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Research Report for ALDOT Project 931-100

DEVELOPMENT OF PIPE PILE TO BENT CAP CONNECTION FOR ALDOT BRIDGES

Submitted to

The Alabama Department of Transportation

Prepared by

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16. Abstract

Steel pipe piles are a promising alternative to traditional H-piles and drilled shafts for ALDOT bridge substructures because they can achieve high force and moment capacity and enable more efficient construction. However, current guidance on steel pipe pile–to–reinforced concrete bent cap connections is limited, and practices among state DOTs vary widely in embedment length, connection detailing, and assumed fixity. This project was undertaken to develop and evaluate practical, code-compliant connection details that can support consistent design assumptions and potential standardization. An extensive survey of state DOTs regarding their steel pipe pile use and pipe pile–to–bent cap connection detailing practices was first conducted. Based on the survey results, an experimental program was designed to quantify the lateral-load behavior of large-diameter steel pipe pile–to–bent cap connections representative of common DOT practices. Five full-scale connection configurations were tested under repeated lateral loading with three rebar-based systems (headed, hooked, and straight bars) utilizing shallow embedment of 1 ft, and two mechanical anchorage systems (headed stud and annular ring) utilizing deeper embedments of 2–3 ft. For the rebar-based configurations, concrete plug depth was governed by reinforcement development/anchorage demands, while the mechanical anchorage configurations were evaluated with reduced plug depths. All configurations sustained the maximum applied loads without failure; all tests were terminated due to equipment limits rather than specimen capacity. The mechanical anchorage systems exhibited higher stiffness than the shallow rebar systems by about 2 times. The rebar-based systems exceeded their calculated nominal flexural strengths by at least 1.8 times. Embedment lengths on the order of one-third of the pile diameter provided a reasonable moment transfer, indicating that idealized “pinned” assumptions may not be applicable for hollow pipe piles. This report provides comparative experimental data and an implementation appendix (Appendix C) that supports selection and detailing of steel pipe pile–to–bent cap connections for ALDOT bridges. The findings further establish a foundation for future research to refine and optimize the design methodology of favorable connection configurations and further extend the methodology to pile-cap connections.

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ABSTRACT

Steel pipe piles are a promising alternative to traditional H-piles and drilled shafts for ALDOT bridge substructures because they can achieve high force and moment capacity and enable more efficient construction. However, current guidance on steel pipe pile–to–reinforced concrete bent cap connections is limited, and practices among state DOTs vary widely in embedment length, connection detailing, and assumed fixity. This project was undertaken to develop and evaluate practical, code-compliant connection details that can support consistent design assumptions and potential standardization. An extensive survey of state DOTs regarding their steel pipe pile use and pipe pile–to–bent cap connection detailing practices was first conducted. Based on the survey results, an experimental program was designed to quantify the lateral-load behavior of large-diameter steel pipe pile–to–bent cap connections representative of common DOT practices. Five full-scale connection configurations were tested under repeated lateral loading with three rebar-based systems (headed, hooked, and straight bars) utilizing shallow embedment of 1 ft, and two mechanical anchorage systems (headed stud and annular ring) utilizing deeper embedments of 2–3 ft. For the rebar-based configurations, concrete plug depth was governed by reinforcement development/anchorage demands, while the mechanical anchorage configurations were evaluated with reduced plug depths. All configurations sustained the maximum applied loads without failure; all tests were terminated due to equipment limits rather than specimen capacity. The mechanical anchorage systems exhibited higher stiffness than the shallow rebar systems by about 2 times. The rebar-based systems exceeded their calculated nominal flexural strengths by at least 1.8 times. Embedment lengths on the order of one-third of the pile diameter provided a reasonable moment transfer, indicating that idealized "pinned" assumptions may not be applicable for hollow pipe piles. This report provides comparative experimental data and an implementation appendix (Appendix C) that supports selection and detailing of steel pipe pile–to–bent cap connections for ALDOT bridges. The findings further establish a foundation for future research to refine and optimize the design methodology of favorable connection configurations and further extend the methodology to pile-cap connections.

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Table of Contents

LIST OF TABLES

LIST OF FIGURES

Chapter 1

INTRODUCTION

1.1 BACKGROUND

One of the most prevalent bridge configurations employed in the United States involves multi-span bridges featuring precast, prestressed concrete girders and pile bents. These pile bents comprise a cast-in-place, reinforced concrete (RC) bent cap, along with either RC drilled shafts or driven steel HP piles, as depicted in Figure 1-1(a) and Figure 1-1(b), respectively. Despite the extensive use of these pile types in bridge construction, they suffer from certain limitations in terms of construction practices and cost-effectiveness. To overcome these challenges, alternative members such as steel pipe piles have emerged as a viable solution. With their potential to deliver superior force and moment capacities, large-diameter steel pipe piles (Figure 1.1(c)) offer a promising alternative to the existing options.

Figure 1-1 (a) RC drilled shaft Steel (Brown et al., 2010); (b) HP piles (Voyiadjis et al., 2016); (c) Large diameter steel piles (Brown et al., 2015)

Large pipe piles with diameters ranging from 3 to 4 ft. hold significant potential as a viable alternative to existing piers in various scenarios. Steel pipe piles are extensively utilized in bridge construction due to their numerous advantages. These advantages include: i) Enhanced vertical bearing strength and lateral load resistance, enabling the possibility of accommodating larger bridge superstructures, ii) Elimination of the need for battered members or inclined braces, iii) Offering a practical and cost-effective solution for reaching the bearing strata when deep beneath soft soil, iv) Suitable for deployment in areas with hard ground or lacking distinct bearing layers, v) Demonstrating excellent environmental performance by requiring minimal earth removal, vi) Facilitating accelerated bridge construction compared to alternative options, vii) Availability in various lengths, diameters, and thicknesses, enabling customization to suit specific bridge requirements in an economical manner, viii) Easy production of longer piles through welding joint connections, and ix) Lightweight and high toughness, facilitating easy handling and transportation.

1.2 RESEARCH OBJECTIVES AND TASKS

The main objectives of this research study is to examine the feasibility of utilizing steel pipe pile bridges as an alternative to the current pile bents in Alabama. Additionally, the study aims to experimentally evaluate robust connection details and design methodologies for various steel pipe pile to bent cap connections for generic Alabama Department of Transportation (ALDOT) bridges, ensuring compliance with the current LRFD specifications and relevant ALDOT construction practices. The research findings will emphasize the advantages of steel pipe piles and serve as preliminary work for future studies, potentially leading to their adoption by ALDOT. The adoption of steel pipe piles has the potential to offer an efficient and effective substructure construction method for bridges of various sizes, resulting in reduced construction costs and improved utilization of taxpayer resources. The tasks involved in achieving these primary objectives include:

i. Conducting a comparative design study to assess the benefits of steel pipe piles over existing piles,
ii. Investigating the current state of practice regarding steel pipe and bent cap connections among state DOTs,
iii. Experimentally evaluating the behavior and capacity of potential connections and evaluating design methodologies,
iv. Proposing the necessary anchorages for the connection between steel pipe piles and reinforced concrete bent caps, as well as establishing design methodologies for the tested connections.

To meet these objectives, five full scale tests were performed on a variety of pipe-pile to bent cap connections were built and tested in the Advanced Structural Engineering Laboratory (ASEL) at Auburn University.

1.3 RESEARCH SCOPE

The scope of work to accomplish the research objective includes collecting and reviewing previous studies on steel pipe pile to bent cap connections, as well as obtaining other state DOT approaches to designing and detailing these connections. Based on the literature findings, a comparative study against steel pipe piles and existing alternatives was conducted to highlight the potential advantages. Lastly, experimental studies were conducted for load testing different connection details to evaluate several alternative approaches for connecting pipe piles to bent caps, and developing design methodologies for the calculation of strength of proposed connections based on the design code provisions.

1.4 ORGANIZATION OF THE REPORT

- Chapter 2: **Literature Review and DOT Survey** – Reviews steel pipe pile–to–bent cap (and foundation) connection configurations, summarizes prior research and available strength-calculation approaches, and documents key parameters affecting connection behavior. This chapter also presents the state DOT survey, focusing on typical details, drawings, embedment lengths, and design assumptions used for steel pipe pile applications.
- Chapter 3: **Experimental Program Design** – Provides experimental investigation details in terms of test setup and specimen designs to evaluate the structural behavior of five steel pile-to-bent cap connection configurations.
- Chapter 4: **Specimen Construction and Details** – Details the construction drawings of test specimens, outlining specimen details and intended purpose within the research.
- Chapter 5: **Experimental Setup and Instrumentation** – Presents the experimental testing program, including the testing procedures and execution details.
- Chapter 6: **Experimental Results and Discussions** – Provides a comparative analysis of results to assess the impact of different steel-pipe pile to bent cap connection configurations.
- Chapter 7: **Summary, Conclusions, and Recommendations** – Concludes the report by presenting a comprehensive discussion on the connection details between steel pipe piles and bent caps. Additionally, the chapter offers recommendations for future research aiming to optimize steel pipe pile connections for improved structural performance and implementation.

Chapter 2

LITERATURE REVIEW

2.1 BACKGROUND

This chapter provides an overview of the research programs conducted on the connection between steel pipe piles and bent caps/foundations. The review begins by examining the use of concrete-filled steel tubes (CFST) embedded within reinforced concrete bent caps/foundations. It then explores various types of anchorage methods utilized in these connections, along with the corresponding strength calculation methodologies

2.2 PILE BENTS

Similar to many other states, ALDOT commonly employs drilled shafts and H-piles as the primary pile types for bridge construction. These piles serve as bearing elements for deep foundations and extend into the bridge cap to function as piers. They are typically used to support structures with significant axial and lateral loads. However, discussions with ALDOT engineers have revealed that the department is exploring alternatives to RC drilled shafts and HP-piers due to a range of issues, which will be detailed in the respective sections below.

2.2.1 DRIVEN H-PILES

H-piles are a commonly utilized pile type in bridge construction, specifically designed for deep foundations. These piles are extensively employed as substructures for small to mid-size bridges constructed on soft soils. The typical approach involves utilizing cast-in-place reinforced concrete bent caps supported by driven steel HP shapes. Unlike other piling systems that face challenges in penetration, H-piles can be driven deep into the ground to reach the necessary soil properties required for supporting buildings and bridges.

ALDOT-funded previous research studies (Marshall et al., 2017) have shown that in such bridge configurations, a combination of simply supported precast, prestressed concrete girders and pile bents is adopted. These pile bents consist of a cast-in-place reinforced concrete bent cap and driven steel HP piles. Typically, the two outermost HP piles are inclined at a slope of 1.5:12 to provide lateral resistance against overturning moments induced by winds, waves, and other lateral loads, as illustrated in Figure 2-1. Moreover, additional bracing is necessary to ensure sufficient lateral stiffness and strength against these lateral loads.

Figure 2-1 Multi-span bridge with cast-in-place reinforced concrete bents supported on H-piles (Anderson et al., 2018)

In cases where the bridge bent is positioned within a flow channel, additional measures are taken to protect H-piles. This involves either providing encasement for the H-piles or applying galvanization for section protection. The extension of HP piles into the bent cap by approximately 12 in. can disrupt the arrangement of bottom longitudinal reinforcement in the bent cap. Due to the relatively small size of H-piles, each HP pile system directly supports a single girder located directly above it. This member layout results in minimal transfer of shear force or bending moment into the bent cap, as the force is directly transmitted between the girders and corresponding H-piles. Consequently, smaller bent caps can be employed.

Figure 2-2 showcases a representative ALDOT bridge bent with HP-piles. The bent cap measures 38 ft. 6 in. in length and has a trapezoidal shape, with a smaller base of 2 ft. and a larger base of 3 ft. The bridge substructure comprises six girders positioned on top of six HP-piles of size 14 x 73. In cases where the clear height of the pile exceeds 14 ft., lateral bracing is provided using struts measuring 4 in. x 3 ½ in. When the bent is located within the flow channel, the steel piles are encased in concrete, extending 3 ft. above the mudline. Typically, the pile encasements extend at least 5 ft. below the projected ground elevation.

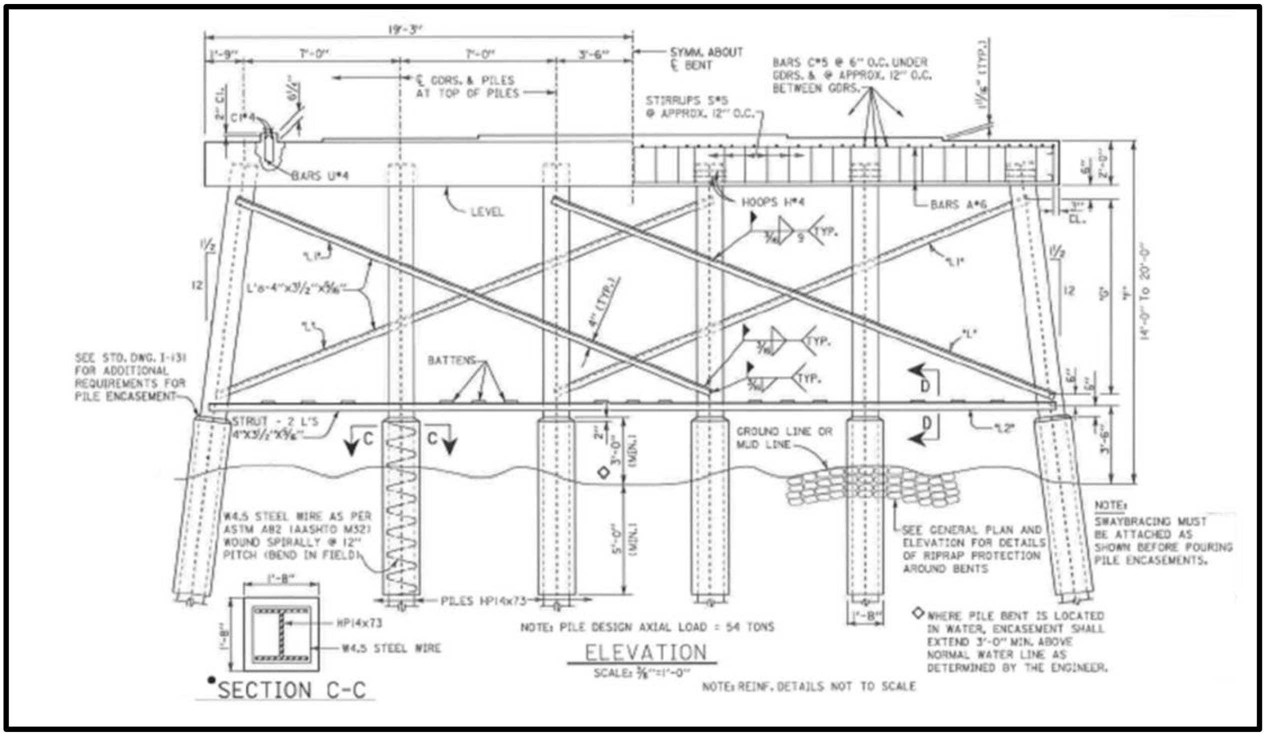


Figure 2-2 Typical multi-span bridge supported on H-piles on flow channel

Despite the widespread use of H-piles in bridge construction, numerous issues have arisen based on field inspections, particularly concerning cracking and damage in the bent cap, specifically near the exterior battered piles. These observations have been supported by previous studies (Marshall et al., 2017). Consequently, the construction of bridges using such piles entails a laborious design process, a complex construction procedure, and ultimately results in long-term performance problems.

2.2.2 DRILLED SHAFTS

Reinforced concrete (RC) piles, commonly known as drilled shafts, are frequently employed to provide deep foundations capable of supporting substantial axial and lateral loads. This involves excavating cylindrical shafts into the ground, inserting reinforcement cages into the shafts, filling them with concrete, removing the permanent casing, and ultimately forming a reinforced concrete pile, as depicted in Figure 2-3.

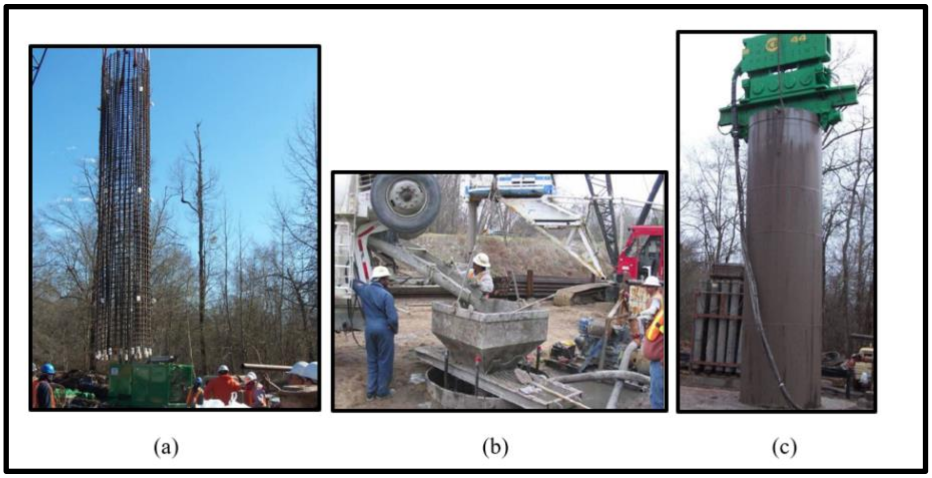

(a) (b) (c)

Figure 2-3 Construction process of drilled shaft; (a) Drilling and reinforcement casing; (b) Concrete pouring; (c) Removal of permanent casing [from Nucor Skyline Catalogue]

RC drilled shafts are commonly utilized for longer spans or wider bridges due to their ability to withstand higher load demands associated with such structures. In the case of a multi-lane bridge featuring a 40 ft. long bent cap with six longitudinal girders, generally, only two large diameter drilled shafts are required. Unlike H-pile bridges, this beam and pier configuration necessitates designing the bent caps to accommodate high moment and shear forces resulting from having fewer piers than girders. As a result, the bent cap beams must be designed to withstand these forces, leading to the utilization of deeper bent caps, such as the 4 ft. 6 in. x 4 ft. bent cap illustrated in Figure 2-4.

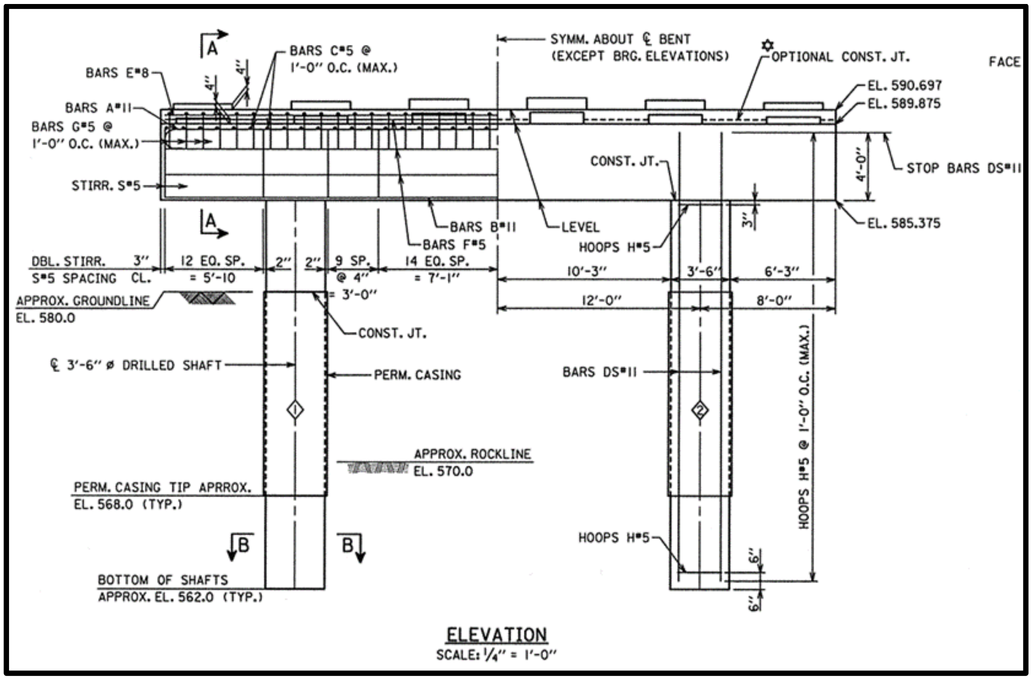


Figure 2-4 RC Drilled shaft use in SR-3 over Cedar Creek in Morgan County, AL

While considered structurally robust and well-established, one of the significant drawbacks of RC drilled shafts lies in their laborious and expensive construction process. This can be attributed to the challenges encountered during drilling, installation, and casting operations.

2.2.3 LARGE DIAMETER STEEL PIPE PILES

Steel pipe piles offer numerous advantages, including their large vertical bearing and bending strength, excellent environmental performance, ability to be customized for each structure, ease of joining with other structures, and convenient handling. These piles come in various dimensions, production methods, strengths, and applications, which are summarized below.

2.2.3.1 Types of Steel Pipe Piles

Various types of steel pipes are manufactured by steel companies, including electric resistance welded (ERW) pipes, spiral welded pipes, rolled and welded pipes, and micropiles. The product catalog of Nucor Pipes, one of the largest steel producers in the US, provides information on the available geometries of steel pipes, which are presented in Table 2-1. The lengths and geometries of the pipes can be customized to suit specific applications and uses.

Table 2-1 Dimension of different types of pipes

Types of pipes	Diameter (in.)	Thickness (in.)	Length(ft.)
ERW	$2^{3}/_{8}$-24	0.179- 5/8	Up to 80
Spiral welded	16-120	0.188-1	Up to 130
Rolled and Welded	24-204	0.25-$2^{1}/_{4}$	Up to 120
Micropile	7-16	0.472-0.5	-

Among the listed pipe piles, all except for spirally welded ones are straight. For straight pipes, the manufacturing process involves rolling up and welding steel plates, which requires larger plates. This process generally increases manufacturing difficulties, and the sizes of straight pipes do not vary significantly. In contrast, spiral welded steel pipe piles are preferred over other types because they experience less stress on the seam, enabling them to withstand greater pressure. Spiral welding also offers advantages such as crush resistance and better control over diameter tolerance. The direction of rolling the skelp is neither perpendicular nor parallel to the longitudinal axis of the pipe, enhancing its crack resistance. Additionally, spirally welded pipes are more flexible and customizable, allowing for larger diameters and longer lengths compared to other types of pipe piles.

2.2.3.2 Manufacturing Process of Spiral Welded Pipe

The production process of steel pipes offers flexibility in manufacturing a wide range of pipe diameters and wall thicknesses. This flexibility is essential due to the diverse applications of steel pipes in both structural and non-structural construction projects. Manufacturers can tailor the production to meet the specific requirements of different construction works.

Steel pipes are manufactured using steel coils and a helical double submerged arc weld (DSAW) process. The manufacturing process involves several steps, as depicted in Figure 2-5. The process begins with a steel coil placed on a horizontal uncoiler mandrel and fed into a straightener. The strip is then passed through a flattener to remove the coil set. As the coil moves through the straightener, the leading and trailing edges of the strip are trimmed in preparation for butt welding. Carbide teeth are used to trim the edges of the coil for welding. The strip enters a three-roll apparatus consisting of lead, buttress, and mandrel roll sets, where it starts to take on a spiral shape, eventually forming a pipe. The welding system performs the welding process, starting from the inside diameter and continuing to the outside diameter using a submerged arc welding technique. After welding, the pipe undergoes visual testing by Quality Control (QC), and if necessary, Ultrasonic (UT) testing is conducted to ensure the absence of defects in the weld. Finally, the pipes are cut to the desired length using a cut-off machine, and specific end properties such as bevel or square cuts can be produced for easier splicing on-site.

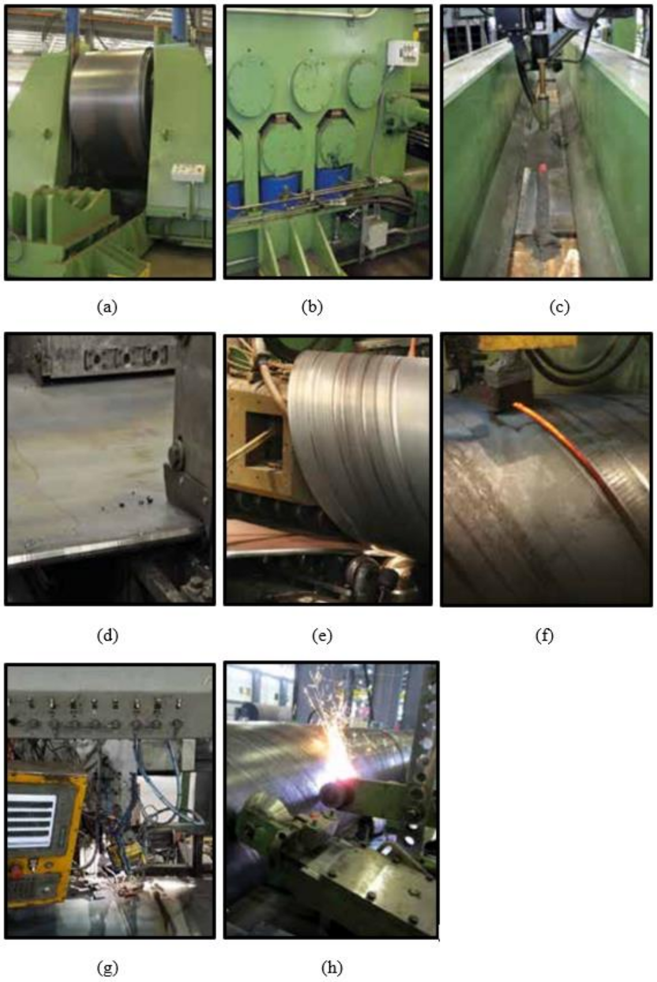


Figure 2-5 Manufacturing process of spiral weld pipe; (a) Uncoiling; (b) Flattening; (c) Joining of the coil ends; (d) Edge milling; (e) Pipe spiraling; (f) Pipe welding; (g) QC; and (h) Pipe cut-off [from Nucor Skyline Catalogue]

2.2.3.3 Pipe Specifications

Steel pipe piles are manufactured according to the pipe specifications expressed in Table 2-2 which provides guidance on manufacturing processes, dimensional requirements, and tolerances. Among all these ASTM 252 Grade 3 having a yield strength of 50 ksi is the most commonly used and readily available steel pipe. Based on the product catalog of Nucor pipes, this specification especially covers steel pipe piles of average wall thickness. It applies to pipe piles in which the steel cylinder acts as a permanent load-carrying member, or as a shell to form cast-in-place concrete piles. The piles should be made by the seamless, electric resistance welded, flash welded, or fusion welded process whereas, the seams of welded pipe piles should be longitudinal, helical-butt, or helical-lap.

Table 2-2 Steel Pipe Grades

ASTM	Yield Strength (ksi)	Manufacturing process		
		Spiralweld	ERW	Rolled and Welded
A 139 Grade A	30	✓	✓	✓
A 139 Grade B	35	✓	✓	✓
A 139 Grade C	42	✓	✓	✓
A 139 Grade D	46	✓	✓	✓
A 139 Grade E	52	✓	✓	✓
A 252 Grade 1	30	✓	✓	✓
A 252 Grade 2	35	✓	✓	✓
A 252 Grade 3	45	✓	✓	✓
A 252 Grade 3	50	✓	✓	✓
A 252 Grade 3	60-80	✓	✓	✓
A 500 Grade B	42		✓	
A 500 Grade C	50		✓	
A 1085	50-70		✓	

2.2.3.4 Application of Steel Pipes as Piles

As discussed earlier, steel pipes possess mechanical and physical properties that make them highly versatile in construction. They provide strength, cost-effectiveness, and ease of installation. Due to the flexibility afforded by the production process and stringent quality control measures, steel pipe piles find applications across a wide range of construction projects. They are commonly utilized in foundation works for marine structures, civil structures, bridge footings, abutments, and even as casings for concrete piles. The following are some examples of the uses of steel pipe piles:

- Bearing piles: Circular steel pile (Figure 2-6 (a)) offers several advantages over other types of driven piles, making them an efficient choice for load-bearing structures. The circular geometry of the pipe eliminates any weak axes, ensuring uniform load distribution. Additionally, the interior of the pile can be driven into the ground to remove obstructions, further enhancing their effectiveness. To further enhance the strength of these piles, they can be reinforced with concrete.
- Drilled shaft casing: Steel piles can also serve as casings for drilled shafts, as depicted in Figure 2-6 (b). These casings encompass a reinforcement cage and concrete. By using casings, the flow of concrete is controlled, preventing the formation of voids within the

structure caused by soil or water intrusion. Furthermore, the casings help mitigate the deterioration of the shafts.

- Combination walls: Large diameter steel pipes exhibit high bending strengths, making them ideal for constructing walls. A highly efficient system can be achieved by combining pipe piles and steel sheet piles, as shown in Figure 2-6 (c). In this system, the pipe piles bear the majority of the loads, while the steel sheet piles transfer the loads to the surrounding soil. This combination results in a robust and effective structural solution.
- Structural sections: The geometry of steel pipes offers uniform bending strength in all directions, making it an excellent material for resisting buckling and ensuring structural stability. This characteristic makes steel pipes well-suited for use in structural sections, as exhibited in Figure 2-6 (d).
- Sign poles, Towers, and Transmission lines: Sign poles and towers, as depicted in Figure 2-6 (e), are specifically designed to withstand significant bending loads at their bases. The extensive availability of large diameter pipes, coupled with a wide range of thicknesses and lengths, offers designers the flexibility to select the precise size required for their specific project. Additionally, the use of reduction collars enables efficient splicing of different pipe diameters, further enhancing the overall design efficiency.
- Mining: Air shafts are crucial for mining operations conducted in hazardous underground environments. Steel pipe piles, illustrated in Figure 2-6 (f), offer a versatile solution for constructing shafts of various diameters, thicknesses, and lengths, tailored to specific project requirements.
- Jacked and Bored: Jacked and bored pipes, depicted in Figure 2-6 (g), are employed for the installation of underground utilities. This process involves several steps. Initially, sections of piles are pushed into the ground using hydraulic jacks, and subsequent pipes are spliced onto the first section to continue the jacking process. Once jacking is completed, the pipe serves as a conduit for utility installation, eliminating the need for extensive excavation.
- Line pipe: Spirally welded pipes, illustrated in Figure 2-6 (h), serve as excellent construction materials for transporting liquids, air, and gas. These pipes exhibit exceptional strength compared to other options and are specifically designed to withstand internal and external pressures in various applications.

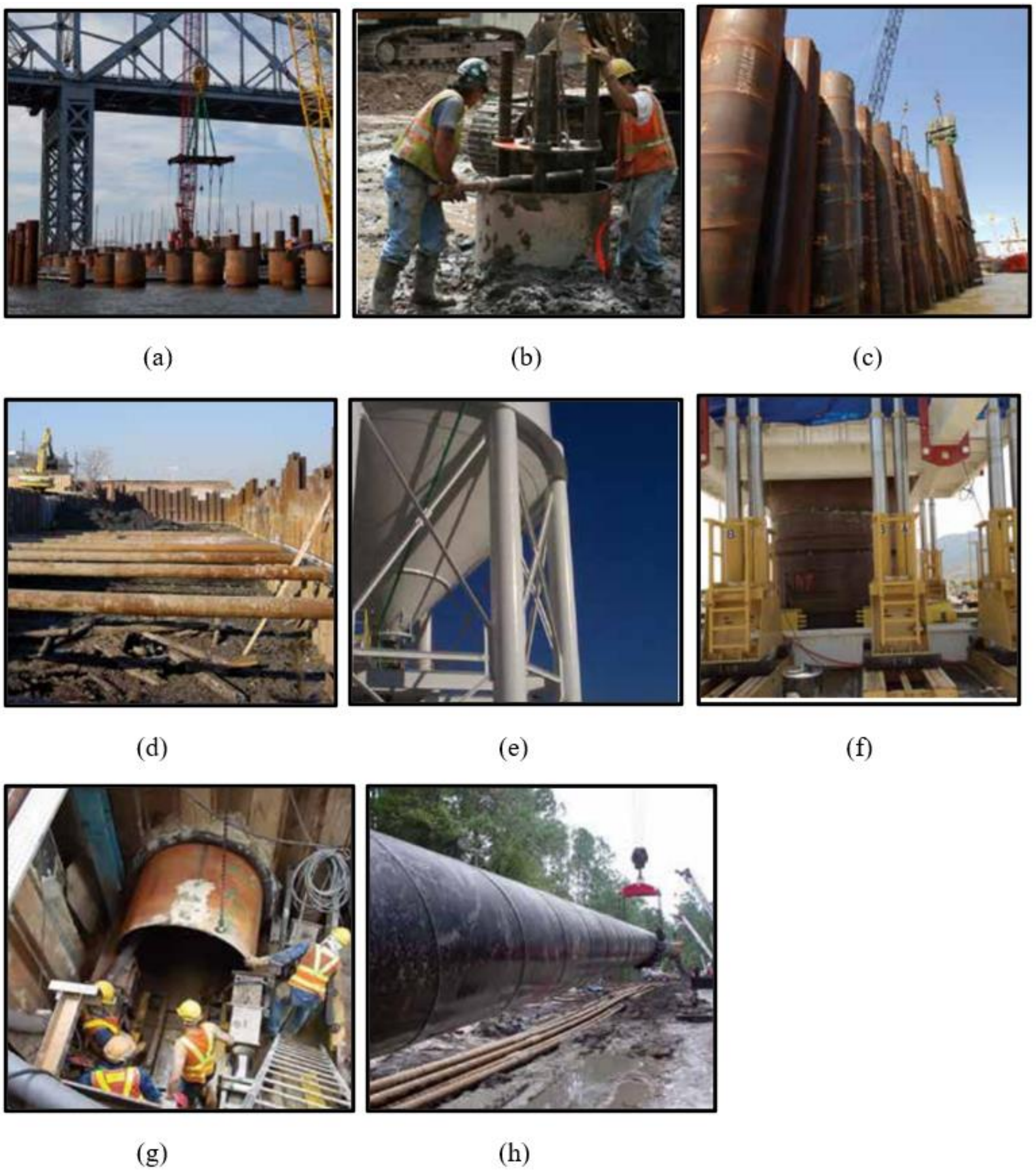


Figure 2-6 Uses of steel pipe; (a) Bearing piles; (b) Drilled shaft casing; (c) Combination walls; (d) Structural sections; (e) Sign poles, Towers & Transmission line; (f) Mining; (g) Jacked and Bored; and (h) Line pipe [from Nucor Skyline Catalogue]

2.2.3.5 Limitations of Steel Pipe Piles

Steel pipe piles have numerous advantages for foundation applications; however, they also come with certain drawbacks. One significant disadvantage is their susceptibility to corrosion, particularly in marine environments or areas with high moisture content. The corrosive effects can gradually erode the structural integrity of the piles, leading to potential issues over time and necessitating expensive maintenance or even replacement. Based on the Nucor Skyline catalogue, various corrosion rates have been provided for steel pipes. These rates are compiled and presented in Table 2-3, providing valuable information about the expected corrosion tendencies of steel pipes under different conditions and environments. This data can help in making decisions regarding the selection and application of steel pipe piles, considering the specific corrosion challenges they may face.

Table 2-3 Corrosion rate of steel pipe in various locations

Location of piles	5 years	25 years	50 years	75 years	100 years
	(in)				
Undisturbed natural soils	0	0.012	0.024	0.035	0.047
Polluted natural soils	0.006	0.03	0.059	0.089	0.118
Aggressive natural soils	0.008	0.039	0.069	0.098	0.128
Non-compacted and non-aggressive fills	0.007	0.028	0.047	0.067	0.087
Non-compacted and aggressive fills	0.02	0.079	0.128	0.177	0.226
Common fresh water	0.006	0.022	0.035	0.045	0.055
Very polluted fresh water	0.012	0.051	0.091	0.13	0.169
Sea water	0.022	0.074	0.148	0.22	0.295

Another drawback is the installation process. While HP-piles are more numerous, their smaller size makes installation easier. However, when dealing with large-diameter steel pipe piles, the installation process can become challenging due to the large size of the structures involved. The size difference affects various aspects, including the logistics involved in handling and transporting the piles, the specific equipment required for installation, and the overall complexity of the installation procedure.

These drawbacks related to steel pipe piles should be considered during the design and construction of bridge structures.

2.3 RESEARCH STUDIES ON PIPE PILES

Although the primary load case typically governing the design of pipe-to-cap connections is the lateral load and moment acting parallel to the roadway, one of the challenges in designing pile-to-cap connections is to prevent failure modes such as pipe pullout or punching shear under axial-load-dominated cases (Moon et al., 2013). Both failure modes are closely associated with pipe embedment depth or lack of other anchorage elements. Insufficient pipe embedment could lead to pullout failures, while inadequate base depth increases the risk of punching shear. Another challenge is the estimation of connection fixity in bridge foundations, typically characterized as "pinned" (allowing rotation with minimal moment transfer), "fixed" (preventing rotation and transferring moment alongside axial and lateral forces), or partially restrained with intermediate rotational stiffness. This characterization significantly impacts stress distribution modeling in bridge substructures. In a synthesis of guidelines from 24 U.S. state manuals, Zapata et al. (2022) found variability in pile-to-cap fixity definition, with three states specifying embedment depths (0.5–1 ft [152–305 mm]) for pinned connections, six states recommending embedments (1.0–4.0 ft [305–1219 mm]) for fixed connections, and the remaining 17 not clearly defining connection behavior. Given the impact of connection fixity on structural design, the lack of consistent fixity guidance highlights the need for experimental research.

Kingsley et al. (2005) investigated three steel pipe pile connections with annular end plates, finding that deep embedments (0.9D) achieved full composite action, while shallower embedments (0.6D) experienced footing damage but maintained acceptable performance when vertical reinforcement was present. In other studies (Stenlund, 2007; Rollins & Stenlund, 2010; Richards et al., 2011), four full-scale specimens with embedment depths of 0.5D, 1.0D, and 2.0D demonstrated that even shallow embedments develop significant moment capacity, suggesting that the standard "pinned" assumption is often unconservative. Additionally, these authors suggested that longer embedment lengths might eliminate the necessity for shear reinforcement in bent caps. Similarly, Eastman (2011) tested embedments of 0.4D, 0.5D, and 1.5D with end plates, confirming that shallow embedments provide substantial rotational stiffness.

Since prior research confirms that embedment depth is a primary driver of stiffness and capacity, assessing the practical viability of different connection hardware requires testing them at the specific embedment depths dictated by their respective design codes (e.g., deeper embedment for shear studs to accommodate spacing rules) rather than at an arbitrary constant depth.

Much of the existing literature on pile-to-bent cap connections has focused on concrete-filled steel tubes (CFTs), which integrate a continuous concrete core with a steel outer tube to provide composite action and improved ductility. Lehman and Roeder (2012) demonstrated that adequate embedment depth in CFTs significantly improves drift capacity and limits footing damage. However, hollow piles with concrete plugs introduce a stiffness discontinuity at the plug termination that continuous CFTs do not experience. While the recent literature reviews (Arockiasamy and Arvan, 2022; Arvan and Arockiasamy, 2023) analyzed a wide range of studies, concluding that while shallow embedments provide capacity, codes need revisions to recommend embedments of 1.2D to 1.5D.

Kappes et al. (2016) refined CFT design by identifying four failure modes: plastic hinging, concrete crushing, rebar yielding, and cap splitting. They found that dense transverse reinforcement was effective in preventing premature cap failure. Montejo et al. (2012) studied seismic behavior of RCFT pile-columns, finding that in-ground hinges remain elastic while the top hinge governs failure. Brown et al. (2015) showed that while the D/t ratio affects local buckling, it does not impact the ultimate limit state. Kenarangi and Bruneau (2020) highlighted that friction between the steel tube and concrete core enables composite action even with minimal interior reinforcement. Further studies on marine structures (Wan-Wendner et al., 2022; Belfroid, 2015) evaluated frictional interaction in piles with concrete plugs, highlighting the benefits of shear keys.

Few studies have directly compared welded and rebar configurations. Stephens et al. (2016b) evaluated embedded ring, welded dowel, and headed rebar connections for CFTs, finding that the embedded ring was the most efficient for structural performance and constructability.

Design guidelines provided by AISC (2022), ACI 318 (2019), and AASHTO LRFD (2020) place varying emphasis on steel, concrete, and composite systems. Roeder et al. (2010, 2014) compared these codes, finding that the plastic stress distribution method provided the most reliable moment capacity prediction, while ACI and AISC methods yielded scattered results. They proposed modified stiffness models (Roeder et al., 2014) and design equations for embedment and footing dimensions (Lehman and Roeder, 2012; Stephens et al., 2016a), which were later incorporated into the WSDOT Bridge Design Manual (WSDOT, 2024).

Collectively, prior studies establish that embedment depth and anchorage detailing significantly influence connection stiffness and moment transfer, with even shallow embedments providing greater fixity than idealized pinned assumptions suggest. The literature review highlights three critical gaps: (1) limited experimental data on hollow pipe pile connections compared to extensively studied CFTs; (2) a lack of experimental validation for 'standard implementation systems'—specifically, direct performance comparisons between code-compliant rebar connections (typically shallow embedment) and code-compliant mechanical anchorage systems (typically deep embedment); and (3) uncertain applicability of existing code provisions (developed for CFTs or composite beams) to hollow pipes with concrete plugs. Common DOT practices (e.g., headed rebars, hooked rebars, shear studs, and annular rings) have not been experimentally validated for hollow pipes. This study aims to address these gaps through full-scale testing of five representative connection typologies.

2.4 CURRENT DOT STATE OF PRACTICE FOR STEEL PIPE PILE TO BENT CAP CONNECTION DESIGNS

This section includes an overview of the current state of practice among state departments of transportation (DOT) regarding the use and experience of steel pipe piles for bridge structures. It provides a summary of a survey conducted to determine the prevailing practices and any variations in state standards and drawings related to the connection between pipe piles and reinforced concrete bent caps.

2.4.1 SURVEY OVERVIEW

A survey was conducted among state DOTs to investigate the utilization of steel pipe piles and their current design practices and methodologies. The main objective of the survey was to gather comprehensive data on the design and connection details used by different states for steel pipe piles connected to reinforced concrete bent caps, as well as to learn about their post-construction experiences with the use of steel pipe piles.

