed on abutment piles compared to piers located in the middle of the bridge.

To formulate the design tables of midspan piles or piers, the capacity of the timber pile was calculated first. Conservative values were chosen for material properties that will not be easily measured, such as the modulus of elasticity and compressive strength. In this case, being conservative when calculating the capacity of the timber pile results in a higher figure since it is desired to design a steel replacement that will support a higher (conservative) load. Lateral loads were ignored when calculating the capacity of the timber pile in order to achieve a conservative capacity. The following equations were used to calculate the capacity of a timber pile.

$$k = 0.671\sqrt{\frac{E}{F_c}} \qquad (3\text{-}4)$$

Where k is the slenderness factor, E is the modulus of elasticity, and F_c is the strength is compression parallel to the grain in the timber pile. A pile is considered either a short, intermediate, or long column based on the following:

$$0 < l_e/d < 11 \quad \text{Short Column} \qquad (3\text{-}5)$$

$$11 < l_e/d < K \quad \text{Intermediate Column} \qquad (3\text{-}6)$$

$$K < l_e/d < 50 \quad \text{Long Column} \qquad (3\text{-}7)$$

Where l_e is the length of the pile and d is the diameter of the pile.

For short columns, the following is true:

$$F_c' = F_c \qquad (3\text{-}8)$$

Where F'_c is the allowable compressive stress parallel to the grain.

For intermediate columns, the following is true:

$$F_c' = F_c * C_p \tag{3-9}$$

And

$$C_p = 1 - \frac{1}{3} * \frac{\frac{L_e}{d}}{K}^{4} \tag{3-10}$$

For long columns, the following is true,

$$F_c^1 = \frac{0.30E}{\frac{L_e}{d}^{2}} \tag{3-11}$$

The nominal axial capacity of a pile is calculated by using:

$$\emptyset P_n = \emptyset F_c' * A_{pile} \tag{3-12}$$

Where ø = 0.9, and A_{pile} is the calculated cross sectional area of the timber pile.

As stated before, some material properties were estimated as conservative values in order to calculate the axial capacity of the timber. The modulus of elasticity was chosen to be E=1,500,000 psi, the timber was assumed to stay in an elastic state, and the compressive strength parallel to the grain was chosen to be f_c=1,450 psi. The material properties were chosen from tables in the appendix of *Structural Design In Wood* by Judith J. Stalnaker and Ernest C. Harris. Due to the variability in the field in regards to the length of timber that needs to be removed from under the bridge, the L used in the capacity equations relates to the distance from the bottom of the pile cap to the ground plus 5 feet. This enables a conservative design with any length of pile that is cut less than 5 feet below grade.

The capacity of steel pipes and H-Piles were designed with both axial and lateral loads in mind. After communicating with multiple engineers that work with country bridges, it was concluded that the design would be sufficient when including only debris and water load from creeks or rivers flowing under the bridge. Other lateral loads used in new design include wind and traffic loads (braking), however it was decided that given the small surface area exposed to wind and limited traffic that timber county bridges are typically exposed to, the debris load would be satisfactory.

The design capacity of the installed piers can be treated at beam-columns. The lateral load applied to these beam-columns is a "worst-case" load calculation. A distributed load is applied to the face of the pier, which is simply supported at both ends. This distributed force extends one third up the height of the exposed pier. After communicating with county engineers, this was determined to be a conservative assumption for a worst case scenario loading situation. This loading situation is illustrated in figure 36.

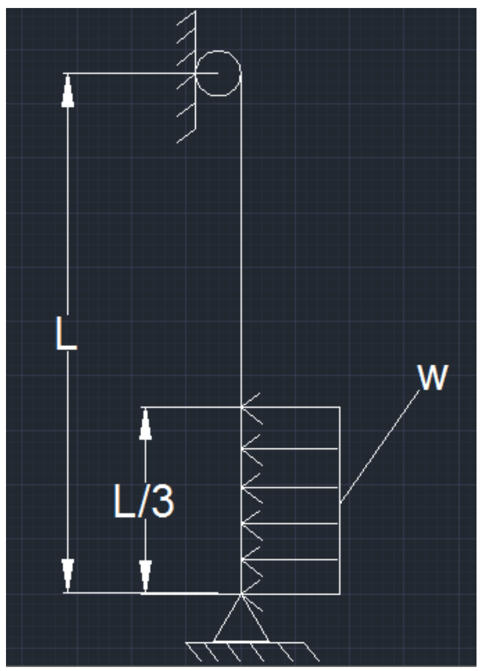


Figure 36. Worst Case Loading Scenario for Piers

The following equations describe how this lateral loading was calculated and applied to the repair:

$$w = dP \tag{3-13}$$

Where d is the diameter or depth of the replacement steel section in feet, and P is the stream pressure in psf.

$$P = KV^2 \tag{3-14}$$

And

$$V = \frac{Q}{A} \tag{3-15}$$

Where K is a coefficient, being 1.4 for square ended piers and 0.7 for circular piers, V is the velocity of water in feet per second, Q is flow rate in cubic feet per second, and A is the flow area in square feet. For this repair:

Q=7,500 ft^3/second

And

$$A = 50ft.* \frac{L}{3} ft.$$

These figures were chosen based on surveying various engineers whom are familiar with county bridge design. The flow rate is a high number based on small creeks in Oklahoma.

From this load, the following maximum moment and shear calculations were derived:

$$M_{max} = M_u = \frac{25wL^2}{648} \tag{3-16}$$

$$V_{top} = \frac{5wL}{18} \tag{3-17}$$

$$V_{bottom} = \frac{wL}{18} \tag{3-18}$$

Due to the fact that the repair will be under a combined load the following interaction equations were used to check if the design was satisfactory:

If $\frac{P_u}{\emptyset P_n} \geq 0.2$,

$$\frac{P_u}{\emptyset P_n} + \frac{8}{9}\frac{M_u}{\emptyset M_n} \leq 1 \tag{3-19}$$

And

If $\frac{P_u}{\emptyset P_n} \geq 0.2$,

$$\frac{P_u}{2\emptyset P_n} + \frac{M_u}{\emptyset M_n} \leq 1 \qquad (3\text{-}20)$$

The axial load on the member, P_u, is the axial capacity of the timber pile being replaced. Using this method of loading, the beam-column can be designed for many general applications, rather than by a case-by-case basis. The following equations were used to calculate the capacity of each steel member considered:

$$\emptyset P_n = \emptyset F_{cr} A_g \qquad (3\text{-}21)$$

Where ø=0.9, A_g is the gross area of the steel and

$$F_{cr} = 0.658^{\frac{F_y}{F_e}} * F_y \qquad (3\text{-}22)$$

If $\quad \frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$

And

$$F_{cr} = 0.877 * F_e \qquad (3\text{-}23)$$

If $\quad \frac{KL}{r} \geq 4.71\sqrt{\frac{E}{F_y}}$

And

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \qquad (3\text{-}24)$$

Where K=1, L is the length of the steel in inches, r is the radius of gyration in inches, F_y is the yield strength of the steel (50 ksi), and E is the modulus of elasticity for steel at 29,000 ksi.

The calculations used to determine the flexural capacity depended on whether the replacement steel was a pipe or an H-pile. For an H-pile, which is undergoing a lateral load parallel to the flanges, the flexural capacity is about the minor (weak) axis. For weak axis bending:

$$\emptyset M_n = \emptyset M_p = F_y Z_y \leq 1.6 F_y S_y \quad \text{(yielding failure)} \tag{3-25}$$

if $\lambda < \lambda_{pf}$ (beam is compact),

or

$$\emptyset M_n = \emptyset \quad M_p - \quad M_p 0.7 F_y S_y \quad \frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \tag{3-26}$$

if $\lambda < \lambda_{pf} < \lambda_{rf}$ (beam is non-compact)

Where ø=0.9, Z_y is the plastic section modulus in inches cubed, and S_y is the elastic section modulus in inches cubed. The flexural capacity cannot exceed $øM_p$. There is another possibility, which includes beams with slender flanges; however no steel sections with slender flanges were used in the repair design. Also,

$$\lambda + \frac{b_f}{2t_f} \tag{3-27}$$

$$\lambda = 0.38 \quad \overline{\frac{E}{F_y}} \tag{3-28}$$

$$\lambda_{rf} = \quad \overline{\frac{E}{F_y}} \tag{3-29}$$

Where b_f is the flange width in inches and t_f is the flange thickness in inches.

For a pipe the flexural capacity was calculated by using the following:

For all pipes $D \quad t \leq \frac{0.45E}{F_y}$

$$\emptyset M_n = \emptyset M_p = F_y Z \quad \text{(yielding failure)} \tag{3-30}$$

if L < L_p (compact sections)

or

$$\emptyset M_n = \emptyset \quad \frac{0.021E}{D \quad t} \quad S \quad \text{(Local Buckling failure)} \tag{3-31}$$

if L > L_p (Non-compact sections)

Where ø=0.9, D is the outside diameter of the pipe in inches, and t is the thickness of the pipe wall in inches. Again, the flexural capacity cannot exceed $øM_p$. There is also a separate calculation used for pipes with slender walls, however no pipes with slender walls were considered in the design. Also,

L = length of pipe to be installed

$$L_p = 1.76r\sqrt{\frac{E}{F_y}} \tag{3-32}$$

Both ends of the steel member are non-moment transferring connections. They need to transfer shear loading and axial loading. Given the piles will be under a compressive axial load, only the plate thickness needs to be checked in order to transfer the end bearing load. The lateral (shear) loads will need to be able to be transferred through each connection to the pile cap and the existing pile underground. Again, the "worst-case" lateral load which was used in designing the beam-column will be used to design the capacity of the connections. The following equation was used to determine the strength of the welds used on each connection:

$$\emptyset R_n = 0.02\ F_{exx} DI \tag{3-33}$$

Where F_{exx}=the electrode used in ksi, D=the leg height of the weld, and I= the length of the weld. Any bolts used to connect the steel to the pile cap will be checked for bolt shear from the following equation:

$$\emptyset R_n = \emptyset R_v * N \tag{3-34}$$

Where $øR_v$ is the shear capacity of the bolt used and N is the number of bolts used. The strength of the connection at each end of the member will be sufficient to transfer all loads to the foundation.

Compared to piers, abutment piles undergo more lateral loads from soil and roadway forces. The amount of lateral load placed on these abutment piles depends on the type of soil used behind the abutment, the height of the abutment, the type and size of roadway traveling over the abutment, and the spacing between piles.

Given that there are many more variables that affect the lateral loading on abutment piles compared to midspan piers, a different approach was used to create the replacement design. A prescriptive method is typically used to design new abutment piles on county bridges. This idea will be utilized in the design of the repair. All replacement piles for abutment components will be specified as one shape with limitations on other variables in the structure. For example, the span length has to be less than a certain value, the distance between piles needs to be less than a certain value, and the height cannot exceed a certain value.

County bridges repeatedly use HP10X42 for abutment piles on new bridges. This shape has also been said to be readily available for the use of this repair across the state. The design for abutment piles will use a HP10X42 with limitations of span length, spacing between piles, and span height. These variables coincide with the prescriptive method used in the design of new bridges and the use of the HP10X42 pile. The following diagram in figure 37, provided by Jimmy Watson of ACCOK, supports the use of this pile and its limitations. This figure is intended to provide an overview of the repair in this report and show that it has been employed rather than to be used for detailed design information.

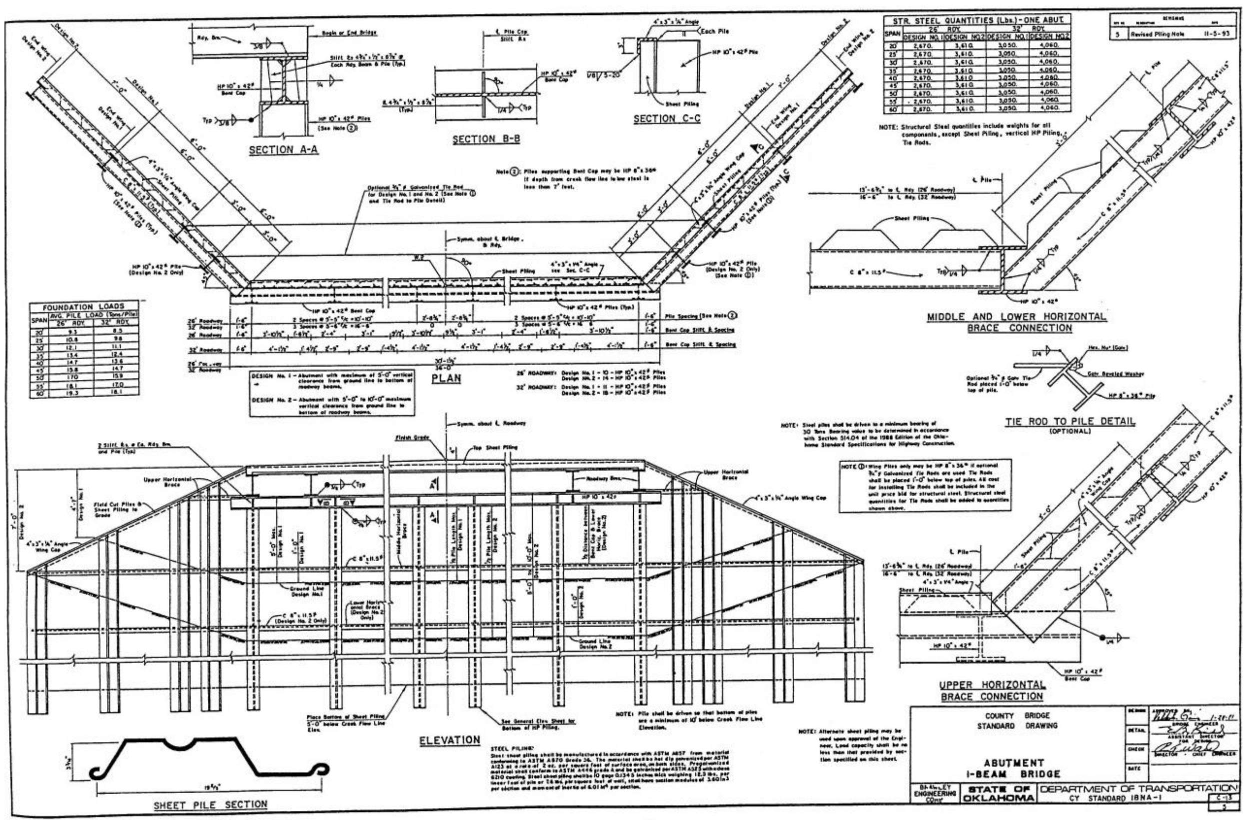


Figure 37. Example Abutment Design of New County Bridge

Depending on the top connection, the bottom connection of the abutment piles may or may not need to be a moment connection. Ideally, the steel replacement member will be able to be connected to the pile cap with some sort of shear connection. If however this is not the case, the pile will act as a cantilever, requiring a moment transfer at the connection to the existing timber pile underground. The only plausible design for this moment connection is to extend the length of the sleeve and insert a bolt through both the side of the sleeve and the existing timber pile. With the extension of the pipe underground, the sleeve would be able to transfer moment to the timber pile. The following diagram depicts the reasoning behind the need for a moment transferring connection.

The amount of moment transferred will depend on the length of the pipe of the sleeve that surrounds the timber pile. This moment must not exceed the flexural strength of the timber pile. Given that there will not be a guaranteed snug fit between the timber pile and the sleeve, there is no guarantee that the full potential of this moment connection design will be achieved. Further work is needed to be done on this connection, but is beyond the scope of this project. Ideas involving this connection may include epoxy injection, concrete encasement, or additional steel placement in the connection.

If the abutment pile is connected to the pile cap with a shear connection, the sleeve connection will mimic that of the pier sleeve connections. The capacity of these connections will exceed those designated for the connections used on new structures.

Other general concerns explored in the design of the repair must be considered. Due to load being distributed by stiffness, it was discovered that the amount of load carried to the existing pile under a repaired pier was higher than the original m theoretical design load. One problem this may cause is an increased settlement of the timber piles beneath the steel splice. While at first thought this may seem like a significant problem, it is believed that with the settlement of the existing timber pile will come a loss of stiffness. With this loss of stiffness, the load applied to the pile cap will redistribute to other piles and safely find its way to the foundation. Repaired piles should be inspected for amount of settlement, and a drastic amount may require further investigation.