Bridge engineers from all states, except Alabama DOT, were contacted to gather their experiences and information regarding the use of steel pipe piles and the connection between the piles and bent caps. The bridge engineers were asked whether their respective states permitted the use of steel pipe piles. In states where steel pipe piles were used, additional questions were posed regarding the typical applications and sizes of commonly employed steel pipe piles. Furthermore, drawings, whether they were standard or specific to certain bridges, along with connection details and related design methodologies, were also collected from the bridge engineers.

Responses were obtained from engineers in 35 out of the 49 states that were contacted, while the remaining states did not respond. A summary of the responses received from each state is depicted in Figure 2-7. Among the contacted states, seven states indicated that they did not adopt steel pipe piles for bridge structures and preferred other types of piles such as HP piles and concrete piles. The remaining twenty-eight states acknowledged the use of steel pipe piles in their state. Among those, seven states had standard drawings and details for the connection, while the other twenty-one states designed the connection based on the specific uses and applications of each bridge. Detailed information gathered from the responses is presented in the next section.

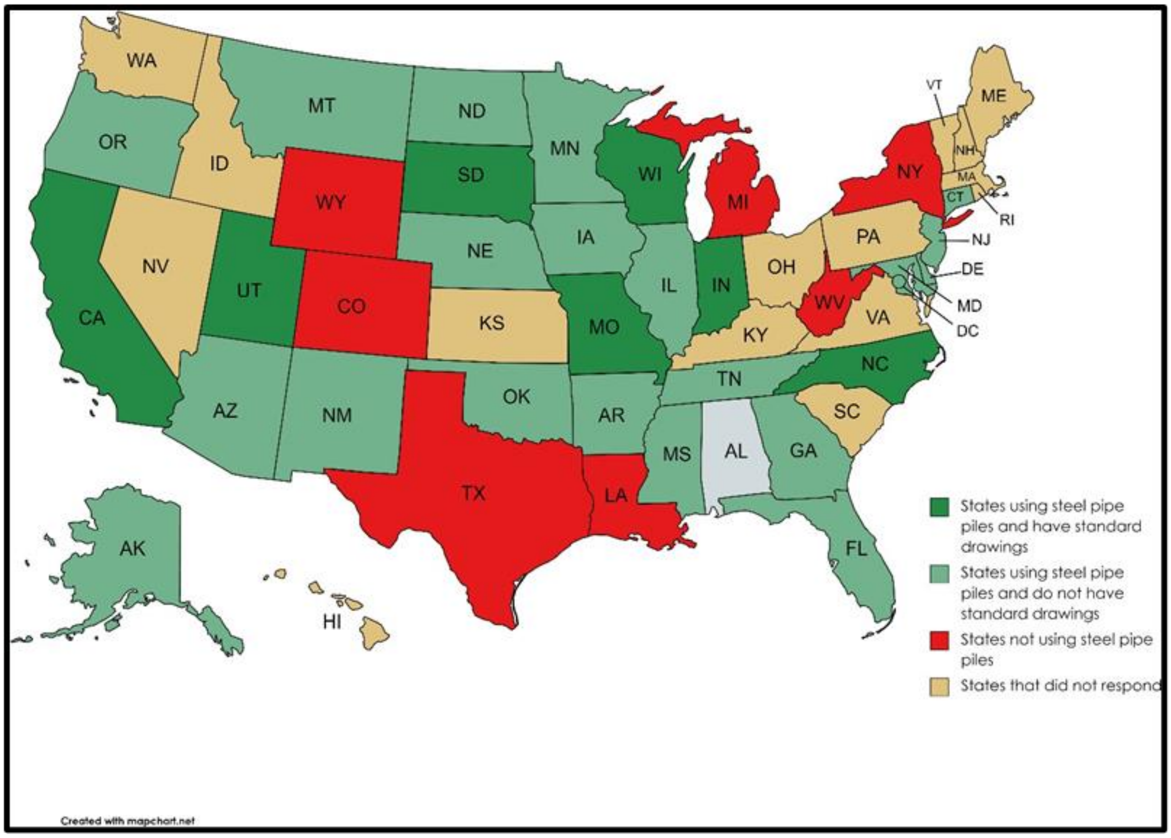


Figure 2-7 Map of response

2.4.2 SURVEY OUTCOMES: GENERAL OBSERVATIONS AND INFORMATION PROVIDED

Detailed design methodologies specifically used by state DOTs for the connection between steel pipe piles and reinforced concrete bent caps were not found. However, various states provided some information, standard drawings, and examples related to such connections. Table 2-4 below serves as a master table summarizing the collected information on steel pipe pile connections used by different states. It includes parameters such as pile diameter, pile thickness, type of anchorage, pile embedment into the bent cap, pile type (hollow steel pipe or concrete-filled), concrete plug depth (if applicable), and references to drawings (standard or implemented example bridges). The elaboration of these parameters used by different states is documented below.

Table 2-4 State DOT Survey Summary

State	Pile diameter (in)	Pile thickness (in)	Type of anchorage	Pile embedment	Pile type		Concrete plug	Implementation		Bridge Implementation Examples
					Steel Pipe	CFT		Standard Dwg or Specifications	Permitted by DOT	
California	20, 24, 25.75 20	0.25	Annular ring Welded dowel Headed rebars	18”, 20”, 20.5” 1 inch below cap -	- - -	✓ ✓ ✓	-	✓	-	-
Delaware	14-18	min 0.1875	180° Hooked rebars	min 1'	-	✓	-	-	✓	-
Florida	24	0.5	90° Hooked rebars	-	-	✓	-	-	✓	Hicks road over west Pittman creek
Georgia	24	0.25-0.50	Rebars	1'	-	✓	-	-	✓	Bridges over Canooche river Dry Creek
Illinois	12-16, 36	-	90°, Headed Rebars	2'	-	✓		-	✓	-
Minnesota	42	-	Straight Rebars Shear studs	- -	✓ ✓	-	10' below bent cap -	-	✓ ✓	Lafayette Bridge Hastings Bridge
Mississippi	24-36	0.5-0.75	V-Rebars	min 1'	✓	-	1'-6" below bent cap	-	✓	Bridges over Cassidy Bayou Wolf River I-59
Missouri	14-24	0.5	180° Hooked rebars Shear studs Annular ring	1'-6"	-	✓	-	✓ - -	- ✓ ✓	- Champ Clark bridge Rocheport Bridge
Montana	16	0.5	Rebars & U-bars	1’-8”	-	✓	-	-	✓	Bridge over Swan River
Nebraska	42	-	180° Hooked rebars	min 1'	-	✓	-	-	✓	Freemont southeast beltway
New Mexico	16, 24	0.5	Studs	min 1' & max 2'	-	✓	-	-	✓	Yaple Canyon Bridge replacement Rio Puerco
North Carolina	14, 16, 18, 24, 30	0.5	90° Rebars	-	✓	-	Min 5' on top of pile	✓	-	-
Oklahoma	24	0.625	Annular ring	2'-6"	-	✓	-	-	✓	Bridge over pine creek
South Dakota	16	0.25	90° Hooked rebars	2'-3"	-	✓	-	✓	-	-
Tennessee	16	0.5	180° Hooked rebars	min 1'-6"	-	✓	-	-	✓	Over Norfolk southern railroad station
Utah	min 12.75	-	90° Rebars	2'	-	✓	-	✓	-	-
Washington	30,36	1	Headed rebars	3'6"-5'	✓	-	12'3"-14'	-	✓	Seatle Trestle Bainbridge island
Wisconsin	12.75, 14	0.375	Rebars	2'	-	✓	-	✓	-	-

2.4.3 SURVEY OUTCOMES: STEEL PIPE PILE TO BENT CAP CONNECTION DETAILS OF DOTS

Information regarding connection details and related drawings was collected from eighteen state DOTs that indicated the adoption of steel pipe piles. This section presents the responses received, which include the usage of steel pipe piles, sizes and types of commonly used steel pipe piles, design considerations for the connection, and associated drawings.

2.4.3.1 California (Caltrans)

Use of steel pipe piles

Steel piles are commonly used in accelerated bridge construction projects. Studies and research were conducted at the University of Washington (D. E. Lehman et al., 2015) in collaboration with Caltrans. These studies focused on the connection between concrete-filled steel tubes and precast reinforced concrete bent caps.

Size and types of commonly used pipes

Piles of various diameters, ranging from 20 in. to 25.75 in., were utilized in the conducted studies. These piles had a thickness of 0.25 in. Typically, these piles were filled with concrete to enhance their performance, especially for enhanced seismic design.

Connection design/Detailing consideration

Three types of anchorage were utilized in the study: i) Annular ring: An anchorage ring was welded on the top of the pile, and the pile was embedded 18 in. to 20 in. into the bent cap. ii) Welded dowel: Rebars were welded on the inside circumference of the pile, with an embedding depth of 12 times the diameter of the rebar (12db), into the bent cap. The pile itself was not embedded inside the bent cap. (iii) Headed rebars: A reinforcement cage was provided in the concrete core of the piles, extending 12 times the diameter of the rebar (12db) into the bent cap and 30 times the diameter of the rebar (30db) into the column.

Connection Drawing

Figure 2-8, Figure 2-9, and Figure 2-10 illustrate the adopted details for the specimens of the annular ring, welded dowel, and headed rebars connections, respectively. These details were successfully employed in the experimental programs and yielded satisfactory results.

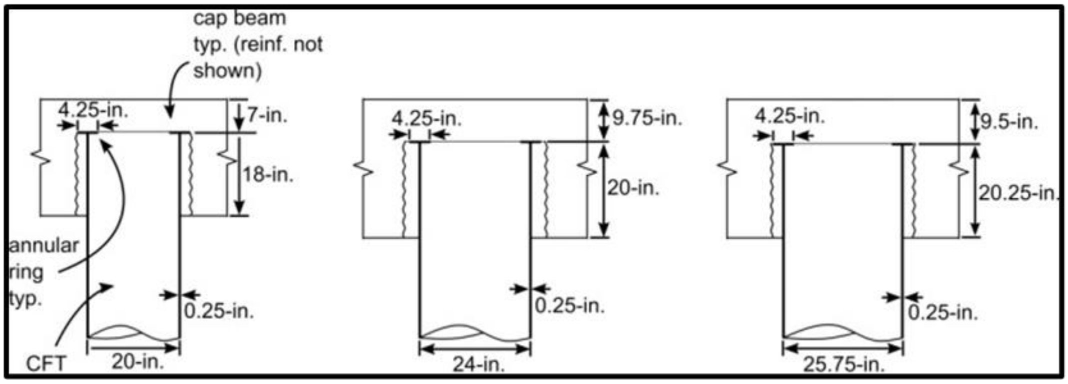


Figure 2-8 Caltrans: Annular ring detail for pile to bent cap connection with annular ring

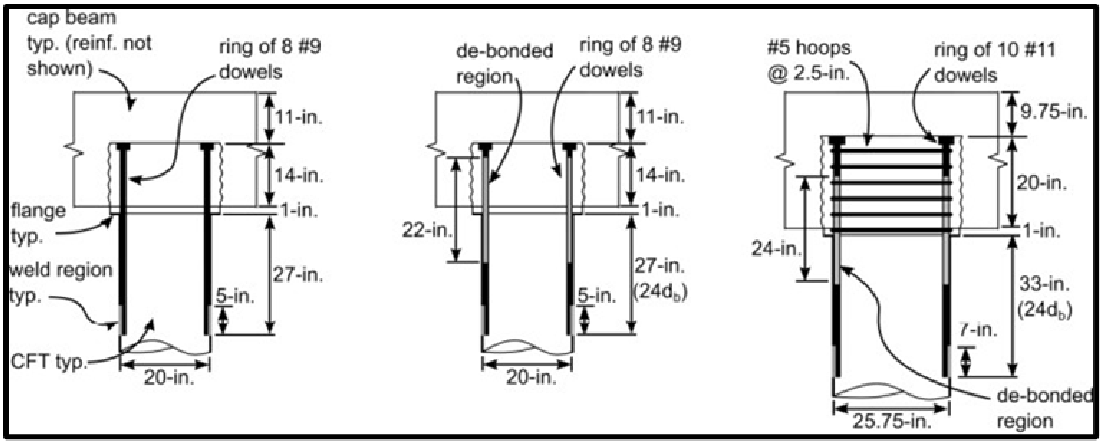


Figure 2-9 Caltrans: Annular ring detail for pile to bent cap connection with welded dowels

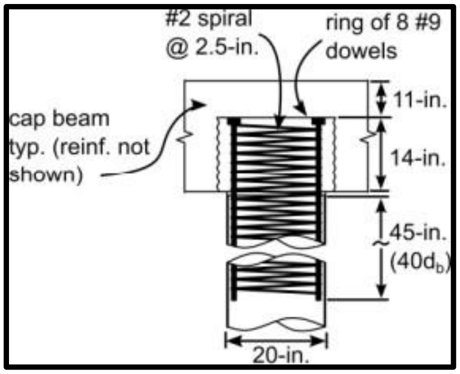


Figure 2-10 Caltrans: Annular ring detail for pile to bent cap connection with headed bars

2.4.3.2 Delaware (DelDOT)

Use of steel pipe piles

Cast-in-place steel piles, which are essentially filled with concrete on-site, are commonly used by DelDOT.

Size and types of commonly used pipes

DelDOT predominantly utilizes small-diameter pipe piles and fluted pipe piles, ranging from 14 in. to 18 in. in diameter, with a minimum thickness of 0.1875 in. for bridge piers. These piles are typically filled with concrete to enhance their strength.

Connection design/Detailing consideration

The connection between the pile and cap is established using a combination of 180° hooked rebars and straight rebars. To ensure a strong connection, a minimum of 1 ft. of pile embedment is required for the hooked bars. In the case of larger diameter steel piles, deeper bent caps can be employed, allowing for greater embedment of reinforcement into the cap and eliminating the need for hooked bars.

Example bridge

DelDOT does not have standard drawings for these details; however, they provide example details in their Bridge Design Manual, as depicted in Figure 2-11.

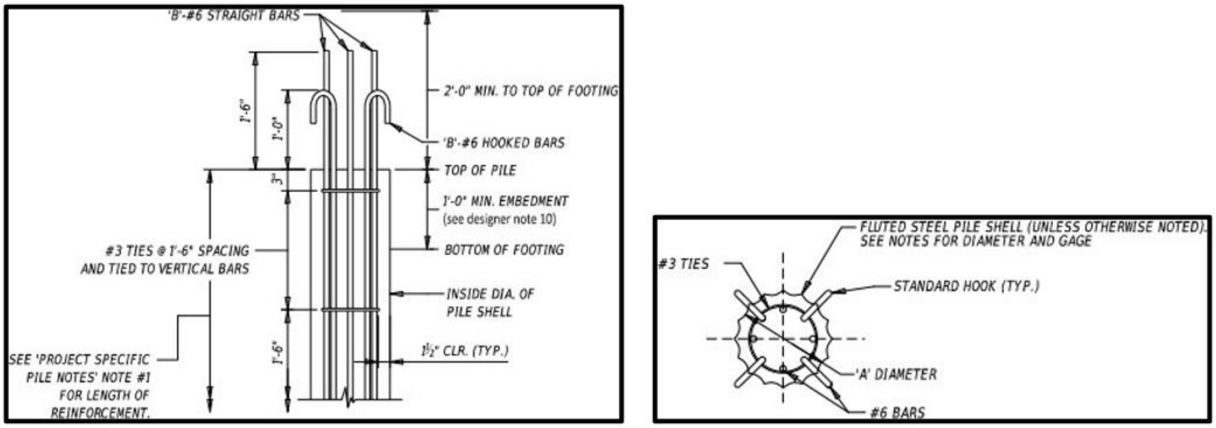


Figure 2-11 DelDOT: Rebar detail for pile to bent cap connection

2.4.3.3 Florida (FDOT)

Use of steel pipe piles

According to FDOT, the state primarily utilizes closed-end steel pipe piles that are filled with concrete.

Size and types of commonly used pipes

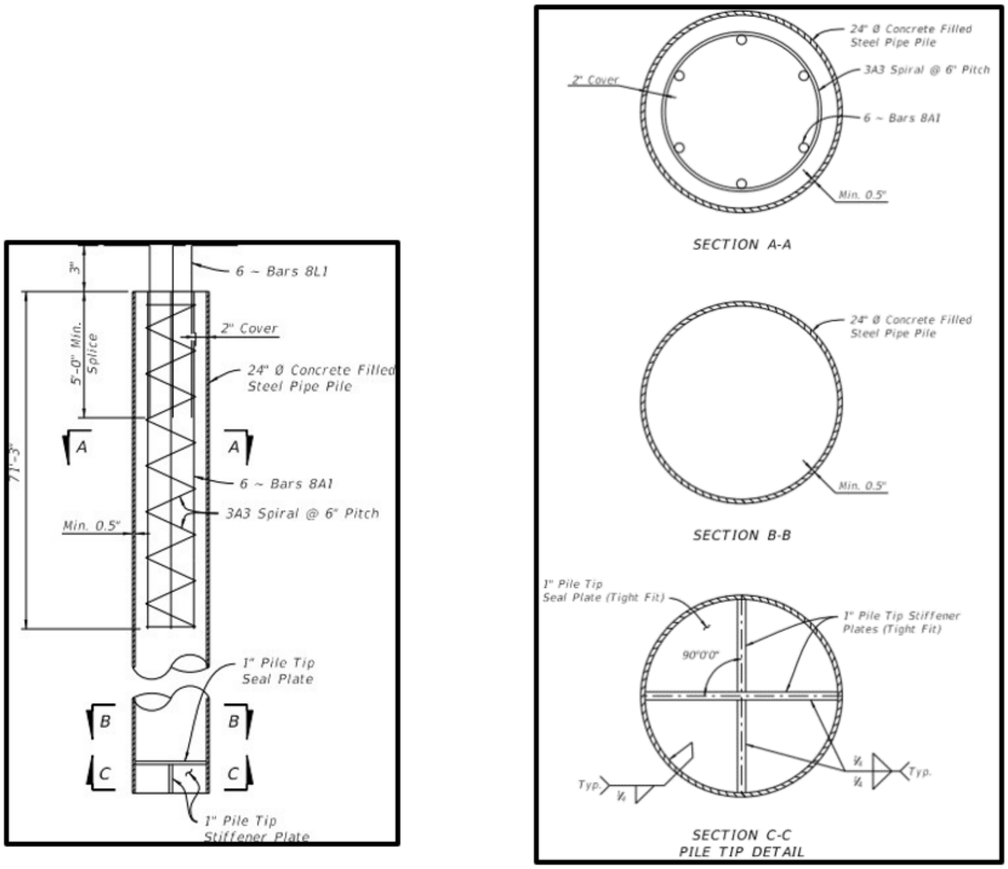
The typical size of the piles used by FDOT is 24 in. in diameter and 0.5 in. in thickness. These piles are constructed with a cast-in-place concrete core that includes reinforcement details for permanent applications.

Connection design/Detailing consideration

The connection between the pile and bent cap is achieved by extending and hooking the reinforcing steel into the bent cap, making the connection easier. These rebars are also properly confined with spiral reinforcement to ensure their effectiveness. Additionally, an end stiffener plate is installed at the end of the pile to increase rigidity and enhance the structural integrity.

Example bridge

The details of the desired connection for Hicks Road over West Pittman Creek are shown in Figure 2-12.



(a)Sectional elevation view

(b) Sectional pile plan view

Figure 2-12 FDOT: Rebar detail for pile to bent cap connection, Hicks Road over west Pittman creek (Bridge No. 524219)

2.4.3.4 Georgia (GDOT)

Use of steel pipe pile

GDOT frequently utilizes steel pipe piles, commonly referred to as metal shell piles.

Size and types of commonly used pipes

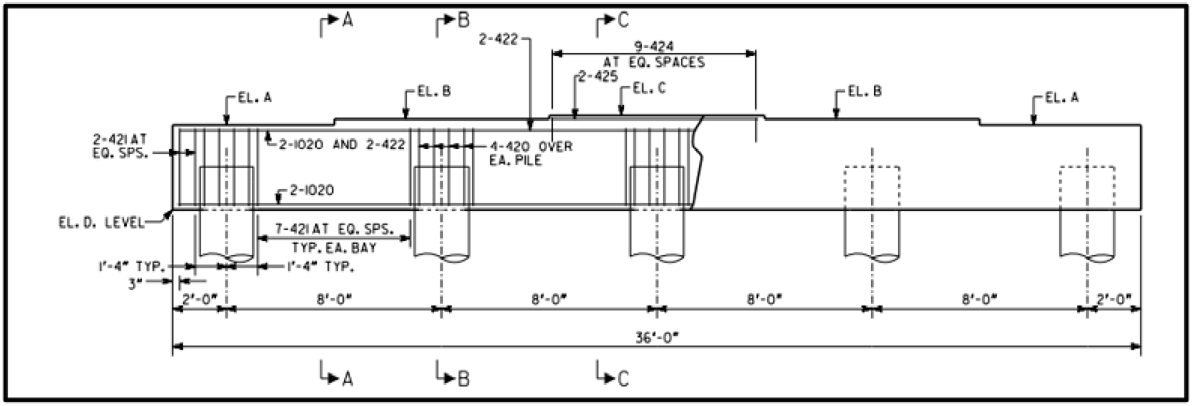
The largest diameter of metal shell piles used by GDOT is 2 ft., with thickness ranging from 0.25 in. to 0.5 in. These piles are filled with a concrete core to prevent pile failure.

Connection design/Detailing consideration

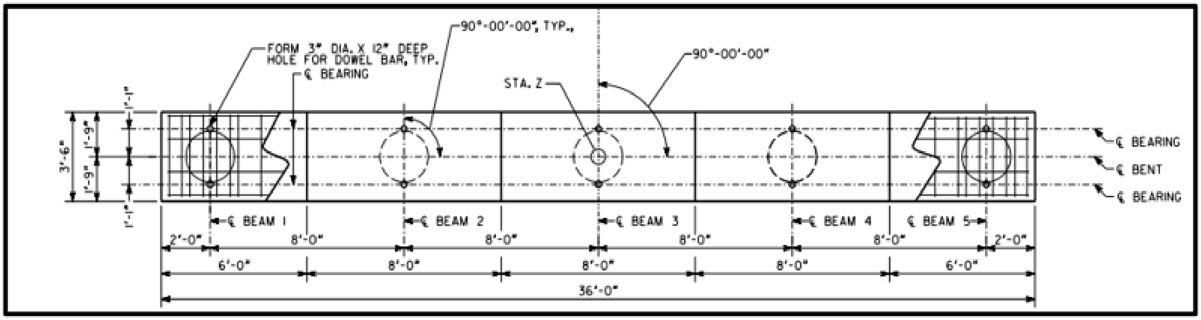
GDOT does not have standard details for the connections of metal shell piles, but they have certain considerations: i) The current practice is to embed the pile a minimum of 1 ft into the cap, ii) Placing a pile under the beam helps reduce moments, iii) Span lengths are typically limited to approximately 60 feet, iv) The fixity of the pile is maintained at a depth of 10 ft into the ground, the braced length, v) Piles exceeding a height of approximately 25 ft, including the braced length, tend to fail or plot outside the limits of the interaction diagram, vi) Lateral load is distributed based on tributary lengths and the stiffness of the bent, vii) A K (Effective length) value of 1.2 is used in the transverse direction, and a value of 2.1 is used in the longitudinal direction, viii) Rebar is not placed in the metal shell pile that supports an end bent, and ix) The metal shell itself does not contribute to the structural capacity.

Example bridge

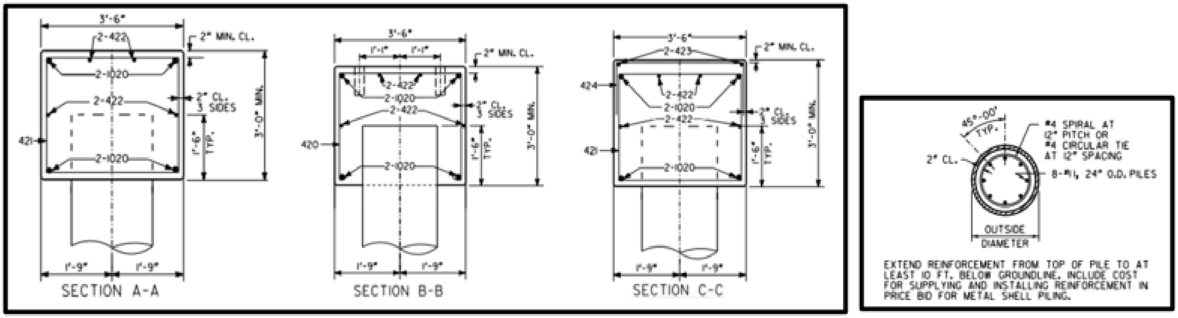
GDOT does not have standard drawing details, but the connection details of the bridge over the Canoochee River are shown in Figure 2-13.



(a) Elevation view



(b) Plan view



(c) Sectional elevation view (d) Pile plan view

Figure 2-13 GDOT: Rebar detail for pile to bent cap connection, Canooche River

2.4.3.5 Illinois (IDOT)

Use of steel pipe pile

IDOT commonly utilizes steel pipe piles, also referred to as metal shell piles, for various bridge structures.

Size and types of commonly used pipes

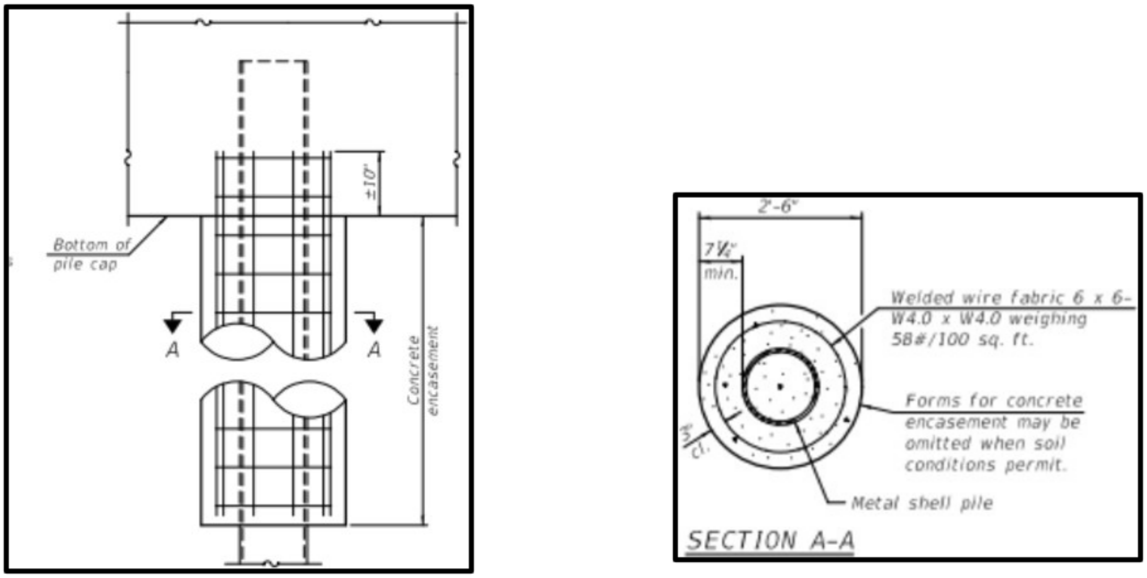
Different diameter sizes, such as 12 in., 16 in., and 36 in., are used for the piles by IDOT. These piles are filled with a concrete core and reinforced with a reinforcement cage to enhance their strength.

Connection design/Detailing consideration

For the connection between the pile and cap, the standard practice for IDOT is to embed the pile 24 in. into the cap. This detail applies to large diameter pipe piles (36 in. and greater) as well. In the case of a cast-in-place pile cap, hooked reinforcement is utilized to extend into the cap. For a precast pile cap, the same connection details are used, but bar terminators are employed instead of hooks to minimize the size of the grout socket.

Example bridge

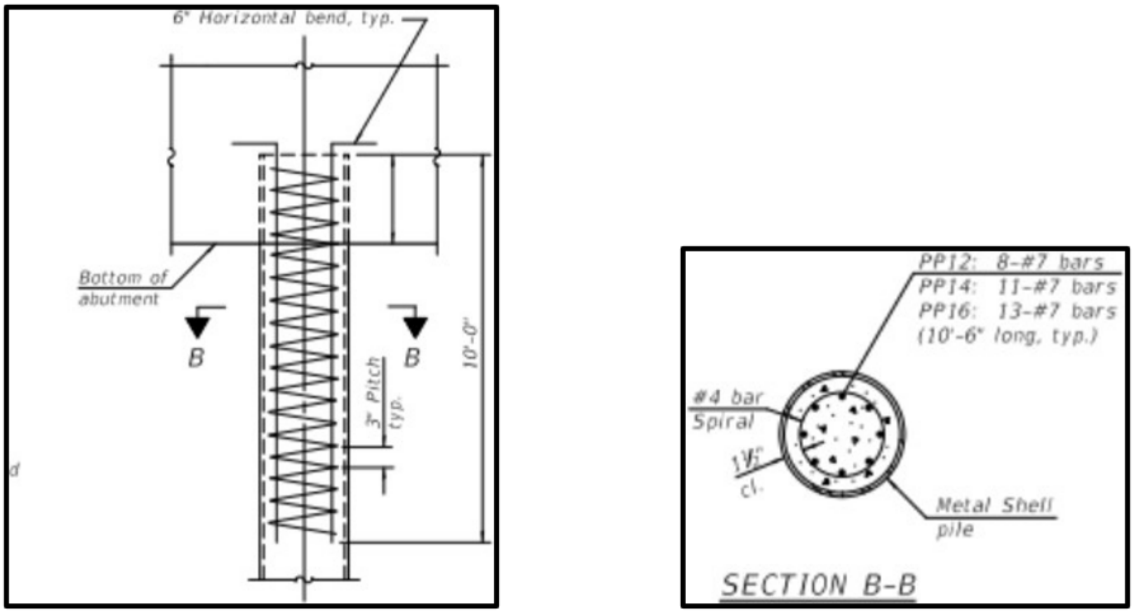
For metal shell piles, specifically those with encasement (12 in. – 16 in. in diameter, spiral-wound pipe), the connection detail is depicted in Figure 2-14. On the other hand, for metal shell piles without encasements, the corresponding detail can be found in Figure 2-15.



(a)Sectional elevation view

(b) Pile plan view

Figure 2-14 IDOT: Rebar detail for pile to bent cap connection with concrete encasement



(a) Sectional elevation view (b) Pile plan view

Figure 2-15 IDOT: Rebar detail for pile to bent cap connection without concrete encasement

2.4.3.6 Minnesota (MnDOT)

Use of steel pipe pile

According to the 2015 NCHRP report [X], MnDOT commonly utilizes steel pipe piles with a foundation at the bottom. These piles are not directly driven into the soil.

Size and types of commonly used pipes

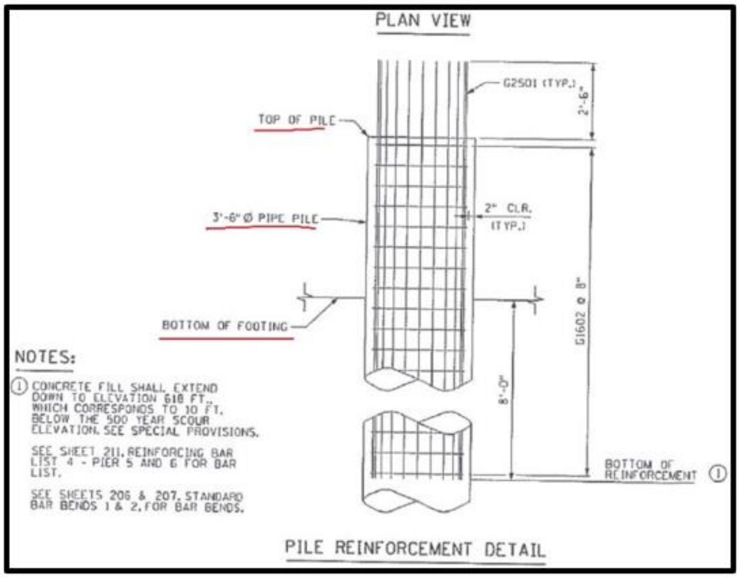
Large diameter piles, such as 42 in., are used in these cases. Due to the large diameter, a concrete core is not provided throughout the entire length of the pile.

Connection design/Detailing consideration

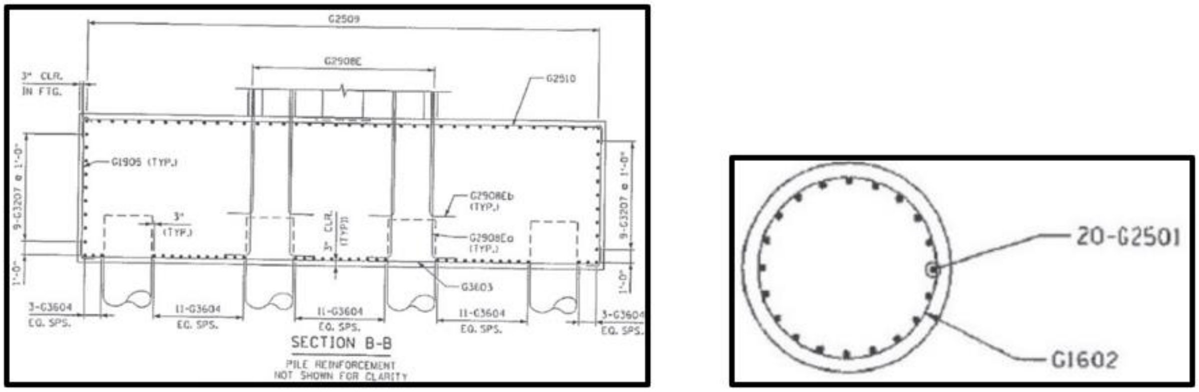
To ensure a strong connection, a concrete plug of approximately 10 ft. is placed below the bent cap. The piles are embedded within the bent cap, and two types of anchorage are used for the connection. Firstly, the longitudinal reinforcement of the piles extends into the pile cap, providing additional strength. Secondly, shear studs are welded to the steel pipe itself, enhancing the structural strength of the steel pipe at the connection point.

Example bridge

Figure 2-16 illustrates the connection details used in the Lafayette Bridge of Minnesota, which involves rebars for the connection. On the other hand, Figure 2-17 presents the connection details adopted by the Hastings Bridge, which utilizes shear studs for the connection.

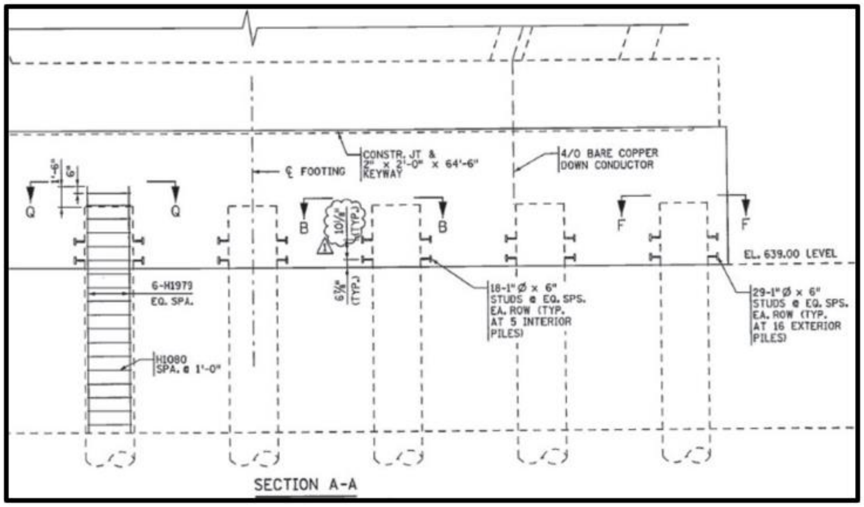


(a) Sectional elevation view

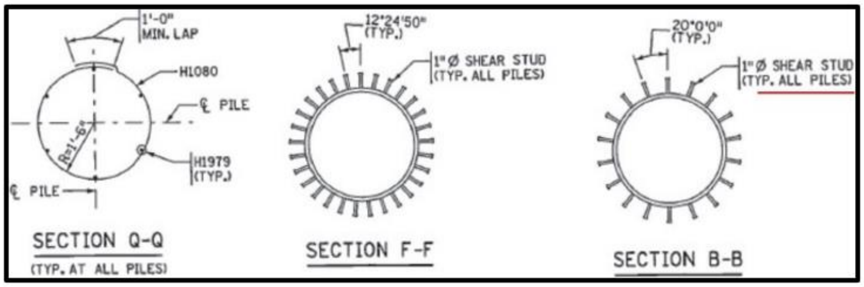


(b) Sectional elevation view (c) Pile plan view

Figure 2-16 MnDOT: Rebar detail for pile to bent cap connection, Lafayette Bridge



(a) Sectional elevation view



(b) Sectional pile plan view

Figure 2-17 MnDOT: Shear studs detail for pile to bent cap connection, Hastings Bridge

2.4.3.7 Mississippi (MDOT)

Use of steel pipe pile

According to MDOT, steel pipe piles are commonly utilized in bridge structures.

Size and types of commonly used pipes

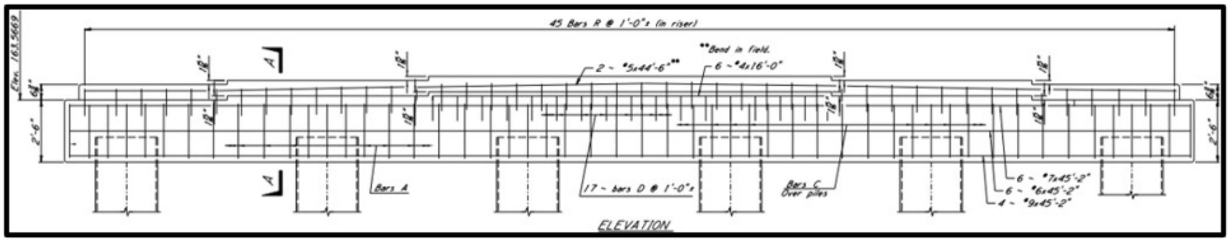
Piles with diameters ranging from 24 in. to 36 in. are used, with a thickness of 0.5 in. to 0.75 in. Due to their larger size, these piles do not have a concrete core.

Connection design/Detailing consideration

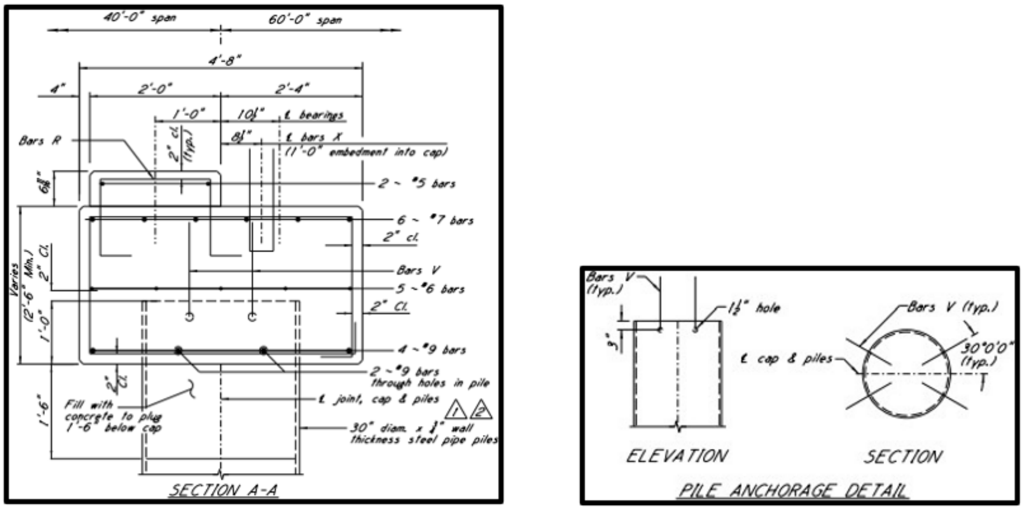
MDOT does not have standard design methodologies but has certain considerations. A minimum embedment of 1 ft. is provided for steel pipe piles inside the concrete bent cap. Additionally, through holes are provided in the piles to ensure that the embedment does not affect the layout of reinforcements in the bent cap. To enhance the strength of the connection between the bent cap and pile, a concrete plug of 1 ft. 6 in. is provided above and below the bent cap inside the steel pipe piles.

Example bridge

Figure 2-18 below depicts the details of the connection between the steel pipe piles and the concrete bent cap of the bridge over Cassidy Bayou. Figure 2-19 also illustrates the most commonly used details for the concrete plug.



(a) Elevation view



(b) Sectional elevation view (c) Connection detail

Figure 2-18 MDOT-Rebar detail for pile to bent cap connection, Cassidy Bayou

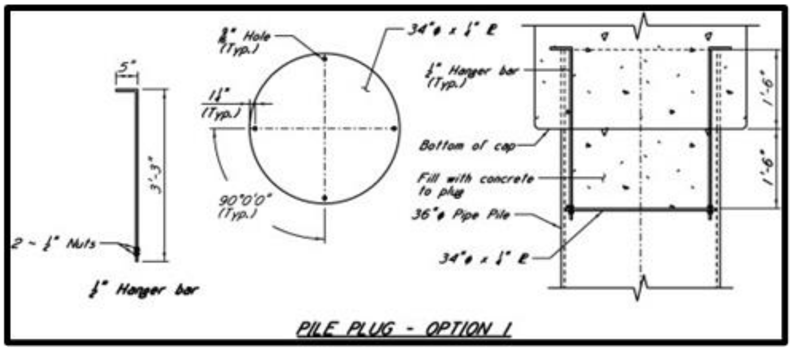


Figure 2-19 MDOT-Concrete plug detail, Wolf River

2.4.3.8 Missouri (MoDOT)

Use of steel pipe pile

Steel pipe piles, commonly referred to as Cast-In-Place (CIP) Piles, are utilized by MoDOT for intermediate bents in bridge construction.

Size and types of commonly used pipes

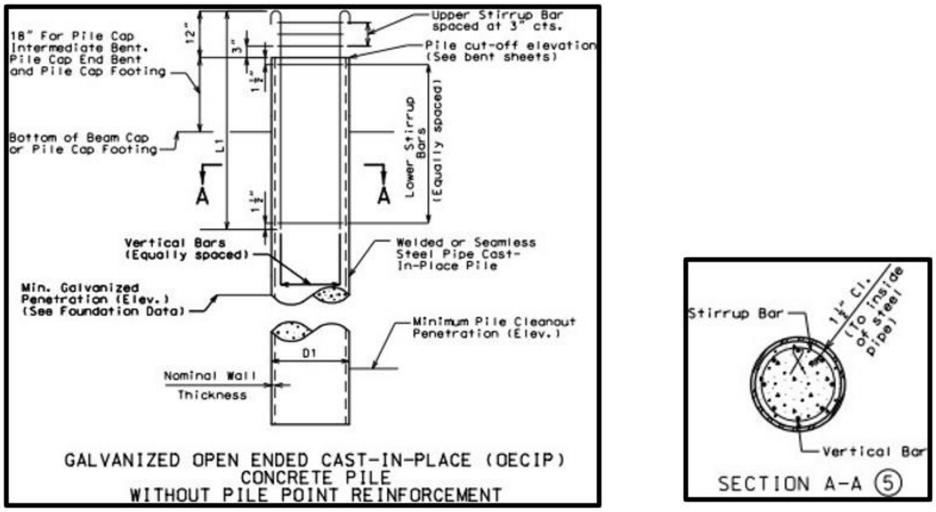
Missouri utilizes steel pipe piles with diameters ranging from 14 in. to 24 in. and thicknesses of 0.5 in. These steel shells are driven into the ground, either with a closed end or open end, and subsequently filled with concrete.

Connection design/Detailing consideration

The steel pipe piles are embedded within the reinforced concrete bent cap. Various anchorages, including hooked rebars, annular rings, and shear studs, are employed for the connection. To ensure a robust connection, a minimum embedment of 18 in. for the steel piles is required.

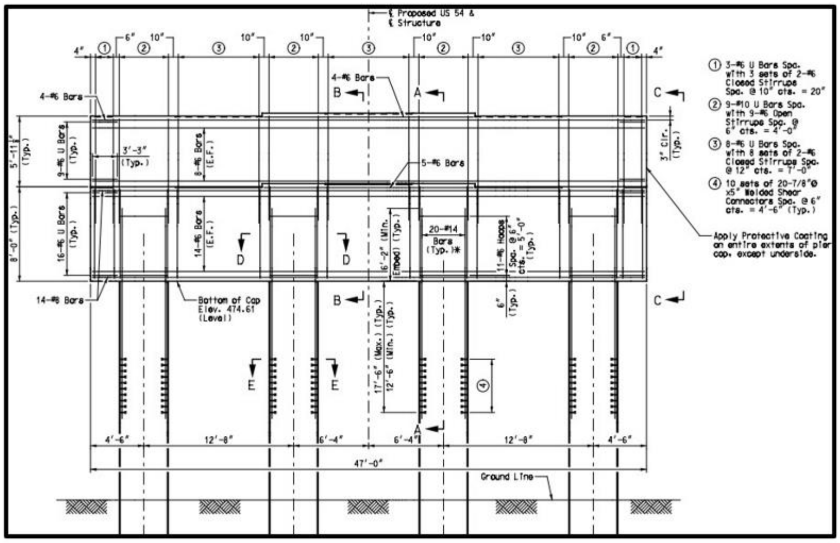
Standard/Example bridge

The standard drawings for the connection, as provided by the MoDOT, are depicted in Figure 2-20. Additionally, Figure 2-21 illustrates the specific details of the connection used in the construction of the Champ Clark bridge in Missouri. For the connection, shear connectors were welded inside the steel pipe piles. The details of the Rocheport bridge, constructed in Missouri, are presented in Figure 2-22. An annular ring was welded at the top end of the embedded steel pipe piles.

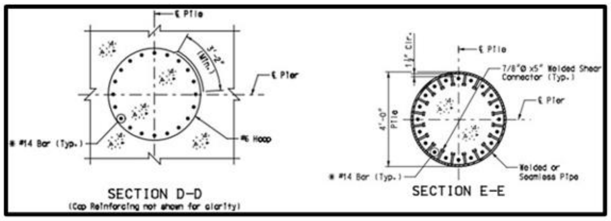


(a)Sectional elevation view (b) Sectional Pile plan view

Figure 2-20 MoDOT: Rebar detail for pile to bent cap connection (Bridge No. PILE02)

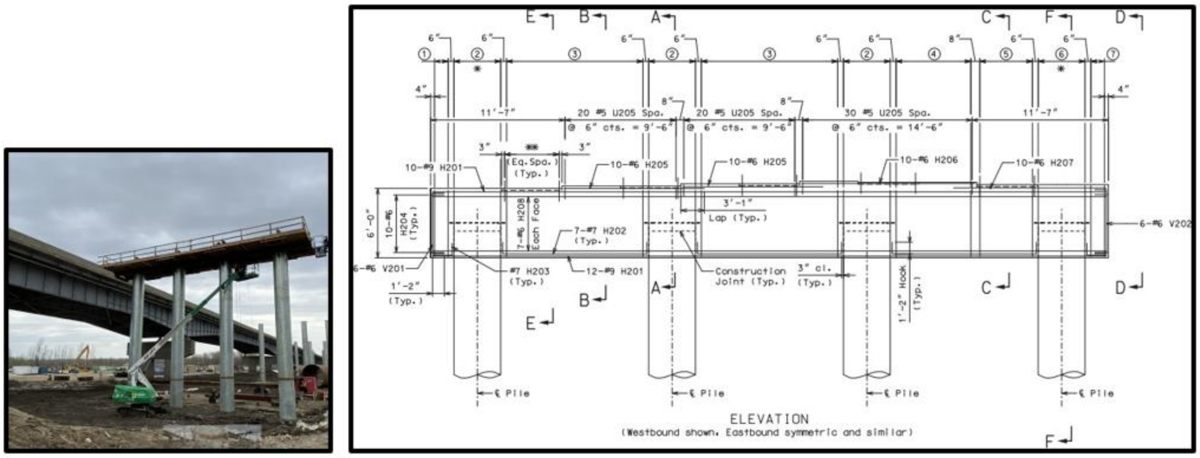


(a) Sectional elevation view



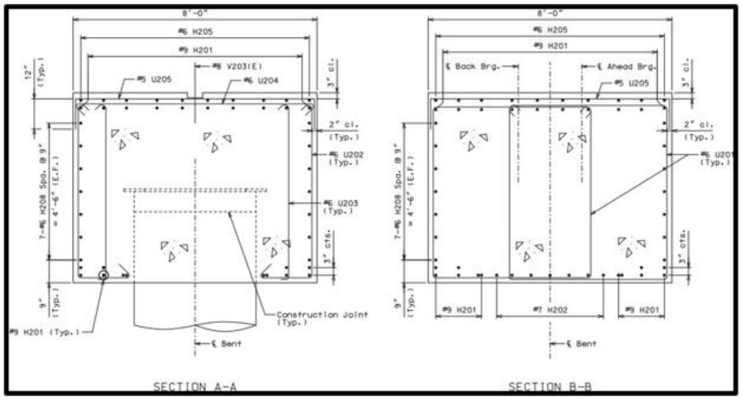
(b) Sectional Pile plan view

Figure 2-21 MoDOT: Shear studs detail for pile to bent cap connection, Champ Clark bridge



(a) Site view

(b)Elevation view



(c) Sectional elevation view

Figure 2-22 MoDOT: Annular ring detail for pile to bent cap connection, Rocheport Bridge

2.4.3.9 Montana (MDT)

Use of steel pipe pile

Steel pipe piles are commonly used by MDT. They are known to be simple, cost-effective, and relatively easy to construct, making them suitable for many situations.

Size and types of commonly used pipes

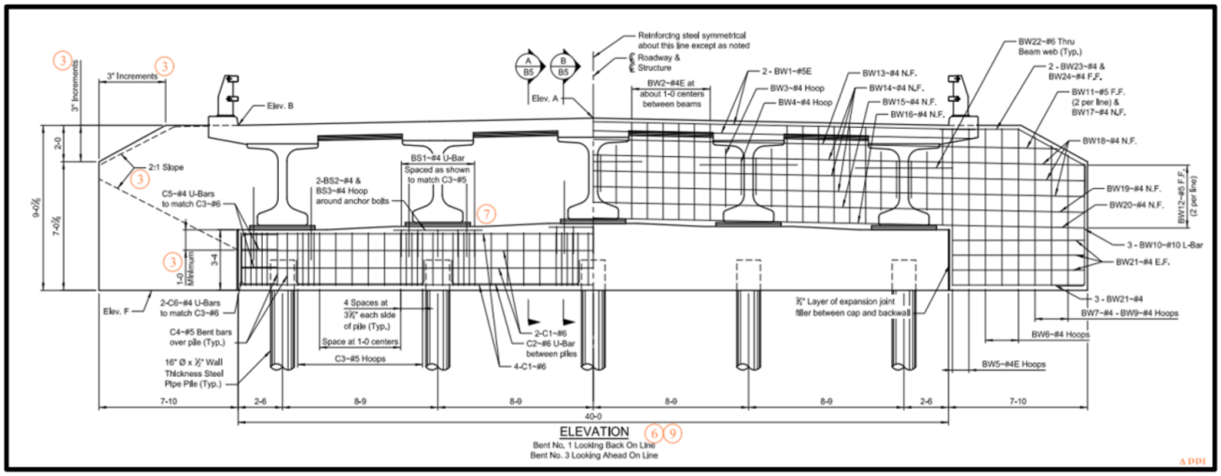
Piles with a small diameter of 16 in. and a thickness of 0.5 in. are used by MDT. These piles are filled with concrete to enhance their strength.

Connection design/Detailing consideration

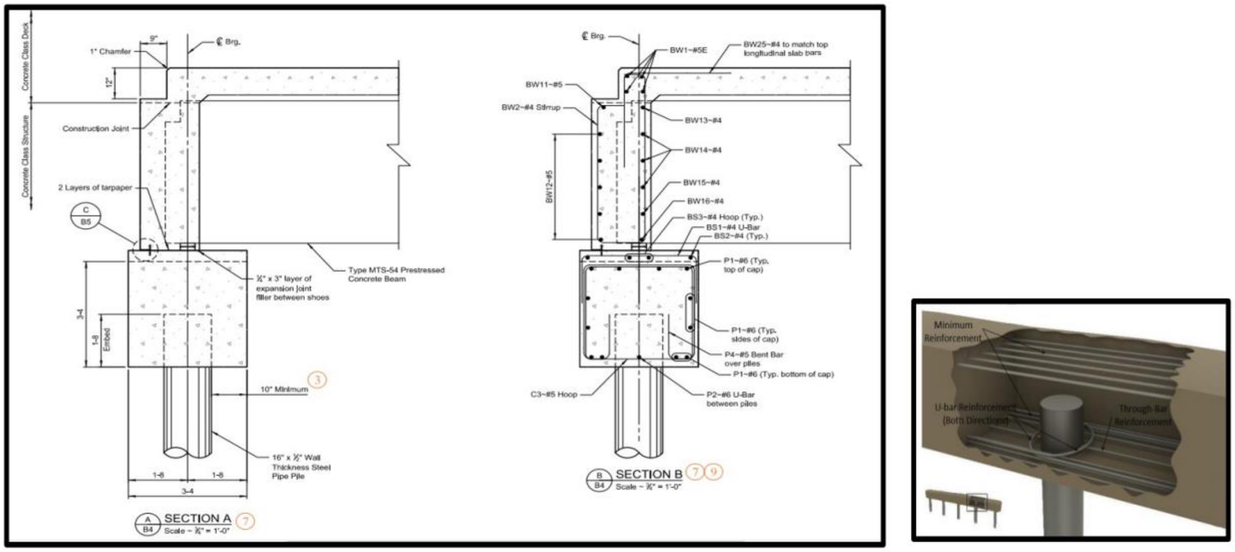
The piles are embedded approximately 1 ft. 8 in. into the cap. To create the anchorage, special U-bars are installed in the bent cap around the pile.

Example bridge

The detailed connection used on the Swan River bridge is presented in Figure 2-23.



(a) Elevation view



(b) Sectional elevation view

(c) Reinforcement details

Figure 2-23 MDT: Rebar detail for pile to bent cap connection, Swan River

2.4.3.10 Nebraska (NDOT)

Use of steel pipe pile

Steel pipe piles are utilized by NDOT when soil displacement bearing is required. In similar cases, HP-piles are also treated as substitutes. These piles are not directly driven into the soil; instead, a foundation is provided at the bottom of the pile.

Size and types of commonly used pipes

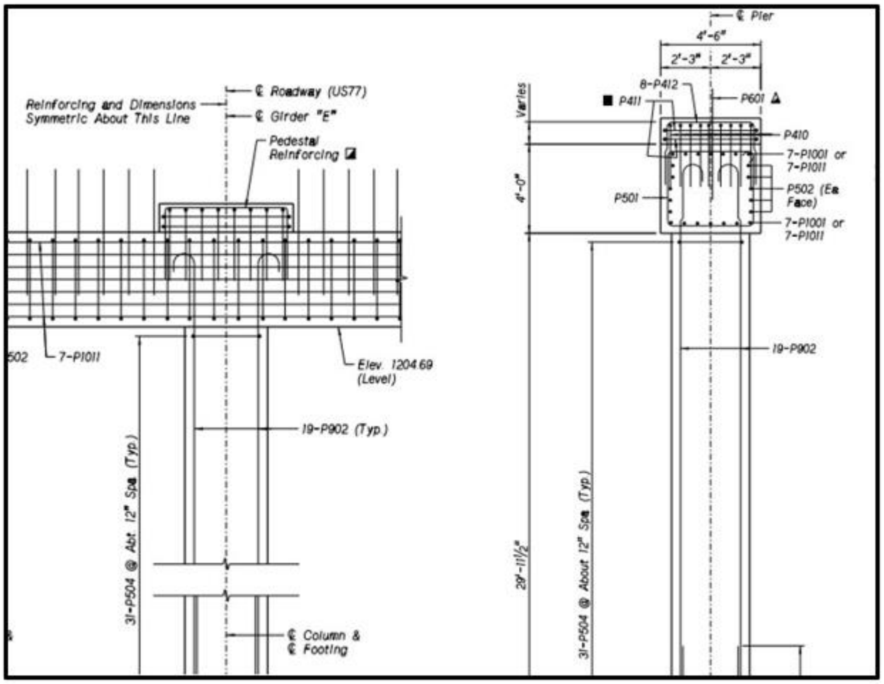
Steel pipe piles with a large diameter of 42 in. are used, and they are equipped with a concrete core and reinforcements.

Connection design/Detailing consideration

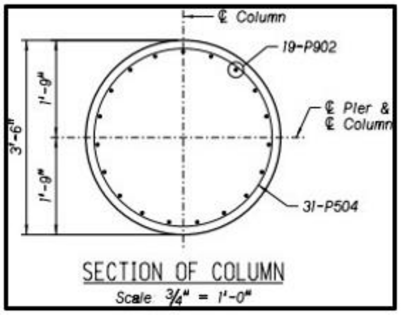
For the required connection, a minimum of 1 ft. of embedment is provided. The longitudinal rebars, which are hooked inside the cap, are placed within the concrete core of the piles.

Example bridge

Nebraska does not have specific connection details; however, Figure 2-24 provides an example of the connection details used at Fremont Southeast Beltway.



(a) Sectional elevation view



(b) Pile plan view

Figure 2-24 NDOT: Rebar detail for pile to bent cap connection, Freemont southeast beltway

2.4.3.11 New Mexico (NMDOT)

Use of steel pipe pile

NMDOT has a long history of frequently using and continues to utilize steel pipe piles for new bridge construction.

Size and types of commonly used pipes

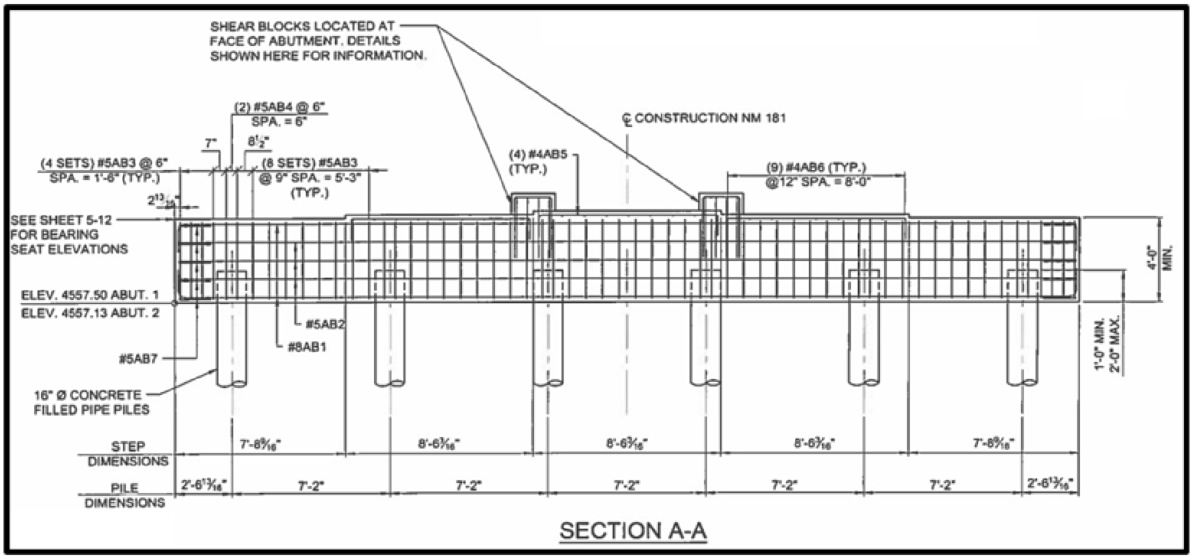
In New Mexico, steel pipe piles of various diameters, such as 16 in. and 24 in., with a thickness of 0.5 in., are commonly used. These piles are filled with concrete for added strength and stability.

Connection design/Detailing consideration

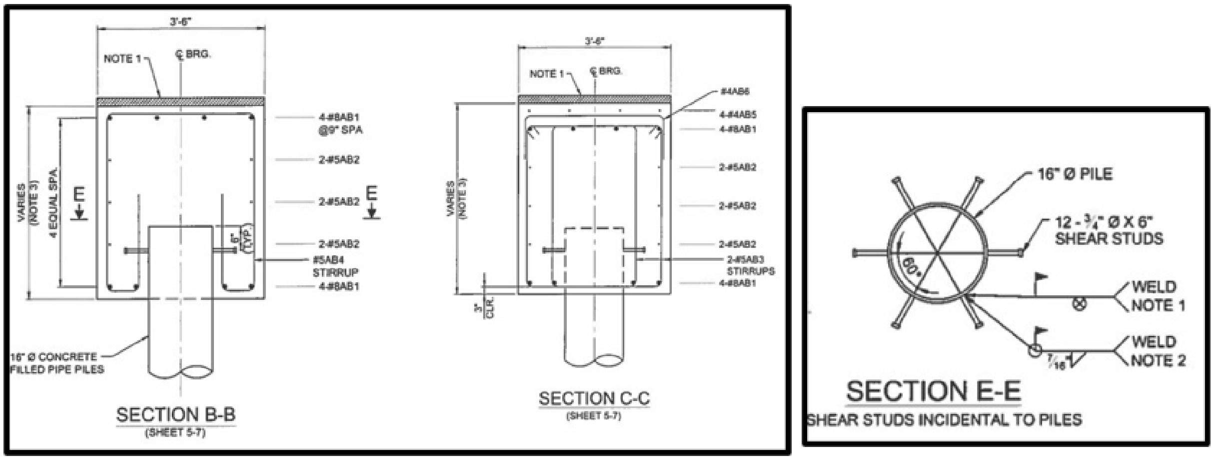
For the connection, a minimum of 1 ft. and a maximum of 2 ft. of the pile are embedded inside the bent cap. Shear connectors are welded to the outer wall of the pile, which is embedded within the cap. This allows for the transfer of force and moment between the pile and the cap.

Example bridge

The detailed connection used in the replacement of Yaple Canyon Bridge is shown in Figure 2-25.



a) Sectional elevation view



(b) Sectional elevation view (c) Pile plan view

Figure 2-25 NMDOT- Rebar detail for pile to bent cap connection, Yaple Canyon Bridge

2.4.3.12 North Carolina (NCDOT)

Use of steel pipe pile

According to NCDOT, steel pipe piles are commonly used in bridge structures.

Size and types of commonly used pipes

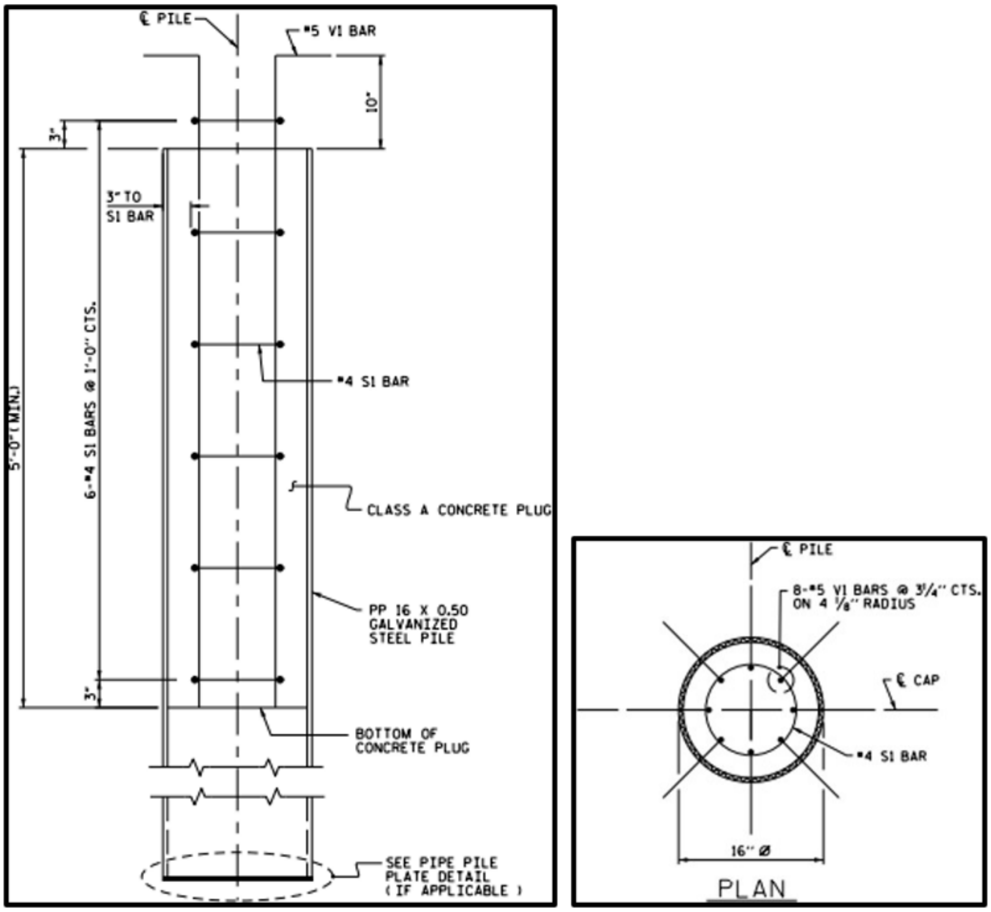
NCDOT utilizes steel pipe piles of various diameters, ranging from 14 in. to 30 in., with a thickness of 0.5 in. These piles, being of larger diameter, are hollow and do not have a concrete core.

Connection design/Detailing consideration

The pile is filled with concrete up to a minimum height of 5 ft. A concrete plug with a reinforcement cage is installed inside the pile, which facilitates the attachment with the bent cap. To minimize the required development length of the rebars inside the cap, they are hooked at a 900 angle. Additionally, transverse ties are incorporated to enhance the rebar confinement and overall strength.

Standard bridge

The standard drawings for steel pipe piles are presented in Figure 2-26.



(a) Sectional elevation view (b) Pile plan view

Figure 2-26 NCDOT- Rebar detail for pile to bent cap connection (STD. NO. SPP2)

2.4.3.13 Oklahoma (ODOT)

Use of steel pipe pile

ODOT has utilized steel pipe piles at only one location, which is a couple of years old but remains in good condition. However, ODOT does not commonly employ such piles and lacks standardized details for their use.

Size and types of commonly used pipes

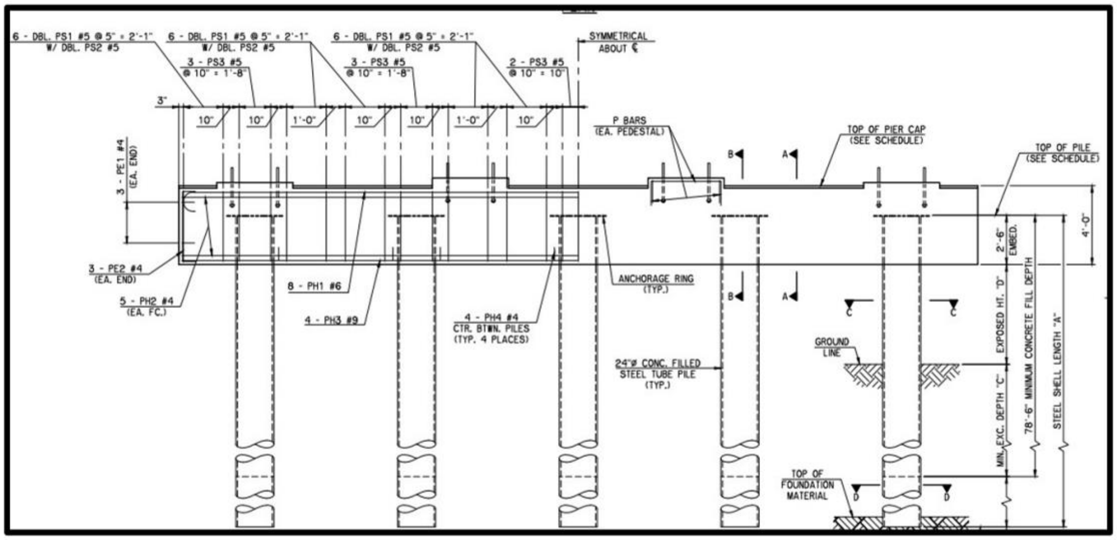
The commonly used size of steel pipe piles is 24 in., with a thickness of 0.625 in. Additionally, these piles are filled with concrete throughout their entire length.

Connection design/Detailing consideration

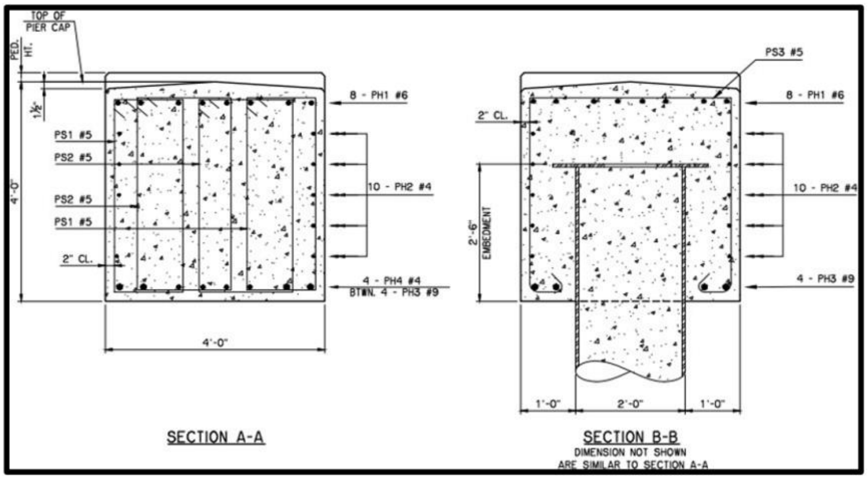
An anchorage ring is employed at the end of the embedded steel pipe pile for the connection. Such attachments enhance anchorage and establish a pathway for transferring forces, stresses, and moments from the concrete to the pile.

Example bridge

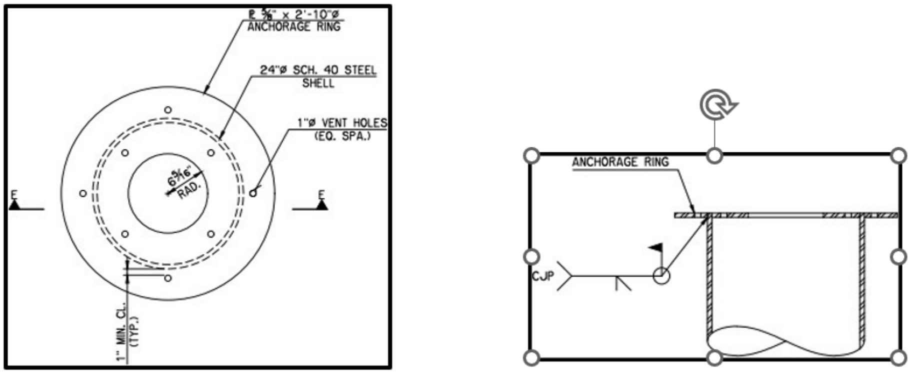
The connection details of the bridge structure over Pine Creek and the unnamed tributary are depicted in Figure 2-27.



(a) Elevation view



(b) Sectional elevation view



(c) Pile plan view

(d) Anchorage detail

Figure 2-27 ODOT: Annular ring detail for pile to bent cap connection, Pine Creek and unnamed tributary

2.4.3.14 South Dakota (SDDOT)

Use of steel pipe pile

Steel pipe piles are frequently utilized by SDDOT for relatively short concrete slab structures. In this scenario, the concrete slab serves as a bent cap, and the steel pipe piles are attached to the concrete slab.

Size and types of commonly used pipes

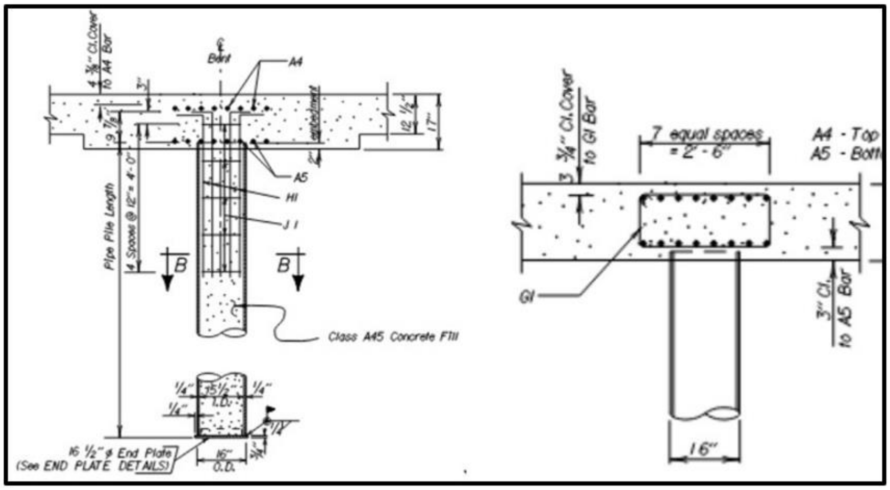
Steel pipe piles with a diameter of 16 in. and a thickness of 0.25 in. are utilized. These piles are filled with concrete to enhance their strength.

Connection design/Detailing consideration

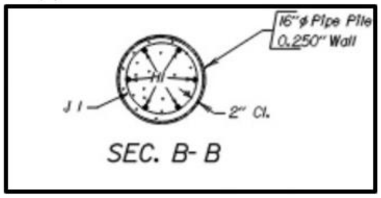
A pinned connection is typically assumed between the pipe pile and the slab. To achieve this connection, either 6 or 8 #6 bars are employed, depending on the diameter of the pipe pile. These bars are hooked at 900 angles and developed inside the cap for the connection. Additionally, the bars in the slab over the top of the pipe piles, acting as a "cap," vary depending on the spacing of the piles.

Standard bridge

The Base detail for the connection of the pile to the cap is illustrated in Figure 2-28.



(a) Sectional elevation view



(b) Pile plan view

Figure 2-28 SDDOT: Rebar detail for pile to bent cap connection

2.4.3.15 Tennessee (TDOT)

Use of steel pipe pile

According to TDOT, steel pipe pile bents are commonly used in bridges, particularly in West Tennessee.

Size and types of commonly used pipes

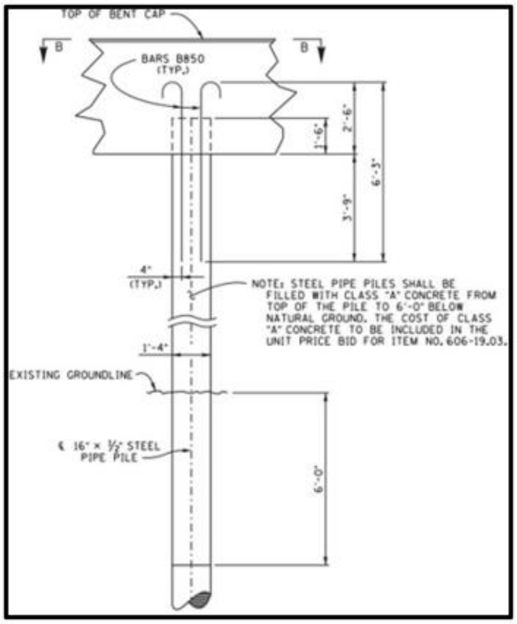
TDOT typically utilizes small diameter (16 in.) steel pipe piles with a thickness of 0.5 in. These piles are filled with concrete due to their small dimensions.

Connection design/Detailing consideration

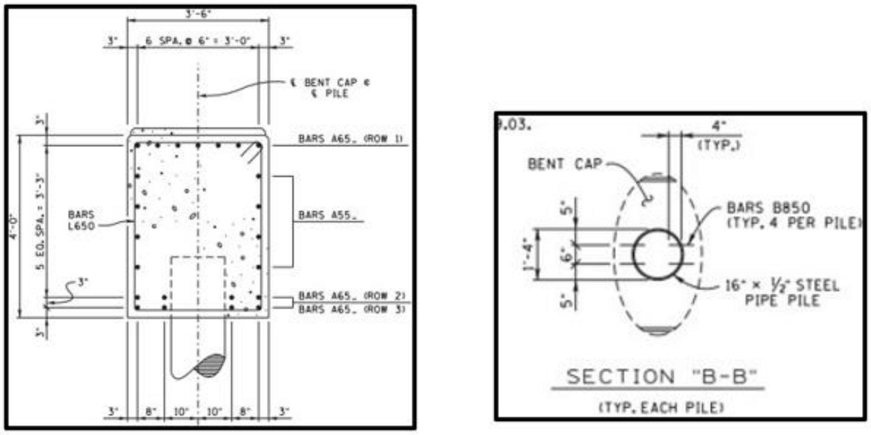
For the connection with the cap, longitudinal rebars are placed inside the concrete core of the steel pipe piles. These rebars are developed inside the cap and hooked at the end. However, they are not provided throughout the entire length of the pile. Additionally, a minimum embedment of 1 ft. 6 in. is required for the connection to the cap.

Example bridge

Figure 2-29 below illustrates an example of the details regarding the connection between the steel pipe pile and concrete bent cap at the Norfolk Southern Railroad Station.



(a)Elevation view



(b) Sectional elevation view (c) Pile plan view

Figure 2-29 TDOT: Rebar detail for pile to bent cap connection, Norfolk southern railroad station

2.4.3.16 Utah (UDOT)

Use of steel pipe pile

According to UDOT, exposed pile bents are not commonly used in bridge structures, where the piles extend directly from the ground to the bent cap. However, in specific cases, this practice may be allowed but requires approval on a case-by-case basis. Typically, bents in bridge structures comprise the bent cap, columns, footings, and piles.

Size and types of commonly used pipes

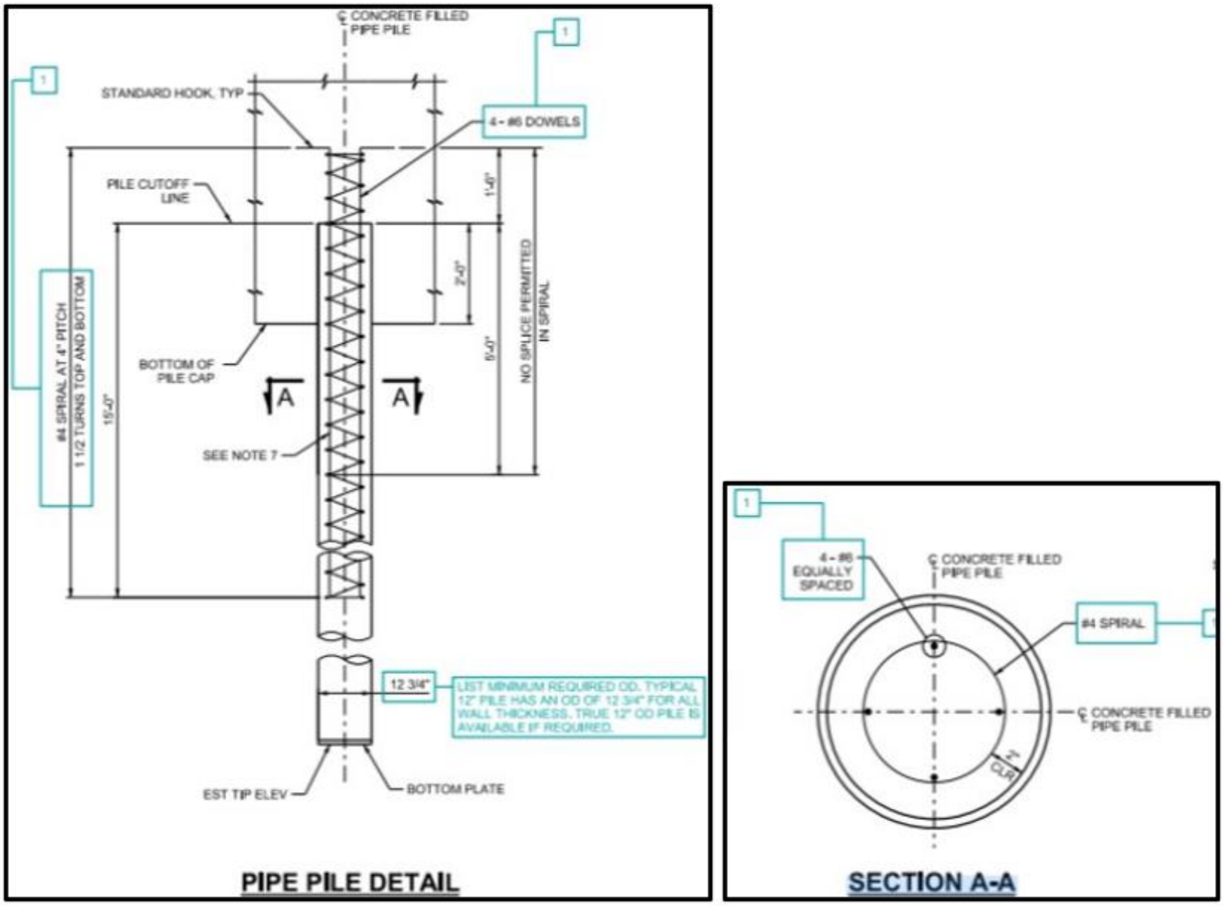
UDOT utilizes piles of various diameters for their bridge structures. The commonly used pile diameter is 12.75 inches. Since these are smaller diameter piles, they are filled with concrete during the construction process.

Connection design/Detailing consideration

UDOT's standard details depict piles embedded in a pile cap, such as an abutment or column footing. The same detailing applies to an exposed pile bent. UDOT provides standard details for both pinned pile heads and fixed pile heads, with an embedment depth into the pile cap of 2 ft. A reinforcement cage is placed inside the steel pile and developed within the bent cap, with hooked ends for a strong connection. A railroad bridge constructed a couple of years ago followed this exact approach.

Standard bridge

Figure 2-30 depicts the bridge standard drawings associated with the pipe piles.



(a) Sectional elevation view

(b) Pile plan view

Figure 2-30 UDOT: Rebar detail for pile to bent cap connection (Drawing No.: WS-2A)

2.4.3.17 Washington (WSDOT)

Use of steel pipe pile

According to WSDOT, steel pipe piles, and cast-in-place concrete caps are commonly employed in bridge structures. In the past, precast concrete piles were utilized to support terminal structures, but in the 1990s, a transition to steel piles occurred due to drivability and seismic considerations. The encountered soil is often highly dense and glacially consolidated, necessitating compliance with increasingly stringent seismic criteria, which include accounting for liquefaction and tsunami effects.

Size and types of commonly used pipes

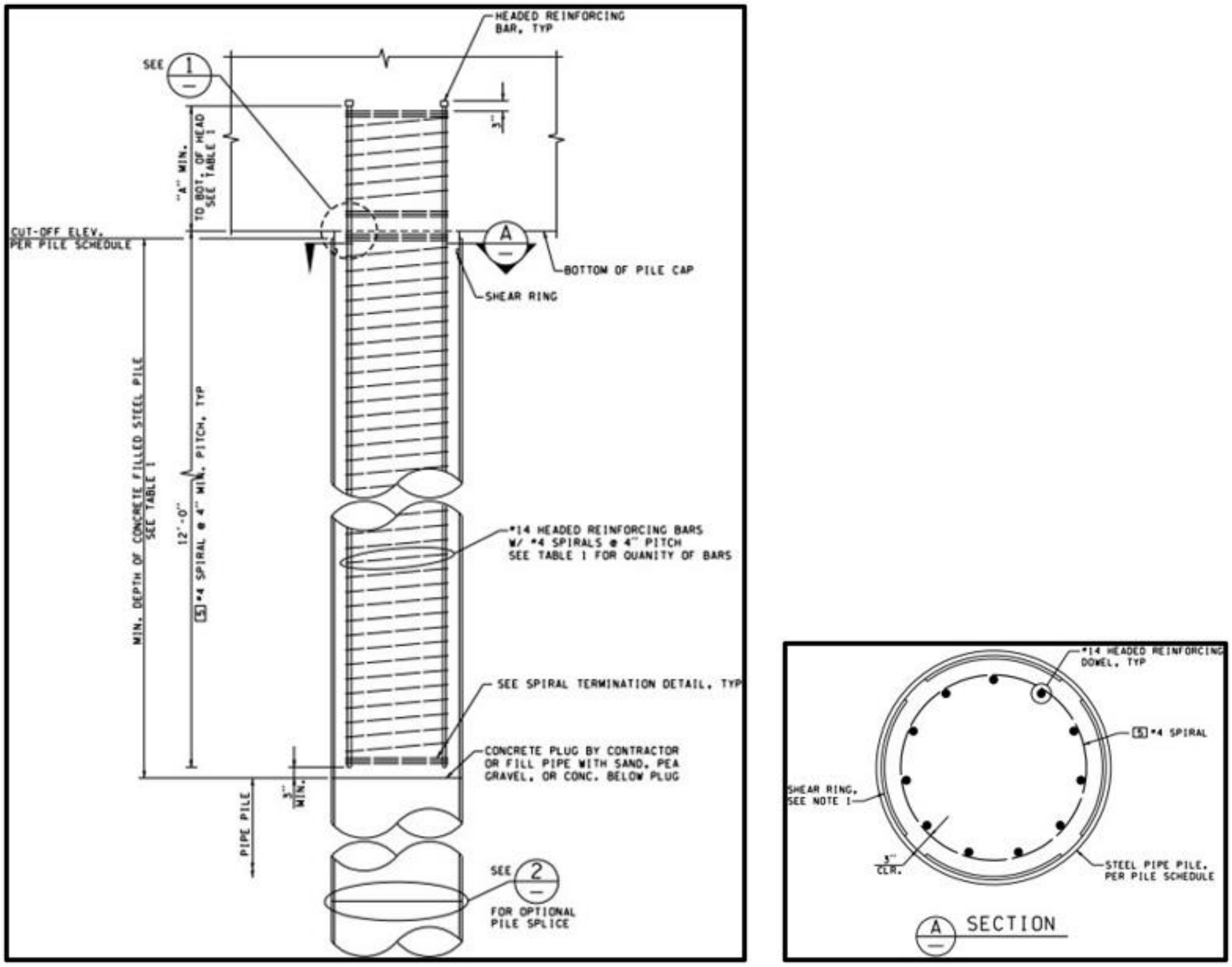
Piles with larger diameters, such as 30 in. and 36 in., with a thickness of 1 in., are commonly used. Due to their larger size, hollow piles are often chosen for these applications.