Another issue involving the steel splice is the corrosion of the member. While these steel members may be exposed to moisture and other environments that cause corrosion, the members used will not be extremely thin in wall thickness, therefore corrosion should not result in a significant amount of section loss. In addition, this repair is only to be used as a temporary fix, providing a few years of service life to a structure before it can be replaced.

3.4.3 INSTALLATION PROCEDURE METHODOLOGY

The installation procedure was modeled after those used in the Grant County repair. Extensive communication with Max Hess, the county commissioner of Grant County, provided installation details of the repair that have been used across the state. This installation procedure has been tried and tested. It is cost efficient and should not impede traffic. It is simple and utilizes machinery commonly found within county maintenance resources. It can also be completed with as few as two workers in less than eight hours. The procedure, which is found in the results section, calls for the pile cap to be "jacked up". The reasoning behind this lies in the theory that a member which is under stress and strain must be relieved of that stress and strain before being replaced or repaired. If the bridge was not jacked up before the new steel member was placed, it would be nearly impossible to provide a load path for the stresses to transfer to the foundation. This is common sense and a theory discussed in Dr. Kyle Riding's Concrete Pavement and Bridge Repair course from Kansas State University.

3.5 RESULTS

3.5.1 FIELD TESTING FINDINGS

The findings from the field testing performed in Grant Country suggest the successful flow of load from the pile cap of the bridge, through the spliced repair, to the existing timber pile in the foundation. The following figure illustrates that as the truck drives across the bridge, the replaced pile takes up to 4.658 Kip (all values are averaged from three data sets).

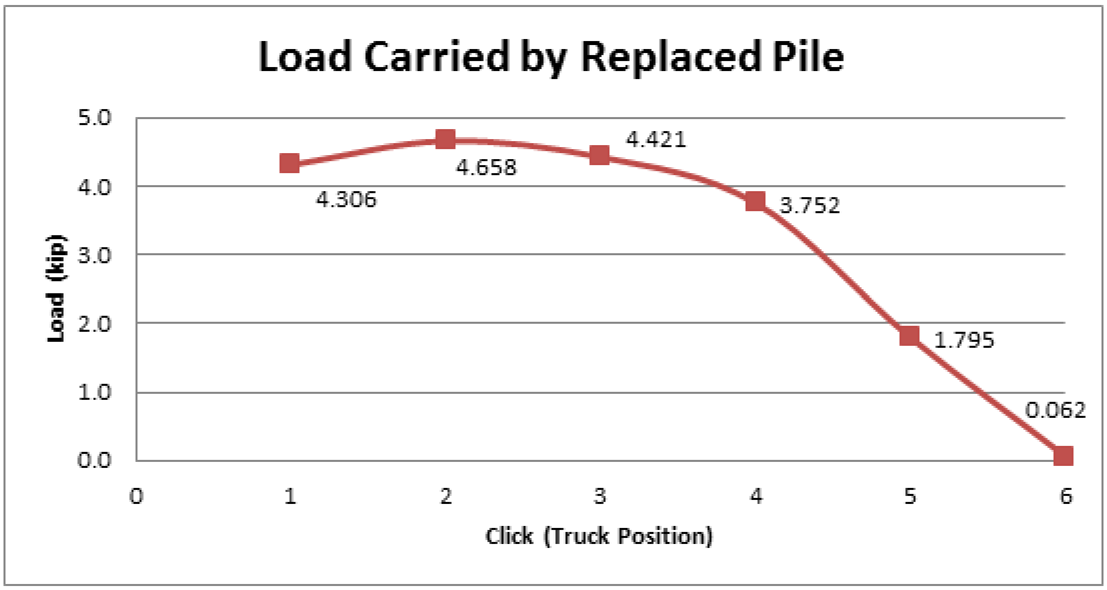


Figure 38. Load Carried by Replaced Pile

In addition, the repaired pile took a large percentage of the total load of the truck, as shown in the following figure:

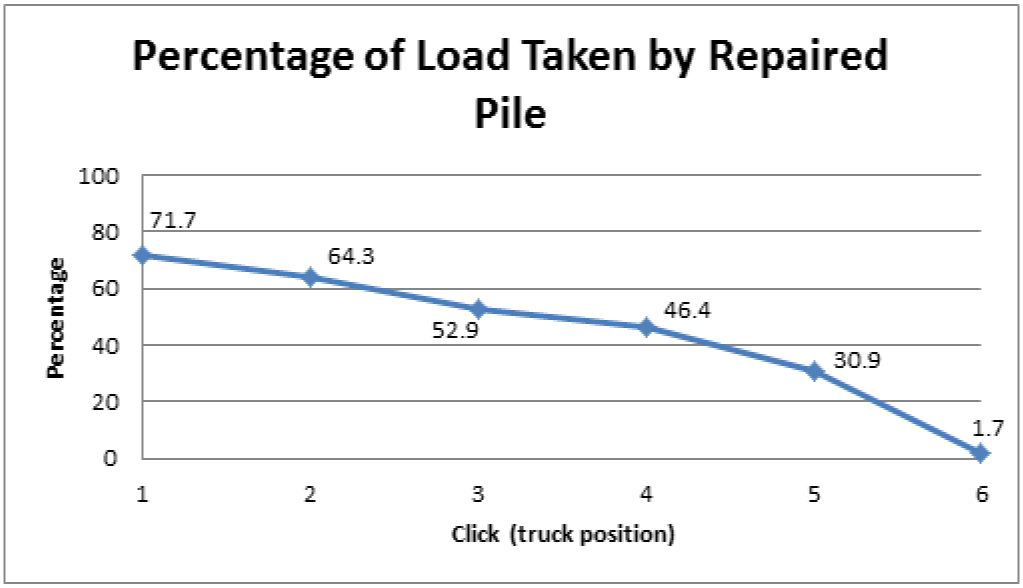


Figure 39. Percentage of Load Taken by Repaired Pile

While the location of the repaired pile does affect the amount of load it will take, this is another example of how stiff the repaired pile actually is.

Figures on the following pages illustrate the actual load registered in each pile, measured from the field test, compared to the theoretical load the pile should see based on structural analysis. It should be noted that in indeterminate structures like a pile system, load is distributed by stiffness. Therefore, if an element of a structure is undergoing a high load, then that element is stiff and provides strength to the system. Recall the location of each pile, illustrated in figure 40, and that the truck was traveling from South to North during the testing runs. Figures 41 through 46 show the comparisons between theoretically modeled loads and experimentally determined loads carried by piles S1, S2, S3, N1, N2, and N3. Pile S2 is the pile that was repaired with a steel replacement. It is also the only pile that actually carried more load than what was theoretically predicted via structural modeling.

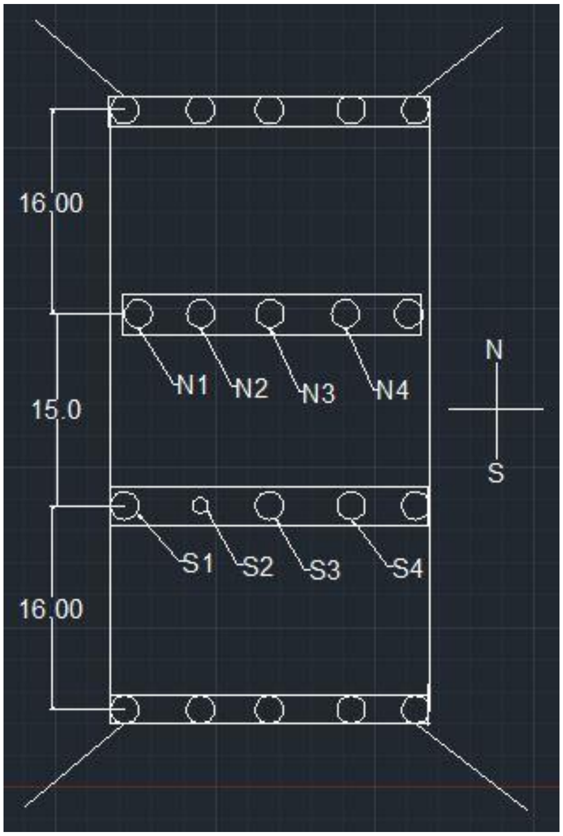


Figure 40. Plan View of Grant County Bridge

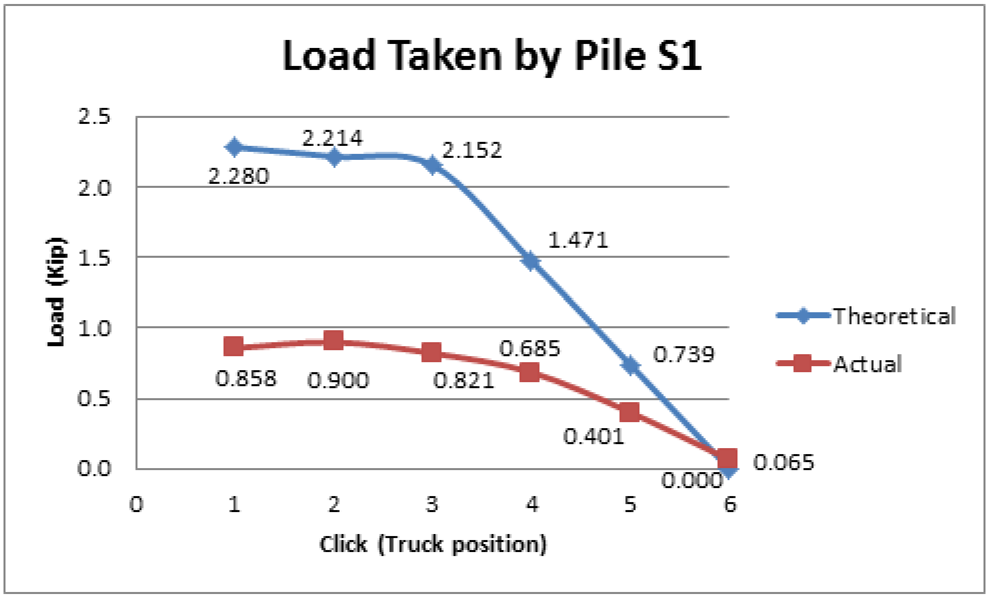


Figure 41. Load Taken by Pile S1

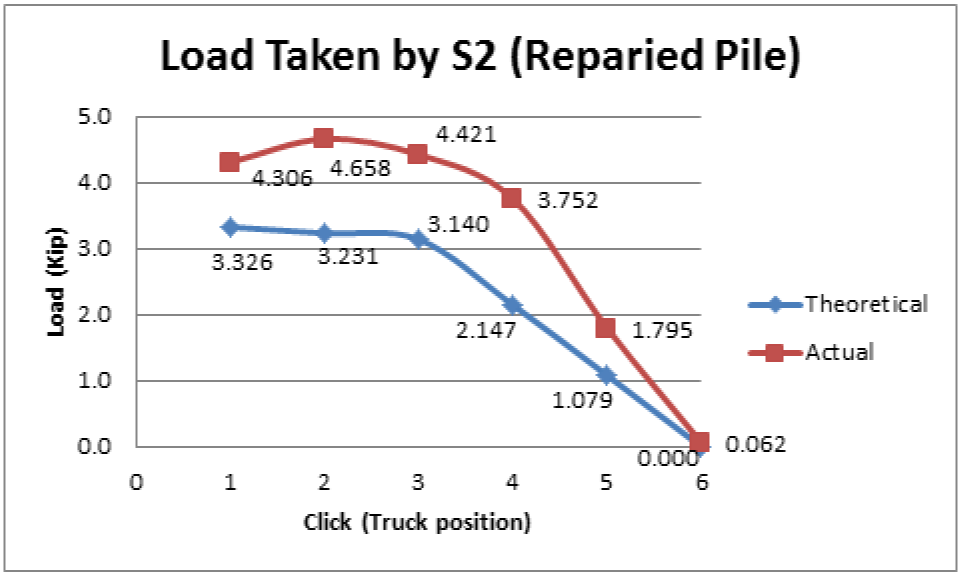


Figure 42. Load Taken by S2 (Repaired Pile)

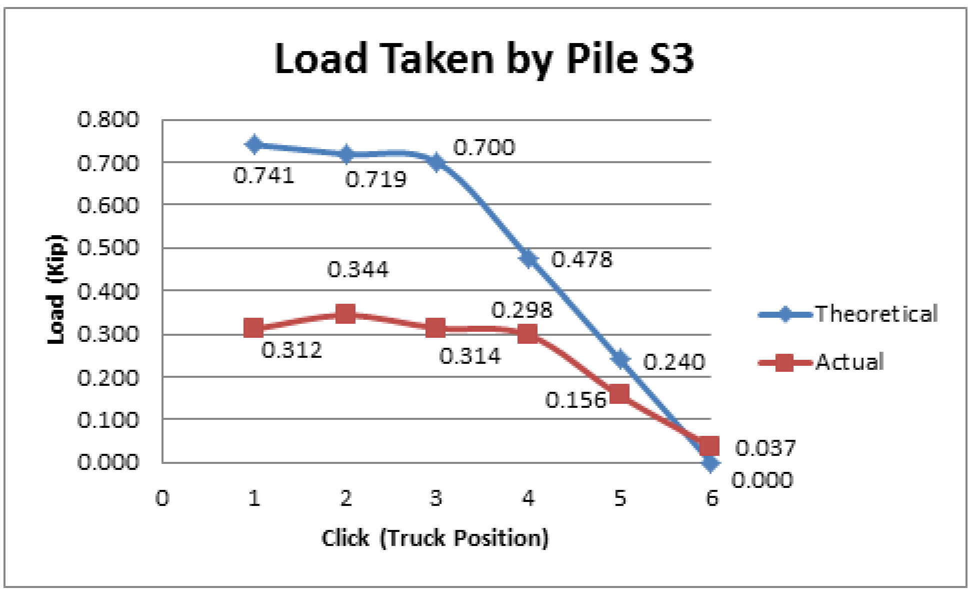


Figure 43. Load Taken by Pile S3

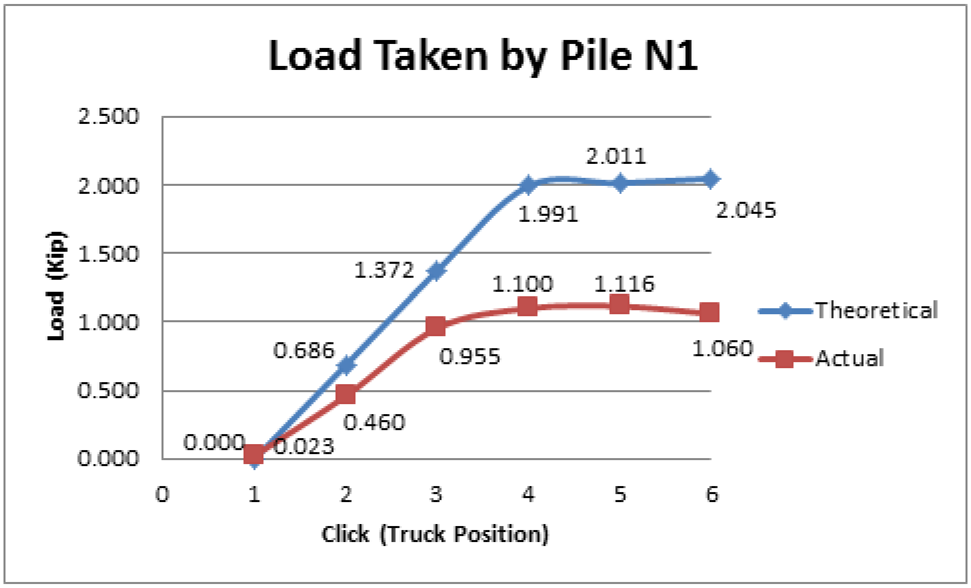


Figure 44. Load Taken by Pile N1

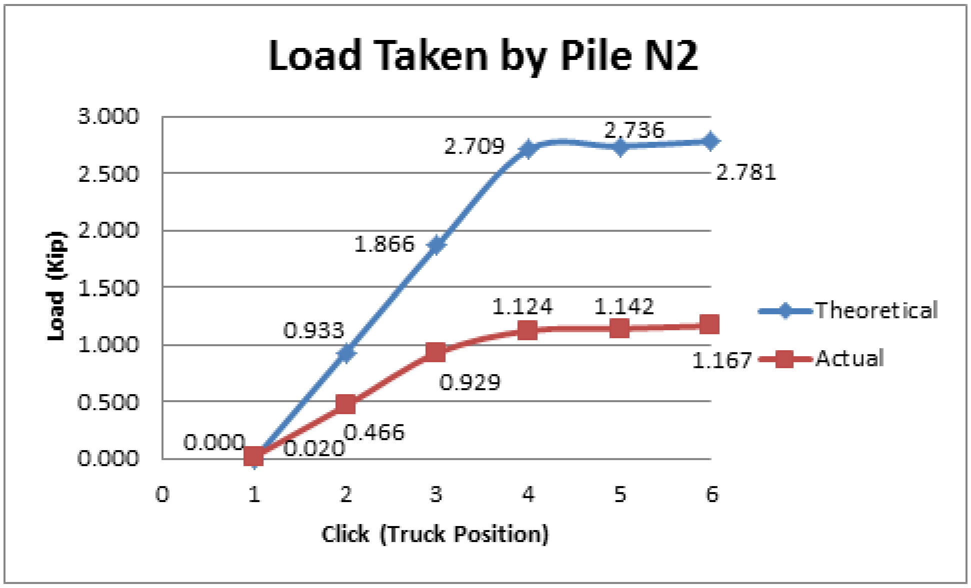


Figure 45. Load Taken by Pile N2

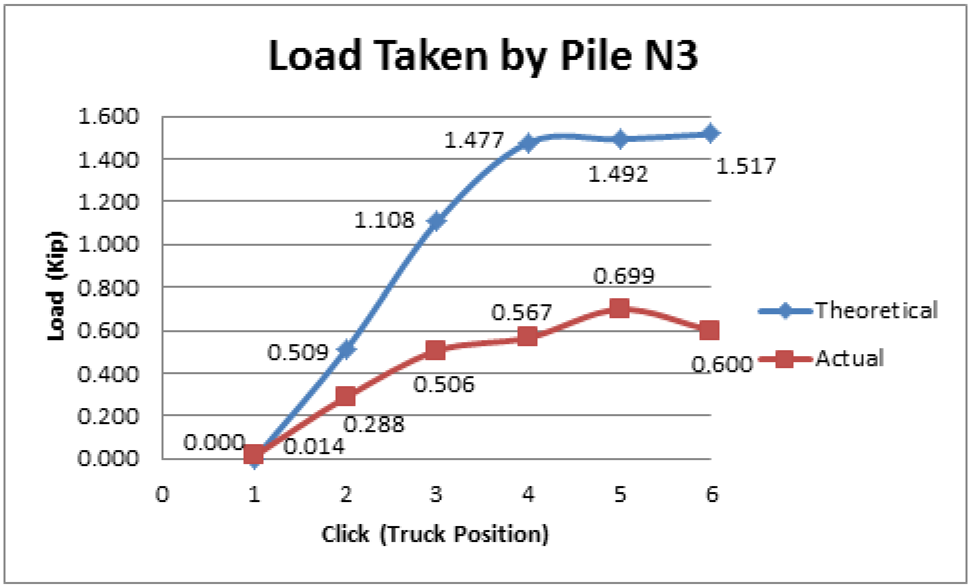


Figure 46. Load Taken by Pile N3

Piles S4 and N4 were negated from these illustrations because they underwent a very small theoretical and actual load. Again, the results of the field testing prove that the installed repair is successful in transferring load from the pile cap to the foundation.

3.5.2 REPAIR DESIGN FINDINGS

Design Tables

The following tables are the design tables derived using the methods discussed in the methodology section. These tables are to be used when selecting a replacement steel section for the splice repair. For example, if a 10 inch diameter, 15 foot long section of timber piling is removed, it needs to be replaced with either an HP10X57 (from table 7), an 8 pipe standard (from table 8), or a 7 5/8 inch diameter, 0.450 inch thick pipe (from table 9). These tables are to be used on piers only, not abutment piles. Design recommendations include using pipe instead of H-Pile on any exterior piers. This recommendation extends from the fact that a pipe will shed debris build up better than an H-Pile, resulting in less lateral load on both the repaired pile and the structure as a whole.

Table 7 Replacement Steel HP Selection Design Table

Replacement Steel HP Selection								
Height(ft)	Original Timber Pile Diameter (in.)							
	6	7	8	9	10	11	12	13
5	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
6	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
7	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
8	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
9	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
10	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
11	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
12	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
13	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
14	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
15	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
16	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
17	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
18	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57	HP10X57
19	HP10X42	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57
20	HP10X42	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57
21	HP10X42	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57
22	HP10X42	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57
23	HP10X42	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57
24	HP10X42	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57
25	HP10X42	HP10X42	HP10X42	HP10X42	HP10X42	HP10X57	HP10X57	HP10X57

Table 8 Replacement Steel Pipe Selection Design Table

Replacement Steel Pipe Selection								
Height (ft)	Original Timber Pile Diameter (in.)							
	6	7	8	9	10	11	12	13
5	Pipe 3 x-strong	Pipe 3 x-strong	Pipe 4 std.	Pipe 5 std.	Pipe 5 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.
6	Pipe 3 x-strong	Pipe 4 std.	Pipe 4 std.	Pipe 5 std.	Pipe 5 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.
7	Pipe 3 x-strong	Pipe 4 std.	Pipe 5 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.
8	Pipe 3 x-strong	Pipe 4 std.	Pipe 5 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.
9	Pipe 4 std.	Pipe 4 std.	Pipe 5 std.	Pipe 5 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 8 std.
10	Pipe 4 std.	Pipe 5 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 8 std.
11	Pipe 4 std.	Pipe 5 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.
12	Pipe 4 std.	Pipe 5 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.
13	Pipe 4 std.	Pipe 5 std.	Pipe 5 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.
14	Pipe 4 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.
15	Pipe 4 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.
16	Pipe 4 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.
17	Pipe 4 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.	Pipe 10 std.
18	Pipe 4 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.	Pipe 10 std.
19	Pipe 4 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.	Pipe 10 std.
20	Pipe 4 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.	Pipe 10 std.
21	Pipe 4 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.	Pipe 10 std.
22	Pipe 4 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.	Pipe 10 std.
23	Pipe 4 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.	Pipe 10 std.
24	Pipe 4 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.	Pipe 10 std.
25	Pipe 4 std.	Pipe 5 std.	Pipe 6 std.	Pipe 6 std.	Pipe 8 std.	Pipe 8 std.	Pipe 10 std.	Pipe 10 std.

Table 9 Replacement Steel Pipe (County Pipe Shapes) Selection Design Table

Replacement Steel Pipe Selection								
Considering Only County Standard Shapes								
Height (ft)	Original Timber Pile Diameter (in.)							
	6	7	8	9	10	11	12	13
5	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450
6	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450
7	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450
8	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450
9	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450
10	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450
11	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450
12	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450
13	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450
14	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450
15	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450	D=9 t=0.450
16	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450	D=9 t=0.450
17	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450	D=9 t=0.450
18	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450	D=9 t=0.500
19	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450	D=9 t=0.500
20	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450	D=9 t=0.500
21	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.500	D=9 t=0.450	D=9 t=0.500
22	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.500	D=9 t=0.450	D=9 t=0.500
23	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450	D=9 t=0.450	D=9 t=0.500
24	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=9 t=0.450	D=9 t=0.450	D=9 t=0.500
25	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.450	D=7 5/8 t=0.500	D=9 t=0.450	D=9 t=0.450	D=9 t=0.500

For abutment piles, the following table displays the replacement steel selection and its limitations in the field.

Table 10 Replacement Steel Selection and Limitations for Abutment Piles

Replacement Steel Selection and Limitations			
Shape	Maximum Span Length (ft.)	Maximum Height (ft.)	Maximum Pile Spacing (ft.)
HP10X42	60	25	5.5

As shown in table 10, only an HP10X42 can be used to splice into abutment piles. The span length starting from that replacement pile may not exceed 60 feet, the height of the pile may not exceed 25 feet, and the spacing between the adjacent piles may not exceed 5.5 feet.

Connection Design

The following diagrams depict the design of the sleeve connection.

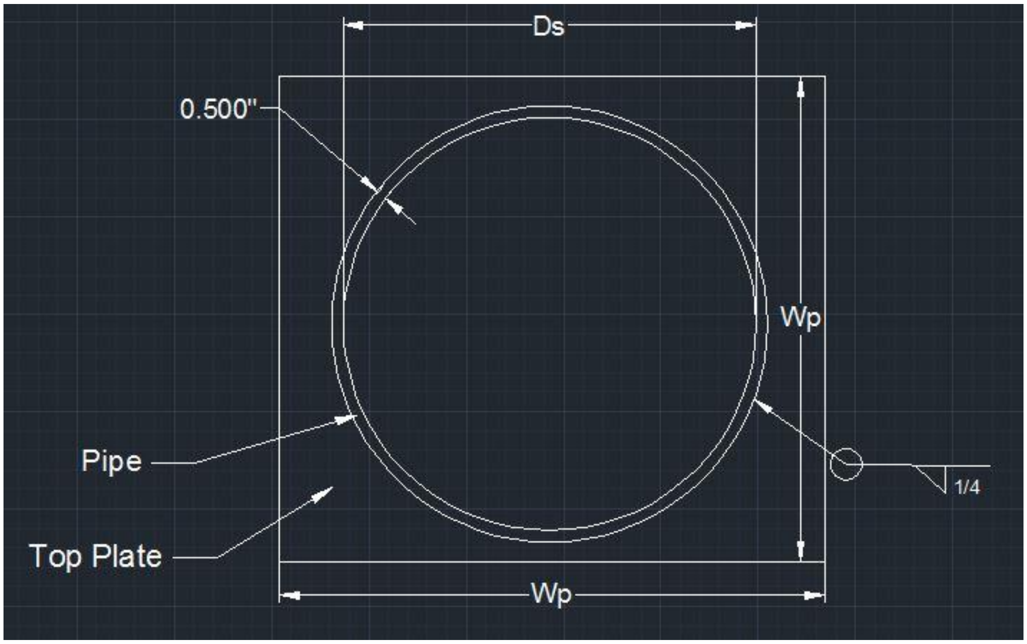


Figure 47. Plan View of Sleeve Design

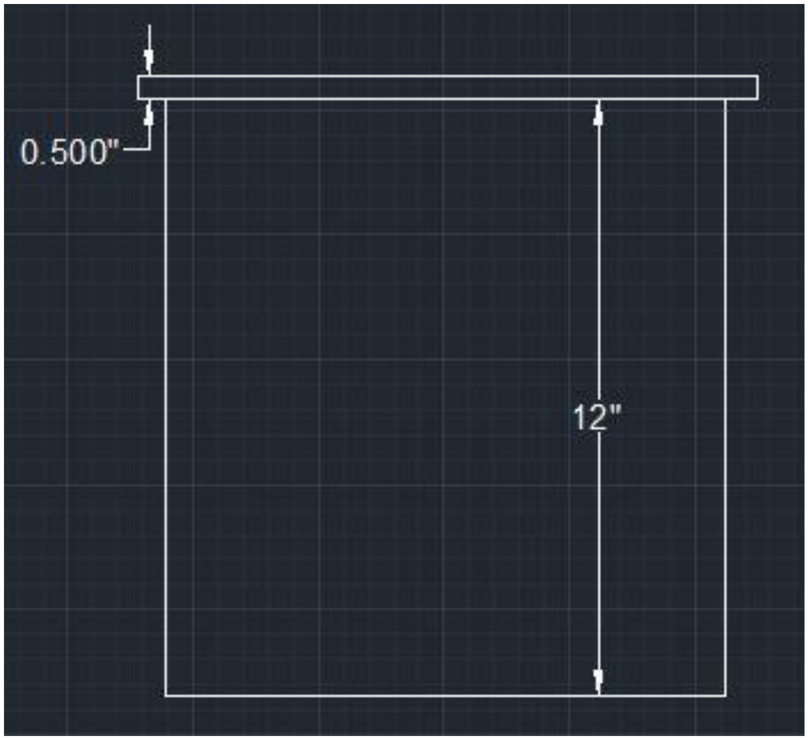


Figure 48. Elevation View of Sleeve Design

W_p and D_s will depend on the diameter of the timber piling the sleeve is going over. For this design, D_s should be one inch larger than the diameter of the timber pile. W_p should be 3 inches larger than D_s. All welds are to be E80 electrode.

The steel splice is welded to the top of the sleeve with E80 electrode and a ¼ inch leg height. For pipes, the entire circumference of the pipe is to be welded to the plate. For H-Piles, only the lengths of the flanges are necessary to weld to the top of the sleeve.

The following tables represent proof that the bottom connection has enough capacity in the worst case scenario for multiple repairs.

Table 11 Bottom, Pipe to Plate Connection

Bottom, Pipe to Plate Connection				
Pipe	Diameter (in)	Connection Capacity (K)	Maximum Shear Load (K)	OK?
Pipe 3 x-Strong	3.5	52.75	2.30	yes
Pipe 4 Std.	4.5	67.82	2.95	yes
Pipe 5 Std.	5.56	83.80	3.65	yes
Pipe 6 Std.	6.63	99.93	4.35	yes
Pipe 8 Std.	8.63	130.07	5.66	yes
Pipe 10 Std.	10.8	162.78	7.09	yes
Pipe 12 Std.	12.8	192.92	8.40	yes

Table 12 Bottom, Pipe to Plate Connection

Bottom, Pipe to Plate Connection				
Pipe	Diameter (in)	Connection Capacity (K)	Maximum Shear Load (K)	OK?
D=7 5/8 t=0.450	7.625	114.92	5.00	yes
D=7 5/8 t=0.500	7.625	114.92	5.00	yes
D=9 t=0.450	9	135.65	5.91	yes
D=9 t=0.500	9	135.65	5.91	yes

Table 13 Bottom, H-Pie to Plate Connection

Bottom, H-Pile to Plate Connection				
HP	Flange Width (in)	Connection Capacity (K)	Maximum Shear Load (K)	OK?
HP10X42	10.1	96.96	12.73	yes
HP10X57	10.2	97.92	13.10	yes

Table 14 Worst Case Scenario for Sleeve Weld Connection

Worst Case Scenario for Sleeve Weld Connection			
	Case	Shear Load (K)	OK?
Maximum shear load	HP10X42, 5 ft. long	12.73	YES
Minimum Shear Capacity	6 in. timber pile	90.57	

The top connection design depends on the material the pile cap is constructed from. If the pile cap is steel, then the steel pipe or H-Pile can simply be welded (E80 electrode and ¼ inch leg height) to the pile cap. For pile caps made of either timber or concrete, the steel splice is welded onto a ½ inch plate, which is then bolted into the pile cap. The following tables represent the strengths of these welds in their respective worst case scenarios.

Table 15 Top Pipe to Plate Connection

Top Pipe to Plate Connection				
Pipe	Diameter (in)	Connection Capacity (K)	Maximum Shear Load (K)	OK?
Pipe 3 x-Strong	3.5	52.75	0.46	yes
Pipe 4 Std.	4.5	67.82	0.59	yes
Pipe 5 Std.	5.56	83.80	0.73	yes
Pipe 6 Std.	6.63	99.93	0.87	yes
Pipe 8 Std.	8.63	130.07	1.13	yes
Pipe 10 Std.	10.8	162.78	1.42	yes
Pipe 12 Std.	12.8	192.92	1.68	yes

Table 16 Top Pipe to Plate Connection

Top Pipe to Plate Connection				
Pipe	Diameter (in)	Connection Capacity (K)	Maximum Shear Load (K)	OK?
D=7 5/8 t=0.450	7.625	114.92	1.00	yes
D=7 5/8 t=0.500	7.625	114.92	1.00	yes
D=9 t=0.450	9	135.65	1.18	yes
D=9 t=0.500	9	135.65	1.18	yes

Table 17 Top H-Pile to Plate Connection

Top H-Pile to Plate Connection				
HP	Flange Width (in)	Connection Capacity (K)	Maximum Shear Load (K)	OK?
HP10X42	10.1	96.96	2.55	yes
HP10X57	10.2	97.92	2.62	yes

The connection between the top plate to the pile cap is designed with 4, ¾ inch diameter, 5 inch long screws. According to McMaster-Carr, the screws shown in table 18 each supply a shear strength of 11.4 kips. These screws are designed for timber or concrete use.