Connection design/Detailing consideration

Since the piles are hollow, a concrete plug measuring 12 ft. 3 in. to 14 in. is placed on top of the piles. Longitudinal rebars are incorporated within the plug to enhance the strength of the connection. WSDOT has also implemented headed rebars, which are developed inside the cap. The purpose of these rebars is to reduce the depth of the bent cap while maintaining the same level of strength.

Example bridge

Figure 2-31 below displays an example of the connection details between the steel pipe pile and concrete bent cap at Bainbridge Island.



(a) Sectional elevation view

(b) Pile plan view

Figure 2-31 WSDOT: Rebar detail for pile to bent cap connection, Bainbridge Island, Washington

2.4.3.18 Wisconsin (WisDOT)

Use of steel pipe pile

WisDOT commonly utilizes steel pipe piles (ASTM A252 Grade 3) that are filled with concrete to provide support for the substructures.

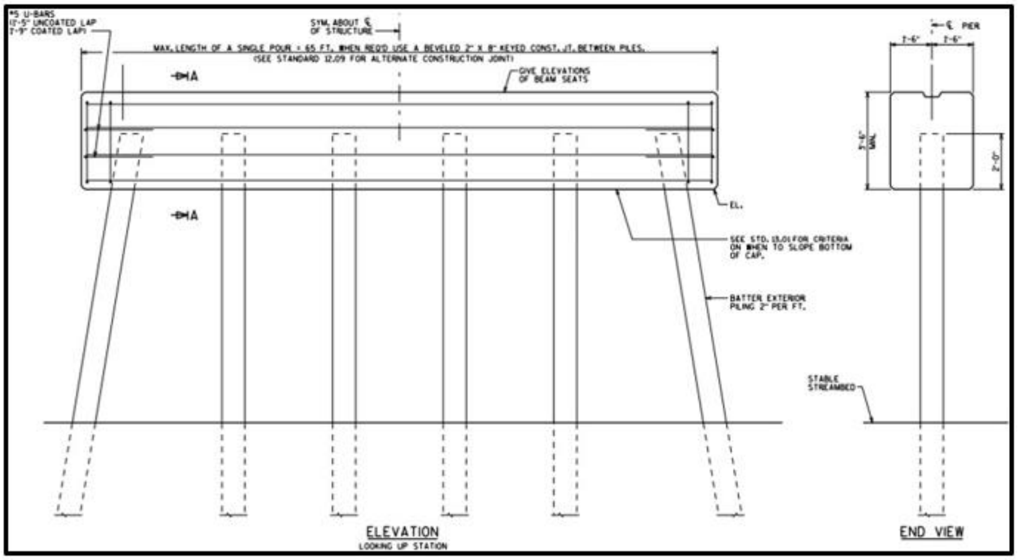
Size and types of commonly used pipes

WisDOT commonly employs steel pipe piles with various diameters, such as 12.75 in. and 14 in. The minimum thickness of these piles is 0.375 in. To enhance their load-bearing capacity, concrete is used to fill the steel piles. This concrete filling significantly increases the strength of the piles.

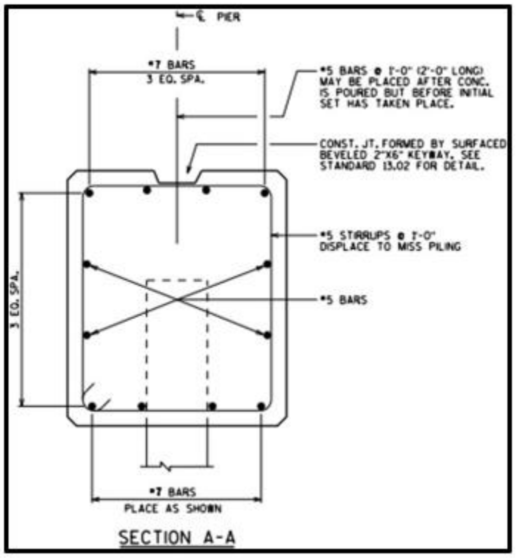
Connection design/Detailing consideration

The steel pipe piles used by WisDOT are internally reinforced with #7 bars, which are terminated 10 ft. below the groundline or streambed. For the necessary connection, a minimum pile embedment of 2 ft. is provided, and the longitudinal rebars extend up to 1 ft. 2 in. into the cap.

Standard bridge



(a) Elevation view



(b) Sectional elevation view

Figure 2-32 WisDOT: Rebar detail for pile to bent cap connection (STD. NO. 13.04)

2.4.4 BRIEF DETAILS OF STEEL PIPE PILE TO BENT CAP CONNECTIONS OF OTHER DOTS

Several states, namely Iowa DOT, North Dakota DOT, Arkansas DOT, Maryland DOT, Oregon DOT, Connecticut DOT (2.4.4.1), Michigan DOT (2.4.4.2), and Alaska DOT, have indicated the use of steel pipe piles. However, these states do not have standard design methodologies or example bridge drawings for the connection between steel pipe piles and concrete bent caps. On the other hand, Indiana DOT (2.4.4.3) also utilizes steel pipe piles and has a standard drawing for the connection details, which will be discussed.

2.4.4.1 Connecticut (CTDOT)

CTDOT rarely utilizes steel pipe piles; therefore, they do not have specific details regarding such piles. They do not have any provisions for steel pipe pile bents where the superstructure rests directly on a bent cap. However, the details concerning steel pipe piles with a wall pier on top of a pile cap are presented in Figure 2-33. Cast-in-place steel piles are employed, and they are embedded 1 ft. inside the cap.

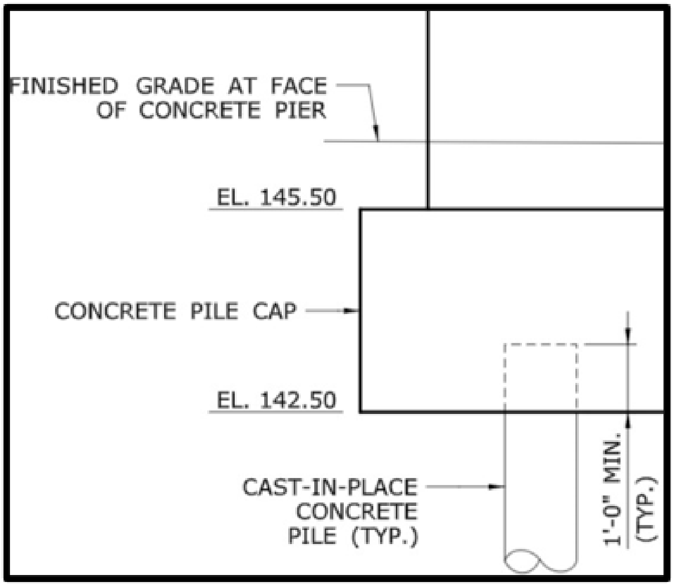


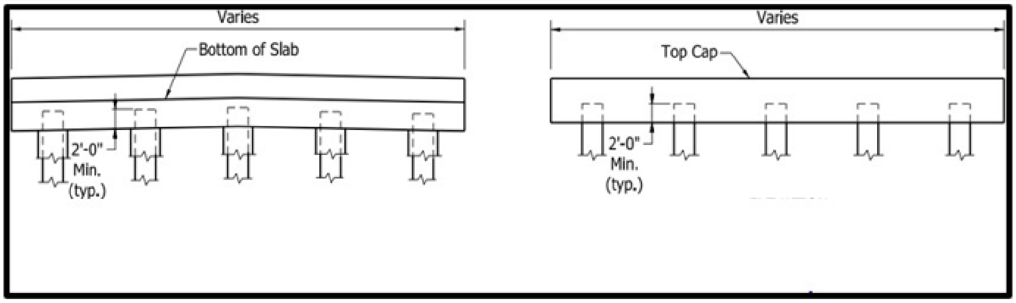
Figure 2-33 CTDOT: Pile to cap connection

2.4.4.2 Michigan (MDOT)

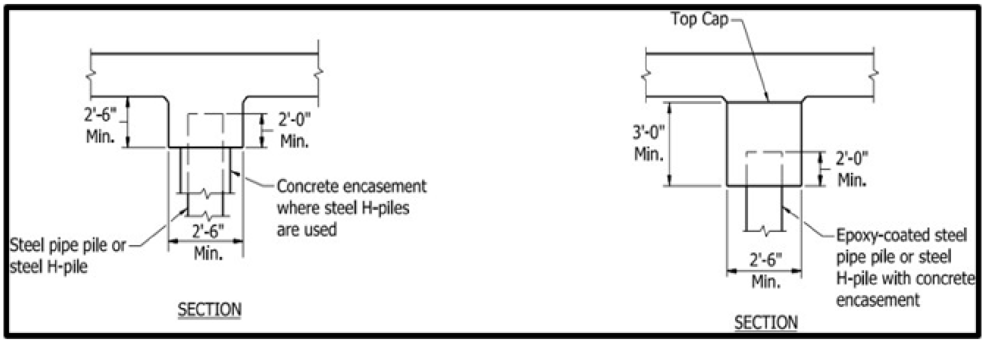
MDOT utilizes steel pipe piles with footings on the bottom to design foundations that prevent uplift, and as a result, the piles are not typically anchored to the foundation. Moreover, MDOT does not possess any standard details specifically for this type of connection.

2.4.4.3 Indiana (INDOT)

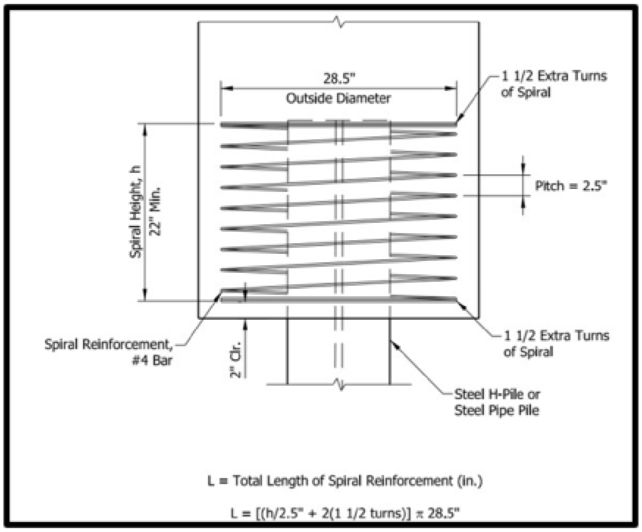
Indiana also utilizes small diameter steel pipe piles filled with a concrete core. While they do not have specific details related to connection, a minimum embedment length of 2 ft. is required. Spiral reinforcement is provided around the steel piles to enhance the connection strength and transfer force, stress, and moment from the concrete to the piles. The standard details provided in the Indiana Bridge Design Manual are showcased in Figure 2-34.



(a) Elevation view



(b) Sectional elevation view



(c) Sectional elevation view

Figure 2-34 INDOT: Spiral rebar detail for pile to bent cap connection

2.4.4.4 Louisiana (LADOT)

Louisiana Department of Transportation (LADOT) used steel pipe piles in the past and has several bridges in the state with exposed steel pipe piles supporting bent caps. All the bents were designed to be in compression, resulting in less critical connections. For connections requiring tension, studs were welded on the outside perimeter of the embedded pile. The steel pipe piles had an embedment length ranging from 6 in. to 9 in. inside the bent cap. A concrete plug was provided in the upper portion of the pile, and its length was determined based on the cohesion value between concrete and steel. For the connection, a reinforcement cage was implemented between the concrete plug and the bent cap. Unfortunately, standard details for these connections are not available. Lastly, it is worth noting that Louisiana no longer employs steel pipe piles for bridge structures.

2.4.5 SUMMARY OF DOT SURVEY

Based on the survey of state DOTs, it is evident that steel pipe piles are widely used by many states, and these states have extensive experience with such piles. The majority of states commonly utilize small diameter steel pipe piles, typically ranging from 14 in. to 24 in., which are filled with concrete. In certain states, a concrete plug is provided only on the top portion of the piles (usually around 1 ft. 6 in.) to enhance the connection's strength against buckling. Different states employ various methods to connect the steel pipe piles to the concrete bent cap, as discussed in the respective sections above. The piles are embedded inside the bent cap to a specific depth to establish a robust connection. The most commonly used method involves attaching the piles to the rebars, utilizing straight rebars, rebars with 1800 hooks, or rebars with 900 hooks (as seen in Caltrans, DelDOT, FDOT, GDOT, IDOT, MnDOT, MDOT, MoDOT, MDT, NDOT, NMDOT, NCDOT, SDDOT, TDOT, UDOT, WSDOT, WisDOT, INDOT). The longitudinal rebars of the piles are integrated into the concrete of the cap to ensure structural continuity. Moreover, in some states (such as Caltrans, MoDOT, and ODOT), an annular ring is welded to the top of the embedded pile to enhance anchorage, facilitating the transfer of stress, force, and moment to the surrounding rebars and concrete. Additionally, some states (MnDOT, MoDOT) utilize shear connectors that are welded to the outer perimeter of the embedded tube, providing strong resistance against applied forces, and ensuring robust punching resistance. These states have been employing steel pipe piles with various connection details for an extended period, and their structures have remained in good condition and performed well. Their extensive understanding and experience with these interconnections further support the suitability of steel pipe piles for bridge structures.

The survey conducted to gather information on the provisions related to different states of the US had certain limitations. The study encountered challenges in reaching out to all the bridge engineers of the respective states due to the nature of communication through emails. As a result, responses from all the states were not obtained, which could be attributed to the constraints on engineers' time and personal circumstances. While the engineers who did respond were very helpful, the study was still somewhat constrained, and in-depth discussions were not feasible due to the chosen mode of communication. Moreover, the privacy concerns of certain states were taken into consideration, and some engineers expressed discomfort in sharing their standard data, details, and drawings. This further restricted the extent of information that could be obtained for the study. Despite these limitations, the available responses and insights from the participating engineers provided valuable information for the study. The findings and observations derived from the obtained data still contribute to understanding the general practices and considerations related to steel pipe piles in bridge structures across different states.

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Chapter 3

EXPERIMENTAL PROGRAM DESIGN

3.1 BACKGROUND

This section details the experimental investigation details in terms of test setup and specimen designs to evaluate the structural behavior of five steel pile-to-bent cap connection configurations. The test program aimed to assess their structural performance under combined lateral load and moment.

3.2 TEST MATRIX

The test matrix was developed to investigate five distinct full-scale connection configurations adapted from the plans of an Alabama DOT bridge that uses deep RC drilled shafts (Bridge SR 3 over Cedar Creek Station in Morgan County, as original bridge plans shown in Appendix A). A full-scale specimen design was targeted, based on the reference project. All specimens have the same bent cap design and material properties, designed after the reference project's bent cap. Key features of the test matrix include:

- Bent cap dimensions: 4 ft (1219 mm) wide × 4.5 ft (1372 mm) deep.
- Steel pipe piles: 3 ft (914 mm) diameter, 0.5 in (12.7 mm) wall thickness
- Girder configuration: 12-ft (3658-mm) long with a pipe pile connected at midlength.

As detailed information regarding the tested specimens provided in Table 3-1, the specimens were categorized into five types of connections, with drawings of each specimen. These configurations were selected based on their common adoption among state DOTs, as discussed by Sharma (2023): three rebar-based connections that relied on non-contact lap splices as the primary force transfer mechanism and two non-rebar-based connections that utilized mechanical anchorage elements. The rebar-based connections straight, hooked, and headed rebar ends, each with distinct development lengths calculated using standard development length equations to ensure effective bond strength and mechanical interlock at the pile-to-cap interface. These specimens were named according to rebar type: (1) Headed, (2) Hooked, and (3) Straight. The mechanical anchorage-based connections incorporated an annular ring plate and a stud as anchorage elements. Correspondingly, these specimens were named based on the anchorage type: (4) Stud and (5) Ring.

Another notable difference among the specimens is the pipe pile embedment depth. Specimens 1–3 (Headed, Hooked, and Straight) had an embedment depth of 1 ft (305 mm), while Specimen 4 (Stud) was embedded to 3 ft (914 mm), and Specimen 5 (Ring) had an embedment depth of 2 ft (610 mm). The embedment depths of the mechanical anchorage specimens were not uniform and required specific justification based on design intent and detailing constraints. The Stud required greater depth to accommodate three rows of shear studs, while the Ring's depth was determined through an iterative design process following WSDOT (2024) guidelines. These embedment lengths meet or exceed typical code minimums for fixed behavior; Zapata et al. (2022) noted that many DOTs consider 1.0–4.0 ft (305–1219 mm) sufficient for such fixity. This experimental design reflects a deliberate choice to evaluate practical connection systems rather than isolated parameters. Isolating embedment effects would require testing each connection type at multiple embedment depths, additional specimens beyond the scope of this study. The practical value lies in comparing configurations as DOTs would actually implement them.

Table 3-1 Test Matrix

Specimen	**Connec. Type**	**Anchorage Type**	**Concrete Plug Depth**	**Pipe Pile Embed. Depth**	**Rebar Embedment Depth**	**Cap Beam**	**Notes**
Test 1 – Headed	Non-contact lap splice	Mechanical headed rebars	3 ft	1 ft	1 ft 10 in	Original Design	Eight #11 rebars uniformly spaced
Test 2 – Hooked		180° hooked rebars	3 ft	1 ft	2 ft 3 in		
Test 3 – Straight		Straight rebars	4 ft	1 ft	3 ft 6 in		
Test 4 – Stud	Non-rebar options	Shear studs	3 ft	3 ft	*No rebars within the pile plug*		Three rows of 12 Nelson-type shear studs (6 in × ¾ in)
Test 5 – Ring		End plate	2 ft	2 ft	*No rebars within the pile plug*		Outer/inner diameters of 44/28 in. with a thickness of 0.5 in.

Note: Concrete plug depth refers to the depth of concrete cast inside the pipe pile; pipe pile embedment depth refers to the depth of the pipe embedded into the bent cap. See Figure 3-3 for visual definition of Plug Depth vs. Embedment Depth.

Note: Concrete plug depth refers to the depth of concrete cast inside the pipe pile; pipe pile embedment depth refers to the depth of the pipe embedded into the bent cap. See Figure 3-1 for visual definition of Plug Depth vs. Embedment Depth.

To ensure experimental validity, each connection system was detailed to meet its specific code-mandated geometry. The Stud connection utilized a 3 ft (1.0D) embedment to satisfy AASHTO spacing requirements for three rows of shear studs. Conversely, the rebar connections utilized a 1 ft (0.33D) embedment, representing the 'pinned' assumption threshold in DOT practice. Therefore, the variable embedment depths are an inherent characteristic of the systems being compared, rather than an independent variable. Each configuration was tested once (n=1), precluding statistical assessment of variability.

3.3 BENT CAP DESIGN

The bent cap was designed with dimensions of 4 ft × 4.5 ft (1219 mm × 1372 mm) and reinforced longitudinally using twenty #11 rebars arranged with eight each in the top and bottom layers, and two additional rebars on each side, as provided in Figure 3-1.

The structural analysis of the bent cap was performed in accordance with AASHTO (2020), with the governing load case identified from the Strength-III limit state envelope, as summarized in Sharma (2023). Based on revised calculations, the axial and flexural demands at the joint were subsequently idealized as 210 kips (934 kN) and ±320 kips-ft (343 kNm) for one case, and 790 kips (3514 kN) and ±320 kips-ft (343 kNm) for the other. The envelope shear demands in the longitudinal and transverse directions of the bent cap remained below 25 kips. The bent cap design was deemed adequate to resist the applied demands based on P-M curve (Sharma, 2023).

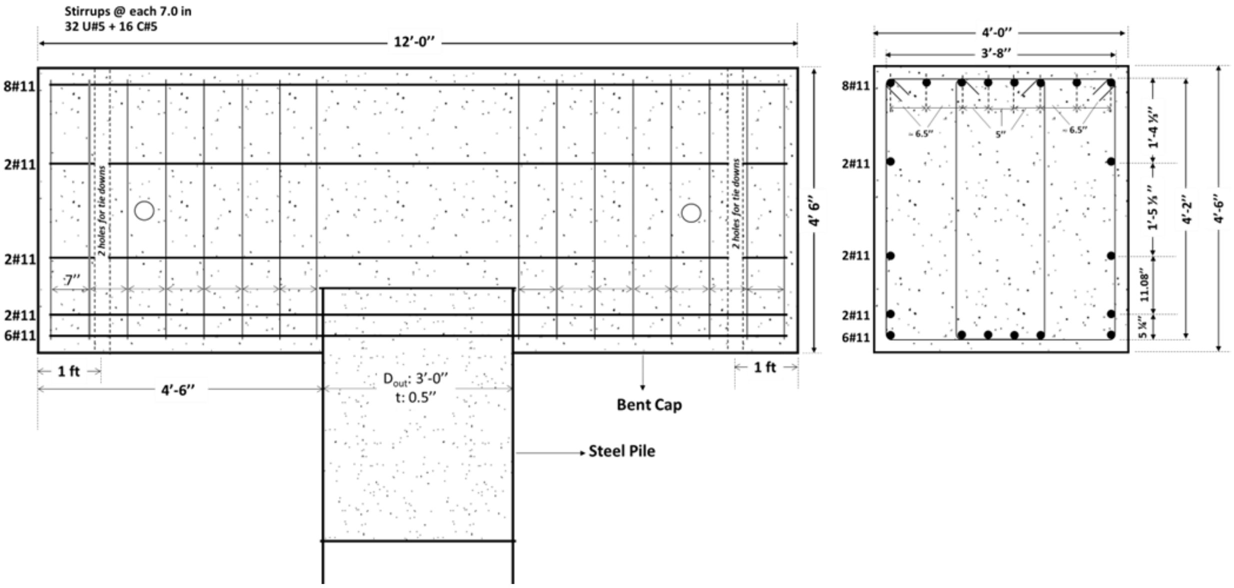


Figure 3-1 Bent Cap Design

Figure 3-2 Photos of Bent Cap Rebar Cage: (a) Isometric View, (b) Face View, (c) Close-up of Headed Rebars Crossing the Pipe Pile, and (d) Bent Cap Rebar Cage with Pipe Pile

3.4 DESIGN OF HEADED, HOOKED AND STRAIGHT REBAR CONNECTIONS

The rebar connections consisted of eight #11 rebars detailed with headed, hooked, and straight rebar ends, as illustrated in Figure 3-3a–c. The rebar embedment lengths were determined by their respective development length requirements in accordance with the applicable AASHTO LRFD specifications (2020). Headed rebars reduce the required development length by transferring a portion of the rebar force directly into the concrete through bearing at the head. The specifications in ACI 318-19 (2019) have been followed as the AASHTO LRFD (2020) does not have standard methodology applicable for calculating their development length.

Embedment lengths were calculated in accordance with ACI 318-19 (headed) and AASHTO LRFD (hooked/straight). For the materials used (f'_c= 4000 psi [27.6 MPa], f_y = 60000 psi [413.7 MPa]), the required development lengths were determined as 19 in. (Headed), 27 in. (Hooked), and 41 in. (Straight). Details are summarized in Table 3-1.

The stirrups consisted of closed hoops spaced at intervals of 10 to 11 in (254 to 279 mm), with the quantity for each specimen detailed in Figure 3-3a–c. The specimen design in this study was based on AASHTO (2020). The photos of constructed rebar cages for headed, hooked, and straight type connections are given in Figure 3-4a–c.

The concrete plug depth was selected as 3 ft (914 mm) for headed and hooked configurations and 4 ft (1219 mm) for straight rebars. The pile embedment depth was chosen as 1 ft (305 mm) for the rebar-based connections to minimize the contribution from pile embedment, ensuring forces were transferred primarily through the vertical rebars at the connection. The 1-ft embedment was chosen as a lower bound depth of the specimens based on practical constructability considerations.

Moment capacities of the tested specimens were estimated based on cross-section P-M curve analysis, assuming flexural failure at the RC section where the pile enters the cap, neglecting axial force. This included eight #11 rebars equally spaced within the circular concrete section, representing the RC section. Calculations were performed using Response2000 (Bentz, 2000), with concrete compressive strength taken as the 28-day cylinder strength from headed specimens and rebar yield strength based on manufacturer-reported test values. Assumptions included plane sections remaining plane, strain compatibility between concrete and reinforcement, and full development of reinforcement. Reinforcing steel was modeled as linear elastic-perfectly plastic, and concrete behavior was represented by the Popovics/Thorenfeldt/Collins model (Bentz, 2000). All reinforcing bars were embedded to at least the required development length (headed, hooked, or straight). Another cross-section, representing a hollow steel pile and incorporating manufacturer-reported steel properties, was used to estimate the capacity of pipe pile alone. The flexural capacities are listed below:

- For reinforced concrete core representing the headed, hooked, and straight rebar connections: M_n=937.5 kip-ft (1271 kNm) as a lower bound, and
- For hollow steel pipe pile considering that the pile can develop full capacity: M_n =2370 kip-ft (3213 kNm).

3.5 DESIGN OF SHEAR STUD CONNECTION

The stud specimen incorporates shear studs, which act as mechanical connectors welded to the outer surface of the steel pipe to facilitate shear transfer between the steel and the surrounding concrete, without any additional reinforcement at the joint. Three rows of 6 in × ¾ in (152 mm × 19.1 mm) shear studs are used. Within each layer, studs are spaced at 30° along the perimeter. The spacing between layers is chosen as 11 in., satisfying minimum requirements of AASHTO (2020). Installation was carried out using proper welding equipment to replicate field conditions, with manufacturer representatives present to guide the process. The drawings of the Stud connection are provided in Figure 3-3d, with the photo of welded studs as given in Figure 3-4d.

Using a force couple approach with linearly distributed stud forces and lever arms based on the sectional arrangement shown in Figure 3-3d, the following configuration was idealized when calculating moment capacity: one stud positioned at the full pile radius, r_{pile}, reaching its full shear capacity; two studs located at sin(60°) × r_{pile} each contributing sin(60°) of their full shear capacity; and two studs at sin(30°) × r_{pile} each contributing sin(30°) of their full shear capacity.

As specified in Section 6.10.10.4.3 of AASHTO (2020), the nominal shear resistance of an individual stud connector is determined using following equation:

$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} \leq A_{sc} F_u \quad \text{(Eqn. 4)}$$

where, A_{sc} is the cross-sectional area of a stud shear connector (in^2), E_c is the modulus of elasticity of concrete (ksi), and F_u is the specified minimum tensile strength of a stud shear connector (ksi). For A_{sc} = 0.44 in2 (284 mm2, i.e., ¾ in. studs), f'_c= 4 ksi (27.6 MPa), E_c = 3600 ksi (24821 MPa), and F_u = 65 ksi (448.2 MPa), the nominal shear capacity, Q_n, is calculated as 28.5 kips (126.7 kN).

Finally, the flexural capacity contributed by a single row of studs is calculated as 6 × Q_n × r_{pile}. With three rows in total, the resulting flexural capacity is calculated as: 3 × 6 × Q_n × r_{pile} = 18 × 28.5 kips × 1.5 ft = 769.6 kip-ft (1043 kNm).

The concrete plug depth was chosen as 3 ft to be consistent with rebar-based connections. The pile embedment of 3 ft (equivalent to one pile diameter) was selected primarily to provide adequate spacing between the three rows of shear studs during installation. Given the deeper embedment in the Stud specimen, the embedment itself is expected to significantly enhance the connection capacity, resulting in a flexural capacity well above the lower bound of 769.6 kip-ft (1043 kNm) based on shear stud contribution alone.

3.6 DESIGN OF ANNULAR RING CONNECTION

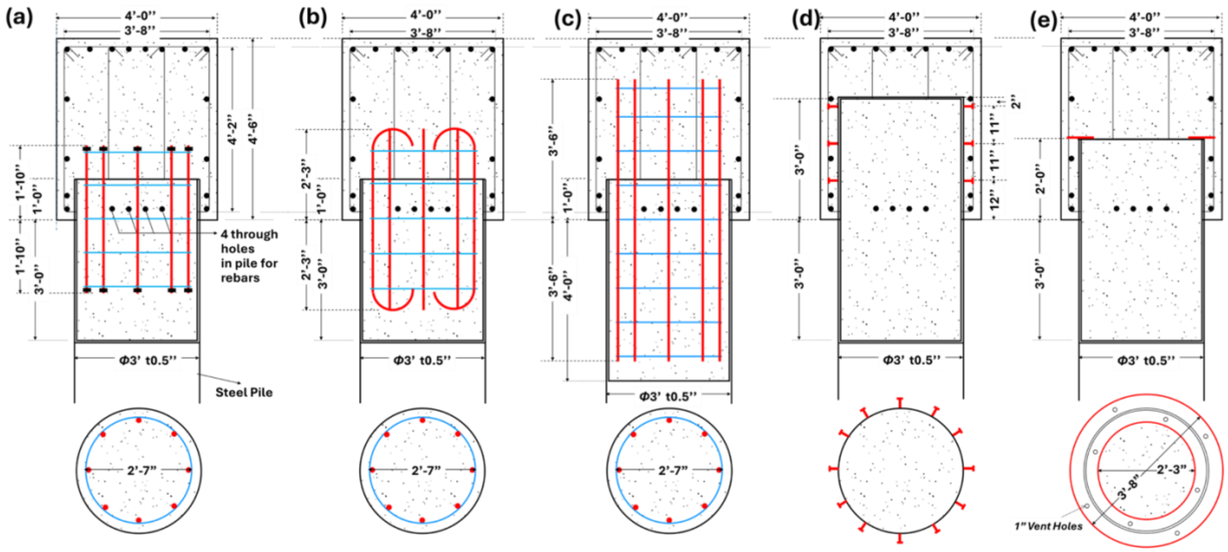
The annular ring welded to the end of the steel tube functions as an anchorage element, distributing stresses on the concrete body to resist flexural loads transferred from the pile. The plate was sized with an outer diameter of 44 inches and an inner diameter of 28 inches, corresponding to the pile diameter ±16 times the wall thickness, and a thickness of 0.5 inches (12.7 mm), corresponding to the thickness of the pipe pile. The welding followed the guidelines specified in Section 7.10.3.A of the WSDOT Design Manual (2024). The wedge-shaped profile at the base of the pipe pile enabled a complete joint penetration weld between the annular plate and the pile, applied continuously around the joint. Additionally, eight 1-inch-diameter (25.4 mm) vent holes were punched into the annular ring—four located outside and four inside the pipe wall, equally spaced to have proper concrete consolidation beneath the ring. The vent hole locations were also intentionally offset to avoid alignment with the load direction. Drawings of the specimen featuring the annular ring are shown in Figure 3-3e, with the connection photo presented in Figure 3-4e.

The embedment length of the pipe pile was optimized using the following equation, as specified in the WSDOT Design Manual (2024):

$$l_e \geq \sqrt{\frac{{D_0}^2}{4} + \frac{5.27DtF_u}{\sqrt{f'_{cf}}}} - \frac{D_0}{2} \quad \text{(Eqn. 5)}$$

Where le is the embedment length (in), D_0 is the outer diameter of the annular ring (in), D is the diameter of the pile (in), t is the wall thickness of the steel tube (in), f'_{cf} is the specified 28-day compressive strength of the cap concrete (ksi), and F_u is the specified minimum tensile strength of the steel (ksi). Using D_0 = 44 in (1118 mm), D = 36 in (914 mm), t = 0.5 in (12.7 mm), f'_{cf} = 4 ksi (27.6 MPa), and F_u = 60 ksi (414 MPa), the required embedment length, l_e, to achieve full plastic capacity is calculated as 36 in (914 mm). For a hollow steel pipe pile capable of developing its full moment capacity, the corresponding flexural strength is 2370 kip-ft (3213 kNm). To achieve comparable moment capacities between the annular ring connection and other connection configurations, the pile embedment depth was adjusted.

An iterative design approach, utilizing Equation 5, was used to refine the embedment length so that concrete failure would occur before the annular ring yielded and reached its full capacity. This was accomplished by targeting a stress level in the pipe pile equal to 50% of its ultimate tensile strength (0.5 F_u). Accordingly, an embedment length of 24 inches (610 mm) was selected, resulting in a flexural capacity of 2225 kip-ft (3017 kNm) based on the pipe pile cross-section reaching 0.5 F_u.



8#11 headed rebars
5#5 Hoops

8#11 hooked rebars
5#5 Hoops

8#11 straight rebars
9#5 Hoops

Studs: 6 in x ¾ in, 1 ½ in head
3 rows x 12 studs per row

Ring
thickness: 0.5 in.

Figure 3-3 Specimen Connection Design and Drawings–(a) Headed, (b) Hooked, and (c) Straight, (d) Stud and (e) Ring.

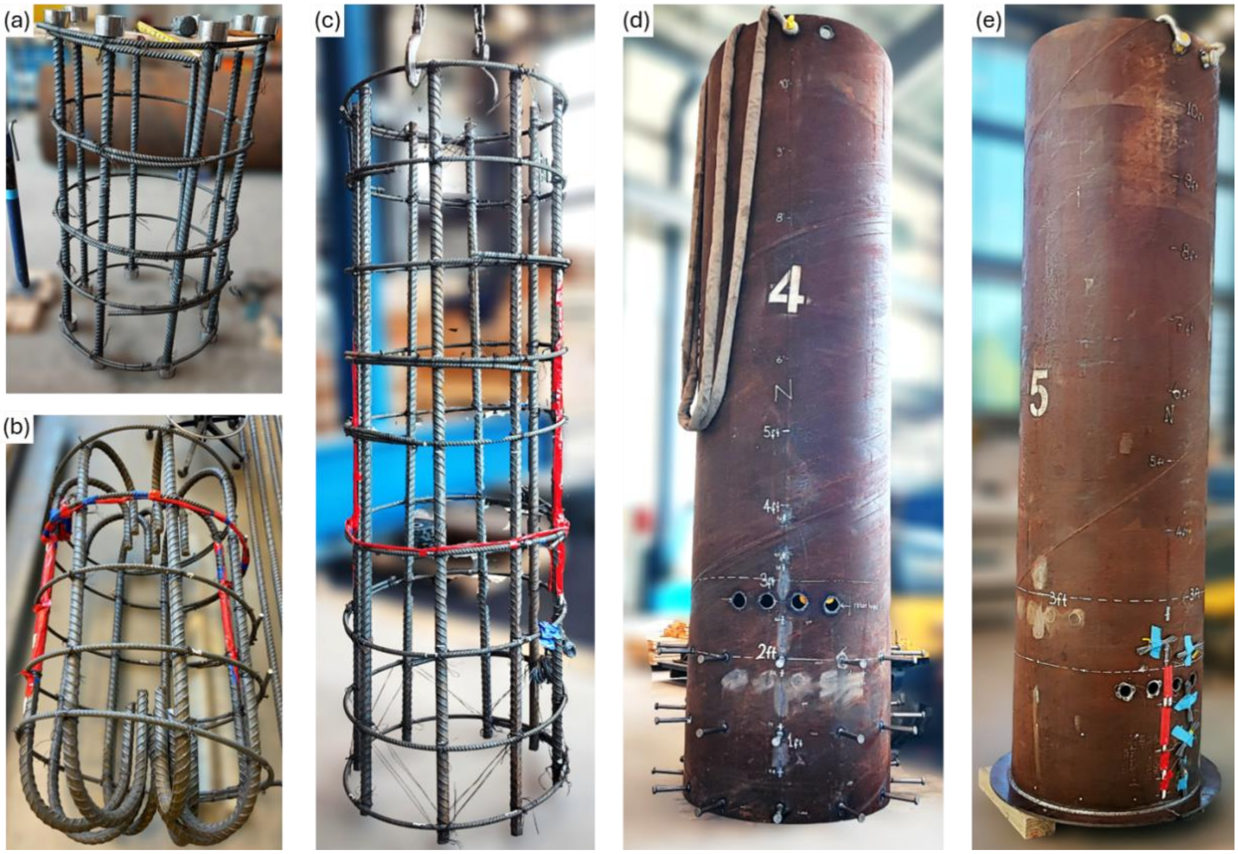


Figure 3-4 Rebar Cages–(a) Headed, (b) Hooked, and (c) Straight; and Welded Specimen Connections–(d) Stud and (e) Ring

Estimated moment capacities for each connection were calculated and compared to test results. The calculation methods used to estimate moment capacity varied by connection configuration to reflect their distinct load transfer mechanisms. Rebar-based capacities were derived from the concrete section within the plug using traditional RC assumptions, excluding steel pipe contribution. The Stud connection used a force-couple approach based on stud location relative to the neutral axis, while the Ring capacity was estimated as 0.5×F_u acting on the developed portion of the pipe section. The flexural capacities and corresponding actuator load levels for each specimen group are calculated by dividing the flexural demand by the distance from the joint interface to the actuator of 6.375 ft (1943 mm) as follows:

- Rebar-based specimens (headed, hooked, and straight) had a flexural capacity of 937.5 kip-ft (Section 3.4), resulting in an actuator load of approximately 147 kips (653.9 kN);
- The stud specimen had a flexural capacity of 769.6 kip-ft (Section 3.5), corresponding to 121 kips (538.2 kN); and

- The annual ring specimen had a flexural capacity of 2,225 kip-ft (Section 3.6), corresponding to an actuator load of 349 kips (1552.4 kN).

The service load level was defined as 88.2 kips (392.3 kN), corresponding to 60% of the rebar-based connection capacity. These simplified approaches provided comparative baselines and were not intended to capture the full plastic capacity. The estimated capacities do not account for potential contributions from embedment, such as bearing resistance and friction between the pile surface and concrete surfaces. Specifically, for the rebar connections, the contribution of the steel pile cross-section was excluded. Similarly, for the non-rebar (mechanical anchorage) connections, the contribution from concrete core filled inside the steel pile was not considered.

Chapter 4

SPECIMEN CONSTRUCTION AND DETAILS

4.1 STEEL-PIPE PILE SPECIMEN DESIGN OVERVIEW

This chapter focuses on the construction of laboratory specimens. The description of the construction process is included here as well as the material properties of the laboratory specimens.

4.2 CONSTRUCTION OF TEST SPECIMENS

The construction of all five pipe pile-to-bent cap connection specimens followed similar procedures, but they each had their own set of challenges to adapt to. The space in the lab only allowed for the construction and testing of one specimen at a time, and this provided ample opportunity to learn from the construction process each time. It is important to note the location within the lab changed for each specimen due to size restraints and the available space, but this did not affect the construction process.

Because the overall specimen geometry remained the same, the formwork was reused for all five bent caps, with only necessary, usually minor changes to the pipe pile connection, which saved both time and resources. The formwork also was efficiently designed to support the load from the uncured concrete. To facilitate integration with the pipe pile, the bottom reinforcement layout was adjusted by placing two rebars along the sides and routing four rebars through pre-punched holes in the pile. The four rebars passing through the pile were headed rebars, secured with mechanical anchorage washers installed post-placement and ensured requirements in accordance with ACI 318-19 (2019) for headed rebars. The holes in the steel pile were made large enough to minimize placement issues, prevent concrete gaps around the openings, and reduce potential interlocking effects. In all specimens, the bent cap design was kept constant while the connection mechanism and corresponding rebar embedment depths were varied to achieve comparable load transfer and moment capacity. Photos of the bent cap rebar cage are provided in Figure 3-4.

4.3 MATERIAL PROPERTIES

The material properties for the experimental specimens were specified to ensure that the test specimens represented nominal strengths of realistic bridge components and conformed to the associated ASTM standards. Both the concrete and reinforcement strengths are specified in ALDOT Standard Specifications for Highway Construction (ALDOT 2022). Bent caps in Alabama are to be constructed with Class B concrete mixtures, and the requirements for this are provided in Table 4-1.

Table 4-1 ALDOT prequalification requirements for class B concrete mixtures (ALDOT 2022)

Minimum 28-Day Compressive Strength (psi)	4000
Maximum Water/Cementitious Materials Ratio	0.45
Range of Total Air Content (%)	2.5 - 6.0
Slump (inches)	3.5
Maximum 28-Day Drying Shrinkage (%)	0.04
Largest Nominal Maximum Aggregate Size (inches)	1.0

The required minimum steel grade is grade 60 as stipulated by the ALDOT standard. Upon the delivery of the reinforcement, material test reports (MTR) documenting the yield strength of the reinforcing steel were provided. Bars of the same size were fabricated with steel from the same heat, so each bar size has its own value. The measured yield strengths from the manufacturer provided material test reports and the calculated yield strains, which are based on assuming an elastic modulus for steel of 29,000 ksi, are provided in, and they all exceed the required nominal strength. The MTRs for the reinforcement of all five specimens are provided in Appendix B.

The steel pipe piles were fabricated from 0.5 in (12.7-mm) thick high-strength steel conforming to ASTM A252 Grade 3 standards, ensuring the adequate structural capacity and ductility for both lateral and axial loads. The rebars, which serve as the key element in the rebar-based connection configurations (#11 bars in the headed, hooked, and straight connection configurations) and as the reinforcement in the bent cap beam (#5 and #11 bars) were specified according to A615 Gr60/AASHTO M31 requirements. Headed rebar terminators used in this study were BPI® BARSPLICER DoughNUT™ Term models, complying with ASTM A970 Class A and Class HA. Each terminator provided a projected bearing area in tension at least four times the full cross-sectional area of the rebar, with dimensions including a diameter of 3¼ in (82.6 mm), width of 1⅞ in (47.6 mm), and thread size of 1⅜ – 6 UNC.

The other specific components used in the connections complied with the relevant material standard: for the shear stud connection (Specimen 4), three rows of 12 Nelson®-type shear studs (6 in × ¾ in) conforming to ASTM A379 were welded to the outer surface of the pipe, while the annular ring connection (Specimen 5) utilized an annular ring plate conforming to A709 Grade 50. The welding specifications in AWS D1.1/D1.1M were followed for the specimens with the shear studs and annular ring plate. The material type and mechanical properties obtained from manufacturer-provided material test reports (MTR) for the steel components used in the tests are summarized in Table 4-2.

Table 4-2 Steel Material Properties

Material	**Type/Grade**	**Yield Strength (ksi)**	**Tensile Strength (ksi)**	**Notes**
Steel Pipe Piles	Spiralweld Pipe ASTM A252 Gr. 3	65.8	76.2	Headed, Hooked, Straight, and Ring Specimens
		73.6	84.8	Stud Specimen
Reinforcement Bars	A615 Grade 60 / AASHTO M31	67.4	104.6	#11 Rebar (1st Heat)
		67.6	99.9	#11 Rebar (2nd Heat)
		83.3	110.5	#11 Headed Rebars (3rd Heat)
		74.4	111.2	#5 Rebar
Headed Rebar Terminators	ASTM A970	42.3	70.0	Threaded doughnut-shaped terminators
Shear Studs	ASTM A379	59.4	73.6	Nelson® S3L Shear Connectors
Annular Ring Plate	A572 Grade 50	59.0	69.7	Nominal values (A572 Gr. 50) used for capacity predictions.

The bent cap and concrete plug were cast simultaneously in a single pour to ensure monolithic behavior and eliminate cold joints at the pile-to-cap interface. Concrete was vibrated to ensure consolidation around reinforcement and within the annular ring vent holes. The concrete mix design of ALDOT's Class B was used, which stipulates a minimum 28-day concrete strength of 4,000 psi (27.6 MPa), maximum water/cementitious materials ratio of 0.45, range of total air content of 2.5% – 6.0%, slump of 3.5 in (88.9 mm), maximum 28-day drying shrinkage of 0.04 and largest nominal maximum aggregate size of 1.0 in (25.4 mm).

Three 4" x 8" concrete cylinders were tested and averaged according to ASTM C39/C39M-21 for 7-day, 28-day, and test-day strengths to verify that the measured strengths reached the target strengths (ASTM 2023). Additionally, the slump and air content were verified when the concrete was delivered to the laboratory to ensure the mixture design met the required standards. Concrete properties played a critical role in the overall behavior of the bent cap, which was consistently cast using a high-performance mix designed to achieve the required compressive strength and durability. Standard testing was conducted on the concrete mix to validate its workability and strength development. The concrete properties at the time of casting are summarized in Table 4-3, and the average compressive strengths are given in Table 4-4. A slump tolerance of +1 in (25.4 mm) was specified with a target of 3.5 in (88.9 mm); however, some mixtures fell outside this range. This deviation did not impact the 28-day compressive strength or concrete quality at placement for any specimens.

Table 4-3 Concrete Mix Properties

	(1) HEADED	**(2) HOOKED**	**(3) STRAIGHT**	**(4) STUD**	**(5) RING**
Slump (in)	4.5	6.0	5.5	3.5	4.0
Air Content (%)	5.5	4.5	5.6	4.5	5.5
Density (pcf)	140	140	140	142	142
Temperature (F)	70	80	90	70	72

Table 4-4 Average Concrete Strength (psi)

Days from casting	**(1) HEADED**	**(2) HOOK**	**(3) STRAIGHT**	**(4) STUD**	**(5) RING**
7 days	3,820	4,050	3,720	4,350	4,310
28 days	4,950	4,520	3,930	5,380	5,070
Testing Day	4,960	5,400	4,600	5,360	5,380

Chapter 5

EXPERIMENTAL SETUP AND INSTRUMENTATION

5.1 INTRODUCTION

This chapter covers the testing of the five full-scale pipe pile-to-bent cap connection specimens. It includes details on the testing setup, load protocol, and instrumentation. The testing was completed in the high bay at the Auburn University Advanced Structural Engineering Lab, and all five full-scale tests required careful planning and preparation.The test setup and procedures remained relatively constant for each test, but each specimen had its unique aspects to address within the setup. Despite the challenges faced during the construction and testing process, five full-scale tests were completed and provided necessary insight into the behavior of pipe pile-to-bent cap connections.

5.2 TEST SETUP AND LOADING PROTOCOL

The experimental test setup was designed to simulate a substructure testing of a realistic bridge pipe pile-to-bent cap connection while ensuring precise control and measurement of structural responses. A hydraulic actuator system was employed to apply repeated unidirectional lateral loading to the full-scale specimens, with the loading protocol programmed to gradually increased until target forces and displacements were reached. This approach enabled the capture of both the elastic and post-yield behavior of each connection configuration.

The test setup, illustrated in Figure 5-1, includes several custom-fabricated steel fixtures. These fixtures were designed to mount the actuator to the lab's reaction wall on one end, and to secure the specimen using steel collar fixtures on the other end. The loading was applied by the actuator pulling the specimen towards the reaction wall. Each specimen was securely mounted on the strong floor with post-tensioning applied at four points configured to avoid uplifting and sliding of the test specimen. Post-tensioning forces were 50 kips (≈222 kN) per tie-down near the actuator and 175 kips (≈778 kN) per tie-down at the far end, as shown in Figure 5-1a. The tie-down locations were chosen to be near the ends of the bent cap beam to ensure minimal stress was induced to the connection region from the post-tensioning. Therefore, the bent cap and pipe pile assembly functioned as an integrated system with appropriate restraints provided at the base of the piles and along the cap. The tie-down points also correlated well with the girder locations observed in the reference SR 3 bridge drawings (Figure X).

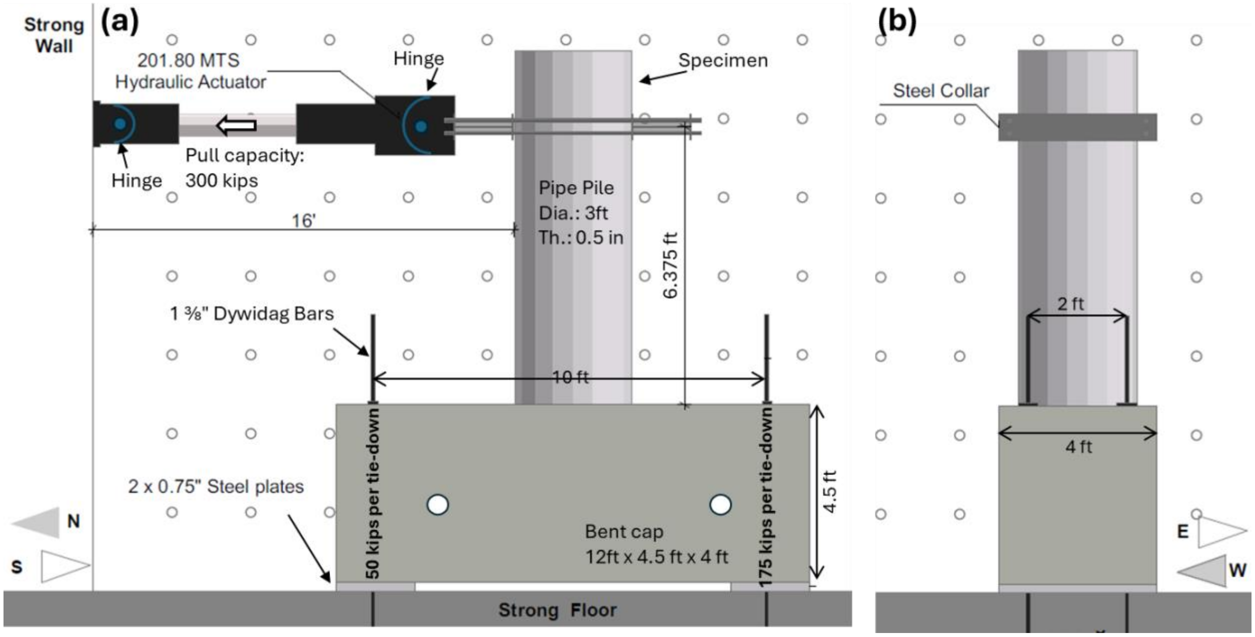


Figure 5-1 Test setup Schematics–(a) Side view, and (b) rear view.

The loading was force-controlled initially up to the point where the loading reached 117.6 kips (523.1 kN) and then changed to displacement-controlled for the remaining of the load cycles to maintain a constant displacement rate and capture any potential reduction in the specimen resistance without causing overloading. Therefore, the last loading cycle of the rebar specimens was displacement-controlled whereas the last three loading cycles of non-rebar specimens were displacement-controlled, as presented in the loading protocol given in Figure 5-2.

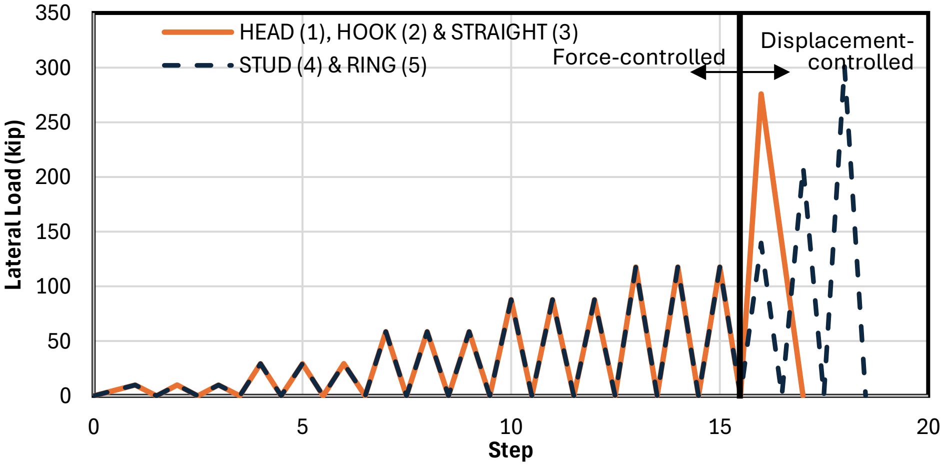


Figure 5-2 Loading Protocol

Several load cycles were performed prior to the final pull of the specimens under displacement control. The first load cycles consisted of an initial 10-kip (44.5 kN) cycle to verify the test setup through assessing any slip, movement, and rotation, and ensuring accurate sensor data. Loading cycles were then applied in increments corresponding to specific percentages of the reinforced concrete core and steel pile capacities. For rebar connection specimens, 29.4 kips (130.8 kN, 20% of concrete core capacity) and 58.8 kips (261.6 kN, 40%) were selected, with the latter approximating service-level loading. Higher load cycles of 88.2 kips (392.3 kN, 60%) and 117.6 kips (523.1 kN, 80%) followed, with each level repeated three times to evaluate the monotonic behavior. The same loading protocol was applied to the non-rebar specimens (Stud and Ring) for consistency with the addition of a single loading-unloading cycle conducted at 139.7 kips (621.4 kN) and 209.3 kips (931.0 kN), corresponding to 40% and 60% of the steel pipe pile connection capacity.

The rebar-connection specimens were loaded up to 276 kips (1228 kN), while the non-rebar specimens were initially tested to the same limit. Based on favorable observations from the rebar tests, the non-rebar specimens were further tested up to 301 kips (1340 kN), the maximum capacity of the actuator. At specific loading levels, the loading was paused, and the crack development on specimen were identified and marked.

5.3 INSTRUMENTATION AND DATA ACQUISITION

The instrumentation strategy for this experimental program was designed to capture both localized and global responses of the steel pipe pile-to-bent cap connections under repeated unidirectional lateral loading. The instrumentation types used in the tests included strain gauges attached to the reinforcement, steel elements and concrete surface as well as displacement transducers and rotation meters positioned at critical locations such as the pile–to-cap interface and different heights. All sensors were integrated into the test setup to monitor and record structural responses, and calibrated to ensure accuracy.

Electrical resistance strain gauges (120 Ω) were bonded to the surface of the #11 rebars in the Headed, Hooked, and Straight specimens, with gauges positioned near the pile-to-cap interface and at mid-embedment depth to monitor tensile and compressive strains and detect yielding or bond slip. The same type of strain gauges was also installed on the outer surface of the steel pipe piles, at various depths above and below the pile-to-cap interface, to measure strains. In addition, strain gauges were welded to the shanks of select shear studs in the Stud specimen to assess load transfer efficiency and stud-to-concrete interaction. The strain gauge layouts are given in Figure 5-3.

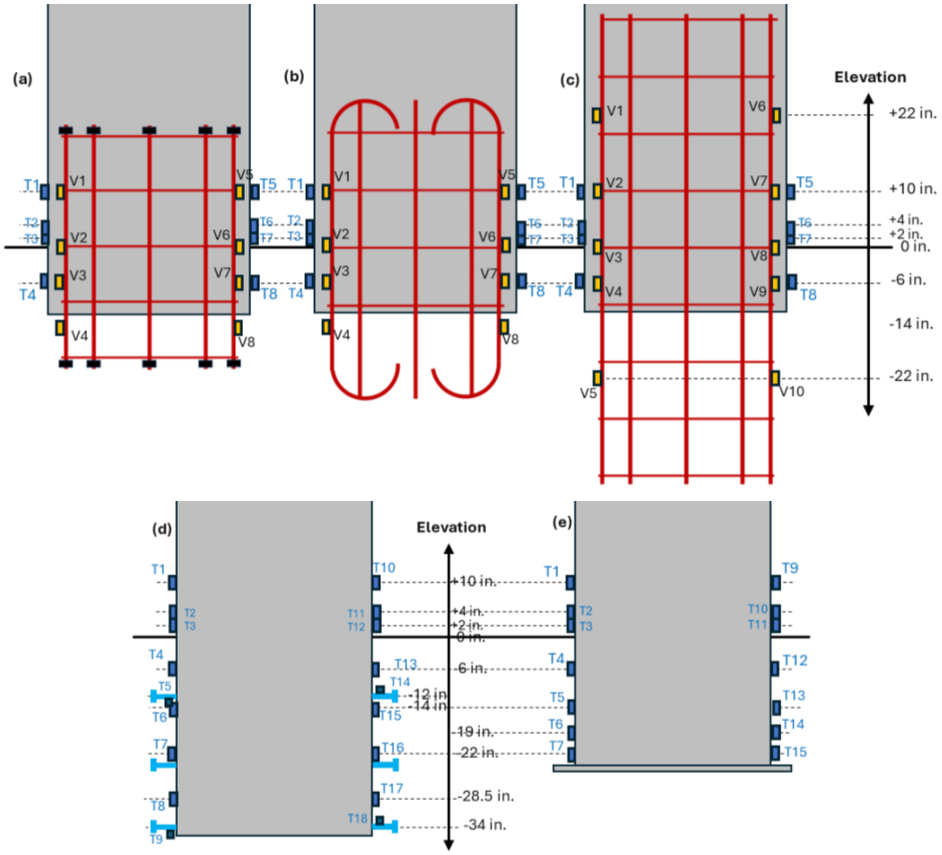
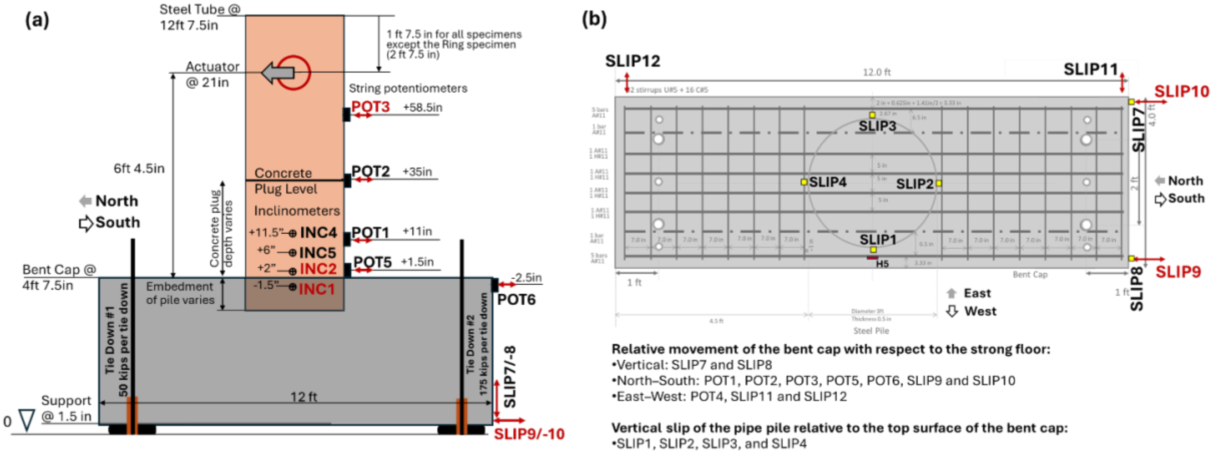


Figure 5-3 Strain gage layout of (a) Test 1 - Headed, (b) Test 2- Hooked, (c) Test 3- Straight, (d) Test 4- Stud, and (e) Test 5- Ring.

Displacement transducers were used to capture global lateral drift by mounting four strings horizontally at the load application plane to measure the lateral displacement relative to the strong floor. Vertical slip meters were installed at the pile tips to record any uplift or settlement during loading, and pairwise slip meters were positioned diagonally across the pile-to-cap interface to detect relative slip between components. High-precision inclinometers attached to the bent cap and pile surfaces near the interface quantified rotational deformations, providing critical moment-rotation relationships to evaluate connection rigidity.

Furthermore, the hydraulic actuator used for applying the loading was equipped with a calibrated load cell and displacement transducers to record the applied lateral forces synchronized with the displacement measurements. Additional strain gauges and displacement transducers were installed on the collar to validate its effectiveness in transferring forces to the pipe pile. A high-speed data acquisition system, sampling all signals at 5 Hz for the Headed specimen and 20 Hz for other specimens (increased based on Test 1 experience to improve resolution during load holds), continuously recorded applied loads, displacements, and strain measurements to capture nonlinear behavior during yielding. This comprehensive instrumentation and data collection framework provided a robust foundation for evaluating the performance of the various connection configurations under conditions that closely simulate actual bridge loading scenarios. A sketch of the instrumentation layout is provided in Figure 5-4, where the displacement transducers are designated as POT, slip meters with S, and inclinometers with INC. Additionally, a picture of the test setup is shown in Figure 5-5.



Relative movement of the bent cap with respect to the strong floor:
- Vertical: SLIP7 and SLIP8
- North–South: POT1, POT2, POT3, POT5, POT6, SLIP9 and SLIP10
- East–West: POT4, SLIP11 and SLIP12

Vertical slip of the pipe pile relative to the top surface of the bent cap:
- SLIP1, SLIP2, SLIP3, and SLIP4

Figure 5-4 Displacement and Rotation Sensor Layouts–(a) Side View, (b) Top View.

Figure 5-5 Test Setup Photo from Test 3- Straight

With the full-scale specimens fabricated and instrumentation configured; the test program proceeded to evaluate each connection under increasing lateral loads. The following section presents the resulting structural responses, including displacements, rotations, and strain data, and compares the performance of the five connection

Chapter 6

EXPERIMENTAL RESULTS AND DISCUSSIONS

6.1 INTRODUCTION

This chapter covers the structural response of the five full-scale pipe pile-to-bent cap connection specimens and provides a comprehensive comparison. The test results of each specimen by reporting the displacements, rotations, and strains against the applied load or moment. The results include all the repeated unidirectional lateral conducted during the tests. Instrumentation included strain gauges, displacement transducers (POTs), and inclinometers as detailed in Figure 5-3 and Figure 5-4.

The reported displacements represent the net deformation which was calculated by subtracting the bent cap translation (POT6) from the pile measurement (POT3). The net deformation was measured at the 58.5-in (1486-mm) level, excluding relative slip at the bent cap interface by subtracting the displacement measured at the bottom of the bent cap from the displacement recorded at 58.5 in (POT3). The reported rotation reflects the net rotation, excluding any relative rotation of bent cap beam. It was measured at +2 in (50.8 mm) above the pile-to-cap interface using sensor INC2. The net rotation was calculated by subtracting the rotation measured at the top of the bent cap beam (INC1) from the rotation recorded at the +2-in level (INC2).

The plots in this section include reference lines representing the service load level and estimated connection capacities. The capacity predictions used as reference lines in the following figures represent conservative lower-bound estimates: rebar-based connection capacities were calculated from the RC core section alone, excluding steel pipe contribution; mechanical anchorage capacities excluded concrete core contribution. Consequently, the observed exceedance of predicted capacities reflects both actual reserve strength and the intentional conservatism of the prediction methodology.

6.2 TESTS 1, 2 AND 3 – HEADED, HOOKED, AND STRAIGHT

The first three tests were conducted on the specimens with rebar-based connections, such as headed, hooked and straight rebars. Figure 6-1a, Figure 6-2a, and Figure 6-3a present the load-displacement responses of the headed, hooked and straight specimens, respectively. The vertical axis represents the force applied by the actuator, while the horizontal axis represents the net deformation at the 58.5-inch level. The specimens were loaded to a maximum force of 276 kips (1228 kN) and reached a net displacement of 0.903 inches (22.9 mm) in Test 1 - Headed, 0.973 inches (24.7 mm) in Test 2 – Hooked, and 0.995 inches (25.3 mm) in Test 3 - Straight. The testing stopped at that level due to the deformations observed in the steel fixtures mounting the actuator to the specimen.

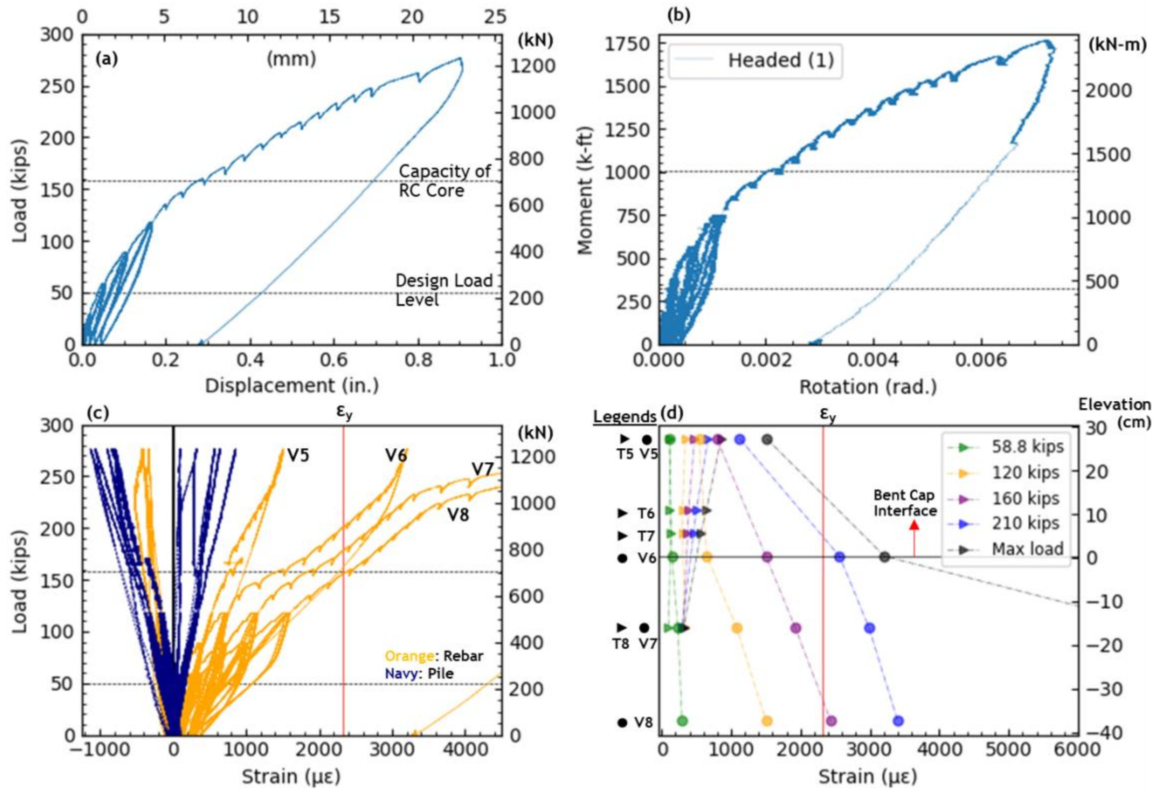


Figure 6-1 Headed (1) specimen Responses: (a) Lateral load-displacement, (b) moment-rotation, (c) load-strain and (d) load-tension side strain. Dashed line: service load; Solid line: estimated nominal strength of connection; Data: net displacement or net rotation response.

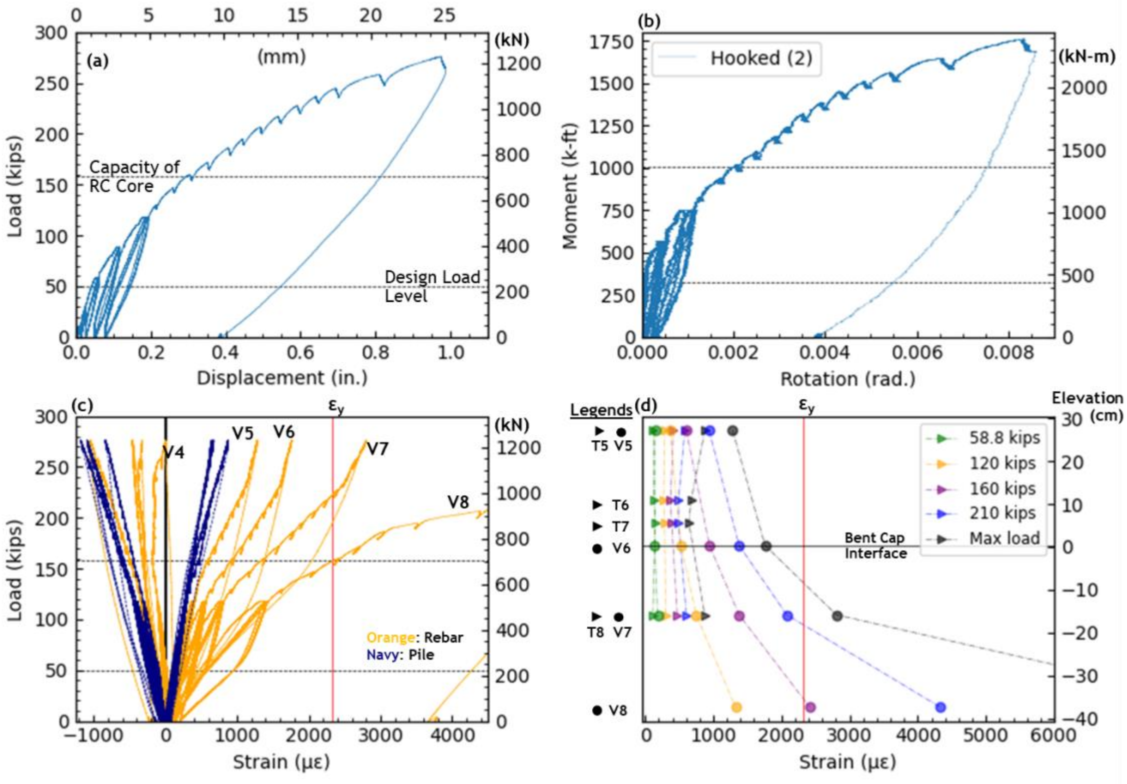


Figure 6-2 Hooked (2) specimen Responses (a) Lateral load-displacement, (b) moment-rotation, (c) load-strain and (d) load-tension side strain. Dashed line: service load; Solid line: estimated nominal strength of the connection; Data: net displacement or net rotation response.

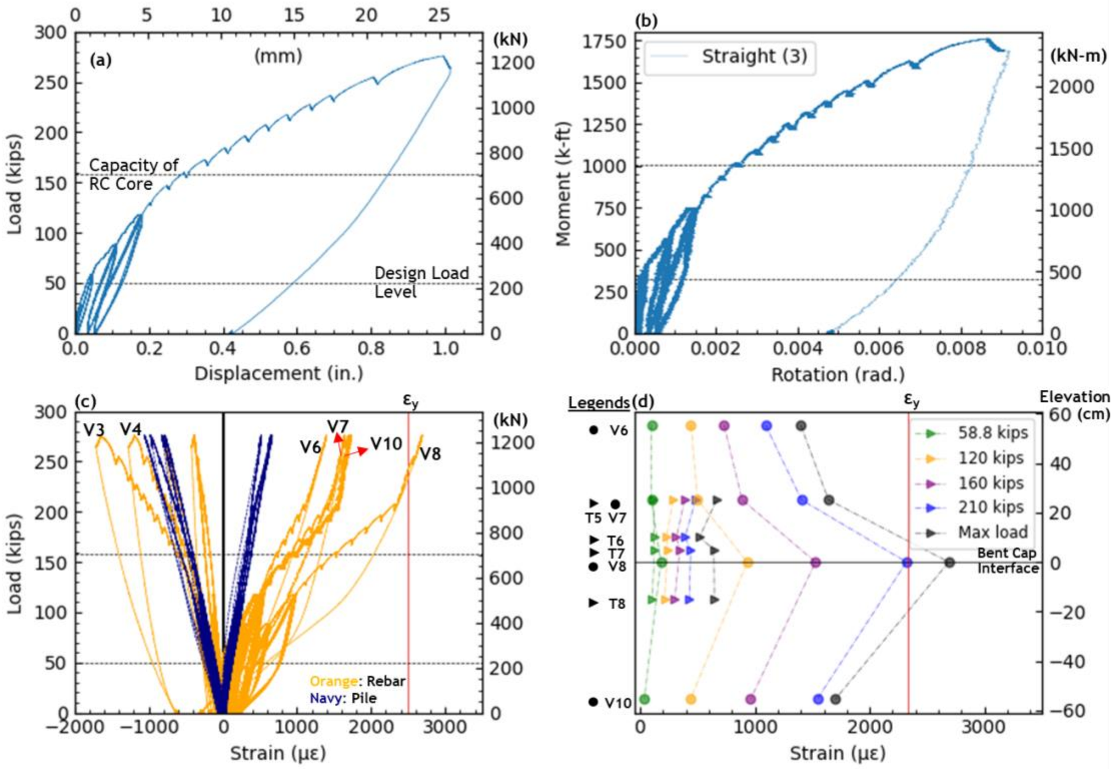


Figure 6-3 Straight (3) specimen Responses (a) Lateral load-displacement, (b) moment-rotation, (c) load-strain and (d) load-tension side strain. Dashed line: service load; Solid line: estimated nominal strength of the connection; Data: net displacement or net rotation response.

The overall response of each specimen exhibited a gradual transition from elastic to nonlinear behavior. The initial parts of the lateral load responses, up to 50 kips, remained nearly linear elastic with minimal residual displacement after completing each load cycle. This response confirmed that the RC section and its rebar connections were within the elastic range. Beyond this load level, the response of each specimen transitioned into the nonlinear regime, characterized by stiffness degradation following the onset of cracking in the RC section and bent cap beam. The sawtooth pattern observed in the nonlinear response was attributed to intermittent load drops during visual crack inspections and markings, as the actuator operated under displacement-controlled loading.

The moment-rotation graph based on inclinometer readings on the pipe pile and bent cap beam are presented in Figures Figure 6-1b, Figure 6-2b, and Figure 6-3b for headed, hooked and straight rebar connections, respectively. The vertical axis represents the reported moment, calculated as the product of the actuator-applied force and the moment arm measured from the pipe-to-cap connection interface, 6.375 ft (1943 mm). The horizontal axis represents the net rotation measured at the +2-inch elevation level on the pile. The moment-rotation response exhibited a similar trend as the load-displacement response, with an initial near linear-elastic response up to 50 kips that was followed by progressive softening due to concrete cracking and yielding of the steel rebars. The reduction in stiffness was more pronounced beyond the calculated nominal strength of the RC section, which coincides with the initiation of yielding in the steel reinforcement.

Figure 6-1c, Figure 6-2c, and Figure 6-3c illustrate the applied load-strain relationships measured on the steel pipe and rebars. The plotted strains include those measured on the outer surface of the steel pipe (navy curves) and rebars (orange curves), where positive values indicate tensile strain and negative strains represent compressive strain. In addition to the horizontal lines corresponding to the calculated service load level and capacity force levels, a vertical line marks the yield strain of the steel rebars (67.5 ksi / 29,000 ksi = 2328 με). Several strain gages on the rebars were labelled on the graph as those approached or exceeded the yield strain, particularly those in the close proximity of the interface between the pipe pile-to-bent cap and resisting tensile stresses. The onset of yielding in the steel rebars aligned with the calculated RC section nominal strength, confirming that the significant stiffness degradation observed at this stage resulted from rebar yielding. Prior to this load level, the nonlinear response primarily attributed to concrete cracking in the pile and bent cap beam, where minor slip of the pipe might have contributed to the stiffness reduction.

The steel pipe strains reached about 1000 με for all three specimens, indicating notable pipe contribution despite the limited embedment depth of 1 ft. The bent cap longitudinal reinforcement passing through the pipe might have contributed to the participation of the pipe in the load. The strain curves of the pipe generally exhibited a linear response, suggesting minimal slip between the outer pipe and the inner RC section.

Figure 6-1d, Figure 6-2d, and Figure 6-3d present the tension-side strain profiles (including both pipe pile and rebar gauges) at certain load levels for the Headed, Hooked, and Straight specimens, respectively. In the Headed and Hooked specimens, rebar yielding begins at 160 kips (711.7 kN), while in the Straight specimen, the yielding starts at 210 kips (934.1 kN). The highest strains for the Headed and Hooked specimens occur at the bottom edge of the rebars, whereas in the Straight specimen they appear at the interface level. Note that the strain gauge (V9), located between the interface and the bottom portion of the Straight specimen, malfunctioned; therefore, no strain data was recorded in that region.

Figure 6-6a–c present the surface crack patterns observed on the bent cap beam for the headed, hooked and straight rebar connections, respectively. The cracks are categorized by different load levels which they formed to observe the evolution of crack patterns. For all rebar-based connection configurations, the crack formation sequence was such that minor vertical cracks appeared at load levels below the service load level, which widened and extended until 120 kips (533.8 kN). As the load further increased, diagonal cracks formed and propagated towards the sides of the steel pipe. This diagonal cracking pattern suggests the potential initiation of a cone-shaped failure mechanism.

Overall, the Headed, Hooked, and Straight specimens demonstrated satisfactory structural performance up to the maximum applied load of 276 kips. The service load level was achieved with minimal damage, and the calculated RC section nominal strength was exceeded by a significant margin. Despite the shallow 1 ft embedment (0.33D), all three rebar systems developed significant moment resistance, confirming that the 'pinned' assumption is conservative for these configurations. However, because testing was terminated before failure due to fixture limitations, ultimate capacity, ductility, and failure modes remain uncharacterized. The connection also effectively transferred the lateral load and moment with adequate stiffness. The enhanced performance beyond the expected strength values was attributed to the steel pipe contribution that was not accounted in the nominal strength calculations. These findings validate the use of headed rebars for pipe pile-to-bent cap connections, demonstrating their suitability for applications requiring robust structural performance.

6.3 TESTS 4 – STUD

The fourth test was conducted on a specimen incorporating an anchorage-based connection utilizing shear studs. Figure 6-4a presents the lateral load-net displacement response of the Stud specimen, encompassing the full range of load cycles. The net displacement was determined using the same methodology as described for the rebar-based specimens. The steel fixtures mounting the actuator to the specimen were reinforced before the test to permit higher actuator loading. The specimen was subjected to a maximum applied force of 301 kips (1340 kN), reaching a net displacement of 0.457 inches (11.6 mm) before the test was terminated due to the actuator reaching its force capacity.

The overall response of the Stud specimen exhibited significantly higher stiffness compared to the rebar-based specimens, as evidenced by the attainment of comparable load levels at considerably smaller displacements. The load-displacement response transitioned gradually from an elastic to a nonlinear regime with increasing loads. The initial portion of the response demonstrated a nearly linear-elastic behavior that extended beyond 50 kips, with minimal residual displacement observed after each load cycle.

Figure 6-4b illustrates the moment-rotation response, as measured by inclinometers mounted on the pipe pile and bent cap beam. The reported rotation was recorded at an elevation of 2 in (50.8 mm) from the bent cap interface. The moment-rotation response followed a similar trend to the load-displacement curve, initially exhibiting a near-linear elastic response up to 50 kips, followed by progressive softening. A notable stiffness softening occurred at an applied load of 220 kips (978.6 kN). Beyond the calculated nominal strength of the hollow pipe, the reduction in stiffness became more pronounced.

Figure 6-4c illustrates the applied load-strain relationships for the steel pipe (navy curves) and shear studs (orange curves). Unlike the rebar-based specimens, the Stud specimen did not exhibit yielding, even at the maximum load level. This absence of yielding suggests that the nonlinearities observed in the load-displacement or moment-rotation responses (e.g., softening at around 220 kips [978.6 kN]) may indicate slip between the pipe and concrete. The steel pipe strains reached approximately 1500 με, highlighting a higher contribution from the pipe compared to the rebar-based specimens, potentially due to the lack of rebars within the core. Strain gauges on the shear studs recorded strain levels exceeding 1000 με, demonstrating the active engagement of the shear studs as primary anchorage elements, with contributions from the deeply embedded (3 ft [914 mm]) steel pipe and longitudinal reinforcement in the bent cap passing through the pipe.

Figure 6-6d presents the surface crack patterns observed on the bent cap beam with projections of the approximate stud locations depicted on the surface. The cracks are categorized by the different load levels at which they formed to observe the evolution of crack patterns. Initially, short vertical cracks appeared at load levels below the service-load threshold, gradually widening and extending to reach the depth of the embedded pile as the load increased to 210 kips (934.1 kN). At higher load levels, steep diagonal cracks developed on the side opposite to the applied load, suggesting the potential initiation of a half-cone failure mechanism if the load had been further increased.

Figure 6-4d shows the tension-side strain profiles. including both pipe pile and shear studs (T14 and T18), at certain load levels. The pile strains at the connection interface are noticeably lower than the those measured approximately seven inches above and below this interface. Meanwhile, the shear stud readings highlight their efficient role in the overall load-transfer mechanism.

Overall, the Stud specimen exhibited relatively higher stiffness and reduced damage accumulation at equivalent load levels. The service load level was achieved with minimal damage, and even at the maximum applied load of 301 kips (1338.9 kN), the connection maintained its integrity. These results validate the effectiveness of shear studs as anchorage elements, confirming their viability for forming robust pipe pile-to-bent cap connections with enhanced strength and stiffness compared to the rebar-based connections.

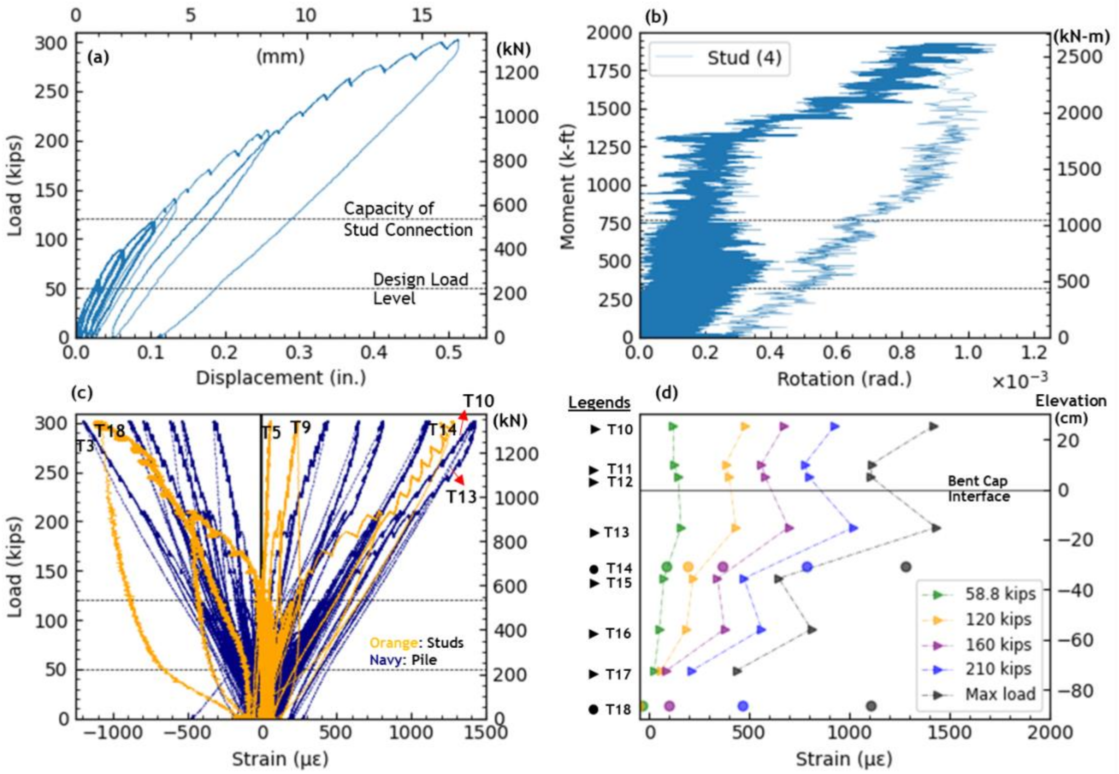


Figure 6-4 Stud (4) specimen Response: (a) Lateral load-displacement, (b) moment-rotation, (c) load-strain and (d) load-tension side strain. Dashed line: service load; Solid line: estimated nominal strength of the connection; Data: net displacement or net rotation response.

6.4 TESTS 5 – RING

The fifth and final test was conducted on a specimen incorporating an anchorage-based connection featuring an annular ring plate welded to the end of the pipe. Figure 6-5a presents the lateral load-net displacement response of the Ring specimen, capturing the full range of load cycles. The specimen was subjected to a maximum applied force of 301 kips (1340 kN), reaching a net displacement of 0.650 inches (16.5 mm) before the test was terminated due to the actuator reaching its force capacity.

Similar to the previous specimens, the load-displacement response transitioned gradually from an elastic to a nonlinear regime as the load increased. The initial portion of the response demonstrated nearly linear-elastic behavior, extending up to approximately 50 kips, with minimal residual displacement observed after each load cycle. Beyond this load level, the response entered the nonlinear regime, where stiffness degradation became apparent following the onset of cracking in the bent cap beam.