Table 18 Screw Properties

			Ultimate Strength lbs.				
Lg.	**Min. Install Dp.**	**Thread Lg.**	**Pull Out**	**Shear**	**PKg. Qty.**	**Pkg.**	
3/4'' Dia. - Head Width : 1 1/8" (Drill Size 3/4")							
4"	2 ¾"	4"	6,500	11,400	1	99795A141	5.92
5"	2 ¾"	5"	6,500	11,400	1	99795A144	6.23
6"	2 ¾"	5"	6,500	11,400	1	99795A146	7.49
7"	3 ¾"	6"	6,500	11,400	1	99795A148	6.65

Table 19 displays the capacities and loading effects of the worst case scenario between a top plate and a pile cap using four of these screws.

Table 19 Worst Case Scenario for Bolted Connection to Pile Cap

	Case	Shear (K)	OK?
Maximum Shear Load	HP10X42, 5 ft. long	12.73 (load)	YES
Standard Bolt Shear Capacity	4, 3/4" diameter 5" long screws	34.20 (capacity)	

The top plate design is illustrated in figure 49.

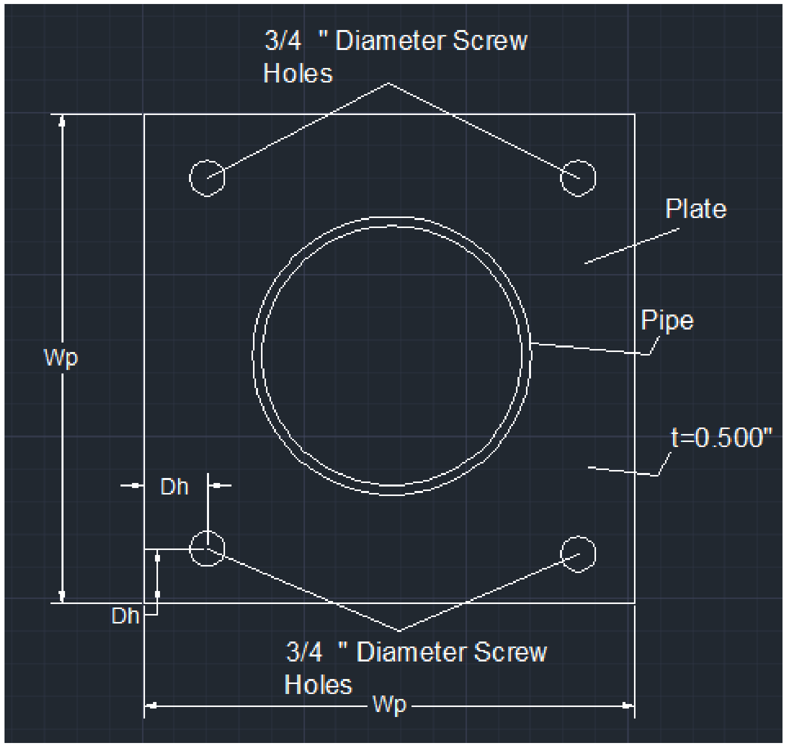


Figure 49. Top Plate Design

W_p is again 3 inches longer than the diameter of the pipe or the flange width of the H-Pile. D_h, the distance from the center of the bolt hole to the edge of the plate should be 2 inches.

For abutment pile connections, it has been stated that more work needs to be done on the design of the bottom sleeve connection. For the top connection, only connections to steel pile caps are acceptable situations which will provide enough shear load transfer between the abutment pile and the pile cap. This connection is specified as a 3/8 inch leg height, E80 electrode weld around the perimeter of the end of the HP10X42 or HP10X57 to the pile cap. This design is used in new county bridges, and has been proven through use in the field that it is an acceptable connection.

The design specified in new county bridges provides a shear strength of 273.6 Kips. This capacity cannot be guaranteed in the weld between the pipe and plate on the bottom sleeve unless the timber pile being replaced has a diameter of 13 inches.

Again, more work needs to be done on the development of a bottom connection for abutment piles.

Installation Procedure

The installation procedure for the splice repair is listed below.

1. Uninstall any connection between timber pile and pile cap if necessary
2. Dig out soil around the decayed timber pile
3. The sound test can be used to estimate a sufficient depth.
4. Jack-Up Pile Cap
5. The pile cap over the bad timber pile needs to be raised using a jack. It is best to raise the pile cap at a point as close to the decayed timber pile as possible.
6. A steel plate should be placed on the ground directly under the pile cap at the location chosen.
7. A 20-50 ton bottle jack is then placed on top of this plate.
8. Between the jack and the pile cap, a steel member is placed. This stele member should be a HP10X57, cut to a length that allows the jack to be raised and the pile cap to be lifted. In between the H-Pile and the jack, a steel plate can be inserted to provide extra stability. This method has been used in the field and provides enough stability in the bridge to install the splice.
9. Lift the pile cap off of the timber pile by activating the jack. The pile cap only needs to rise slightly (< ½ inch) off of the pile.
10. Remove decayed timber pile.
11. Cut timber using a chainsaw. This cut needs to be as horizontal as possible.
12. Check quality of remaining timber pile by using the sound check. A deeper cut may be necessary.
13. Remove pile from under the bridge. This may require some sort of front end loader or other similar machinery.
14. Fabricate and Install Sleeve
15. Weld pipe length to ½ inch plate.
16. Install sleeve by sliding it over the top of the timber pile.
17. Install Steel Splice

18. Install top connection by welding on the plate if necessary. This plate will be bolted to a timber or concrete pile cap.
19. Measure and cut the steel splice. The length of the splice needs to be as close as possible between the top of the sleeve and the pile cap connection.
20. The splice is then installed by welding the bottom to the sleeve plate and connection the top connection to the pile cap (either bolting plate to timber/concrete pile cap or welding splice to a steel pile cap).
21. Backfill Soil
22. Replacing the soil around the bottom connection will prevent further timber decay and steel corrosion.
23. Release Jack
24. Lower jack and disassemble pieces.

3.5.3 CONCLUSION

It has been proven that the steel splice repair technique is an effective method to extend the service life of aging timber bridges. While the repair itself is not ground breaking in terms of new technology, the development of the design tables and standardization of the installation procedures should provide a safe design for local engineers to use on off-system bridges. This information can be distributed to county and tribal governments as a useful tool when repairing bridges.

Benefits Of Repair

The steel splice repair is cost effective. Most of the materials used can be found in county stock piles, limiting the amount of funding needed to install the repair. The use of the splice repair technique will prolong the service life of a bridge for a few years, freeing up funds and resources that local governments can use elsewhere. Although the repair has already been used, this project will provide a safe, simple, inexpensive design for future repairs.

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4 COUNTY BRIDGE REPAIR OF STEEL H-PILES WITH CORROSION DAMAGE THROUGH CROSS SECTION REINFORCEMENT

4.1 INTRODUCTION

Currently, Oklahoma has about 16,000 bridges on the off-system. Nearly 4630 (32%) are structurally deficient. In 2003, to replace the structurally deficient bridges, the estimated cost to replace the bridges by the Oklahoma Department of Transportation was $3.4 billion (proposal). It is obviously not feasible for the counties to replace all of their structurally deficient bridges. The County, city, and Tribal governments look after most of these bridges and they have a very limited amount of resources, manpower and expertise available to maintain these bridges. However, they all do their best to keep the bridges in working conditions. With bridges being expensive and laboring to remove and replace, the counties, city, and tribal governments rely heavily on the ability to repair a bridge in order to increase its life expectancy by another 10 or more years. Currently only about 15 of the off-system bridges are replace each year (Peters). This leaves thousands of bridges that need to be repaired to prolong their life until their inevitable replacement will be implemented.

This research wanted to concentrate on developing simple, inexpensive repair that would increase the superstructure rating of a bridge in order to prolong its life. This thesis concentrates on corroded steel H-piles that have corrosion damage and are repaired by welding supplemental steel plates on the corroded section to increase the strength of its deteriorated cross section. This repair has already been implemented on some county bridges, so field tests were conducted to ensure the effectiveness of the repairs.

Once the effectiveness of the repair was observed, lab tests were conducted at Oklahoma State University to create a repair design and determine ways to decrease the cost and labor time of the bridge repair. The final results of the project consist of a simple equation used for designing the supplement repair plate, determining the plate weld length design tables, and organizing installation instructions for the bridge repair.

4.1.1 REVIEW OF LITERATURE

Meetings with ODOT and the Association of County Commissioners were conduct to determine various common structural bridge repairs across Oklahoma. The ODOT meeting with Walt Peters, P.E., provided information about the bridge ratings and it was determined that the best way to increase a bridge rating is to increase the superstructure instead of the deck of a sub structure. Thus indicating that the piles, pile caps, beams, and abutments are considered to be a higher priority because the repair of these items can raise a bridge rating the most significantly. Bridges with a 50-80 out of 100 rating are candidates to be repaired, while bridges with ratings below 50 are candidates for full replacement. Bridges with a 40 rating or lower are considered structurally deficient and are at the highest need for repair or replacement. There are about 4600 of these structurally deficient bridges in the state of Oklahoma. Mr. Peters also recommended we meet with Randy Robinson of the Association of County Commissioners for further information regarding these structurally deficient bridges. A meeting with Randy Robinson, P.E., Donny Head, and Jimmy Watson was conducted and a few common county bridge repairs were discussed. Mr. Robinson stated that cause for most of the structurally deficient bridges is due to the timber and steel piling deterioration, whereas the bridge beams themselves are normally in fine condition. One specific repair identified was a steel reinforcement plate welded on to corroded steel piles for many county bridges. This technique involves welding supplemental steel plates onto steel H-piles which have significant section loss to increase the cross section of the pile. However, this repair had not been analyzed in an engineering perspective to determine the actual effectiveness of the repair. It was simply done with the best judgment and available materials of the specific counties. Figure 50 and Figure 51 show sample drawings of the corroded bridge piles both before and after the supplemental repair plates have been added. Donny Head provided information on two bridges which had steel corrosion of their H-piles. One bridge had not yet been repaired, while the other was already repaired with the supplement plate welds. It was stressed by Randy Robinson that this repair technique was not to prevent the bridge from needing to be replaced but to prolong the life of the bridge until replacement funding becomes available.

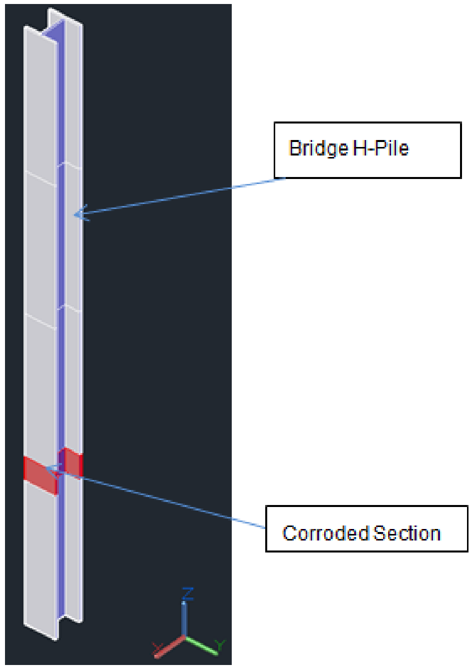

Figure 50. Corroded bridge H-pile with repair plate

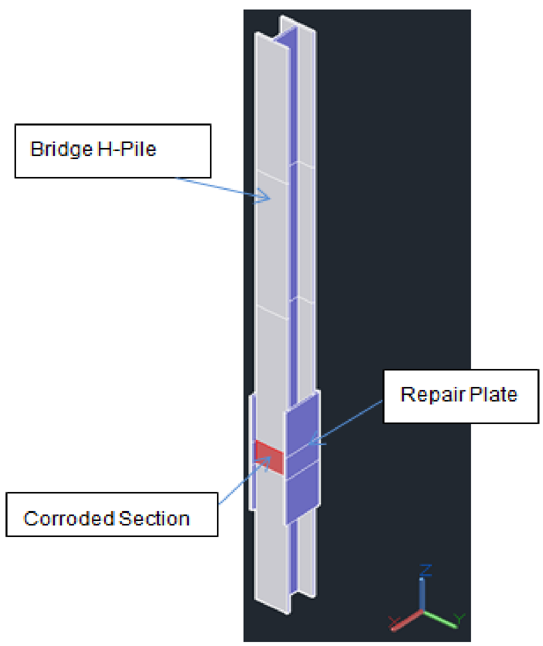

Figure 51. Corroded bridge H-pile without repair plate

4.2 METHODOLOGY

4.2.1 **Equipment Used**

A Bridge Diagnostic Inc. Structural Testing System II (STS II) was used to find the strains of the steel members during testing. Strain sensors, referred to as Intelliducers and shown in Figure 54, are easily attached to the steel members. Two C-clamps were used to attach the sensor to the flanges of the H-piles. To attach the sensors to webs of the members, two tabs and a fast acting adhesive were required. Each sensor had to be connected to the data acquisition boxes before connection to the computer, as shown in Figure 51. The STS II program then takes the data and converts it into understandable strain information. The STS II program outputs the sensor data in two different ways. One is a graphical representation which allows the user to view the data immediately; the other is a full list of all data points. Figure 52 below is a representative graph of one of the sensors showing the strain vs. time for a particular data run.

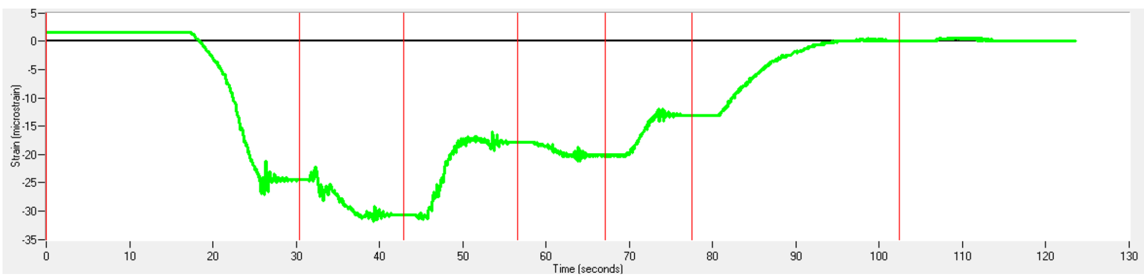

Figure 52. Strain Sensor Data for a Field Test

Each of the sensors used during testing outputs a similar graph. The vertical red lines represent a "click" in the test. A "click" is a data collection point performed by the user to provide information corresponding to the exact location of the truck during the test. This information is then used to determine the load magnitude expected at certain key points during testing. Further discussion on the individual program "clicks" performed during the tests are discussed later. The equipment used during testing is shown in figure 53 and figure 54. Figure 53 shows the data acquisition computer and power supply. Figure 54 shows the assortment of strain transducers. Each of the strain transducers was tested for calibration in the laboratory before used in the field. Figure 55 shows the calibration testing of the transducers.

Figure 53. Computer and Power Supply

Figure 54. Sensors Used in Testing

Figure 55. Initial Lab Tests

4.2.2 Field Testing

With the guidance of Donnie Head, a bridge located about 4 miles N.W. of Okarche with a 15 ton posted limit was found with the steel pile reinforcement repair already in place. The repair was implemented on this bridge in order to reinforce the corroded H-piles. All 5 of the piles were repaired on the east side bent. A schematic of the Kingfisher County Bridge is depicted in figure 56. As displayed in Figure 58, sensors were placed on the middle 3 piles of the east bent and the adjacent middle 3 piles of the west bent for a basis comparison.