Figure 6-5b illustrates the moment-rotation response, measured by inclinometers mounted on the pipe pile and bent cap beam. The reported rotation was recorded at an elevation of 2 in (50.8 mm) from the bent cap interface. Due to cable malfunctions, only the final two load cycles were recorded for this test.

Rotational stiffness values reported for the Ring specimen (Table 6-1 and Table 6-2) are therefore based on limited data and should be interpreted with caution; direct rotational comparisons with other specimens are provisional. The moment-rotation response followed a similar trend to the load-displacement curve, initially exhibiting a near-linear elastic response up to 50 kips, followed by progressive softening. A notable deviation in response occurred at an applied load of 200 kips (889.6 kN), potentially indicating slip between the pipe and concrete, as no yielding was observed in other elements. Beyond the calculated nominal strength of the hollow pipe, stiffness degradation became more pronounced.

Figure 6-5c presents the applied load-strain relationships measured on the steel pipe (navy curves) and the ring plate (orange curves). No yielding was observed in the Ring specimen, as the measured strains remained significantly below the yield strain calculated from the measured material properties. The steel pipe strains approached 1500 με, indicating a substantial contribution from the pipe, attributed to the absence of rebars within the core. Strain gauges mounted on the annular ring plate recorded strain levels approaching 1000 με, demonstrating the active engagement of the ring plate, with potential contributions from the embedded (2 ft [610 mm]) steel pipe and the longitudinal reinforcement in the bent cap passing through the pipe.

Figure 6-5d presents the tension-side strain profiles of the pipe pile at certain load levels. The pile strains at the connection interface are noticeably lower than those measured approximately seven inches above and below this interface. Within the embedded portion, the pile strain distribution shows elevated levels as it nears the interface.

Figure 6-6e presents the surface crack patterns observed on the bent cap beam. The cracks are categorized by the load levels at which they formed, illustrating the evolution of crack patterns. As observed in the previously tested specimens, short vertical cracks initially developed at load levels below the service-level threshold, gradually widening and extending toward the depth of the embedded pile as the load increased to 210 kips (934 kN). This led to the formation of steep diagonal cracks on the side opposite the applied load. As the load increased further, diagonal cracks propagated toward the sides of the steel pipe. This diagonal cracking pattern suggests the potential initiation of a cone-shaped failure mechanism.

Overall, the Ring specimen demonstrated exhibited a slightly more flexible response than the Stud specimen. The Ring specimen displayed high stiffness and reduced damage accumulation at equivalent load levels compared to the rebar-based specimens. The service load level was achieved with minimal damage, and even at the maximum applied load of 301 kips (1340 kN), the connection exhibited robust performance. These results validate the effectiveness of the annular ring plate connection as an anchorage element, confirming its viability for forming robust pipe pile-to-bent cap connections with greater strength and stiffness compared to the rebar-based connections. The superior stiffness of the Stud and Ring connections reflects the synergy of mechanical anchorage and the deeper embedment required to accommodate them, verifying that these 'fixed' detailing standards successfully achieve rigid behavior compared to the shallow 'pinned' rebar details.

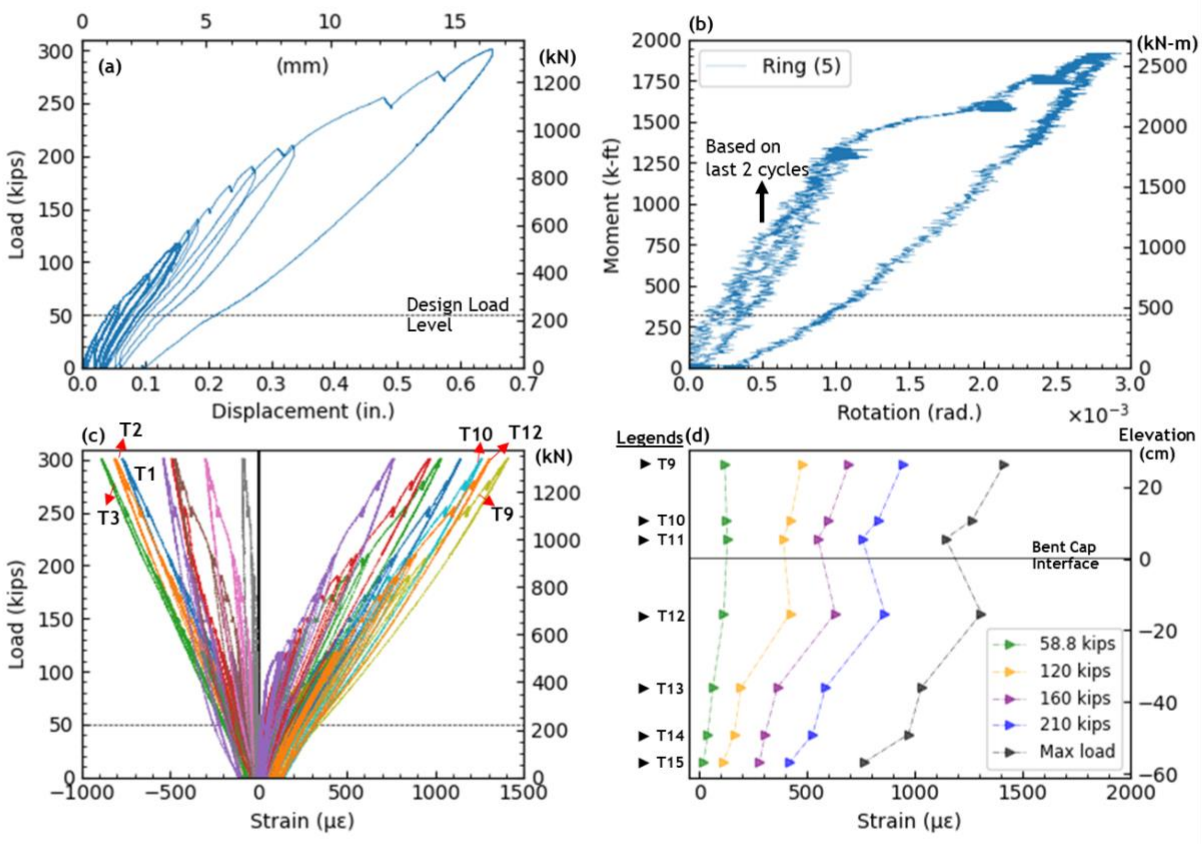
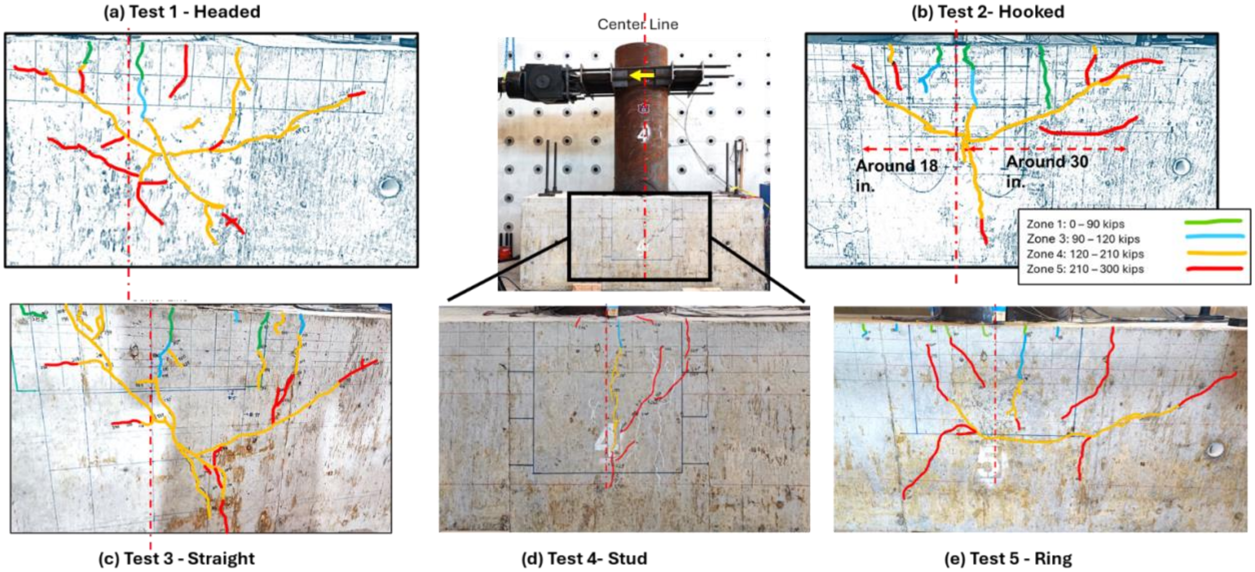


Figure 6-5 Ring (5) specimen Response: (a) Lateral load-displacement, (b) moment-rotation, (c) load-strain and (d) load-tension side strain. Dashed line: service load; Solid line: estimated nominal strength of the connection; Data: net displacement or net rotation response.


(a) Test 1 - Headed

(b) Test 2- Hooked

(c) Test 3 - Straight

(d) Test 4- Stud

(e) Test 5 - Ring

Figure 6-6 Identified crack patterns for tested specimens

6.1 DISCUSSION OF TEST RESULTS

The test results revealed distinct behavioral characteristics for each connection configuration. To better understand the implications of these findings for bridge design and connection detailing, this section summarizes and discusses key performance metrics, explores underlying mechanisms, and links experimental observations to design recommendations.

Figure 6-7a presents the envelope lateral load-displacement responses of each of the five tested connection configurations, namely Test (1) - Headed, Test (2) - Hooked, Test (3) - Straight, Test (4) - Stud, and Test (5) - Ring. The envelope response was obtained by tracking the maximum load response corresponding to each displacement, where the displacement was measured at 58.5 in (1486 mm). As expected from the individual responses, non-linear load-displacement relationships were observed for all connection configurations. Additionally, secant stiffnesses (slopes) obtained from the load-displacement responses were also plotted in Figure 6-7b.

Figure 6-8a illustrates the envelope moment-rotation responses for all the connection configurations. In the plots, the moment was calculated at the pipe-to-cap interface and the net rotation measured at 2 in (INC2-INC1). Figure 6-8b illustrates the secant stiffnesses (slopes) calculated based on the moment-rotation responses, where a non-linear relationship was observed between applied load and rotation. The rotations measured near the base primarily reflect the rotational stiffness of the connection, while displacements recorded near the load level capture the combined stiffness of both the connection and the pipe lateral stiffness. These two distinct comparisons provide valuable insights into the contributions of each stiffness component.

Table 6-1 presents the secant stiffness values based on the displacement or rotation response at applied load levels of 58.8 kips (261.6 kN), 160 kips (711.7 kN) and 276 kips (1228 kN). Table 6-2 presents the differences in secant stiffness at indicated load levels relative to the Straight specimen. The Stud system exhibited the highest secant stiffness. This performance is attributed to the combined contribution of the stiff mechanical anchorage and the deep (3 ft [914 mm], 1.0D) embedment required for its implementation. By contrast, the Rebar systems, limited to the standard 1 ft (0.33D) embedment, exhibited lower stiffness. This highlights the trade-off inherent in DOT designs: the construction efficiency of shallow typologies comes with a reduction in rotational stiffness compared to deeper mechanical anchorage systems. Note that Ring specimen rotational stiffness values are based on incomplete data (final two cycles only) and should be interpreted with caution.

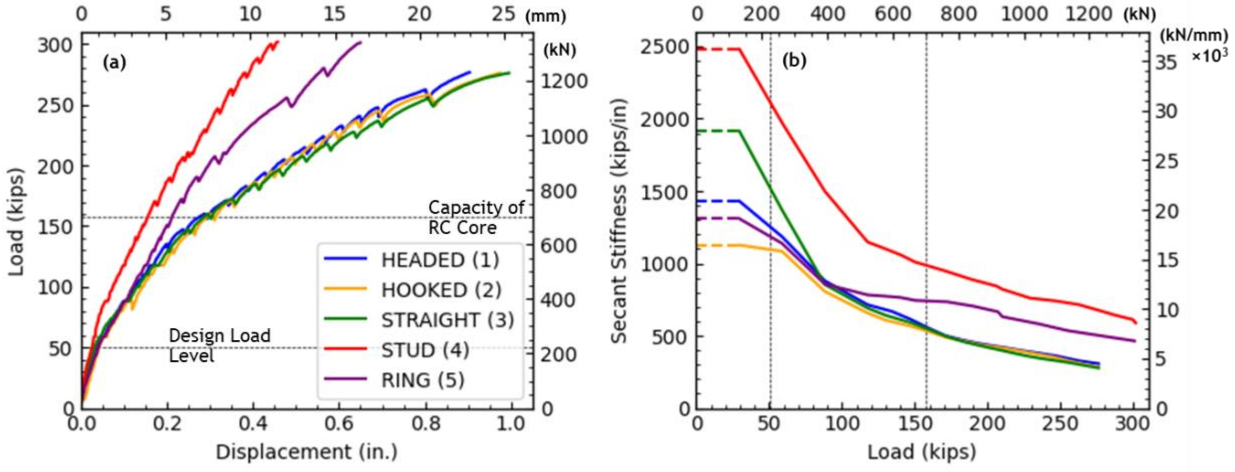


Figure 6-7 (a) Enveloped lateral load-displacement trends and (b) displacement-based secant stiffnesses for all specimens

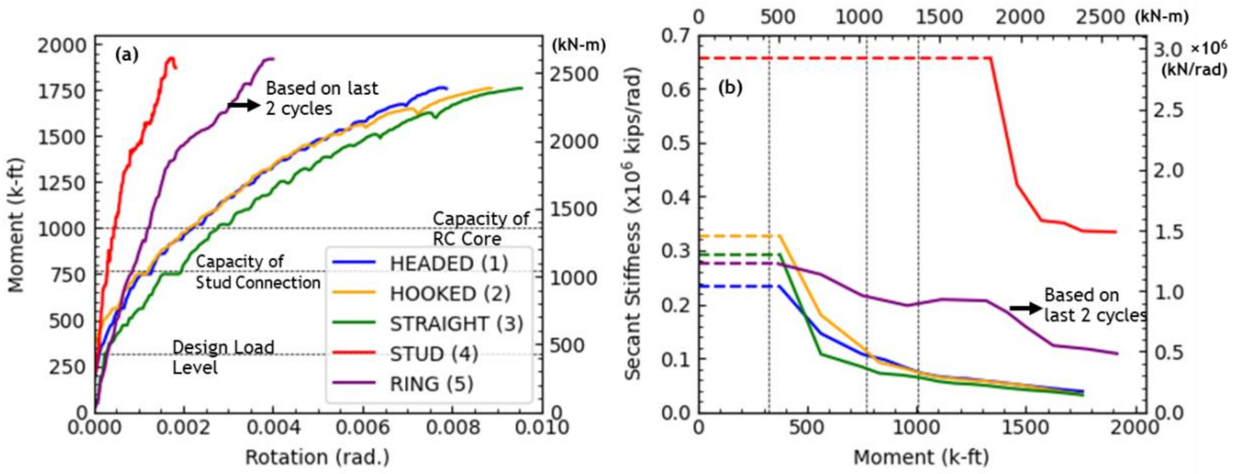


Figure 6-8 (a) Enveloped moment-rotation trends and (b) rotation-based secant stiffnesses for all specimens.

Table 6-1 Secant Stiffnesses at Indicated Load Levels

Secant Stiffness	**At 58.8 kips**		**At 160 kips**		**At 275 kips**	
	Displacement (kN/mm)	**Rotation (kN/rad)**	**Displacement (kN/mm)**	**Rotation (kN/rad)**	**Displacement (kN/mm)**	**Rotation (kN/rad)**
Headed (1) **(1 ft Embed)**	17 297	1 033 584	8 027	321 598	4 482	169 902
Hooked (2) **(1 ft Embed)**	15 821	1 452 945	7 696	319 577	4 137	143 952
Straight (3) **(1 ft Embed)**	20 000	1 300 751	7 904	282 917	4 051	140 681
Stud (4) **(3 ft Embed)**	28 265	2 924 290	13 893	2 924 290	9 380	1 494 421
Ring (5) **(2 ft Embed)**	16 617	1 224 285	10 864	879 004	7 257	520 150

Notes: Secant stiffness calculated as load (or moment) at indicated level divided by corresponding displacement (or rotation) from envelope response. Ring rotational stiffness based on final two load cycles only due to instrumentation malfunction.

Table 6-2 Differences in Secant Stiffness at Indicated Load Levels Relative to Straight Specimen

	At 58.8 kips		**At 160 kips**		**At 275 kips**	
Secant Stiffness	**Displacement**	**Rotation**	**Displacement**	**Rotation**	**Displacement**	**Rotation**
Headed (1) **(1 ft Embed)**	-14%	-21%	2%	14%	11%	21%
Hooked (2) **(1 ft Embed)**	-21%	12%	-3%	13%	2%	2%
Straight (3) **(1 ft Embed)**	0%	0%	0%	0%	0%	0%
Stud (4) **(3 ft Embed)**	41%	125%	76%	934%	132%	962%
Ring (5) **(2 ft Embed)**	-17%	-6%	37%	211%	79%	270%
Note: Ring rotational stiffness based on final two load cycles only; displacement-based values are complete.						

Regarding crack patterns, the rebar-based connections (i.e. Headed, Hooked and Straight) showed earlier development of diagonal cracks, propagating deeper into the bent cap beam and resembling the initiation of a pullout cone. Stress concentrations were particularly noticeable at the embedded ends of the rebars. These patterns indicate progressive load transfer dependent on bar development and concrete bearing. In contrast, Stud and Ring specimens exhibited delayed cracking and localized failures, with lower strains in anchorage elements and greater pipe engagement, confirming a more efficient load transfer relying on the steel pipe and mechanical anchorage in the absence of internal reinforcement. These observations suggest that stirrups should be provided in the connection zone for effective crack control.

In the rebar-based specimens with headed, hooked, and straight rebar connections, higher tensile strains were observed not directly at the pipe–to–cap interface, but further down the embedded length of the rebar within the pile plug. This indicates that load transfer occurred progressively along the rebar, with engagement extending beyond the intended development length. The distribution of strain suggests that the anchorage mechanisms may not have been fully effective near the interface, making the connections more reliant on bond strength alone, an approach that can be less efficient under higher applied loads. Conversely, the Stud and Ring specimens, with their welded connections, exhibited substantially lower strain levels in both the steel pipe and the pipe anchorage elements, all remaining below yield.

Differences in lateral and rotational stiffness among the five connection configurations significantly affect bridge system design. Higher stiffness, as seen in the Stud and Ring specimens, enhances pile–cap fixity, shifting force distribution by increasing moment resistance at the connection and potentially reducing demands on elements like girders or adjacent piles.

Experimental results demonstrated that all connection systems effectively transferred moments, regardless of whether embedment depths were one-third (Rebar), two-thirds (Ring), or one full pile diameter (Stud). Notably, even at the 1-ft embedment (l_e/D = 0.33), rebar-based systems achieved rotational stiffness values of 140,000–170,000 kN·m/rad at maximum load (Table 6-1). This magnitude represents substantial fixity, contradicting the common design assumption that shallow embedments (<0.5*D*) behave as ideal hinges with near-zero stiffness. These results confirm that the presence of the concrete plug and rebar cage generates significant moment transfer capability, even in minimal-embedment details. Ignoring this intrinsic fixity may lead to inaccurate force distribution estimates in the superstructure. Consequently, connections traditionally detailed as 'pinned' should be modeled with partial fixity in refined analyses.

For projects where drift control is critical, the Stud and Ring systems offer a verifiable pathway to high-stiffness connections. While this comes at the cost of deeper embedment (2–3 ft) compared to standard rebar details, the enhanced stiffness is a direct benefit of this deep embedment and mechanical anchor system topology. However, because ultimate capacities were not reached, conclusions regarding strength advantages remain tentative. Additionally, the prefabrication-friendly nature of these connections, which eliminates the need for reinforcement caging, may reduce field reinforcement placement compared to rebar-based alternatives, though construction time comparisons were not evaluated. These potential benefits must be weighed against welding requirements (qualified welders, inspection, field conditions) that were not evaluated in this study.

To ensure the tested specimens remained representative of actual field applications, embedment depths were not held constant but were selected based on the specific geometric requirements of each anchorage type. The Stud specimen utilized a 3.0 ft (1.0*D*) embedment to satisfy AASHTO 6.10.10.4.3 spacing for three rows of connectors, while rebar-based specimens utilized a 1.0 ft embedment to represent a lower-bound 'pinned-to-fixed' transition depth common in DOT standards. While this introduces varying embedment as a parameter, it allows for a comparison of standardized connection systems rather than isolated components.

As testing was terminated at the actuator's capacity limit (276–301 kips) prior to specimen failure, the ultimate strength and ductility limits of these systems remain unquantified. However, the results establish a proven lower-bound capacity for all systems that exceeds conventional section-based predictions by a factor of at least 1.8. This demonstrates a substantial safety margin for design, even without capturing the full post-peak behavior.

Chapter 7

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1 INTRODUCTION

This chapter covers the entirety of the research project that included the DOT survey and test results from the five full-scale pipe pile-to-bent cap connection specimens. It includes a summary of the work performed as well as conclusions and recommendations based on the data collected.

7.2 SUMMARY

This study was carried out to support ALDOT's consideration of steel pipe pile–to–reinforced concrete bent cap connection details for bridge substructures. The work had two main components. First, an extensive survey of state DOTs to document current practice was conducted, including typical embedment lengths, commonly used connection concepts, and the range of fixity assumptions adopted in design. Second, the survey findings were used to develop a full-scale experimental program that evaluates representative connection types under lateral loading.

Five full-scale connection configurations were fabricated and tested, which comprised three rebar-based details (headed, hooked, and straight bars) with shallow embedment, and two mechanical anchorage details (headed stud and annular ring) with deeper embedment. All specimens were detailed to be consistent with code-based requirements and were tested using a controlled lateral loading protocol. The instrumentation was selected to capture both the global response and the internal demand, including displacements/rotations and strain response. For the rebar-based configurations, concrete plug depth was selected to satisfy reinforcement development/anchorage requirements; for the mechanical anchorage configurations, reduced plug depths were evaluated. This report compiles the DOT survey results, specimen details and test procedures, and the measured response data in a consistent format to facilitate comparison across connection types and to support implementation-focused evaluation.

To facilitate implementation, Appendix C compiles the tested connection options and provides a parameterized detailing framework and for adaptation into ALDOT standard details.

7.3 CONCLUSIONS

Testing of five full-scale steel pipe pile–to–reinforced concrete bent cap connection specimens provided a comprehensive comparison of rebar-based and mechanical anchorage connection configurations representative of common DOT practice. The following conclusions are drawn:

1. All tested connection configurations sustained the maximum applied actuator loads up to 300 kips without specimen failure. For the rebar-based configurations, the measured strengths exceeded the calculated nominal flexural strengths by at least 1.8 times. Because testing was terminated due to equipment capacity rather than specimen limit states, the reported strengths represent lower-bound performance measures rather than ultimate capacities.
2. The mechanical anchorage configurations (headed stud and annular ring), which used deeper embedments (2–3 ft) to satisfy geometric and detailing constraints, exhibited substantially higher lateral stiffness than the shallow rebar-based configurations with 1ft. embedment. At peak loading, the headed stud system, having 3 ft embedment, exhibited approximately 2.3 times the stiffness of the straight rebar system, while the annular ring system (2 ft embedment) exhibited approximately 1.8 times. Overall, mechanical anchorage systems provided approximately twice the stiffness advantage relative to the shallow rebar-based systems.
3. Even the minimal 1 ft embedment (≈0.33D) rebar-based specimens demonstrated significant rotational stiffness (on the order of >100,000 kip-ft/rad) and reasonable moment transfer. These results indicate that idealized pinned assumptions may be conservative for hollow pipe piles with concrete plugs and shallow embedment, and that connection fixity should be considered when evaluating substructure force distribution and system response.
4. For the rebar-based configurations, plug depth was governed by reinforcement development/anchorage requirements, whereas the mechanical anchorage configurations were evaluated with reduced plug depths. Accordingly, the plug depths used in this study reflect practical details for the tested systems and can be refined for optimization in future studies.
5. Appendix C provides a draft implementation plan and LRFD-style guidance framework for ALDOT consideration, including parameterized detailing recommendations based on the tested configurations. The appendix is intended to serve as a baseline design framework for future refinement as additional validation becomes available.

7.4 RECOMMENDATIONS

Further work is recommended to refine design guidance, define governing limit states, and extend applicability beyond the tested configurations:

1. Additional testing is recommended to establish ultimate limit states, failure mechanisms, and post-peak ductility for the evaluated connection types. Because the present tests were terminated due to equipment limits, scaled testing with revised test setups is needed to fully quantify ultimate strength and deformation capacity.
2. Refined studies are recommended that target to isolate the effects of key detailing and geometry parameters (e.g., embedment depth, plug depth, and reinforcement

layout/clearance) and to evaluate long-term cyclic/fatigue performance and stiffness degradation under service-representative loading.

3. Nonlinear finite element modeling calibrated to the experimental response is recommended to expand the parameter space and support refined design procedures for constructability-preferred connection configurations.
4. A future testing program is also recommended to extend and validate the methodology for pile-cap (pile-to-footing) connections and foundations, and refine the Appendix C implementation framework through additional experimental or numerical research (ultimate/fatigue/expanded parameter space) before final implementation

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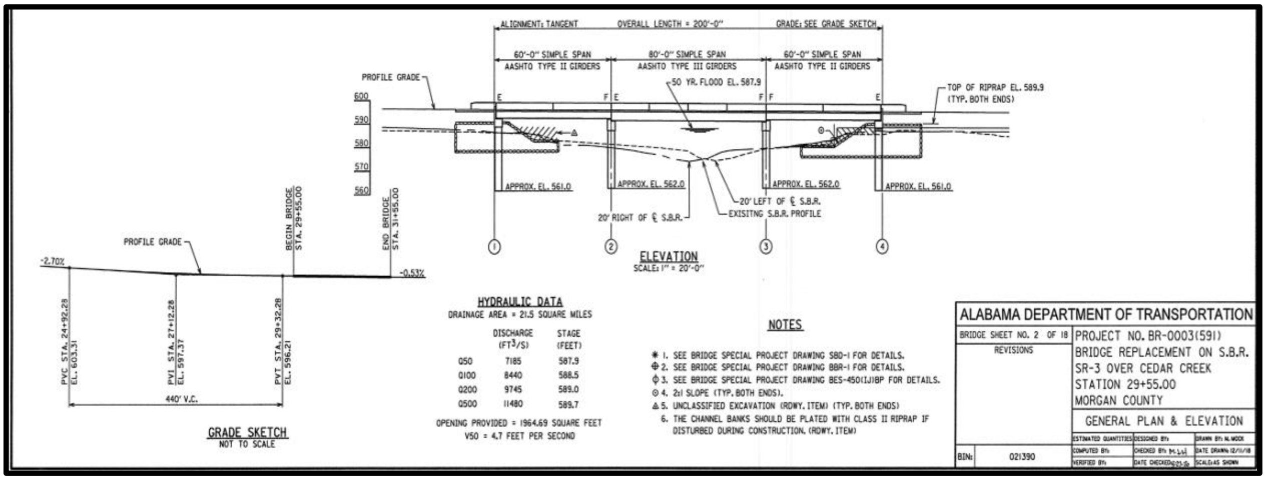
Appendix A: Bridge SR-3 Over Cedar Creek Station in Morgan County



HYDRAULIC DATA

DRAINAGE AREA = 21.5 SQUARE MILES

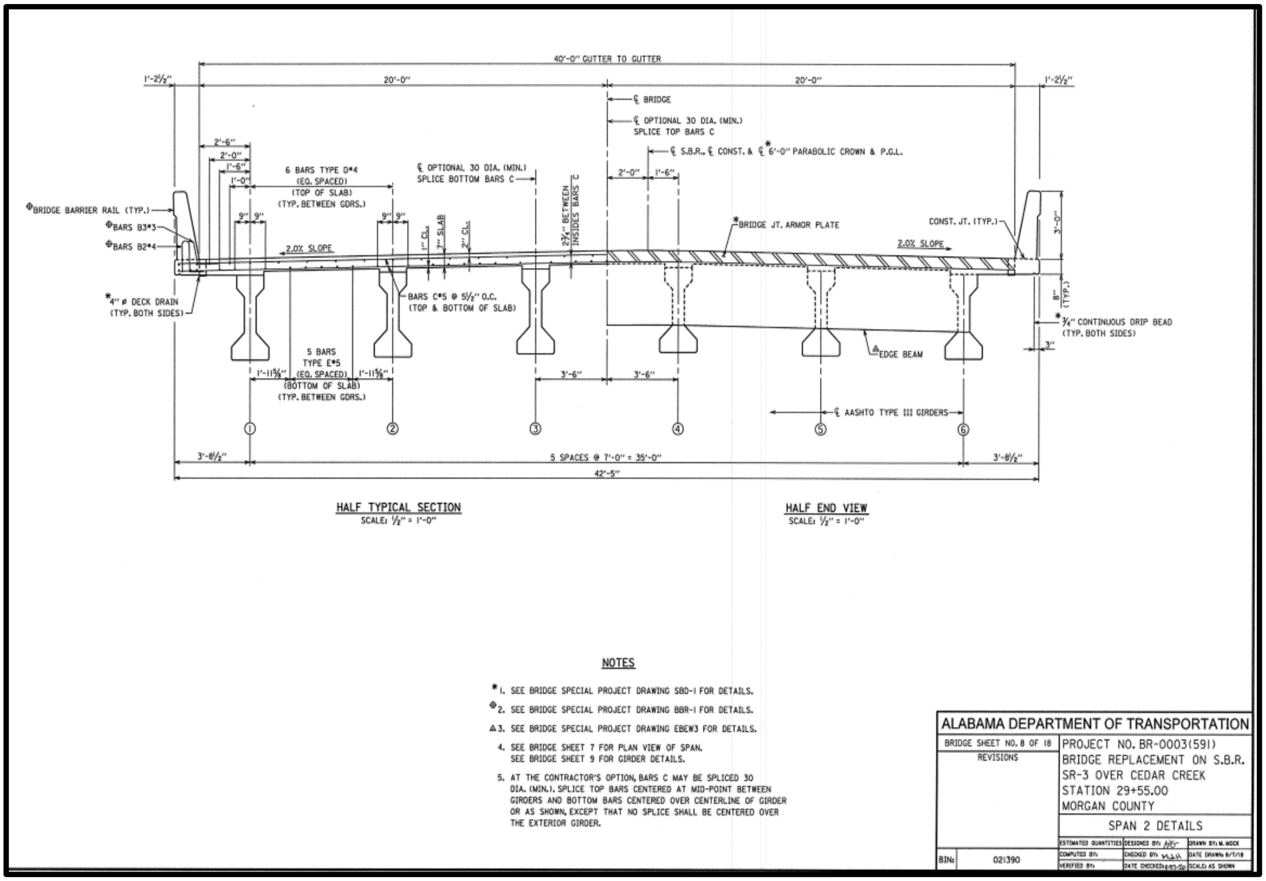
	DISCHARGE (FT³/S)	STAGE (FEET)
Q50	7185	587.9
Q100	8440	588.5
Q200	9745	589.0
Q500	11480	589.7

OPENING PROVIDED = 1964.69 SQUARE FEET
V50 = 4.7 FEET PER SECOND

NOTES

- ✱ 1. SEE BRIDGE SPECIAL PROJECT DRAWING SBD-1 FOR DETAILS.
- ⊕ 2. SEE BRIDGE SPECIAL PROJECT DRAWING BBR-1 FOR DETAILS.
- ϕ 3. SEE BRIDGE SPECIAL PROJECT DRAWING BES-450(IJ)BP FOR DETAILS.
- ⊙ 4. 2:1 SLOPE (TYP. BOTH ENDS).
- △ 5. UNCLASSIFIED EXCAVATION (RDWY. ITEM) (TYP. BOTH ENDS)
- 6. THE CHANNEL BANKS SHOULD BE PLATED WITH CLASS II RIPRAP IF DISTURBED DURING CONSTRUCTION. (RDWY. ITEM)

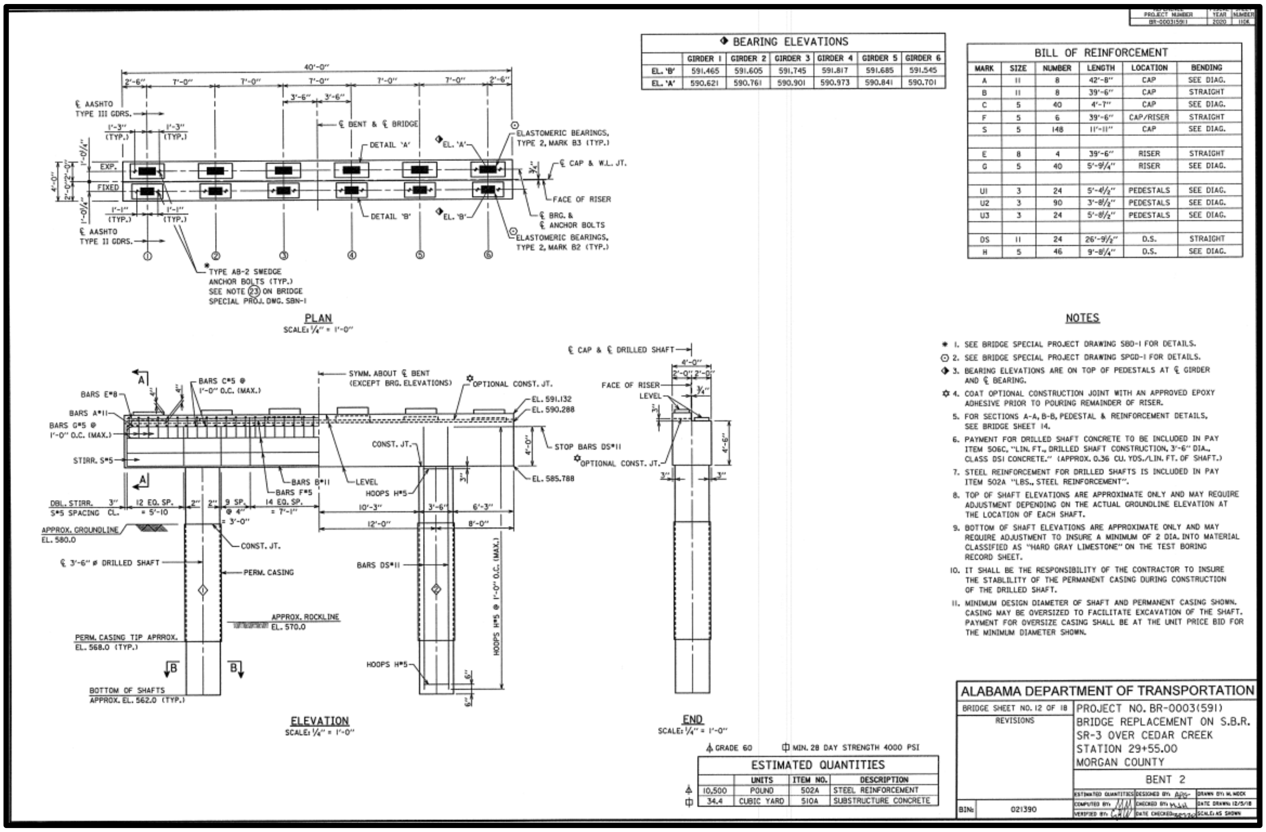
ALABAMA DEPARTMENT OF TRANSPORTATION
BRIDGE SHEET NO. 2 OF 18
REVISIONS
PROJECT NO. BR-0003(591)
BRIDGE REPLACEMENT ON S.B.R.
SR-3 OVER CEDAR CREEK
STATION 29+55.00
MORGAN COUNTY
GENERAL PLAN & ELEVATION
ESTIMATED QUANTITIES
DESIGNED BY:
DRAWN BY: M. MOCK
COMPUTED BY:
CHECKED BY: M.L.H
DATE DRAWN: 12/11/18
BIN: 021390
VERIFIED BY:
DATE CHECKED:
SCALE: AS SHOWN



NOTES

- ✱1. SEE BRIDGE SPECIAL PROJECT DRAWING SBD-1 FOR DETAILS.
- ⊕2. SEE BRIDGE SPECIAL PROJECT DRAWING BBR-1 FOR DETAILS.
- △3. SEE BRIDGE SPECIAL PROJECT DRAWING EBEW3 FOR DETAILS.
- 4. SEE BRIDGE SHEET 7 FOR PLAN VIEW OF SPAN.
  SEE BRIDGE SHEET 9 FOR GIRDER DETAILS.
- 5. AT THE CONTRACTOR'S OPTION, BARS C MAY BE SPLICED 30 DIA. (MIN.). SPLICE TOP BARS CENTERED AT MID-POINT BETWEEN GIRDERS AND BOTTOM BARS CENTERED OVER CENTERLINE OF GIRDER OR AS SHOWN, EXCEPT THAT NO SPLICE SHALL BE CENTERED OVER THE EXTERIOR GIRDER.

ALABAMA DEPARTMENT OF TRANSPORTATION
BRIDGE SHEET NO. 8 OF 18
REVISIONS
PROJECT NO. BR-0003(591)
BRIDGE REPLACEMENT ON S.B.R.
SR-3 OVER CEDAR CREEK
STATION 29+55.00
MORGAN COUNTY
SPAN 2 DETAILS
ESTIMATED QUANTITIES
DESIGNED BY:
DRAWN BY: M. MOCK
COMPUTED BY:
CHECKED BY: M.L.H
DATE DRAWN: 8/7/18
BIN: 021390
VERIFIED BY:
DATE CHECKED:
SCALE: AS SHOWN



PROJECT NUMBER	YEAR	NUMBER
BR-0003(591)	2020	110K

◆ BEARING ELEVATIONS

	GIRDER 1	GIRDER 2	GIRDER 3	GIRDER 4	GIRDER 5	GIRDER 6
EL. 'B'	591.465	591.605	591.745	591.817	591.685	591.545
EL. 'A'	590.621	590.761	590.901	590.973	590.841	590.701

BILL OF REINFORCEMENT

MARK	SIZE	NUMBER	LENGTH	LOCATION	BENDING
A	11	8	42'-8"	CAP	SEE DIAG.
B	11	8	39'-6"	CAP	STRAIGHT
C	5	40	4'-7"	CAP	SEE DIAG.
F	5	6	39'-6"	CAP/RISER	STRAIGHT
S	5	148	11'-11"	CAP	SEE DIAG.
E	8	4	39'-6"	RISER	STRAIGHT
G	5	40	5'-9 1/4"	RISER	SEE DIAG.
U1	3	24	5'-4 1/2"	PEDESTALS	SEE DIAG.
U2	3	90	3'-8 1/2"	PEDESTALS	SEE DIAG.
U3	3	24	5'-8 1/2"	PEDESTALS	SEE DIAG.
DS	11	24	26'-9 1/2"	D.S.	STRAIGHT
H	5	46	9'-8 1/4"	D.S.	SEE DIAG.

NOTES

* 1. SEE BRIDGE SPECIAL PROJECT DRAWING SBD-1 FOR DETAILS.

⊙ 2. SEE BRIDGE SPECIAL PROJECT DRAWING SPGD-1 FOR DETAILS.

◆ 3. BEARING ELEVATIONS ARE ON TOP OF PEDESTALS AT ℄ GIRDER AND ℄ BEARING.

✡ 4. COAT OPTIONAL CONSTRUCTION JOINT WITH AN APPROVED EPOXY ADHESIVE PRIOR TO POURING REMAINDER OF RISER.

5. FOR SECTIONS A-A, B-B, PEDESTAL & REINFORCEMENT DETAILS, SEE BRIDGE SHEET 14.

6. PAYMENT FOR DRILLED SHAFT CONCRETE TO BE INCLUDED IN PAY ITEM 506C, "LIN. FT., DRILLED SHAFT CONSTRUCTION, 3'-6" DIA., CLASS DS1 CONCRETE." (APPROX. 0.36 CU. YDS./LIN. FT. OF SHAFT.)

7. STEEL REINFORCEMENT FOR DRILLED SHAFTS IS INCLUDED IN PAY ITEM 502A "LBS., STEEL REINFORCEMENT".

8. TOP OF SHAFT ELEVATIONS ARE APPROXIMATE ONLY AND MAY REQUIRE ADJUSTMENT DEPENDING ON THE ACTUAL GROUNDLINE ELEVATION AT THE LOCATION OF EACH SHAFT.

9. BOTTOM OF SHAFT ELEVATIONS ARE APPROXIMATE ONLY AND MAY REQUIRE ADJUSTMENT TO INSURE A MINIMUM OF 2 DIA. INTO MATERIAL CLASSIFIED AS "HARD GRAY LIMESTONE" ON THE TEST BORING RECORD SHEET.

10. IT SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR TO INSURE THE STABLILITY OF THE PERMANENT CASING DURING CONSTRUCTION OF THE DRILLED SHAFT.

11. MINIMUM DESIGN DIAMETER OF SHAFT AND PERMANENT CASING SHOWN. CASING MAY BE OVERSIZED TO FACILITATE EXCAVATION OF THE SHAFT. PAYMENT FOR OVERSIZE CASING SHALL BE AT THE UNIT PRICE BID FOR THE MINIMUM DIAMETER SHOWN.