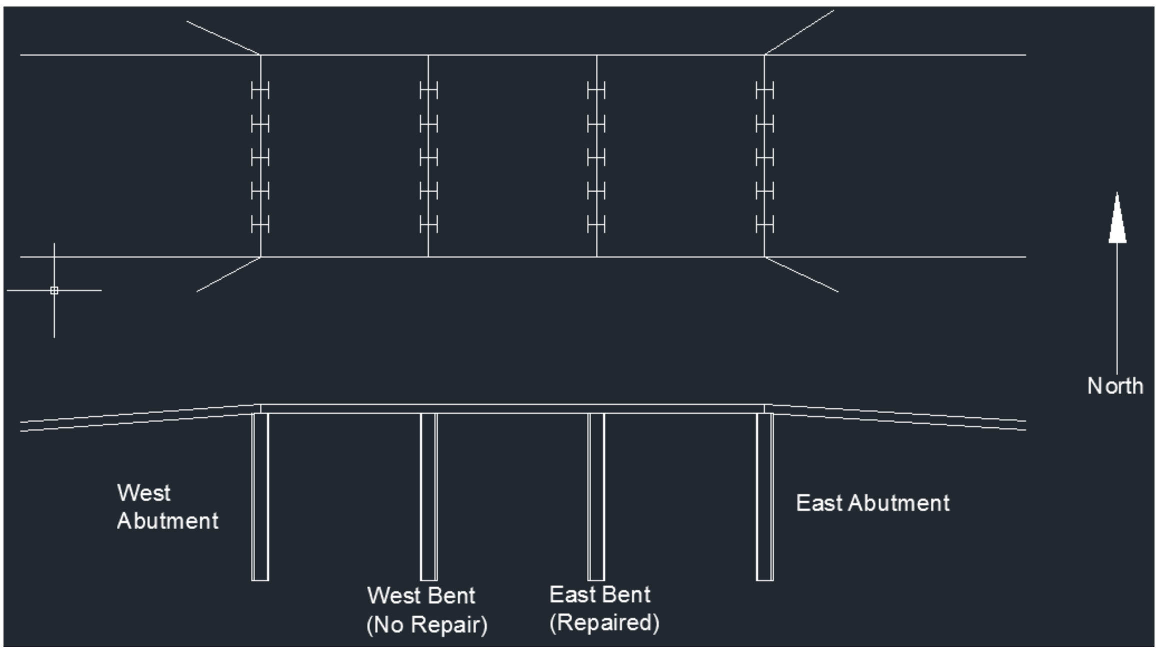

Figure 56. Detailed View of Kingfisher County Bridge

A total of 36 sensors were used to gather the required data. With the help of County Commissioner, Keith Schroeder, a truck with a know weight of 15 tons was used to drive over the bridge while the sensors analyzed the strains within the bridge piles. A picture of the 15 ton truck used during testing is shown in Figure 57 below.

Figure 57. Truck used for Kingfisher County Tests

As the truck drove slowly over the bridge, a “click” was performed within the sensor program to indicate key points as the truck crossed the bridge. The “clicks” were performed as the front axle, center of the truck, and the rear axle crossed over each of the two bents for a total of 6 “clicks”. As shown in Figure 59, three sensors were arranged on a steel plate so that the load path transferring into the supplement plates could be determined.

Figure 58. Kingfisher County Test Set Up

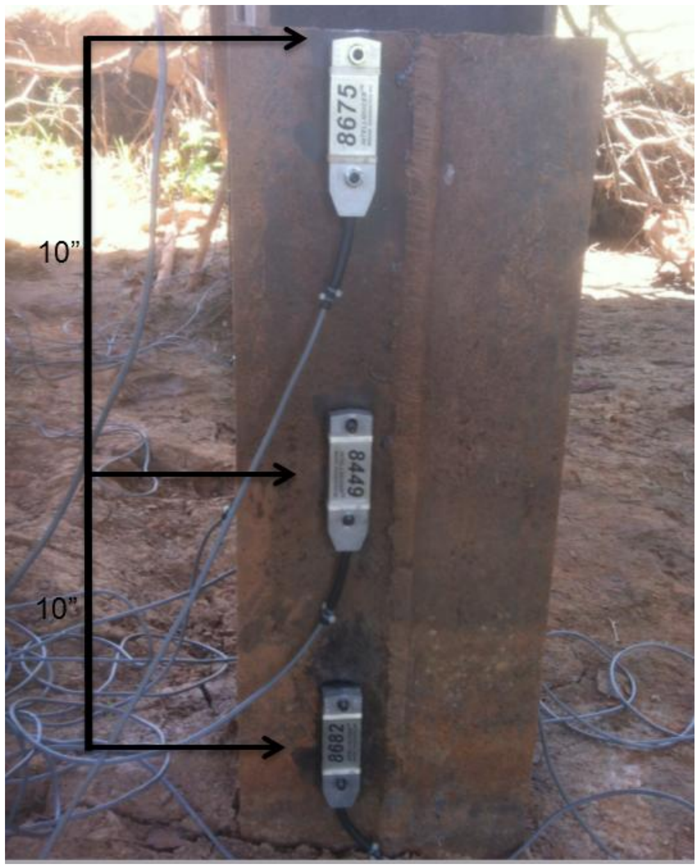

Figure 59. Sensor Configuration on a Repair Plate

4.2.3 Lab Testing

Controlled laboratory tests were performed to supplement the positive results from the Kingfisher County Test. The laboratory tests performed at Oklahoma State University replicated the field repair. Using the same HP-8x36 piles that were used at Kingfisher, 2 separate 3 foot sections were fabricated to create a consistent analysis. For each of the 3 ft. sections, a small area of the web was cut out using a blowtorch to replicate the section loss found in the field pile corrosion. Then two 2 ft. long 1/2” by 10” plates were welded onto the H-piles to simulate the field repair. Refer to Figure 60 for visual representation of the simulated repair plates used in the lab testing.

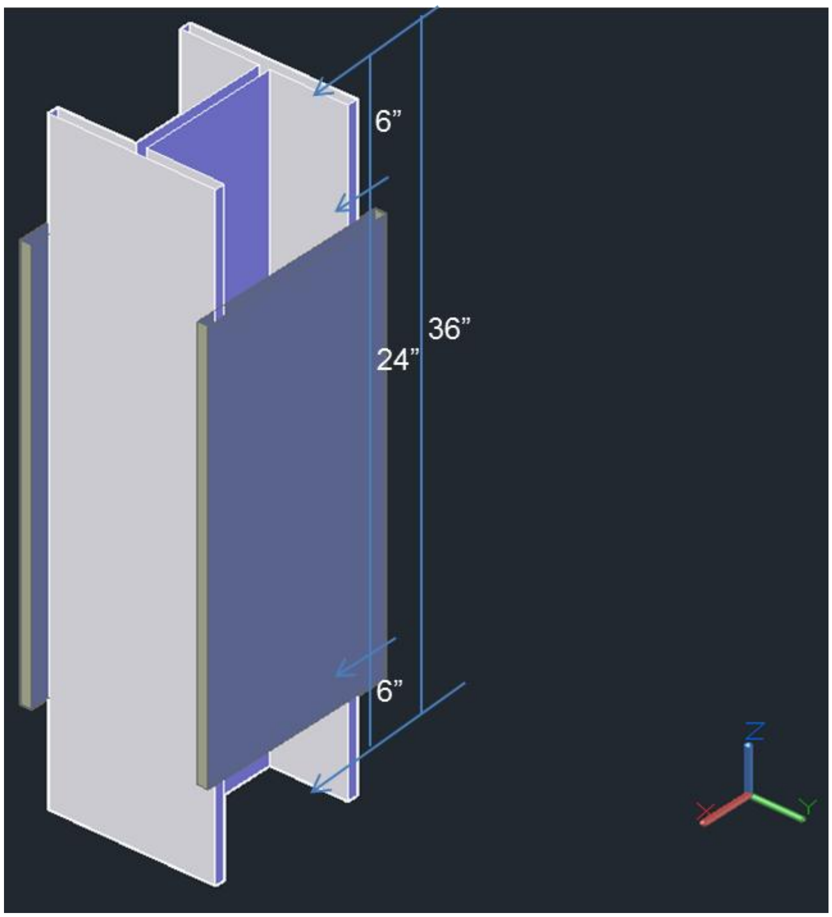

Figure 60. Lab replica of bridge H-pile repair plate with dimensions

The supplement plates were welded from the center to the ends in short increments and then testing was performed after each weld increment to determine the length the potential load is fully transferred to the supplement plate. Figures 61 and 62 show the initial set up of the sensors before any section loss was simulated. This provided base readings for future comparison of the data on later tests.

Figure 61. Specimen D Base Test

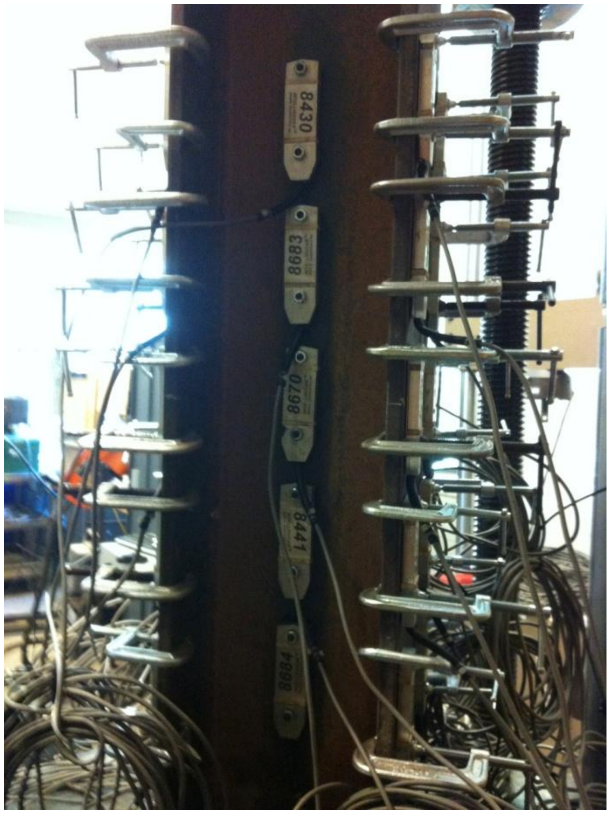

Figure 62. Specimen D Base Test without web cut out

A total of 70 Sensors were used during the lab testing, with each sensor placed on the pile by attaching two metal tabs onto the sensor itself. A fast acting adhesive was used to attach the sensors onto the pile. The sensors could then be carefully removed from the pile while keeping the tabs on the H-pile without having to be removed and replaced after each test. This allowed for consistent data collection since the sensor remains in the exact same position on each subsequent test. Figure 63 shows the tab arrangement on the pile.

Figure 63. Sensor Tab Arrangement on the Pile for Testing

The chosen tab arrangement also allows for 2 additional rows to be analyzed on the pile for a total of 7 rows. At each row there are 10 sensors placed symmetrically around the cross section, 3 on each plate and 2 on each flange of the H-pile.

Compression tests were performed on two 3 foot long steel piles with sections of the web cut out to simulate field corrosion damage. The first specimen tests, Specimen D, were completed in November of 2012. The testing process involved welding ½" steel plates on the flanges of the steel H-pile at various small increments. Therefor the first plate was welded onto the steel pile with only a 3" weld length at the center of both the pile and the plate. The pile was then placed in compression with 30,000 lbs. of force to simulate the loading caused when a truck drives over a repaired bridge. After the data was gathered, the 3" weld length was increased to 7" and then re-tested in the same manner. After the 7” weld length test, additional tests were performed on the specimen at 11”, 13”, 15”, 17.5”, 19.5”, 22”, and 24” weld lengths. For each test, a total of 70 strain gauges were attached to the specimen (42 on the plates, 28 on the pile). With only 30 strain gauges available, it took three separate tests to complete a full test on the pile at its respected length. To ensure consistent data, the compression tests were performed 3 times each time the sensors were moved to cover the complete specimen. Thus there were 9 total tests performed at each weld length. Since 10 different weld lengths tested, a total amount of 90 compression tests on Specimen D were performed. Each individual test had about 30,000 data points, so after 90 tests over 2.5 million data points for specimen D were collected. The same tests (90 in total), with the same sensors at the same locations, were performed on another pile, Specimen C, but with ¼” plates welded onto the pile instead of ½” plates to determine if the plate thickness effected the rate of load transfer into the supplement plate. Testing on Specimen C was completed in December. The data showed very similar results to the ½” plate on Specimen D. The ¼” tests provided nearly the same rate of load transfer as the ½” tests.

4.3 Results

The results from the field testing, lab testing, and 3-D Modeling showed successful transfer of the load from the steel pile to the plates. The field testing provided definitive evidence of the supplementary steel plates being loaded. The results showed that the welded steel supplement plates were indeed receiving a significant load from the bridge piles. The data revealed that the bottom sensor received the most load compared to the other two sensors, while the top sensor received very little load. The middle sensor took about 85% more load compared to the bottom sensor while the top sensor only received about 13% of the load. This showed that the repair was indeed working and that a development length was established before the potential load was fully transferred.

The lab tests provided detailed information on the load transfer rate based on the weld length. The load transfers into the supplementary plate in a linear fashion. This creates a linear line which allows for the determination of the weld length required for the plate to take on the full load for any pile size. Below illustrates how the strains in the piles decrease as the plate weld length increases.

The strain values, displayed in Figure 64, correspond to a 30 kip loading in the specimen in order to show the strain at a consistent load. Figure 64 illustrates the strain values within the specimen associated with each weld length. The data suggests that after the 19.5 inch weld no further strain is transferred from the specimen to the middle of the plate as the plate weld length increases. It can thereby be concluded that the plates welded onto specimen C have accepted all of the potential transfer loading from the specimen to the middle of the plate.

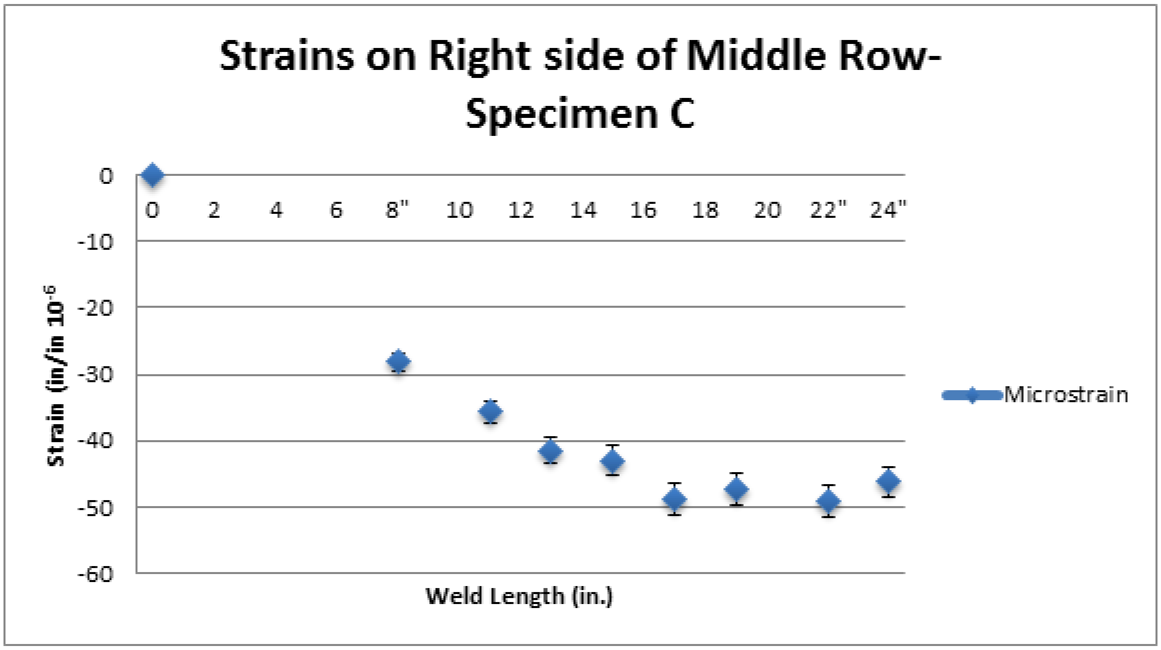

Figure 64. Specimen C strains in the middle of the plate

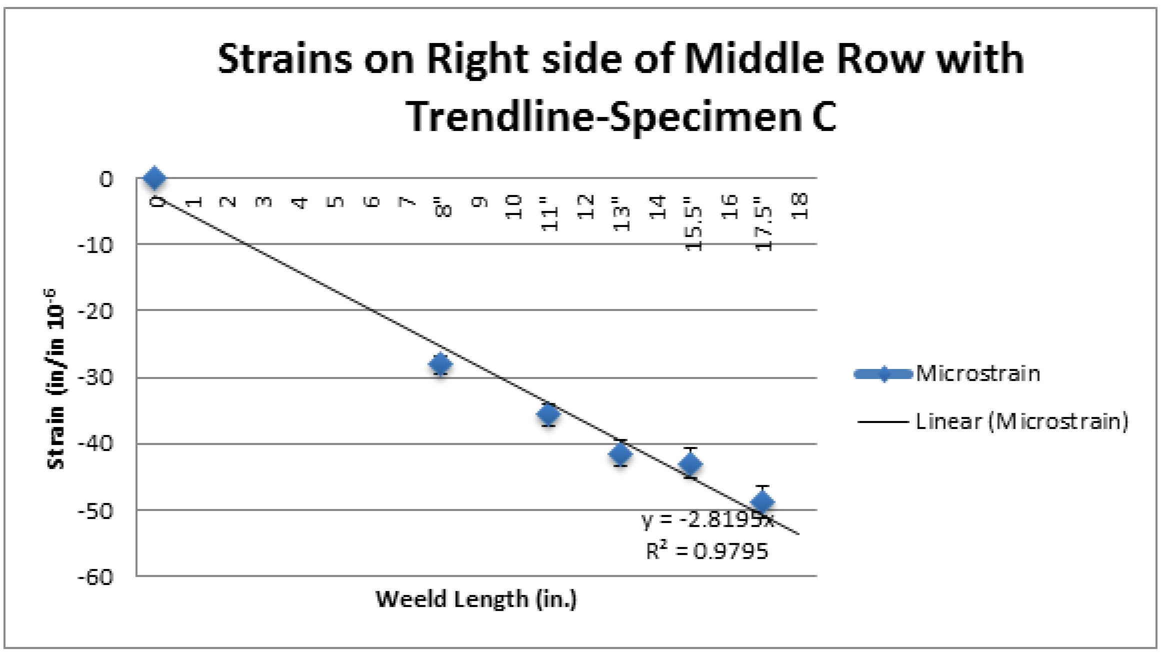

Figure 65. Strains with Linear Trend line

Figure 65 above provides the same information as Figure 65 except that the weld lengths at 22 inches and 24 inches have been excluded since the strain values did not increase after the 17.5 inch weld.