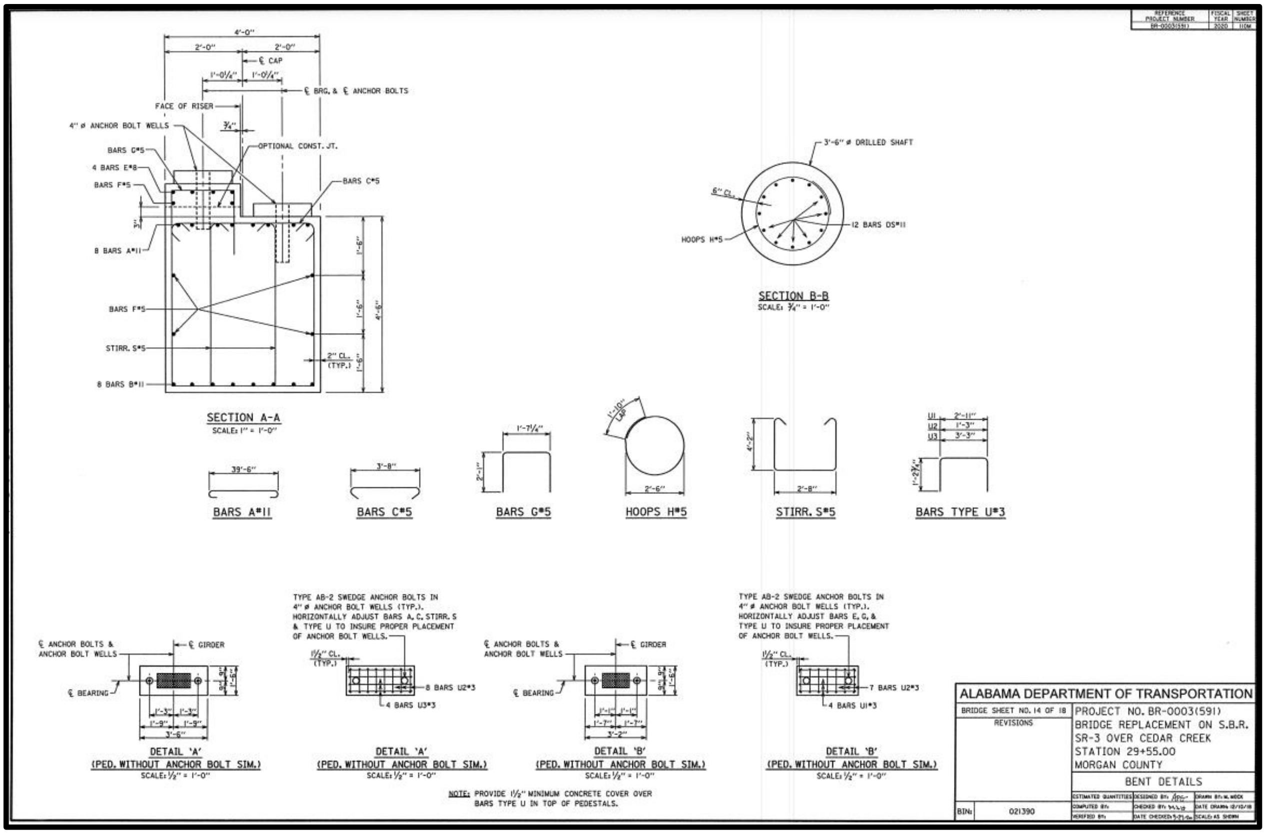
⍋ GRADE 60   ⌽ MIN. 28 DAY STRENGTH 4000 PSI

ESTIMATED QUANTITIES

		UNITS	ITEM NO.	DESCRIPTION
⍋	10,500	POUND	502A	STEEL REINFORCEMENT
⌽	34.4	CUBIC YARD	510A	SUBSTRUCTURE CONCRETE

ALABAMA DEPARTMENT OF TRANSPORTATION
BRIDGE SHEET NO. 12 OF 18
REVISIONS
PROJECT NO. BR-0003(591)
BRIDGE REPLACEMENT ON S.B.R.
SR-3 OVER CEDAR CREEK
STATION 29+55.00
MORGAN COUNTY
BENT 2
BIN: 021390

ESTIMATED QUANTITIES	DESIGNED BY: ARS	DRAWN BY: M. HOCK
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 12/5/18
VERIFIED BY:	DATE CHECKED:	SCALE: AS SHOWN



REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
BR-0003(591)	2020	110M

SECTION A-A
SCALE: 1" = 1'-0"

SECTION B-B
SCALE: ¾" = 1'-0"

BARS A#11

BARS C#5

BARS G#5

HOOPS H#5

STIRR. S#5

BARS TYPE U#3

TYPE AB-2 SWEDGE ANCHOR BOLTS IN 4" ø ANCHOR BOLT WELLS (TYP.). HORIZONTALLY ADJUST BARS A, C, STIRR. S & TYPE U TO INSURE PROPER PLACEMENT OF ANCHOR BOLT WELLS.

TYPE AB-2 SWEDGE ANCHOR BOLTS IN 4" ø ANCHOR BOLT WELLS (TYP.). HORIZONTALLY ADJUST BARS E, G, & TYPE U TO INSURE PROPER PLACEMENT OF ANCHOR BOLT WELLS.

DETAIL 'A'
(PED. WITHOUT ANCHOR BOLT SIM.)
SCALE: ½" = 1'-0"

DETAIL 'A'
(PED. WITHOUT ANCHOR BOLT SIM.)
SCALE: ½" = 1'-0"

DETAIL 'B'
(PED. WITHOUT ANCHOR BOLT SIM.)
SCALE: ½" = 1'-0"

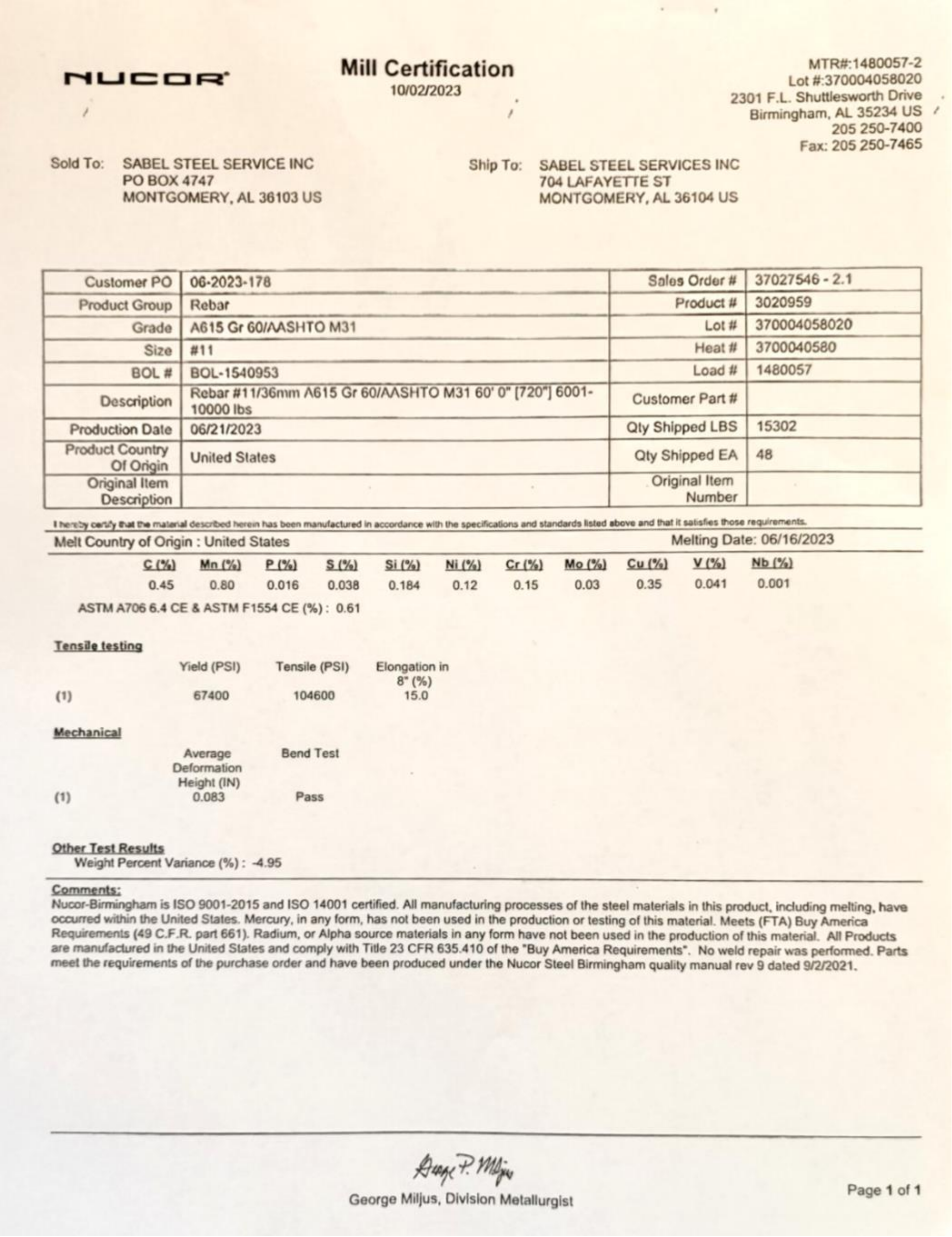
DETAIL 'B'
(PED. WITHOUT ANCHOR BOLT SIM.)
SCALE: ½" = 1'-0"

NOTE: PROVIDE 1½" MINIMUM CONCRETE COVER OVER BARS TYPE U IN TOP OF PEDESTALS.

ALABAMA DEPARTMENT OF TRANSPORTATION
BRIDGE SHEET NO. 14 OF 18
REVISIONS
PROJECT NO. BR-0003(591)
BRIDGE REPLACEMENT ON S.B.R.
SR-3 OVER CEDAR CREEK
STATION 29+55.00
MORGAN COUNTY
BENT DETAILS
ESTIMATED QUANTITIES
DESIGNED BY: APG
DRAWN BY: M. WOCK
COMPUTED BY:
CHECKED BY:
DATE DRAWN: 12/10/18
VERIFIED BY:
DATE CHECKED:
SCALE: AS SHOWN
BIN: 021390

Appendix B : Material Data

MTRs of reinforcing bars used in Steel-pipe Pile Specimens



NUCOR

Mill Certification
10/02/2023

MTR#:1480057-2
Lot #:370004058020
2301 F.L. Shuttlesworth Drive
Birmingham, AL 35234 US
205 250-7400
Fax: 205 250-7465

Sold To: SABEL STEEL SERVICE INC
PO BOX 4747
MONTGOMERY, AL 36103 US

Ship To: SABEL STEEL SERVICES INC
704 LAFAYETTE ST
MONTGOMERY, AL 36104 US

Customer PO	06-2023-178	Sales Order #	37027546 - 2.1
Product Group	Rebar	Product #	3020959
Grade	A615 Gr 60/AASHTO M31	Lot #	370004058020
Size	#11	Heat #	3700040580
BOL #	BOL-1540953	Load #	1480057
Description	Rebar #11/36mm A615 Gr 60/AASHTO M31 60' 0" [720"] 6001-10000 lbs	Customer Part #	
Production Date	06/21/2023	Qty Shipped LBS	15302
Product Country Of Origin	United States	Qty Shipped EA	48
Original Item Description		Original Item Number	

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed above and that it satisfies those requirements.

Melt Country of Origin : United States

Melting Date: 06/16/2023

C (%)	Mn (%)	P (%)	S (%)	Si (%)	Ni (%)	Cr (%)	Mo (%)	Cu (%)	V (%)	Nb (%)
0.45	0.80	0.016	0.038	0.184	0.12	0.15	0.03	0.35	0.041	0.001

ASTM A706 6.4 CE & ASTM F1554 CE (%) : 0.61

Tensile testing

	Yield (PSI)	Tensile (PSI)	Elongation in 8" (%)
(1)	67400	104600	15.0

Mechanical

	Average Deformation Height (IN)	Bend Test
(1)	0.083	Pass

Other Test Results

Weight Percent Variance (%) : -4.95

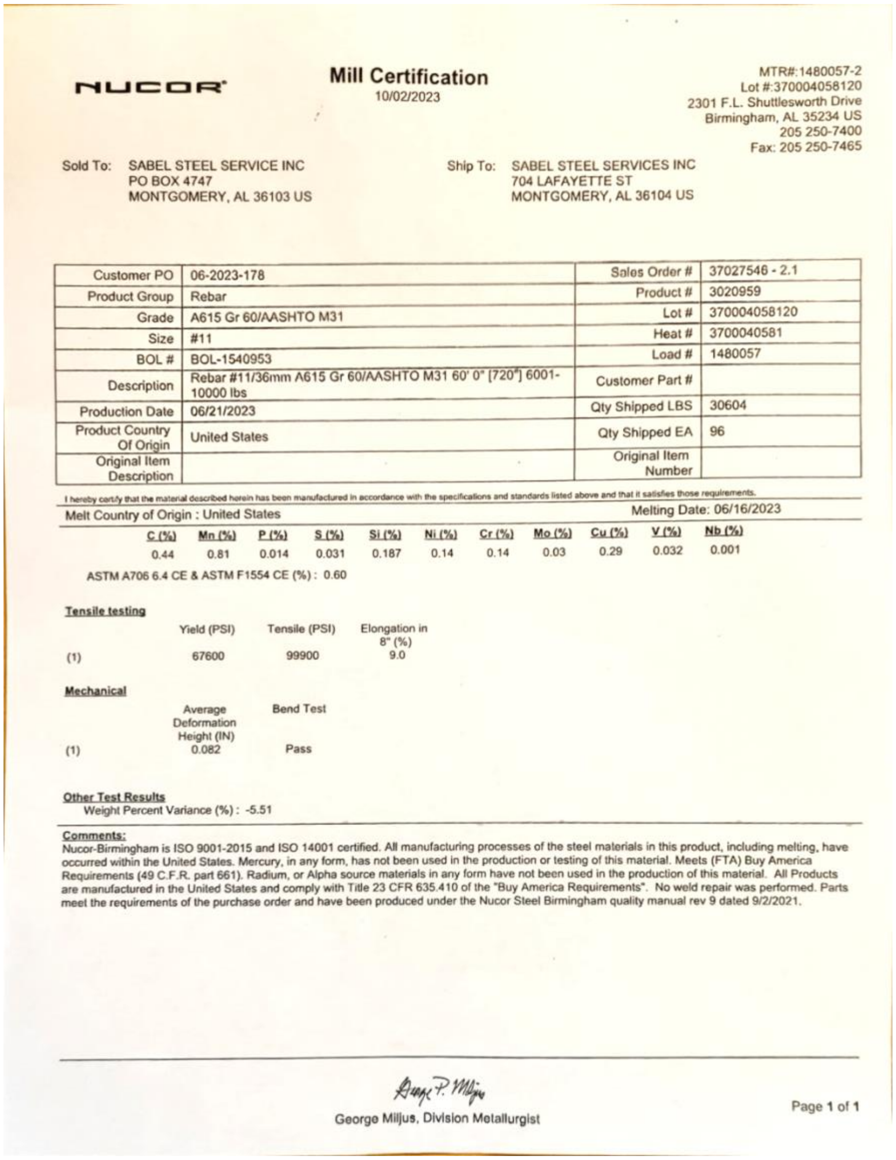
Comments:

Nucor-Birmingham is ISO 9001-2015 and ISO 14001 certified. All manufacturing processes of the steel materials in this product, including melting, have occurred within the United States. Mercury, in any form, has not been used in the production or testing of this material. Meets (FTA) Buy America Requirements (49 C.F.R. part 661). Radium, or Alpha source materials in any form have not been used in the production of this material. All Products are manufactured in the United States and comply with Title 23 CFR 635.410 of the "Buy America Requirements". No weld repair was performed. Parts meet the requirements of the purchase order and have been produced under the Nucor Steel Birmingham quality manual rev 9 dated 9/2/2021.

George P. Miljus

George Miljus, Division Metallurgist

Page 1 of 1



NUCOR

Mill Certification

10/02/2023

MTR#:1480057-2
Lot #:370004058120
2301 F.L. Shuttlesworth Drive
Birmingham, AL 35234 US
205 250-7400
Fax: 205 250-7465

Sold To: SABEL STEEL SERVICE INC
PO BOX 4747
MONTGOMERY, AL 36103 US

Ship To: SABEL STEEL SERVICES INC
704 LAFAYETTE ST
MONTGOMERY, AL 36104 US

Customer PO	06-2023-178	Sales Order #	37027546 - 2.1
Product Group	Rebar	Product #	3020959
Grade	A615 Gr 60/AASHTO M31	Lot #	370004058120
Size	#11	Heat #	3700040581
BOL #	BOL-1540953	Load #	1480057
Description	Rebar #11/36mm A615 Gr 60/AASHTO M31 60' 0" [720"] 6001-10000 lbs	Customer Part #	
Production Date	06/21/2023	Qty Shipped LBS	30604
Product Country Of Origin	United States	Qty Shipped EA	96
Original Item Description		Original Item Number	

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed above and that it satisfies those requirements.

Melt Country of Origin : United States

Melting Date: 06/16/2023

C (%)	Mn (%)	P (%)	S (%)	Si (%)	Ni (%)	Cr (%)	Mo (%)	Cu (%)	V (%)	Nb (%)
0.44	0.81	0.014	0.031	0.187	0.14	0.14	0.03	0.29	0.032	0.001

ASTM A706 6.4 CE & ASTM F1554 CE (%) : 0.60

Tensile testing

	Yield (PSI)	Tensile (PSI)	Elongation in 8" (%)
(1)	67600	99900	9.0

Mechanical

	Average Deformation Height (IN)	Bend Test
(1)	0.082	Pass

Other Test Results

Weight Percent Variance (%) : -5.51

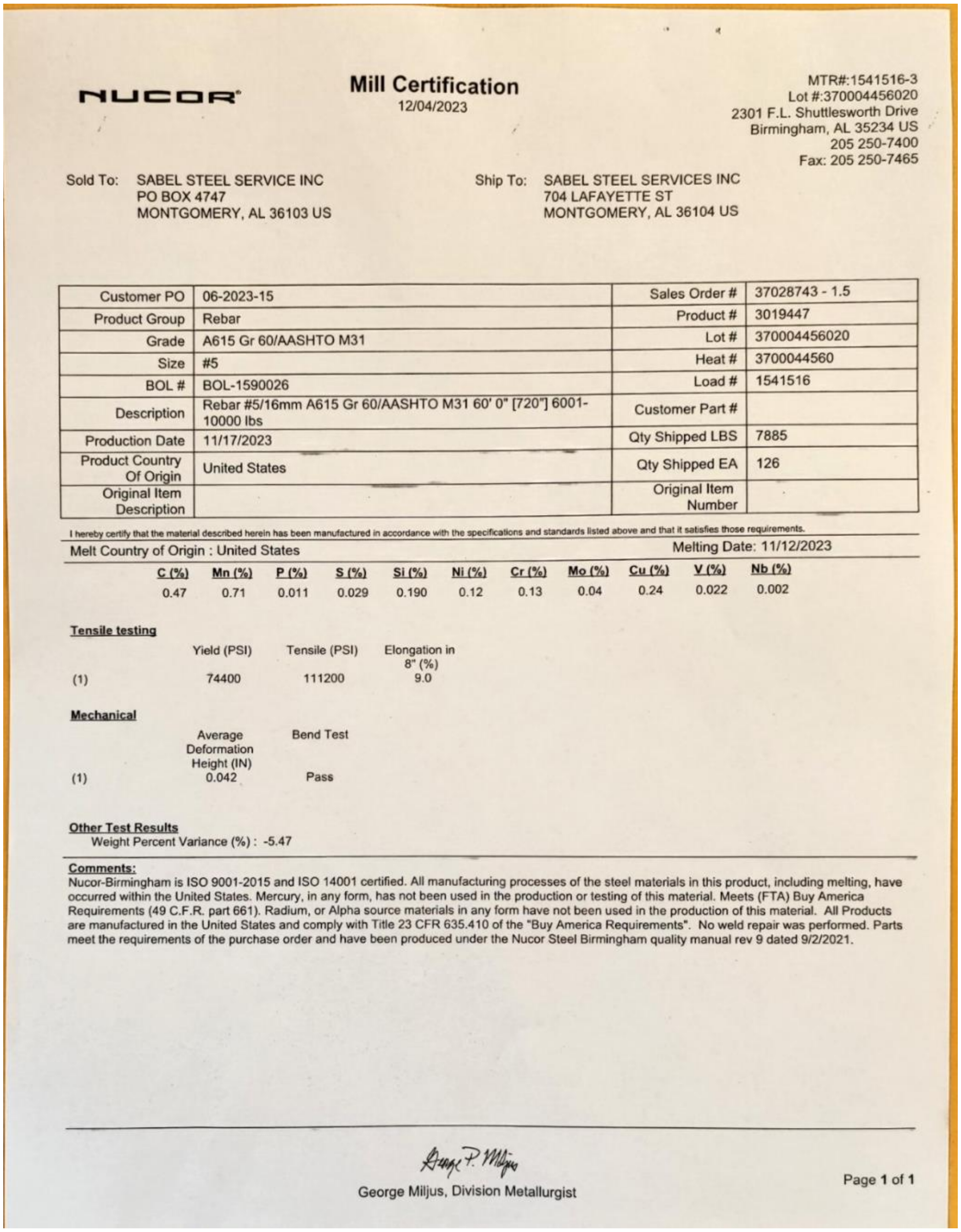
Comments:

Nucor-Birmingham is ISO 9001-2015 and ISO 14001 certified. All manufacturing processes of the steel materials in this product, including melting, have occurred within the United States. Mercury, in any form, has not been used in the production or testing of this material. Meets (FTA) Buy America Requirements (49 C.F.R. part 661). Radium, or Alpha source materials in any form have not been used in the production of this material. All Products are manufactured in the United States and comply with Title 23 CFR 635.410 of the "Buy America Requirements". No weld repair was performed. Parts meet the requirements of the purchase order and have been produced under the Nucor Steel Birmingham quality manual rev 9 dated 9/2/2021.

George P. Miljus

George Miljus, Division Metallurgist

Page 1 of 1



NUCOR

Mill Certification

12/04/2023

MTR#:1541516-3
Lot #:370004456020
2301 F.L. Shuttlesworth Drive
Birmingham, AL 35234 US
205 250-7400
Fax: 205 250-7465

Sold To: SABEL STEEL SERVICE INC
PO BOX 4747
MONTGOMERY, AL 36103 US

Ship To: SABEL STEEL SERVICES INC
704 LAFAYETTE ST
MONTGOMERY, AL 36104 US

Customer PO	06-2023-15	Sales Order #	37028743 - 1.5
Product Group	Rebar	Product #	3019447
Grade	A615 Gr 60/AASHTO M31	Lot #	370004456020
Size	#5	Heat #	3700044560
BOL #	BOL-1590026	Load #	1541516
Description	Rebar #5/16mm A615 Gr 60/AASHTO M31 60' 0" [720"] 6001-10000 lbs	Customer Part #	
Production Date	11/17/2023	Qty Shipped LBS	7885
Product Country Of Origin	United States	Qty Shipped EA	126
Original Item Description		Original Item Number	

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed above and that it satisfies those requirements.

Melt Country of Origin : United States

Melting Date: 11/12/2023

C (%)	Mn (%)	P (%)	S (%)	Si (%)	Ni (%)	Cr (%)	Mo (%)	Cu (%)	V (%)	Nb (%)
0.47	0.71	0.011	0.029	0.190	0.12	0.13	0.04	0.24	0.022	0.002

Tensile testing

	Yield (PSI)	Tensile (PSI)	Elongation in 8" (%)
(1)	74400	111200	9.0

Mechanical

	Average Deformation Height (IN)	Bend Test
(1)	0.042	Pass

Other Test Results

Weight Percent Variance (%) : -5.47

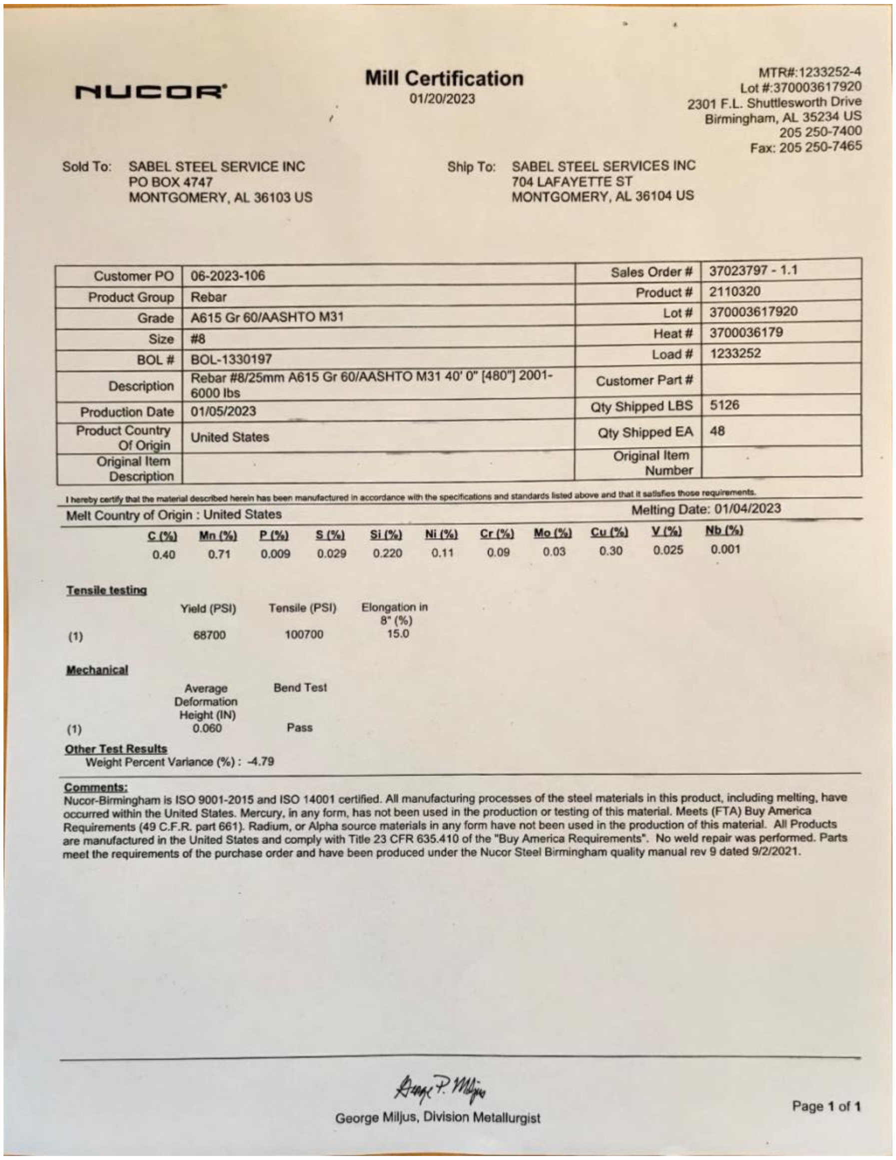
Comments:

Nucor-Birmingham is ISO 9001-2015 and ISO 14001 certified. All manufacturing processes of the steel materials in this product, including melting, have occurred within the United States. Mercury, in any form, has not been used in the production or testing of this material. Meets (FTA) Buy America Requirements (49 C.F.R. part 661). Radium, or Alpha source materials in any form have not been used in the production of this material. All Products are manufactured in the United States and comply with Title 23 CFR 635.410 of the "Buy America Requirements". No weld repair was performed. Parts meet the requirements of the purchase order and have been produced under the Nucor Steel Birmingham quality manual rev 9 dated 9/2/2021.

George P. Miljus

George Miljus, Division Metallurgist

Page 1 of 1



NUCOR

Mill Certification

01/20/2023

MTR#:1233252-4
Lot #:370003617920
2301 F.L. Shuttlesworth Drive
Birmingham, AL 35234 US
205 250-7400
Fax: 205 250-7465

Sold To: SABEL STEEL SERVICE INC
PO BOX 4747
MONTGOMERY, AL 36103 US

Ship To: SABEL STEEL SERVICES INC
704 LAFAYETTE ST
MONTGOMERY, AL 36104 US

Customer PO	06-2023-106	Sales Order #	37023797 - 1.1
Product Group	Rebar	Product #	2110320
Grade	A615 Gr 60/AASHTO M31	Lot #	370003617920
Size	#8	Heat #	3700036179
BOL #	BOL-1330197	Load #	1233252
Description	Rebar #8/25mm A615 Gr 60/AASHTO M31 40' 0" [480"] 2001-6000 lbs	Customer Part #	
Production Date	01/05/2023	Qty Shipped LBS	5126
Product Country Of Origin	United States	Qty Shipped EA	48
Original Item Description		Original Item Number	

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed above and that it satisfies those requirements.

Melt Country of Origin : United States

Melting Date: 01/04/2023

C (%)	Mn (%)	P (%)	S (%)	Si (%)	Ni (%)	Cr (%)	Mo (%)	Cu (%)	V (%)	Nb (%)
0.40	0.71	0.009	0.029	0.220	0.11	0.09	0.03	0.30	0.025	0.001

Tensile testing

	Yield (PSI)	Tensile (PSI)	Elongation in 8" (%)
(1)	68700	100700	15.0

Mechanical

	Average Deformation Height (IN)	Bend Test
(1)	0.060	Pass

Other Test Results

Weight Percent Variance (%) : -4.79

Comments:

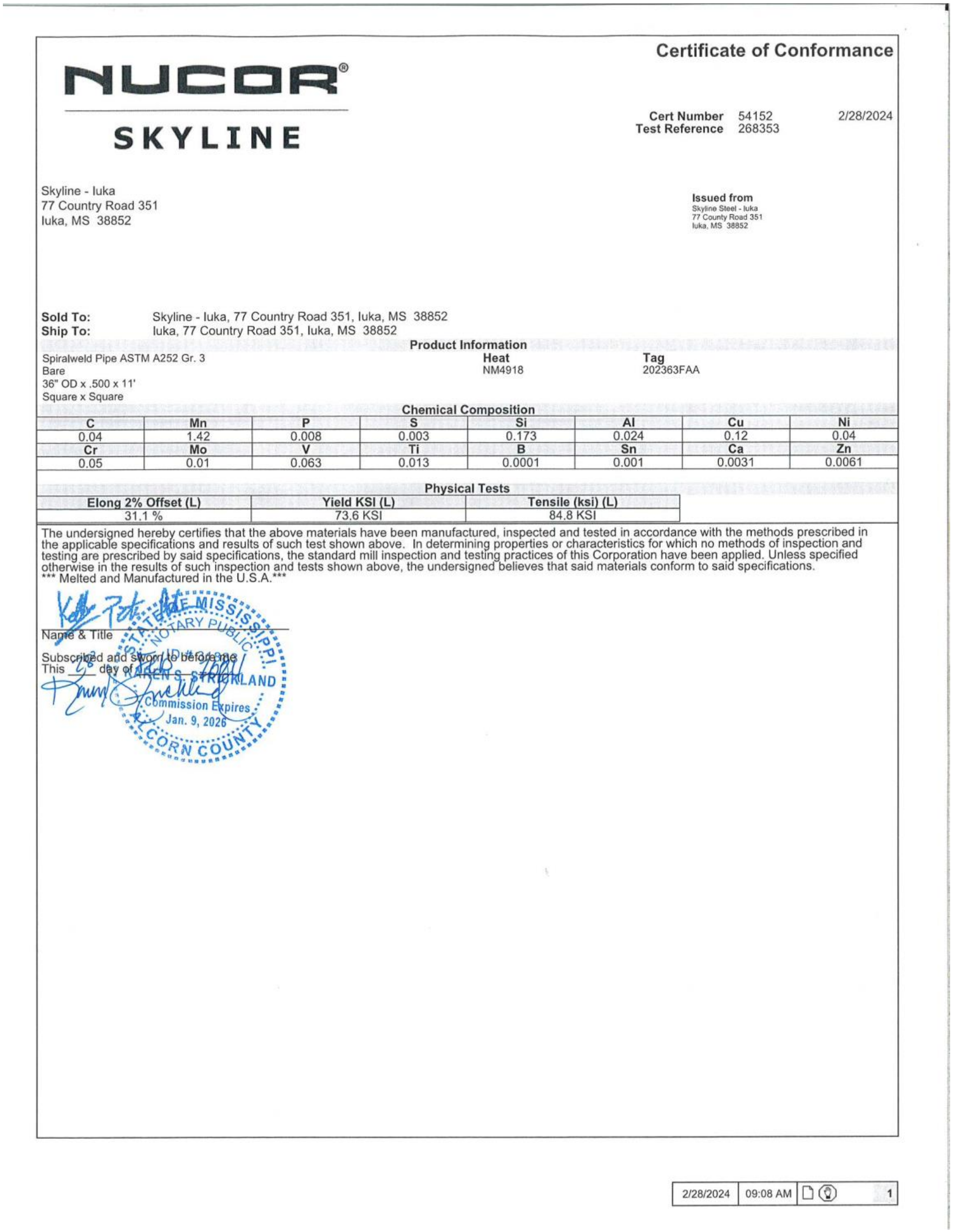
Nucor-Birmingham is ISO 9001-2015 and ISO 14001 certified. All manufacturing processes of the steel materials in this product, including melting, have occurred within the United States. Mercury, in any form, has not been used in the production or testing of this material. Meets (FTA) Buy America Requirements (49 C.F.R. part 661). Radium, or Alpha source materials in any form have not been used in the production of this material. All Products are manufactured in the United States and comply with Title 23 CFR 635.410 of the "Buy America Requirements". No weld repair was performed. Parts meet the requirements of the purchase order and have been produced under the Nucor Steel Birmingham quality manual rev 9 dated 9/2/2021.

George P. Miljus

George Miljus, Division Metallurgist

Page 1 of 1

MTRs of Steel Pipes used in Specimens



NUCOR®

SKYLINE

Certificate of Conformance

Cert Number 54152 2/28/2024
Test Reference 268353

Skyline - Iuka
77 Country Road 351
Iuka, MS 38852

Issued from
Skyline Steel - Iuka
77 County Road 351
Iuka, MS 38852

Sold To: Skyline - Iuka, 77 Country Road 351, Iuka, MS 38852
Ship To: Iuka, 77 Country Road 351, Iuka, MS 38852

Product Information

	Heat	Tag
Spiralweld Pipe ASTM A252 Gr. 3 Bare 36" OD x .500 x 11' Square x Square	NM4918	202363FAA

Chemical Composition

C	Mn	P	S	Si	Al	Cu	Ni
0.04	1.42	0.008	0.003	0.173	0.024	0.12	0.04
Cr	**Mo**	**V**	**Ti**	**B**	**Sn**	**Ca**	**Zn**
0.05	0.01	0.063	0.013	0.0001	0.001	0.0031	0.0061

Physical Tests

Elong 2% Offset (L)	Yield KSI (L)	Tensile (ksi) (L)
31.1 %	73.6 KSI	84.8 KSI

The undersigned hereby certifies that the above materials have been manufactured, inspected and tested in accordance with the methods prescribed in the applicable specifications and results of such test shown above. In determining properties or characteristics for which no methods of inspection and testing are prescribed by said specifications, the standard mill inspection and testing practices of this Corporation have been applied. Unless specified otherwise in the results of such inspection and tests shown above, the undersigned believes that said materials conform to said specifications.
*** Melted and Manufactured in the U.S.A.***

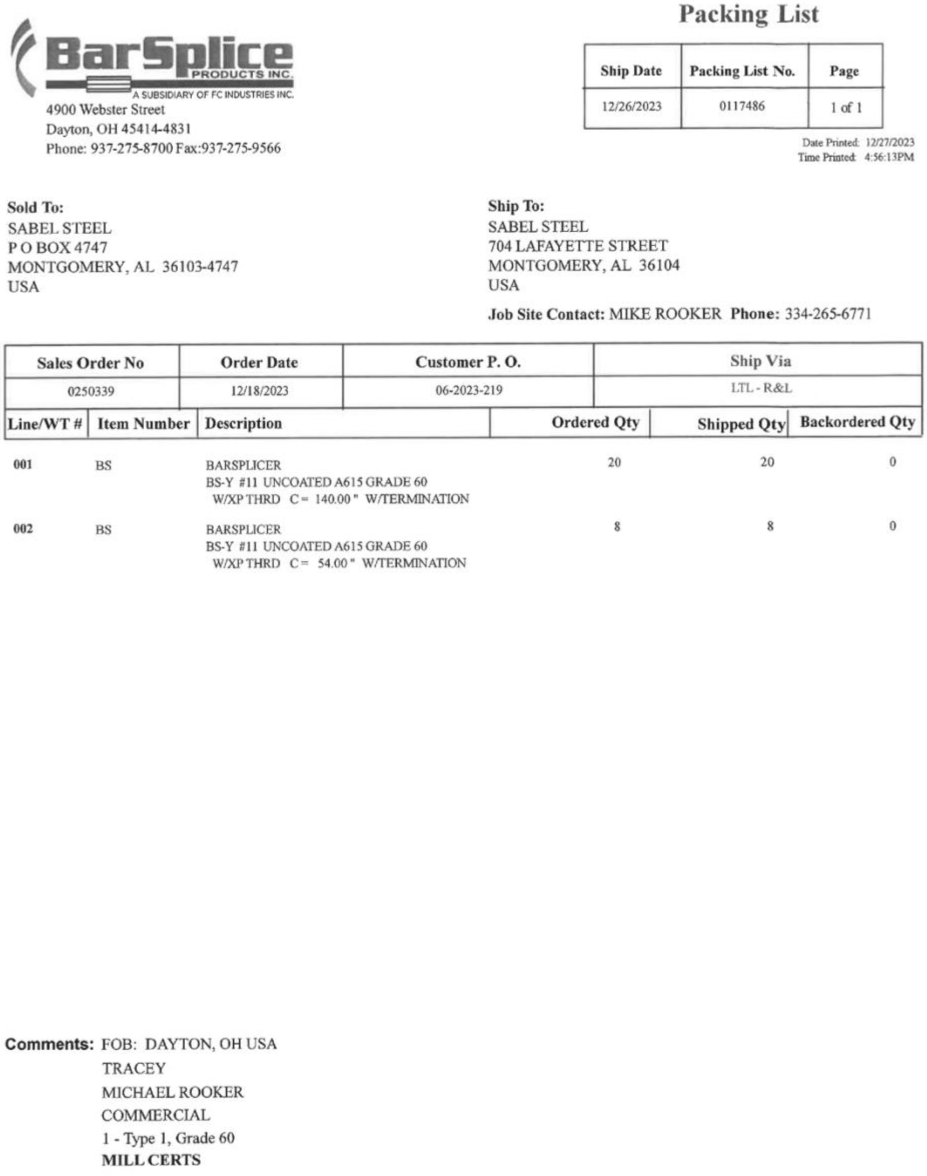
Name & Title

Subscribed and sworn to before me
This 28 day of Feb 2024

KAREN S. STRICKLAND
Commission Expires
Jan. 9, 2026

2/28/2024 | 09:08 AM | 1

MTRs of Rebar Coupler used in Specimens



BarSplice PRODUCTS INC.
A SUBSIDIARY OF FC INDUSTRIES INC.
4900 Webster Street
Dayton, OH 45414-4831
Phone: 937-275-8700 Fax:937-275-9566

Packing List

Ship Date	Packing List No.	Page
12/26/2023	0117486	1 of 1

Date Printed: 12/27/2023
Time Printed: 4:56:13PM

Sold To:
SABEL STEEL
P O BOX 4747
MONTGOMERY, AL 36103-4747
USA

Ship To:
SABEL STEEL
704 LAFAYETTE STREET
MONTGOMERY, AL 36104
USA

Job Site Contact: MIKE ROOKER **Phone:** 334-265-6771

Sales Order No	Order Date	Customer P. O.	Ship Via
0250339	12/18/2023	06-2023-219	LTL - R&L

Line/WT #	Item Number	Description	Ordered Qty	Shipped Qty	Backordered Qty
001	BS	BARSPLICER BS-Y #11 UNCOATED A615 GRADE 60 W/XP THRD C = 140.00 " W/TERMINATION	20	20	0
002	BS	BARSPLICER BS-Y #11 UNCOATED A615 GRADE 60 W/XP THRD C = 54.00 " W/TERMINATION	8	8	0

Comments: FOB: DAYTON, OH USA
TRACEY
MICHAEL ROOKER
COMMERCIAL
1 - Type 1, Grade 60
MILL CERTS

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Comments: FOB: DAYTON, OH USA
TRACEY
MICHAEL ROOKER
COMMERCIAL
1 - Type 1, Grade 60
MILL CERTS

BarSplice PRODUCTS INC.

CERTIFICATE OF COMPLIANCE

FOR **BPI® TYPE 1** MECHANICAL SPLICES & ASTM A970 HEADED DEVICES (GRADE 60)

DATE	CUSTOMER	PURCHASE ORDER	ITEM	PROJECT NAME
12/27/2023	SABEL STEEL	06-2023-219	--	COMMERCIAL

QTY	PRODUCT	IDENTIFICATION	HEAT LOT No.	ASTM Spec or Rebar Mill
269.033 FT	#11 REBAR, UNCOATED A615 GRADE	11RB_11Y	7033300	ASTM A615-22
40 EA	#11 THREADED HEAD 5Ab	11TERM_7W2	A191470	ASTM A576-17
41.508 FT	#11 REBAR, UNCOATED A615 GRADE	11RB_11Y	7033300	ASTM A615-22
16 EA	#11 THREADED HEAD 5Ab	11TERM_7W2	A191470	ASTM A576-17

This certifies the following:

1. Barsplice Products, Inc. (BPI) designs, manufactures, and supplies available mechanical splices and headed devices for reinforcing bars under an approved quality system in accordance with ISO 9001:2015.
2. This Certificate of Compliance was issued upon the date shown above to the customer noted with reference to the customer purchase order and project name if provided. This certifies that the coupler systems and headed devices shipped from BPI comply with general descriptions, dimensions and representations made in published BPI literature and data sheet(s) in effect at the time of issue.
3. Supplied products shown above are broken down by heat lot number for each quantity of product(s) listed. The BPI identification codes uniquely identify the products provided and are marked upon the finished parts where applicable. These codes match the raw material Certified Material Test Reports (CMTR) and the heat lot number. When required by material specifications, chemical analyses, tests, examinations, and heat treatments have been performed by the supplier(s) and the results have been verified by BPI to be included on the CMTR. Any required test not performed (if applicable) is noted on the CMTR, and any operation not performed per specification (if applicable) is reported.
4. Raw materials used in the products listed were obtained from, and processed through, BPI qualified sources/suppliers in accordance with ISO 9001:2015. All ferrous raw materials (iron and steel) used, and their coatings if applicable, have been domestically sourced, processed and manufactured in the United States of America in full compliance with the following: Federal Highway Administration (FHWA) 23 USC § 313 – Buy America; 23 CFR § 635.410, American Recovery and Reinvestment Act of 2009 Section 1605 – Buy American, 41 USC § 8302 – Buy America; American Iron and Steel (AIS) requirements in the EPA State Revolving Fund Programs, and; the Build America, Buy America (BABA) Act.
5. Product and/or material specifications are verified to be met, and products are mechanically tested when appropriate, prior to shipment to ensure the mechanical splices and headed devices provided meet applicable compression and tensile strength requirements. Type 1 mechanical splices provided conform to Section 25.5.7 of ACI 318-19 and develop 125% x specified yield (f_y) strength of the reinforcing bar when installed on Grade 60 (metric Grade 420) reinforcing bar manufactured to the latest requirements of ASTM A615/A615M or A706/A706M. Headed devices provided meet the full tensile strength requirements of ASTM A970 (confirmed with in-air testing) and conform to Class A/HA. Performance claims are only applicable when BPI supplies all components of the mechanical splice system. When components are supplied by others, performance is not included in the scope of this certificate.
6. The above referenced iron and steel product(s) additionally meet all requirements of the following:
   - Arizona Department of Transportation Specification Section 106.05(B), with legal authority to bind BPI
   - Illinois Department of Transportation Specification Section 106.01
   - Michigan Department of Transportation Specification Section 105.10
   - West Virginia Department of Transportation Approval Code #1456220A.

When coated splices and headed devices are installed on galvanized or epoxy coated bars, some touch-up and/or repair of the coating may be required after installation.

NOTICE

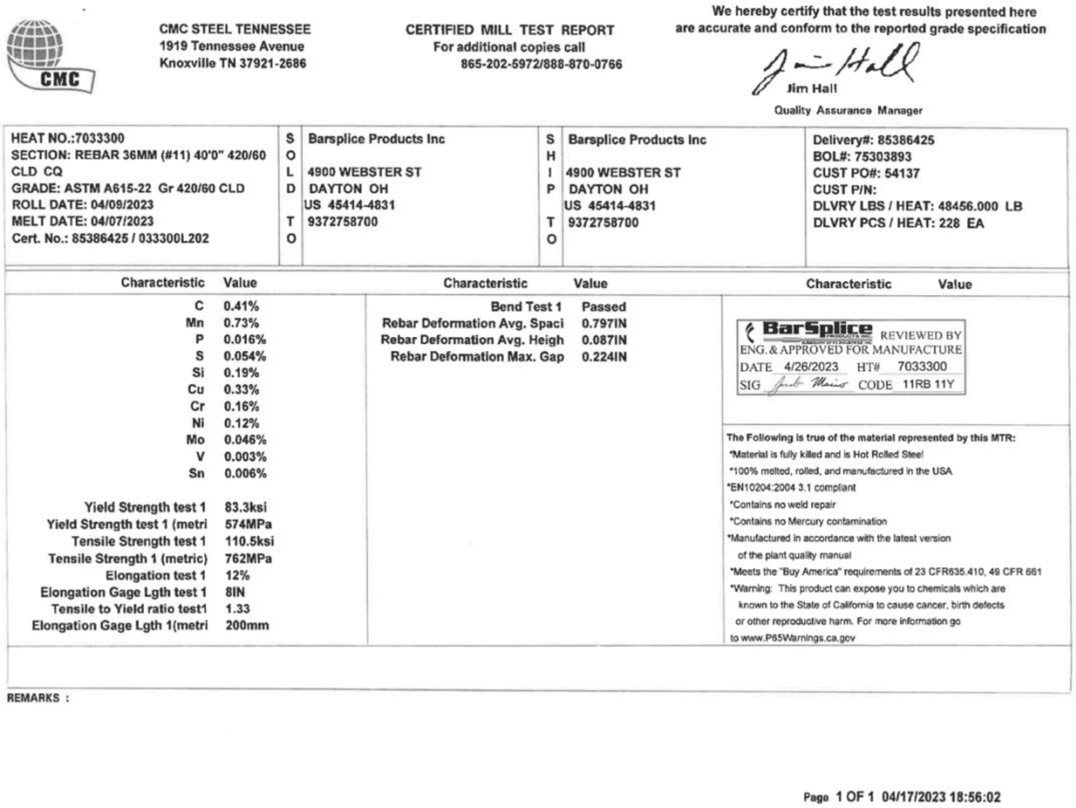
The products described herein must be installed in accordance with the latest edition of the appropriate BPI SPLICING MANUAL and/or INSTALLATION INSTRUCTIONS (and any special supplements) supplied with the product or to the project. These documents must be read and fully understood by the operator before use. In accordance with project specifications, tensile tests may be required before and during production splicing to verify correct usage, rebar grade, and operator proficiency. Other terms and conditions are applicable as may have been previously supplied on quotations and order acknowledgments, either directly or to the dealer, distributor or representative.

SIGNED: ______________________ **DATE:** 12/27/2023 **SHOP ORDER #** 0250339

Amber Hare, Shipping Clerk
BARSPLICE PRODUCTS, INC.

4900 Webster Street, Dayton OH 45414, USA • Tel: (937) 275-8700 • email: bar@barsplice.com

BPI 4.12-08 REV. C Effective 05/01/2023



CMC STEEL TENNESSEE
1919 Tennessee Avenue
Knoxville TN 37921-2686

CERTIFIED MILL TEST REPORT
For additional copies call
865-202-5972/888-870-0766

We hereby certify that the test results presented here are accurate and conform to the reported grade specification

Jim Hall
Jim Hall
Quality Assurance Manager

HEAT NO.:7033300
SECTION: REBAR 36MM (#11) 40'0" 420/60 CLD CQ
GRADE: ASTM A615-22 Gr 420/60 CLD
ROLL DATE: 04/09/2023
MELT DATE: 04/07/2023
Cert. No.: 85386425 / 033300L202

SOLD TO
Barsplice Products Inc
4900 WEBSTER ST
DAYTON OH
US 45414-4831
9372758700

SHIP TO
Barsplice Products Inc
4900 WEBSTER ST
DAYTON OH
US 45414-4831
9372758700

Delivery#: 85386425
BOL#: 75303893
CUST PO#: 54137
CUST P/N:
DLVRY LBS / HEAT: 48456.000 LB
DLVRY PCS / HEAT: 228 EA

Characteristic	Value
C	0.41%
Mn	0.73%
P	0.016%
S	0.054%
Si	0.19%
Cu	0.33%
Cr	0.16%
Ni	0.12%
Mo	0.046%
V	0.003%
Sn	0.006%
Yield Strength test 1	83.3ksi
Yield Strength test 1 (metri	574MPa
Tensile Strength test 1	110.5ksi
Tensile Strength 1 (metric)	762MPa
Elongation test 1	12%
Elongation Gage Lgth test 1	8IN
Tensile to Yield ratio test1	1.33
Elongation Gage Lgth 1(metri	200mm

Characteristic	Value
Bend Test 1	Passed
Rebar Deformation Avg. Spaci	0.797IN
Rebar Deformation Avg. Heigh	0.087IN
Rebar Deformation Max. Gap	0.224IN

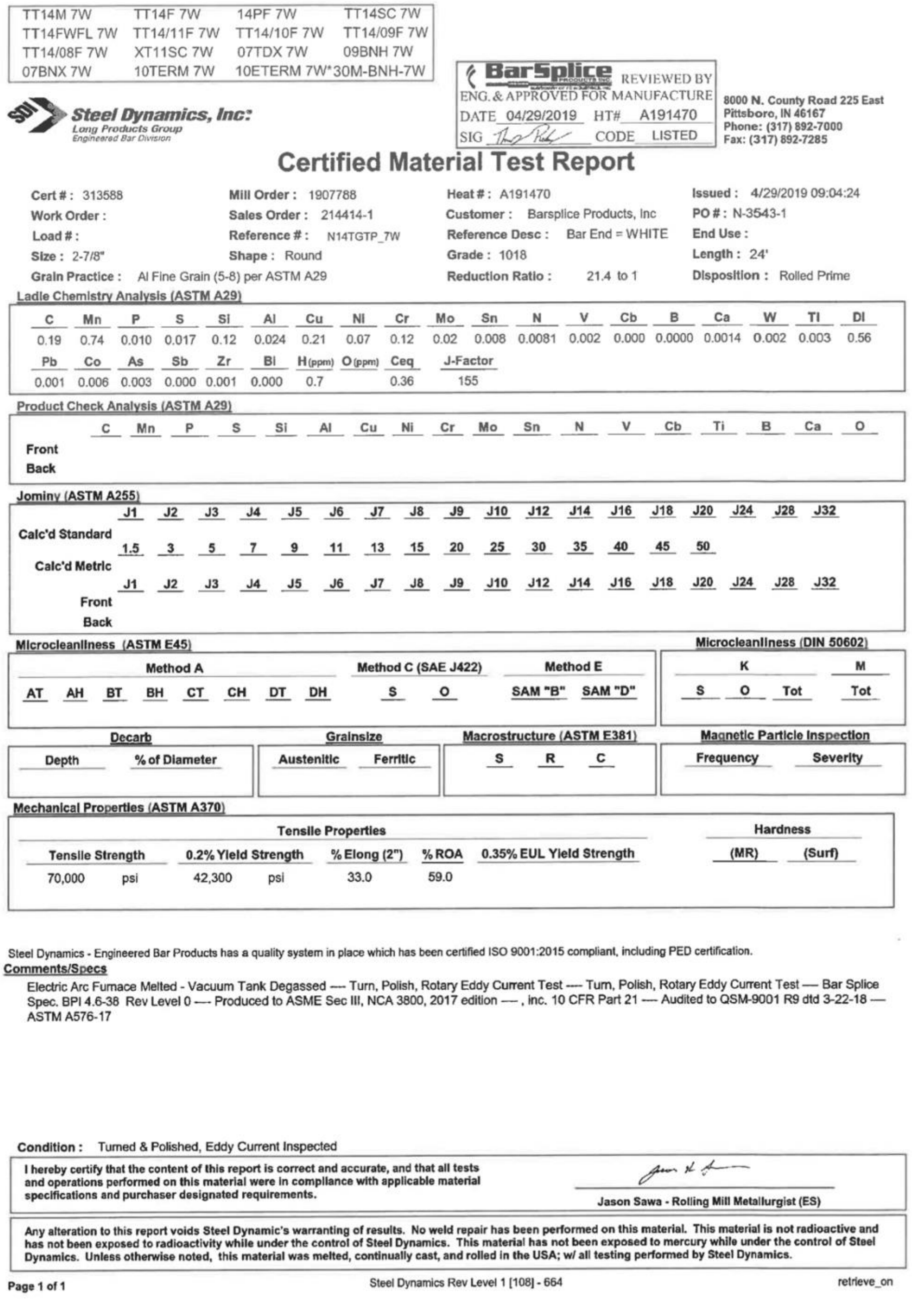
Characteristic	Value

BarSplice PRODUCTS INC REVIEWED BY ENG. & APPROVED FOR MANUFACTURE
DATE 4/26/2023 HT# 7033300
SIG CODE 11RB 11Y

The Following is true of the material represented by this MTR:
*Material is fully killed and is Hot Rolled Steel
*100% melted, rolled, and manufactured in the USA
*EN10204:2004 3.1 compliant
*Contains no weld repair
*Contains no Mercury contamination
*Manufactured in accordance with the latest version of the plant quality manual
*Meets the "Buy America" requirements of 23 CFR635.410, 49 CFR 661
*Warning: This product can expose you to chemicals which are known to the State of California to cause cancer, birth defects or other reproductive harm. For more information go to www.P65Warnings.ca.gov

REMARKS :

Page 1 OF 1 04/17/2023 18:56:02



TT14M 7W	TT14F 7W	14PF 7W	TT14SC 7W
TT14FWFL 7W	TT14/11F 7W	TT14/10F 7W	TT14/09F 7W
TT14/08F 7W	XT11SC 7W	07TDX 7W	09BNH 7W
07BNX 7W	10TERM 7W	10ETERM 7W*30M-BNH-7W	

BarSplice PRODUCTS INC REVIEWED BY
ENG. & APPROVED FOR MANUFACTURE
DATE 04/29/2019 HT# A191470
SIG CODE LISTED

Steel Dynamics, Inc.
Long Products Group
Engineered Bar Division

8000 N. County Road 225 East
Pittsboro, IN 46167
Phone: (317) 892-7000
Fax: (317) 892-7285

Certified Material Test Report

Cert # : 313588	**Mill Order :** 1907788	**Heat # :** A191470	**Issued :** 4/29/2019 09:04:24
Work Order :	**Sales Order :** 214414-1	**Customer :** Barsplice Products, Inc	**PO # :** N-3543-1
Load # :	**Reference # :** N14TGTP_7W	**Reference Desc :** Bar End = WHITE	**End Use :**
Size : 2-7/8"	**Shape :** Round	**Grade :** 1018	**Length :** 24'
Grain Practice : Al Fine Grain (5-8) per ASTM A29		**Reduction Ratio :** 21.4 to 1	**Disposition :** Rolled Prime

Ladle Chemistry Analysis (ASTM A29)

C	Mn	P	S	Si	Al	Cu	Ni	Cr	Mo	Sn	N	V	Cb	B	Ca	W	Ti	DI
0.19	0.74	0.010	0.017	0.12	0.024	0.21	0.07	0.12	0.02	0.008	0.0081	0.002	0.000	0.0000	0.0014	0.002	0.003	0.56

Pb	Co	As	Sb	Zr	Bi	H(ppm)	O(ppm)	Ceq	J-Factor
0.001	0.006	0.003	0.000	0.001	0.000	0.7		0.36	155

Product Check Analysis (ASTM A29)

	C	Mn	P	S	Si	Al	Cu	Ni	Cr	Mo	Sn	N	V	Cb	Ti	B	Ca	O
Front																		
Back																		

Jominy (ASTM A255)

	J1	J2	J3	J4	J5	J6	J7	J8	J9	J10	J12	J14	J16	J18	J20	J24	J28	J32
Calc'd Standard																		
	1.5	3	5	7	9	11	13	15	20	25	30	35	40	45	50			
Calc'd Metric																		
	J1	J2	J3	J4	J5	J6	J7	J8	J9	J10	J12	J14	J16	J18	J20	J24	J28	J32
Front																		
Back																		

Microcleanliness (ASTM E45)

Method A								Method C (SAE J422)		Method E	
AT	AH	BT	BH	CT	CH	DT	DH	S	O	SAM "B"	SAM "D"

Microcleanliness (DIN 50602)

K			M
S	O	Tot	Tot

Decarb		Grainsize		Macrostructure (ASTM E381)			Magnetic Particle Inspection	
Depth	% of Diameter	Austenitic	Ferritic	S	R	C	Frequency	Severity

Mechanical Properties (ASTM A370)

Tensile Properties							Hardness	
Tensile Strength		0.2% Yield Strength		% Elong (2")	% ROA	0.35% EUL Yield Strength	(MR)	(Surf)
70,000	psi	42,300	psi	33.0	59.0			

Steel Dynamics - Engineered Bar Products has a quality system in place which has been certified ISO 9001:2015 compliant, including PED certification.

Comments/Specs

Electric Arc Furnace Melted - Vacuum Tank Degassed ---- Turn, Polish, Rotary Eddy Current Test ---- Turn, Polish, Rotary Eddy Current Test --- Bar Splice Spec. BPI 4.6-38 Rev Level 0 ---- Produced to ASME Sec III, NCA 3800, 2017 edition ---- , inc. 10 CFR Part 21 ---- Audited to QSM-9001 R9 dtd 3-22-18 ---- ASTM A576-17

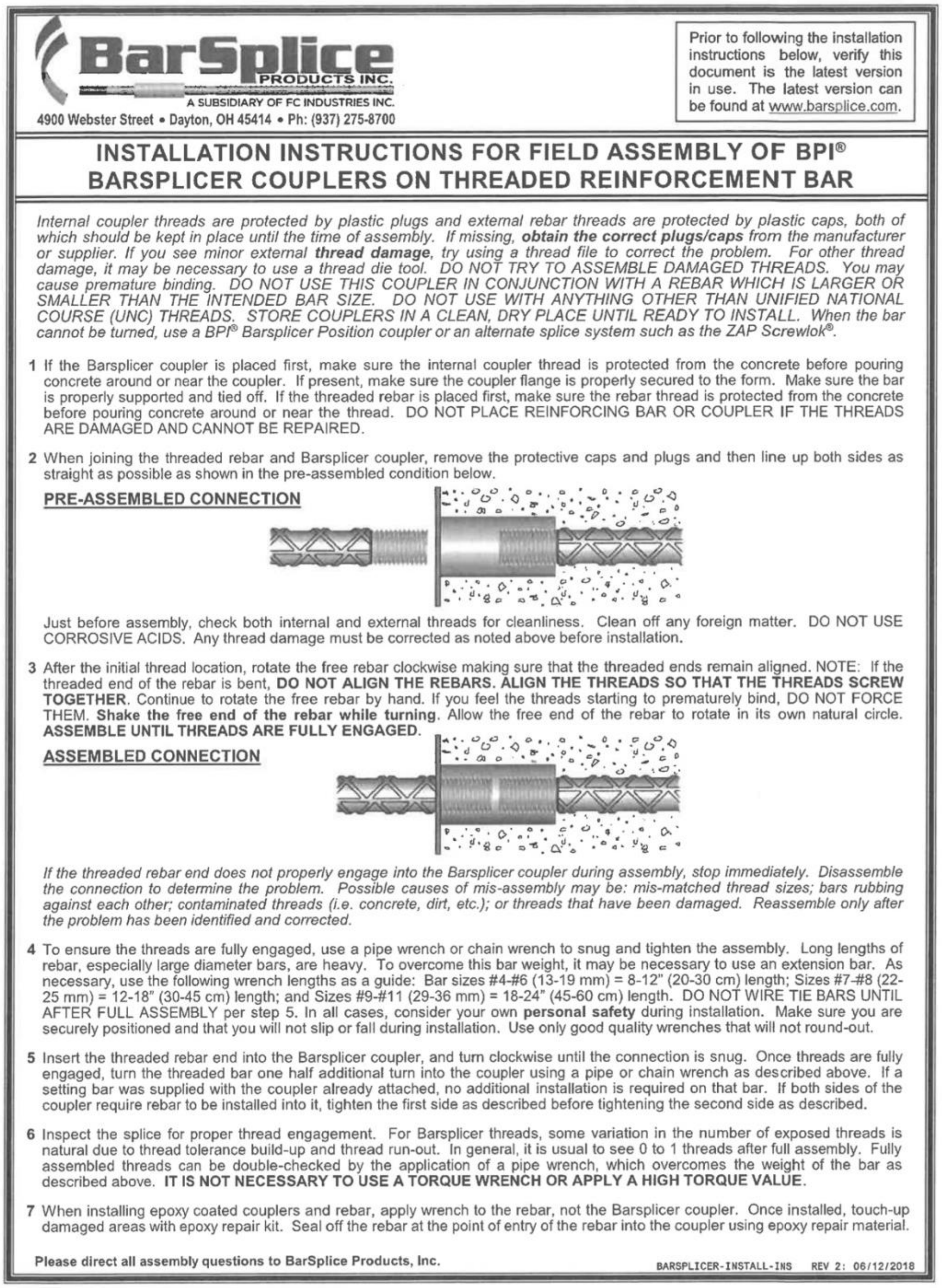
Condition : Turned & Polished, Eddy Current Inspected

I hereby certify that the content of this report is correct and accurate, and that all tests and operations performed on this material were in compliance with applicable material specifications and purchaser designated requirements.

Jason Sawa - Rolling Mill Metallurgist (ES)

Any alteration to this report voids Steel Dynamic's warranting of results. No weld repair has been performed on this material. This material is not radioactive and has not been exposed to radioactivity while under the control of Steel Dynamics. This material has not been exposed to mercury while under the control of Steel Dynamics. Unless otherwise noted, this material was melted, continually cast, and rolled in the USA; w/ all testing performed by Steel Dynamics.

Page 1 of 1 Steel Dynamics Rev Level 1 [108] - 664 retrieve_on



BarSplice PRODUCTS INC.
A SUBSIDIARY OF FC INDUSTRIES INC.
4900 Webster Street • Dayton, OH 45414 • Ph: (937) 275-8700

Prior to following the installation instructions below, verify this document is the latest version in use. The latest version can be found at www.barsplice.com.

INSTALLATION INSTRUCTIONS FOR FIELD ASSEMBLY OF BPI® BARSPLICER COUPLERS ON THREADED REINFORCEMENT BAR

Internal coupler threads are protected by plastic plugs and external rebar threads are protected by plastic caps, both of which should be kept in place until the time of assembly. If missing, ***obtain the correct plugs/caps*** *from the manufacturer or supplier. If you see minor external* ***thread damage****, try using a thread file to correct the problem. For other thread damage, it may be necessary to use a thread die tool. DO NOT TRY TO ASSEMBLE DAMAGED THREADS. You may cause premature binding. DO NOT USE THIS COUPLER IN CONJUNCTION WITH A REBAR WHICH IS LARGER OR SMALLER THAN THE INTENDED BAR SIZE. DO NOT USE WITH ANYTHING OTHER THAN UNIFIED NATIONAL COURSE (UNC) THREADS. STORE COUPLERS IN A CLEAN, DRY PLACE UNTIL READY TO INSTALL. When the bar cannot be turned, use a BPI® Barsplicer Position coupler or an alternate splice system such as the ZAP Screwlok®.*

1 If the Barsplicer coupler is placed first, make sure the internal coupler thread is protected from the concrete before pouring concrete around or near the coupler. If present, make sure the coupler flange is properly secured to the form. Make sure the bar is properly supported and tied off. If the threaded rebar is placed first, make sure the rebar thread is protected from the concrete before pouring concrete around or near the thread. DO NOT PLACE REINFORCING BAR OR COUPLER IF THE THREADS ARE DAMAGED AND CANNOT BE REPAIRED.

2 When joining the threaded rebar and Barsplicer coupler, remove the protective caps and plugs and then line up both sides as straight as possible as shown in the pre-assembled condition below.

PRE-ASSEMBLED CONNECTION

Just before assembly, check both internal and external threads for cleanliness. Clean off any foreign matter. DO NOT USE CORROSIVE ACIDS. Any thread damage must be corrected as noted above before installation.

3 After the initial thread location, rotate the free rebar clockwise making sure that the threaded ends remain aligned. NOTE: If the threaded end of the rebar is bent, **DO NOT ALIGN THE REBARS. ALIGN THE THREADS SO THAT THE THREADS SCREW TOGETHER**. Continue to rotate the free rebar by hand. If you feel the threads starting to prematurely bind, DO NOT FORCE THEM. **Shake the free end of the rebar while turning**. Allow the free end of the rebar to rotate in its own natural circle. **ASSEMBLE UNTIL THREADS ARE FULLY ENGAGED.**

ASSEMBLED CONNECTION

If the threaded rebar end does not properly engage into the Barsplicer coupler during assembly, stop immediately. Disassemble the connection to determine the problem. Possible causes of mis-assembly may be: mis-matched thread sizes; bars rubbing against each other; contaminated threads (i.e. concrete, dirt, etc.); or threads that have been damaged. Reassemble only after the problem has been identified and corrected.

4 To ensure the threads are fully engaged, use a pipe wrench or chain wrench to snug and tighten the assembly. Long lengths of rebar, especially large diameter bars, are heavy. To overcome this bar weight, it may be necessary to use an extension bar. As necessary, use the following wrench lengths as a guide: Bar sizes #4-#6 (13-19 mm) = 8-12" (20-30 cm) length; Sizes #7-#8 (22-25 mm) = 12-18" (30-45 cm) length; and Sizes #9-#11 (29-36 mm) = 18-24" (45-60 cm) length. DO NOT WIRE TIE BARS UNTIL AFTER FULL ASSEMBLY per step 5. In all cases, consider your own **personal safety** during installation. Make sure you are securely positioned and that you will not slip or fall during installation. Use only good quality wrenches that will not round-out.

5 Insert the threaded rebar end into the Barsplicer coupler, and turn clockwise until the connection is snug. Once threads are fully engaged, turn the threaded bar one half additional turn into the coupler using a pipe or chain wrench as described above. If a setting bar was supplied with the coupler already attached, no additional installation is required on that bar. If both sides of the coupler require rebar to be installed into it, tighten the first side as described before tightening the second side as described.

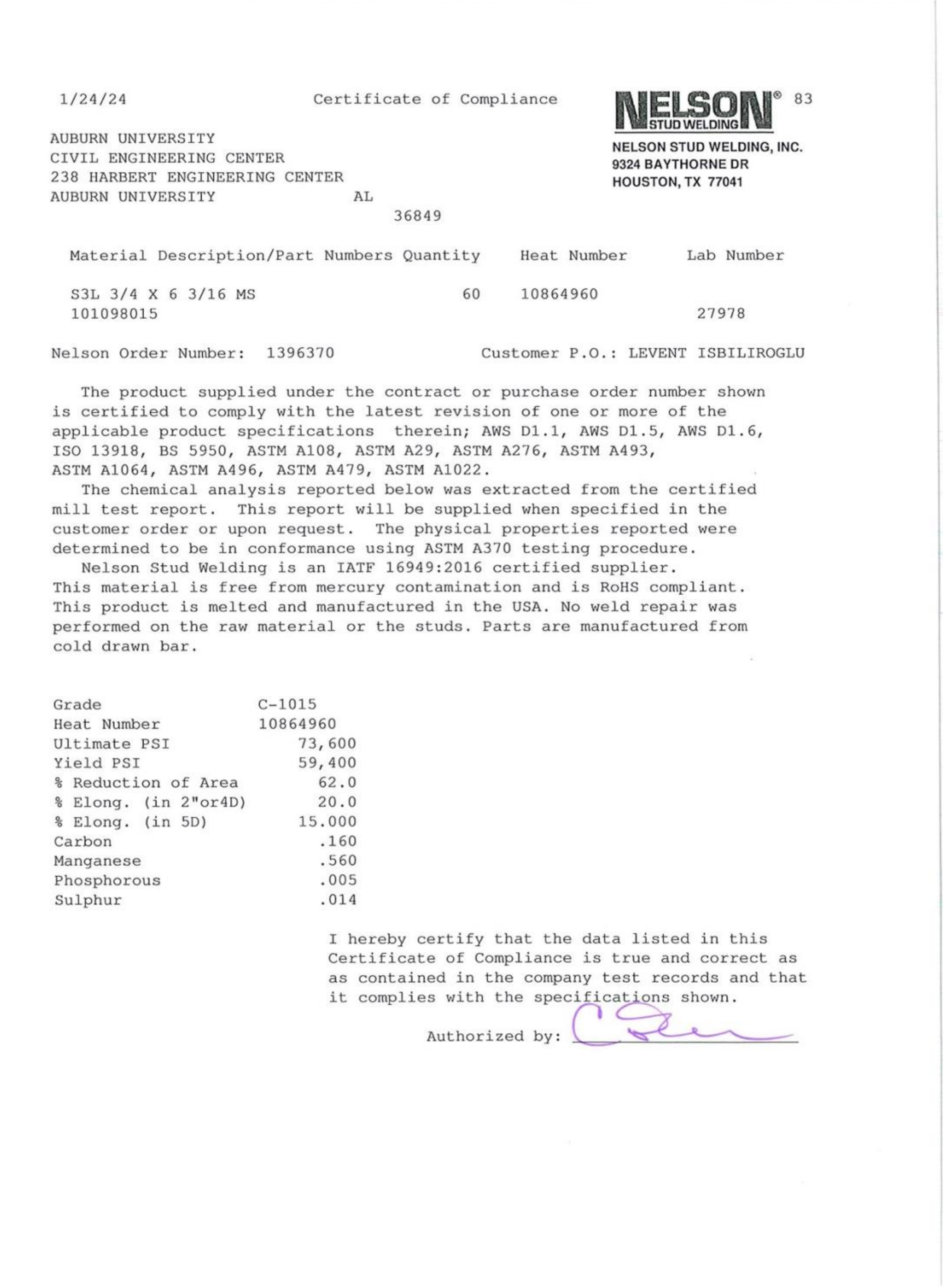
6 Inspect the splice for proper thread engagement. For Barsplicer threads, some variation in the number of exposed threads is natural due to thread tolerance build-up and thread run-out. In general, it is usual to see 0 to 1 threads after full assembly. Fully assembled threads can be double-checked by the application of a pipe wrench, which overcomes the weight of the bar as described above. **IT IS NOT NECESSARY TO USE A TORQUE WRENCH OR APPLY A HIGH TORQUE VALUE**.

7 When installing epoxy coated couplers and rebar, apply wrench to the rebar, not the Barsplicer coupler. Once installed, touch-up damaged areas with epoxy repair kit. Seal off the rebar at the point of entry of the rebar into the coupler using epoxy repair material.

Please direct all assembly questions to BarSplice Products, Inc.

BARSPLICER-INSTALL-INS REV 2: 06/12/2018

MTRs of reinforcing bars used in Steel-pipe Pile Specimens



1/24/24 Certificate of Compliance

NELSON STUD WELDING 83

NELSON STUD WELDING, INC.
9324 BAYTHORNE DR
HOUSTON, TX 77041

AUBURN UNIVERSITY
CIVIL ENGINEERING CENTER
238 HARBERT ENGINEERING CENTER
AUBURN UNIVERSITY AL
36849

Material Description/Part Numbers	Quantity	Heat Number	Lab Number
S3L 3/4 X 6 3/16 MS 101098015	60	10864960	27978

Nelson Order Number: 1396370 Customer P.O.: LEVENT ISBILIROGLU

The product supplied under the contract or purchase order number shown is certified to comply with the latest revision of one or more of the applicable product specifications therein; AWS D1.1, AWS D1.5, AWS D1.6, ISO 13918, BS 5950, ASTM A108, ASTM A29, ASTM A276, ASTM A493, ASTM A1064, ASTM A496, ASTM A479, ASTM A1022.

The chemical analysis reported below was extracted from the certified mill test report. This report will be supplied when specified in the customer order or upon request. The physical properties reported were determined to be in conformance using ASTM A370 testing procedure.

Nelson Stud Welding is an IATF 16949:2016 certified supplier.
This material is free from mercury contamination and is RoHS compliant.
This product is melted and manufactured in the USA. No weld repair was performed on the raw material or the studs. Parts are manufactured from cold drawn bar.

Grade	C-1015
Heat Number	10864960
Ultimate PSI	73,600
Yield PSI	59,400
% Reduction of Area	62.0
% Elong. (in 2"or4D)	20.0
% Elong. (in 5D)	15.000
Carbon	.160
Manganese	.560
Phosphorous	.005
Sulphur	.014

I hereby certify that the data listed in this Certificate of Compliance is true and correct as as contained in the company test records and that it complies with the specifications shown.

Authorized by:

CHARTER STEEL
A Division of
Charter Manufacturing Company, Inc.

1658 Cold Springs Road
Saukville, Wisconsin 53080
(262) 268-2400
1-800-437-8789
Fax (262) 268-2570

CHARTER STEEL TEST REPORT

Melted in USA Manufactured in USA

Nelson Stud Welding - A Nelson Fastener Systems Company
7900 West Ridge Road
QC Department
Elyria,OH-44035

Cust P.O.	429539
Customer Part #	103004320
Charter Sales Order	10370082
Heat #	10864960
Ship Lot #	1386042
Grade	1015 R SK FG RHQ 49/64 RNDCOIL
Process	HR
Finish Size	49/64
Ship date	09-NOV-23

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed below and that it satisfies these requirements. The recording of false, fictitious and fraudulent statements or entries on this document may be punishable as a felony under federal statute.

Test results of Heat Lot # 10864960

Lab Code: 7388

CHEM	C ✓	MN ✓	P ✓	S ✓	SI ✓	NI ✓	CR ✓	MO ✓	CU ✓	SN	V ✓
%Wt	.16	.56	.005	.014	.160	.05	.05	.02	.09	.005	.001
	AL ✓	N ✓	B ✓	TI ✓	CA ✓	NB	SB	AS			
	.025	.0060	.0001	.001	.0001	.001	.001	.003			
	PB										
	.001										

JOMINY(HRC)

J1	J3
42	20

JOMINY SAMPLE TYPE ENGLISH=C

QUALITY CONTROL
180
11-20-23
SEF-NELSON STUD WELDING

Test results of Rolling Lot # 1386042

	# of Tests	Min Value	Max Value	Mean Value	
TENSILE (KSI)	4	64.2	64.6	64.4	TENSILE LAB = 0358-02
REDUCTION OF AREA (%)	4	32	41	35	RA LAB = 0358-02

REDUCTION RATIO=66:1

Specifications: Manufactured per Charter Steel Quality Manual Rev Date 05/12/17
Charter Steel certifies this product is indistinguishable from background radiation levels by having process radiation detectors in place to measure for the presence of radiation within our process & products.
Meets customer specifications with any applicable Charter Steel exceptions for the following customer documents:
Customer Document = MPS-102C Revision = H Dated = 03-JUL-18

Additional Comments: This material meets the chemistry requirements of ASTM-A108 (latest version), ASTM A29 (latest version), and EN 10025-2 S-235-J2G3

Lots 27928

Melt Source:

Charter Steel

Saukville, WI, USA

This MTR supersedes all previously dated MTRs for this order

Douglas Jones Division Mgr. of Quality Assurance

jonesdo@chartersteel.com

Trip: R5282821

Printed Date : 11/09/2023

The following statements are applicable to the material described on the front of this Test Report:

1. The laboratory that generated the analytical or test results can be identified by the following key:

Certificate Number	Lab Code	Laboratory		Address
0358-01	7388	**CSSM**	Charter Steel Melting Division	1658 Cold Springs Road, Saukville, WI 53080
0358-02	8171	**CSSR**	Charter Steel Rolling	1658 Cold Springs Road, Saukville, WI 53080
0358-07	8171	**CSSP**	Charter Steel Processing Division	1658 Cold Springs Road, Saukville, WI 53080
0358-03	123633	**CSFP**	Charter Steel Ohio Processing Division	6255 US Highway 23, Rising Sun, OH 43457
0358-04	125544	**CSCM/ CSCR**	Charter Steel Cleveland	4300 E. 49th St., Cuyahoga Heights, OH 44125-1004
*	*	--	Subcontracted test performed by laboratory not in Charter Steel System	

2. When run by a Charter Steel laboratory, the following tests were performed according to the latest revisions of the specifications listed below, as noted in the Charter Steel Laboratory Quality Manual:

Test	Specifications	CSSM	CSSR	CSSP	CSFP	CSCM/ CSCR
Chemistry Analysis	ASTM E415; ASTM E1019	X				X
Macroetch	ASTM E381	X				X
Hardenability (Jominy)	ASTM A255; SAE J406; JIS G0561	X				X
Grain Size	ASTM E112	X	X	X	X	X
Tensile Test	ASTM E8; ASTM A370		X	X	X	X
Rockwelll Hardness	ASTM E18; ASTM A370	X	X	X	X	X
Microstructure (spheroidization)	ASTM A892		X	X	X	
Inclusion Content (Methods A, E)	ASTM E45		X			X
Decarburization	ASTM E1077		X	X	X	X

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7. Certificate type 3.1 per EN 10204 rev. 2005-01
8. Charter Steel melting is a continuous cast electric arc furnace

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Testing Laboratory

0358-01 Saukville Melting
0358-02 Saukville Rolling
0358-03 Fostoria Processing
0358-04 Cleveland Melt/Roll
0358-07 Saukville Processing

Appendix C : DRAFT ALDOT SPECIFICATION

ALABAMA DEPARTMENT OF TRANSPORTATION

Date: April 1, 2026 Special Provision / Manual Insert No. xx-xxxx

Subject: Steel Pipe Pile–to–Reinforced Concrete Bent Cap Connections (Hollow Pipe Piles with Concrete Plugs)

Alabama Bridge Design/Detailing Manual (current edition) and/or Alabama Standard Specifications (2026 Edition) shall be amended by the addition of a new section as follows:

SECTION XXX

STEEL PIPE PILE-TO-BENT CAP CONNECTIONS (WITH CONCRETE PLUG)

xxx.01 Description.

This section provides LRFD-aligned design and detailing requirements for steel pipe pile–to–reinforced concrete bent cap connections for hollow steel pipe piles filled with concrete plugs. Connection types covered include:

(A) rebar-based non-contact lap splice systems; i) headed, ii) hooked, or iii) straight bars within the plug

(B) mechanical anchorage systems i) headed shear studs or ii) annular ring/end plate systems welded to the pipe.

These connection types were selected to represent common practice and were evaluated in a full-scale experimental program.

Commentary: The project report indicates that even shallow embedment (approximately 0.33D) provided significant moment transfer and that mechanical systems exhibited higher stiffness; however, tests did not reach ultimate failure.

xxx.02 Definitions.

(a) Pipe embedment depth, e_{pipe} : depth of steel pipe embedded into the bent cap.

(b) Concrete plug depth, h_{plug} : depth of concrete placed inside the steel pipe (measured from the bent cap surface level).

(c) Pipe outer diameter, D_{pipe}

(d) Pipe wall thickness, t_{pipe}

(e) Radial rebar clear distance to pipe, c_{clr} : clear distance from the inside face of pipe to the nearest reinforcement surface.

xxx.03 Design requirements (LRFD).

xxx.03.1 General. The designer shall determine factored design demands at the pile–to–bent cap interface per the governing LRFD load combinations and shall design the connection to satisfy Strength, Service, and Fatigue limit states as applicable.

xxx.03.2 Limit states to check.

The following limit states shall be checked, as applicable to the selected connection type:

Flexure and combined axial–flexure (plug region and adjacent cap region).

Shear transfer across the interface and within the connection zone.

Concrete bearing/local crushing at bearing surfaces (bar heads/hooks, ring plate bearing region, stud engagement region).

Anchorage capacities:

rebar development/anchorage (straight and hooked bars per AASHTO LRFD; headed bars per ACI where AASHTO LRFD does not apply),

stud connector nominal resistance (AASHTO LRFD composite stud connector equation),

annular ring plate design, weld capacity, and embedment requirement per WSDOT Design Manual (2024) method cited in the report.

Commentary: The report recommends additional fatigue and ultimate limit state evaluation prior to full standardization of welded/mechanical options.

xxx.03.3 Connection type selection.

(a) Rebar-based connections may be used when constructability is prioritized.

(b) Mechanical connections (stud or annular ring) may be used when higher stiffness/fixity are desired and deeper embedment/prefabrication are acceptable.

xxx.04 Prescriptive detailing requirements.

xxx.04.1 Rebar-based non-contact lap splice connections (Headed / Hooked / Straight)

xxx.04.1.1 Pipe embedment depth e_{pipe}.

For rebar-based connections, the pipe embedment depth shall satisfy:

$e_{pipe} \geq \max(0.33 \cdot D_{pipe}, 12 \text{ in.})$

Demonstrated baseline: For D_{pipe} =36 in, e_{pipe} =12 in. was used for headed/hooked/straight rebar systems.

xxx.04.1.2 Concrete plug depth h_{plug} .

For rebar-based connections, plug depth shall be governed by reinforcement development/anchorage requirements:

$h_{plug} \geq e_{pipe} + (l_{dev,req} + \Delta_{end})$

where $l_{dev,req}$ is the required bar development/anchorage length inside the plug per applicable code (AASHTO LRFD for straight/hooked; ACI for headed when used), and Δ_{end} is an additional allowance to provide end cover/tolerance and to avoid congestion at the plug termination (recommended $\Delta_{end} \geq 3$ in. as a baseline constructability allowance).

Demonstrated baseline (tested D=36 in.):

- Headed and Hooked rebar systems: h_{plug} =3 ft (≈ 1.0D)

- Straight rebar system: h_{plug} =4 ft (≈ 1.33D)

xxx.04.1.3 Required bar embedment inside plug.

Bar embedment inside the plug shall meet or exceed required development/anchorage lengths.

Demonstrated computed requirements for #11 bars at the report baseline ($f'_c \approx 4$ ksi, $f_y \approx 60$ ksi):

- Headed bar: 19 in.
- Hooked bar: 27 in.
- Straight bar: 41 in.

xxx.04.1.4 Rebar-to-pipe clearance and rebar cage diameter.

Rebar in the plug shall be detailed to provide adequate clearance to the pipe inner face for consolidation and to satisfy minimum spacing/cover requirements. The radial rebar clear distance, c_{clr}, shall satisfy the recommended baseline clearance:

$c_{clr} \geq \max(1 \text{ in.}, d_b, 1.33\, d_{agg})$

where d_b is nominal longitudinal bar diameter, and d_{agg} is nominal maximum coarse aggregate size.

Demonstrated baseline geometry: 1.3 in. clear distance to the bar surface was used in testing. This geometry is permitted as a baseline for similar pile sizes.

xxx.04.1.5 Confinement reinforcement.

Closed hoops/stirrups shall be provided in the connection zone to confine the plug and control cracking.

Demonstrated baseline: #5 hoop spacing of approximately 10 in. was used.

xxx.04.2 Headed stud mechanical connection

xxx.04.2.1 Pipe embedment depth e_{pipe}.

For headed stud connections:

$e_{pipe} \geq \max(1.0 \bullet D_{pipe}, e_{rows} + \Delta_{cover})$

where e_{rows} is the required depth to accommodate stud rows and spacing, and Δ_{cover} provides top/bottom clear cover and constructability tolerance.

Demonstrated baseline: For $D_{pipe} = 36$ *in.,* $e_{pipe} = 3$ *ft (1.0* D_{pipe}*) was used to accommodate three stud rows.*

xxx.04.2.2 Concrete plug depth h_{plug}.

Plug depth shall be at least the pipe diameter:

$h_{plug} \geq D_{pipe}$

Demonstrated baseline: $h_{plug} = D_{pipe} = 3$ *ft.*

xxx.04.2.3 Stud nominal resistance and demand check.

Stud connectors shall be designed using the nominal resistance in AASHTO LRFD.

Demonstrated baseline configuration (D=36 in.): 3 rows × 12 studs, each 6 in × ¾ in, studs angularly spaced at 30°, row spacing 11 in.

Commentary: The report's stiffness comparison demonstrates that mechanical systems achieved substantially higher stiffness than shallow rebar systems. Fatigue verification of stud welds is recommended.

xxx.04.3 Annular ring/end plate mechanical connection

xxx.04.3.1 Pipe embedment depth e_{pipe}.

Embedment depth for annular ring connections shall be no less than the required embedment l_e computed from the WSDOT-based equation used in the report (WSDOT Design Manual, 2024):

Demonstrated baseline: For D=36 in., the report documents that the equation yields approximately 36 in. embedment for full plastic capacity targeting, while the tested configuration used 2 ft embedment to target concrete failure before full ring yield.

xxx.04.3.2 Ring geometry and venting.

Ring/end plate geometry, weld detailing, and venting shall follow the WSDOT method cited in the report:

Demonstrated baseline: Ring/end plate geometry OD/ID 44/28 in, thickness 0.5 in., and eight 1-in. vent holes.

xxx.05 Bent cap reinforcement continuity and constructability.

Bent cap longitudinal reinforcement shall be detailed such that bars that pass through the pile footprint remain continuous and develop the required strength. Where the pipe and plug reinforcement cage interferes with longitudinal bars, the design shall detail longitudinal bars to pass through the pipe reinforcement layout.

Demonstrated basis: The tested bent cap included a large amount of longitudinal reinforcement (20 #11 bars) with bars in central regions of the section.