Figures 66 and 67 provide similar information for specimen D.

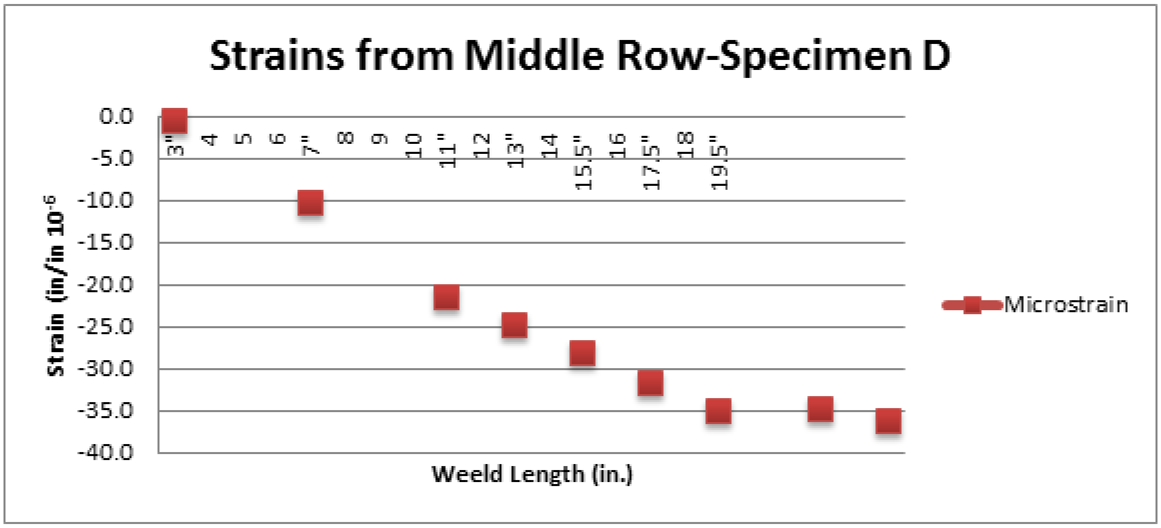

Figure 66. Specimen D Strains in the Middle of the Plate

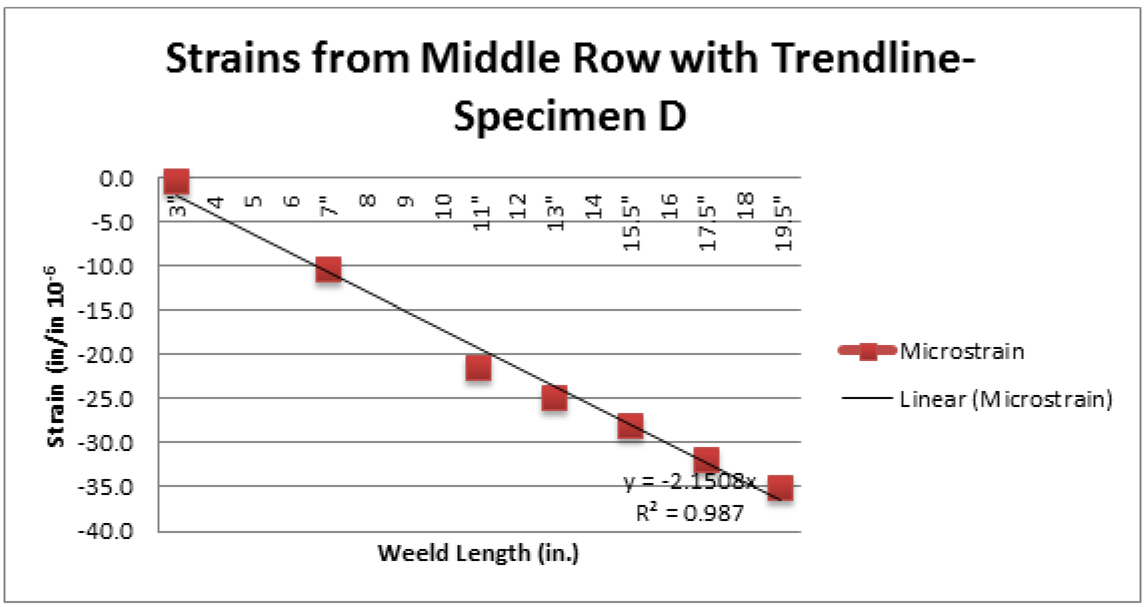

Figure 67. Strains in Middle Row with linear trend line

Figure 66 shows that the strains transferred into the supplement plate increase in a linear fashion as the associated weld length increases. Similar to specimen C, Figure 66 does not include data points from the weld tests at 22" and 24" since the strains transferring into the supplement plate peaked at the 19.5" weld test. Including strain data after the peak point would alleviate the true linear progression from the graphs. Figure 68 on the following page illustrates data from one of the supplement plates that were welded onto Specimen C. Using the program SigmaPlot10, color gradient maps were created for easy analysis and representation. In order to analyze the change in strains after each weld length, the graphs show the supplement plate strains with the specimen was under a 30 kip load. Each graph represents the strains on the same plate at each respective weld length. Purple, blue, and green shades represent lower strains values, while red, orange, and yellow shades represent larger strain values. Purple shows strains of zero while blue, green, yellow, orange, and red show strains of 10, 20, 30, 40, 50, and 60 microstrain respectively. Any strain values that were over 60 microstrain are represented in red.

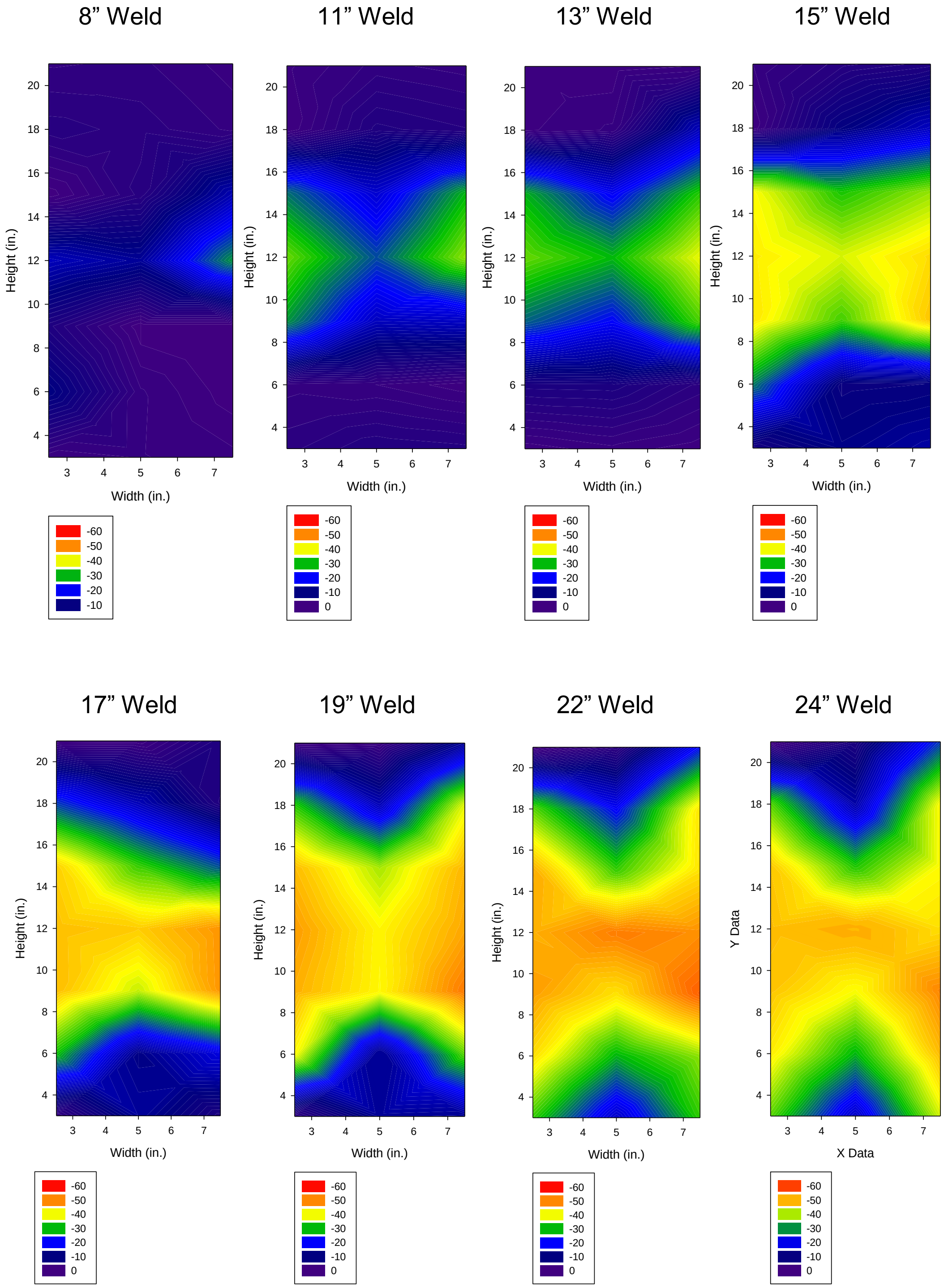

Figure 68. Strains within a repair plate at various weld lengths

Through analysis of the graphs, it can be shown that the strains within the supplement plate peak after the 19.5” weld length and no longer increase in the center of the plate. This confirms that the plates do not need any longer weld length to ensure the full potential transfer of load from the pile to the welded plate has occurred. Another important element to discuss is that the strains within the flanges of the pile decrease as the weld length increase. This demonstrates that the supplement plates are reducing the strains in the pile and transferring load from H-pile to the plates. Figures 70 through 77 below display graphs for each side of Specimen C at the various weld lengths tested. Sides 1 and 3 represent the flanges of the pile while sides 2 and 4 represent the 2 foot long plates welded onto the H-pile. Refer to Figure 69 for visual representation of each side.

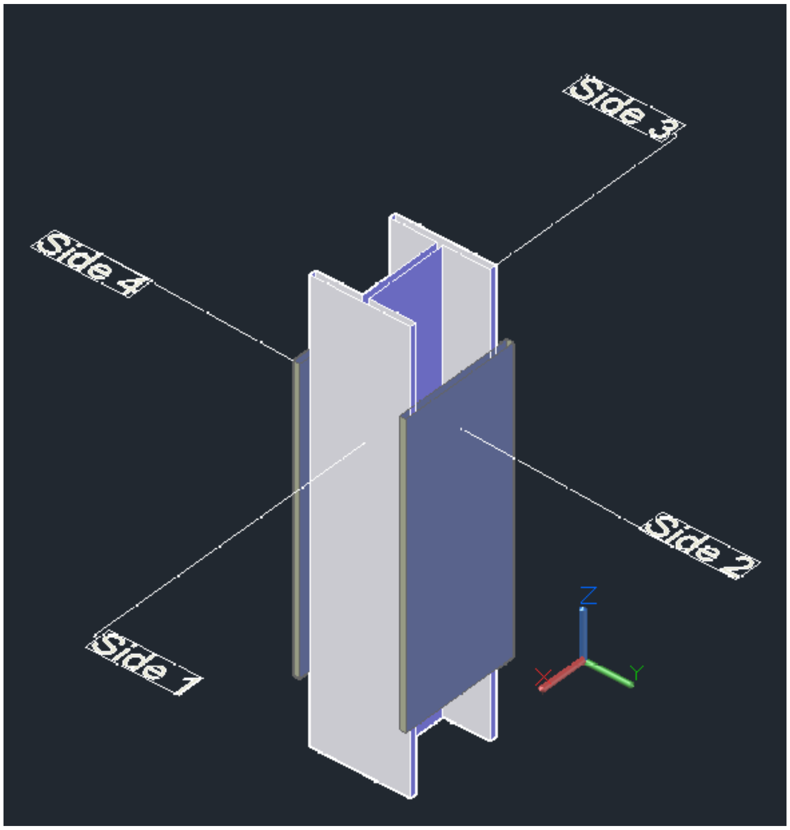

Figure 69. Visual representation of each side on test Specimen C

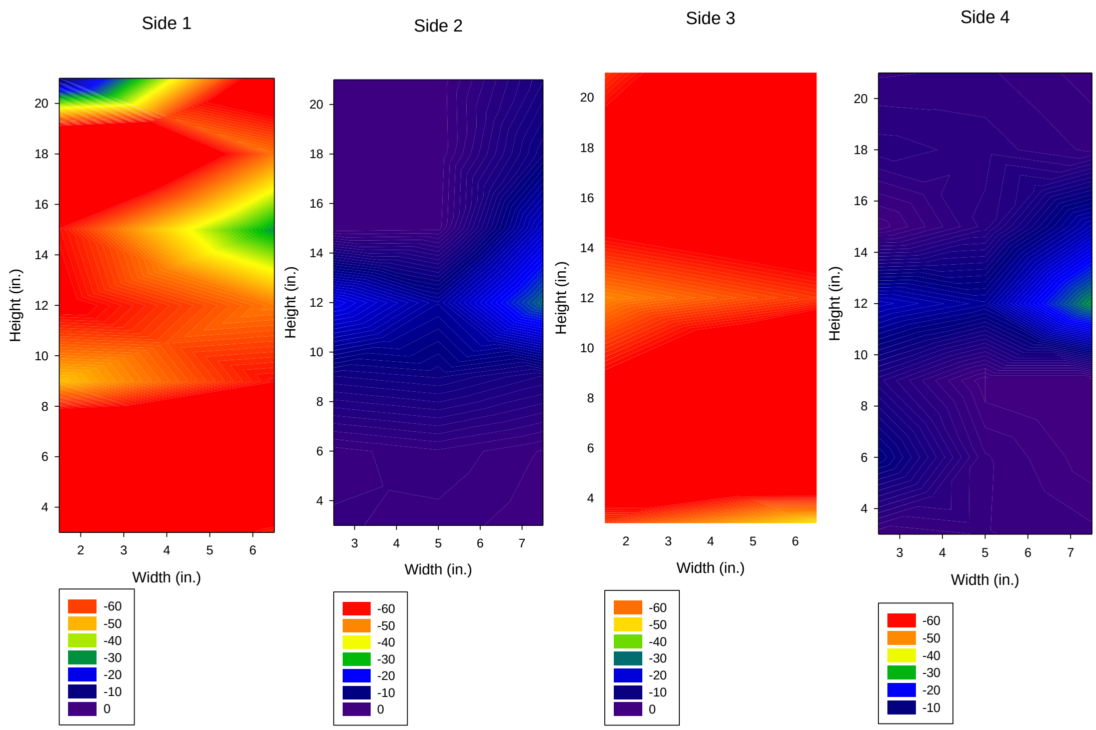

Figure 70. Strains at 30 kips within Specimen C with an 8" repair plate weld length

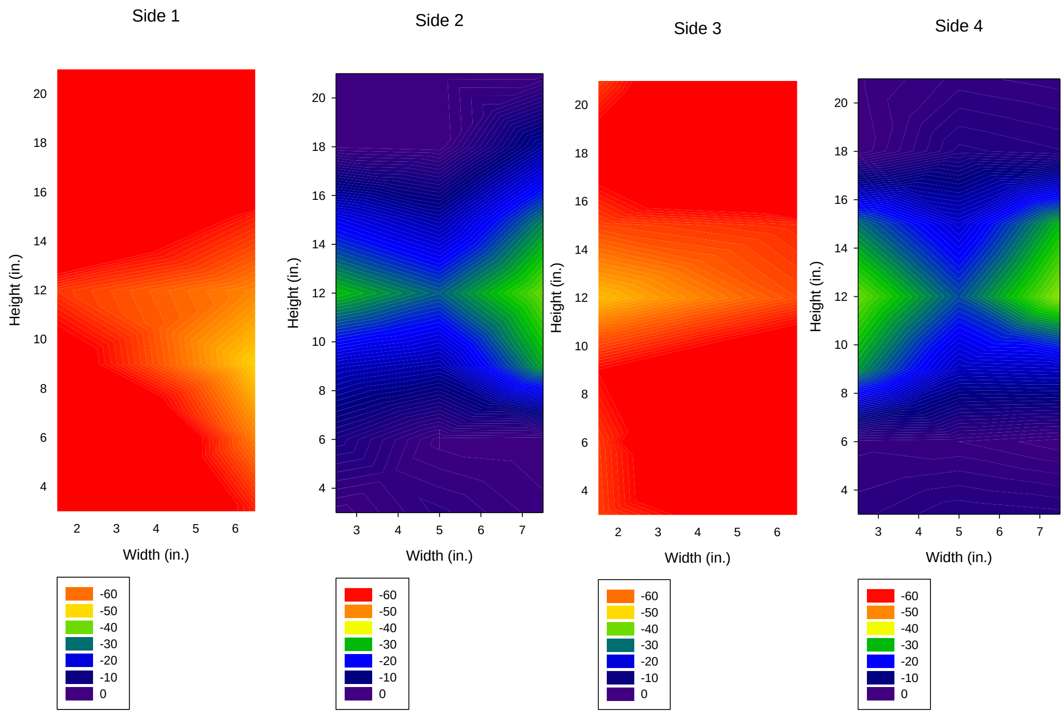

Figure 71. Strains at 30 kips within Specimen C with an 11” repair plate weld length

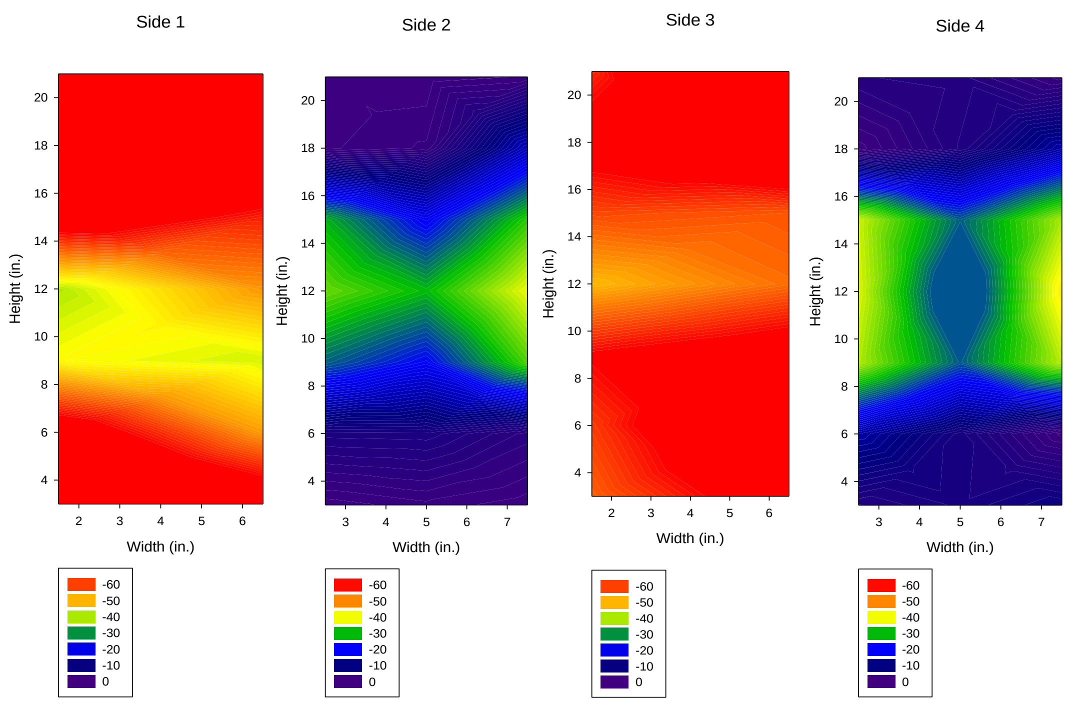

Figure 72. Strains at 30 kips within Specimen C with a 13” repair plate weld length

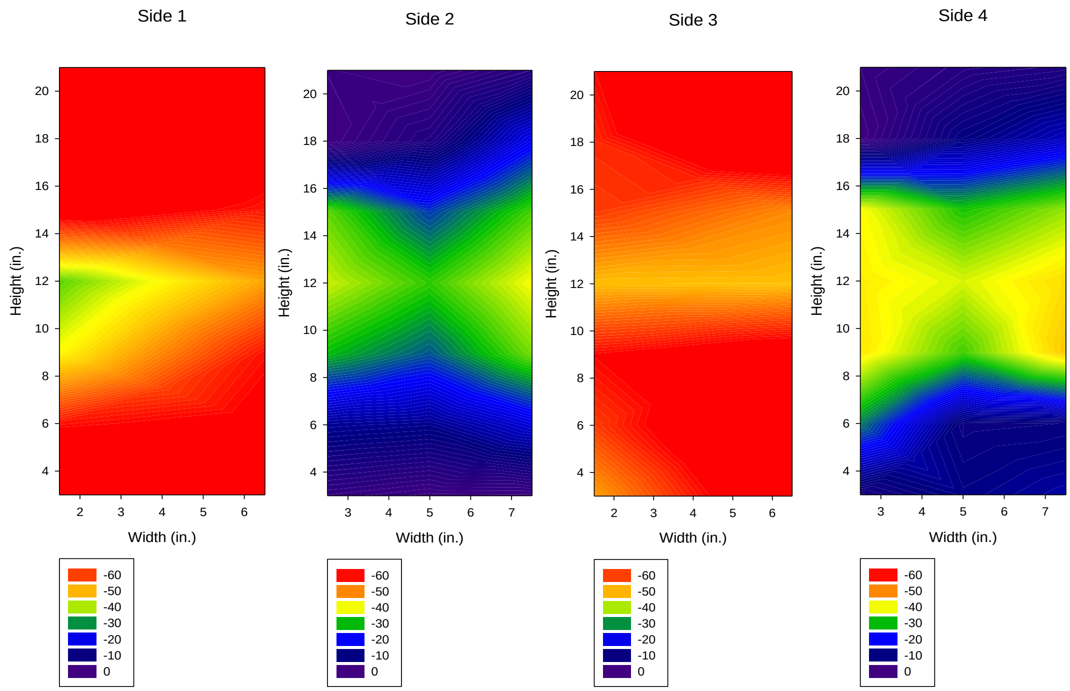

Figure 73. Strains at 30 kips within Specimen C with a 15" repair plate weld length

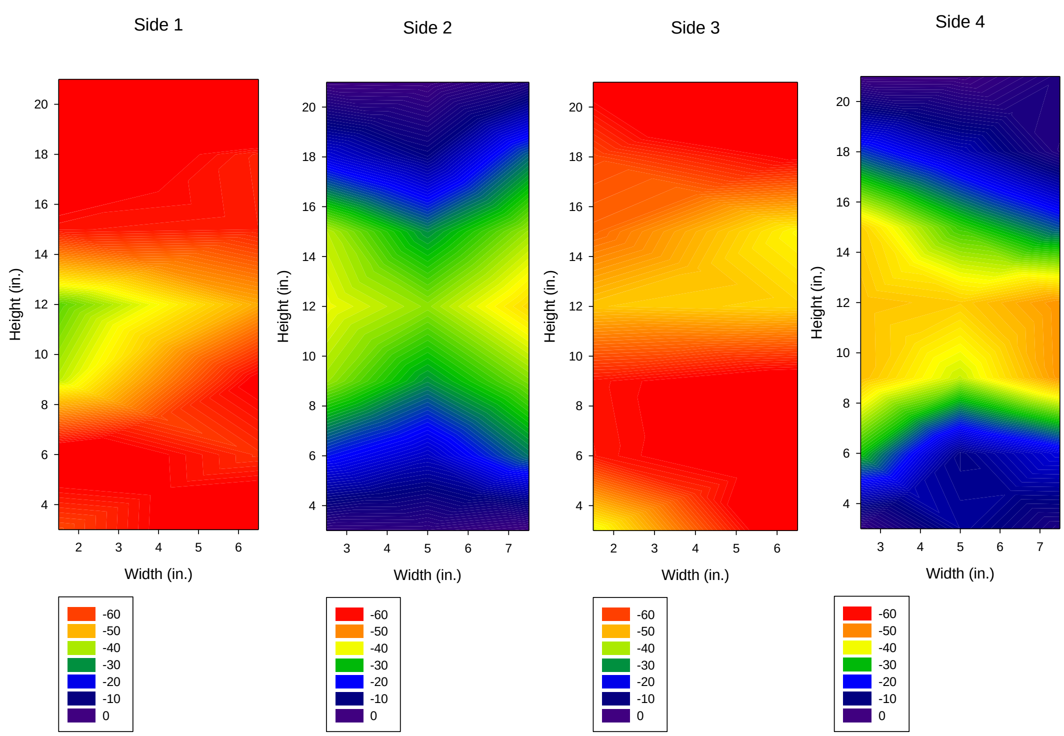

Figure 74. Strains at 30 kips within Specimen C with a 17" repair plate weld length

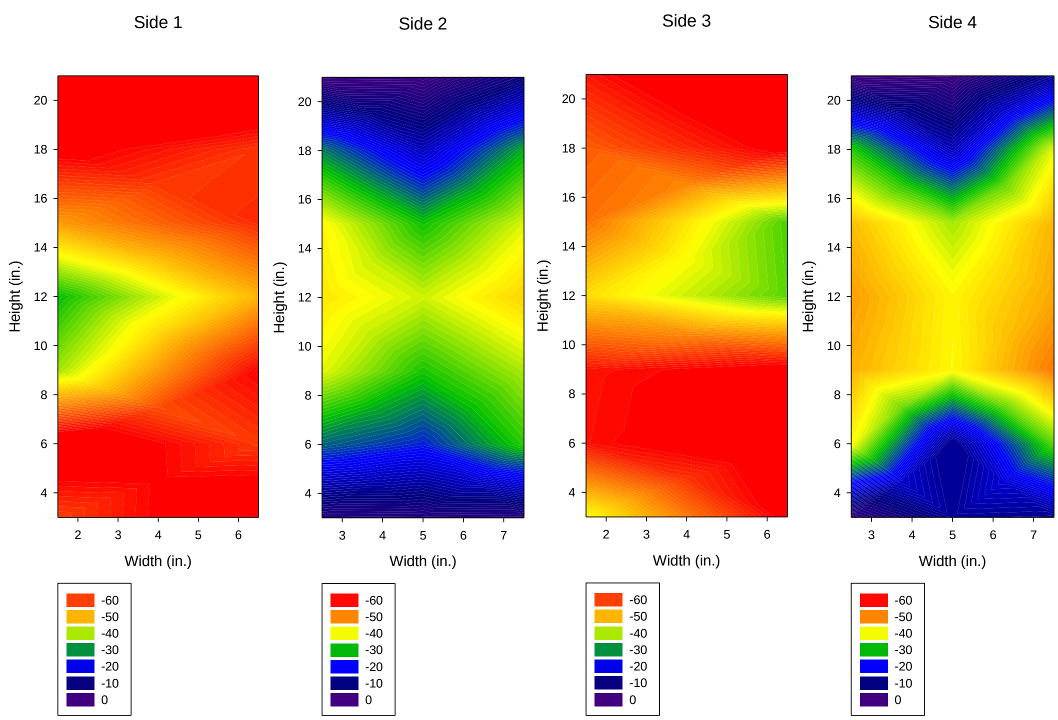

Figure 75. Strains at 30 kips within Specimen C with a 19” repair plate weld length

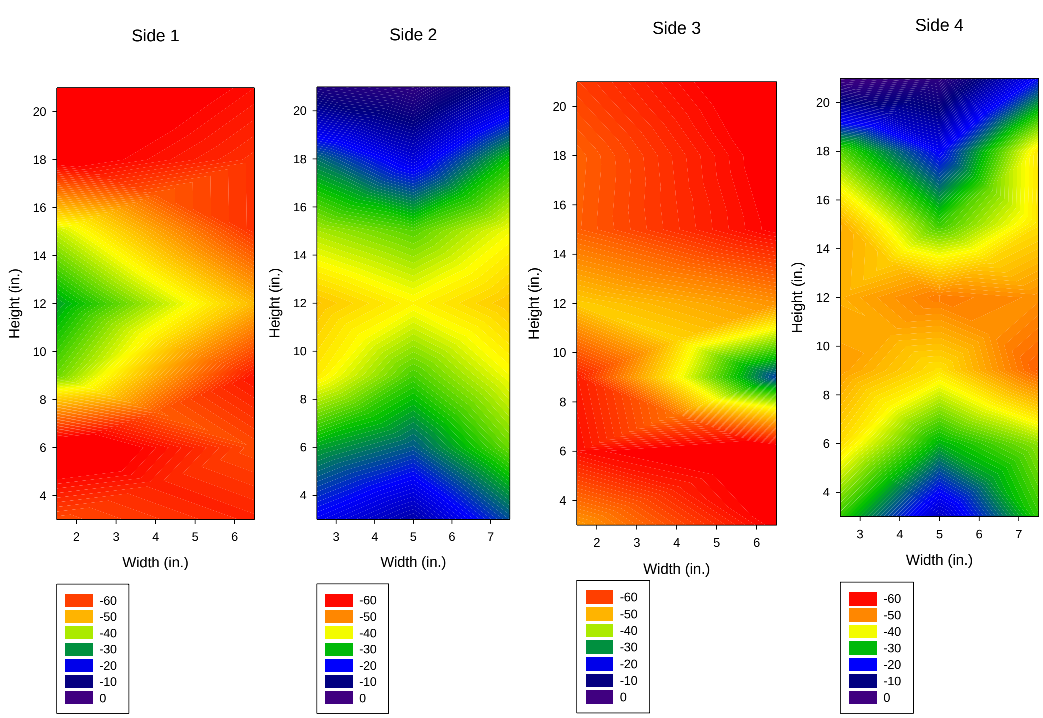

Figure 76. Strains at 30 kips within Specimen C with a 22” repair plate weld length

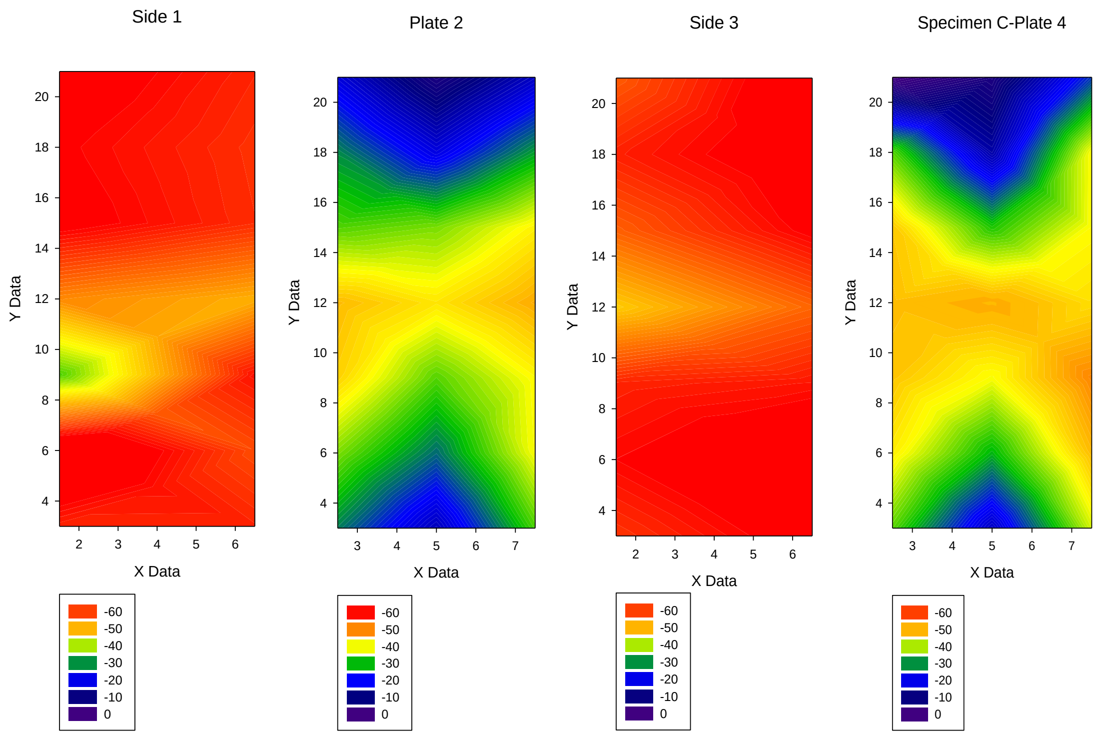

Figure 77. Strains at 30 kips within Specimen C with a 24" repair plate weld length

Comparing the plate and the pile strains shows that the supplement plate takes load from the pile and transfers 40% of the load from the pile to the plates. The fact that only 40% of the load is transferred from the pile to the supplement plates can be attributed to the remaining web that is present in the pile.

3-D modeling done in RISA 3-D has provided additional information for the design of the repair. The specimens tested in the lab were replicated in RISA 3-D to compare to the lab results. The results from the modeling were positive and supported the lab testing analysis. The data from the RISA modeling was converted to the SigmaPlot10 to present the results in the same fashion as the lab data. Figure 78 shows the color gradient strain maps of the modeling results.

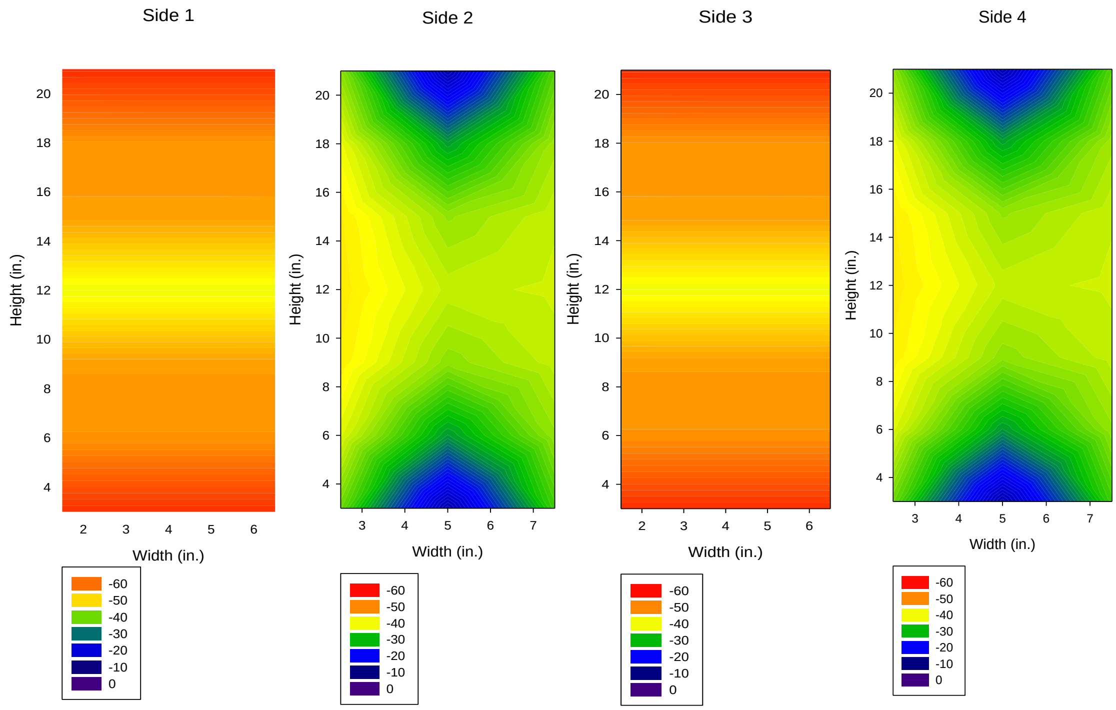

Figure 78. Color Gradient Strain Maps of RISA 3-D Model of Lab Test Specimens

The results of computer modeling depicted in Figure 78 shows nearly the same results as the lab testing. The microstrain values from experimental testing are about 20% less than the modeling strains. These differences can be attributed to error in the strain sensors, minor human error, or assumptions made in the RISA 3-D program. The RISA 3-D had to be drawn using hundreds of individual plates to replicate the lab testing. This causes the program to act like a finite model analysis. Regardless of the minor value differences, the results from modeling provided beneficial results. The contour maps from the RISA modeling provide nearly identical load transfer paths to those shown within the lab testing. These paths allowed for the development of a design equation. To determine whether the transfer path is the same for sizes larger than the HP 8x36 used in lab testing, additional RISA modeling was performed using the dimension of common H-piles used in county bridges. Donnie Head stated that county bridges will normally be HP 10x42's and in some cases HP 12x57's will be used. Any size pile larger than HP

12x57 can be found for federal bridges but these larger sizes are not used in county bridges. (Donnie Head)
The results from the HP 10x42 and HP 12x57 modeling showed that the loads transferred into the plate at the same rate as the HP 8x36. This confirms that a universal design equation will work for any size H-pile.

Using the results from the field testing, the lab testing, and the RISA 3-D modeling, a design equation can be formulated to provide easy use for the field repair designer. The results from the lab testing showed that the load does not increase in the middle of the plate after the 19.5” weld length. Running multiple analyses using RISA 3-D, shows that the load transfers into the plate does not differ based on the size of the pile.

In order to derive a generalized equation which can be used to determine the weld length necessary for any sized H-pile, the following variables were defined for each lab test. The variables used in development of the equation are depicted in Figure 79.

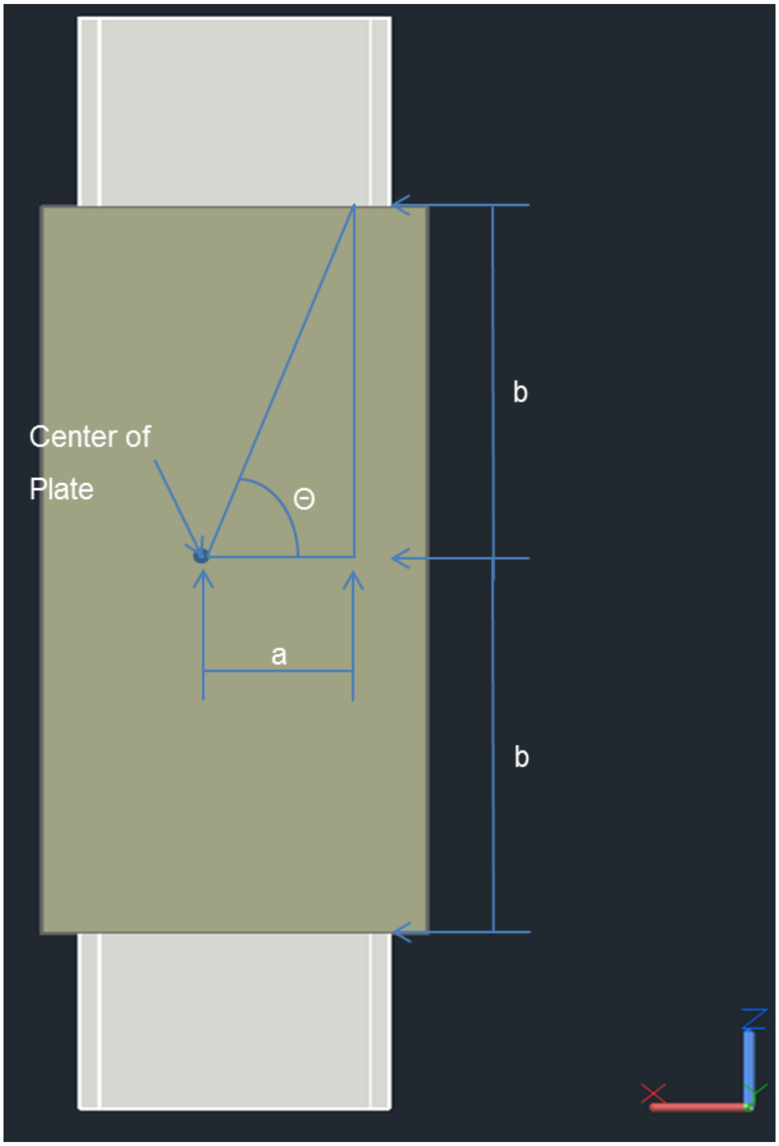

Figure 79. Detail for Design Equation Derivation

Where:

$$a = {}^{1}\!/_{2} * Height\ of\ H\ pile\ (inches) \tag{4-35}$$

$$b = {}^{1}\!/_{2} * Weld\ length\ from\ lab\ test\ (inches) \tag{4-36}$$

$$\theta = Angle\ of\ load\ transfer\ \pi\ (degrees)$$

During data collection it was found that the maximum potential load transfer from the H-pile to the repair plate occurred with the 20” weld length. The data collected from the 24” weld length lab test showed no increase in load transfer, and thus the 20" weld length was determined to be the largest length of weld needed for this particular H-pile size. Data from the 20" weld length test was therefore used to derive the equation which can be used to determine the necessary weld length for maximum potential load transfer on any size H-pile. The following data variables were taken from the 20" weld length test:

Height of Hpile = 8 inches

Weld length from lab test = 20 inches

Using equation (4-35) above, the variable "a" was determined.

a = ½ *8 inches

a = 4 (inches)

Using equation (4-36) above, the variable "b" was determined.

b = ½ * 20 inches

b = 10 (inches)

Equation (4-37), shown below, was then used to calculate the angle of load transfer.

$$\theta = \tan \frac{b}{a} \qquad (4\text{-}37)$$

$$\theta = \tan \frac{10}{4}$$

$$\theta = 68.2\ (degrees)$$

This angle of load transfer can then be substituted back into equation in order to derive a general equation which can be used for any size H-pile.

$$68.2 = \tan^{-1} \frac{b}{a} \qquad (4\text{-}38)$$

Then by solving for the variable "b" from equation (4-38), the following equation (4-39) is developed.

$$b = a * \tan\ 68.2 \qquad (4\text{-}39)$$

By calculating the Tangent of 68.2 to be approximately 2.5 and altering the variable names to make equation (4-39) more user friendly, the generalized equation for determining the required weld length for any given corroded H-pile size was derived. The finalized equation is shown below as equation (4-40).

$$W_L = h * 2.5 + L_c \tag{4-40}$$

Where:

$$h = Height\ of\ H\ pile\ (inches)$$

$$L_c = Length\ of\ corroded\ H\ pile\ section\ (inches)$$

$$W_L = Weld\ Length\ Required\ (inches)$$

Based on the design equation, design tables were developed for quick reference in determining the weld length required based on H-pile size and the length of corroded area.

Table 20 Design Table

	Weld Length Required (inches)		
Corroded Area Length (inches)	H-pile Size		
	HP 8x36	HP 10x42	HP 12x57
1	21	26	31
2	22	27	32
3	23	28	33
4	24	29	34
5	25	30	35
6	26	31	36
7	27	32	37
8	28	33	38
9	29	34	39
10	30	35	40
11	31	36	41
12	32	37	42
13	33	38	43
14	34	39	44
15	35	40	45
16	36	41	46
17	37	42	47
18	38	43	48
19	39	44	49
20	40	45	50
21	41	46	51
22	42	47	52
23	43	48	53
24	44	49	54

4.4 CONCLUSIONS

Through field testing of currently implemented bridge repairs on corroded H-piles, it was determined that the supplement plate type of repair is indeed working sufficiently to help prolong the life of off-system bridges. Further lab testing determined a more cost efficient and time efficient method to repair these structurally deficient bridges using the supplemental plate repair system. Currently the repair workers are welding the full length of 4 to 5 foot long supplement plate on to the H-piles which takes up additional time and resources. Through lab modeling, a user friendly equation was derived for county personnel to utilize when determining the necessary weld length for the repair plates. By determining the actual weld length necessary, repair workers can save both labor time and resources by limiting the amount of field welding and plate necessary to perform on each repair bridge. This will allow more time for the county officials to perform repairs on subsequent structurally deficient bridges within their counties. The supplement plate repair system will help to lengthen the life of these structurally deficient bridges.

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5 CONCLUSIONS AND RECOMMENDATIONS

Repair techniques were evaluated and developed for two common county bridge problems, decay of timber piles and corrosion of steel H-piles. The repair techniques are effective for restoring strength and extending the lifespan of county and local bridges. Relatively simple design guidelines were developed that county and local engineers and repair personnel can use for designing effective and safe repairs.

The repair technique for the repair of decayed timber piles consists of replacing the existing decayed timber pile with a steel pipe or H-pile section from the pile cap down to sound wood that exists below grade. The pile cap is temporarily lifted via jacking. The decayed pile is removed. A steel cap is placed on the sound wood below grade. A section of steel pipe or H-pile is put into place and the pile cap is lowered back into place effectively transferring its load. The repair was evaluated in the field through bridge load testing and was found to be effective in transferring loads from the pile cap to the remaining wood pile below. Design equations and design tables were developed for implementation of the repair at the county and local level.

The repair technique for the repair of corroded H-piles consists of welding steel plates to the flanges of the H-pile in the locations where the webs are corroded. Typically, the web of the H-pile has experienced the greatest deterioration. The steel plates are used to restore lost cross-sectional area and axial strength. The repair was evaluated in the field and in the laboratory via load testing. Through strain analysis it was found that the repair was fully effective when the plate extended and was welded out to ten inches beyond the location of corrosion. Simple design equations and tables were developed for future design of the repairs.

Both repairs are suitable for transferring axial forces. However, it is recommended that the connection between the below grade timber pile and the steel replacement should be evaluated and developed for moment resistance. Until then, the design procedures should only be assumed to resist axial forces and not bending moments effectively.

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6 IMPLEMENTATION AND TECHNOLOGY TRANSFER

The repair techniques that were evaluated and developed during the project have been shared and implemented for county and local bridges within the state. The repair techniques have been passed on to the county and state personnel that participated in the project. The repair techniques have also been presented to engineering students in the timber engineering course taught at Oklahoma State University. Technology transfer efforts continue in a variety of forms. Results are shared in the form of research reports through the Oklahoma Transportation Center. The technology will be transferred to county commissioners and county bridge personnel through the Association of County Commissioners of Oklahoma. The technology will be transferred to other local and tribal officials through the Center for Local Government Technology at OSU. The technology will also be continued to be shared with students in appropriate courses at OSU.

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