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Strength, durability, and application of grouted couplers for integral abutments in accelerated bridge construction projects

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Strength, durability, and application of grouted couplers for integral abutments in accelerated bridge construction projects

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ABSTRACT

In areas of high traffic, long term bridge construction can have significant impacts on the traveling public and surrounding communities. To minimize this impact, engineers and contractors prefabricate bridge elements and utilize technologies that facilitate rapid bridge assembly. This strategy is known as Accelerated Bridge Construction (ABC) and has gained the attention of the bridge community as information and the benefits of ABC projects has been shared. There is untapped potential in this movement as some advantages of certain bridge types, like the integral abutment bridge, have seen limited use. Integral abutment bridges were developed as a means of eliminating the expansion joint from the bridge superstructure which present long term maintenance concerns. To eliminate the joint, integral abutments rigidly connect the superstructure and foundation so that the entire structure experiences thermal expansion and contraction as one. For this reason, the integral abutment is often large and heavily reinforced which present challenges for use in ABC projects. The size of the abutment presents weight issues and mechanical splicing of the abutment to the deep foundation presents tight construction tolerances.

This research investigated integral abutment details for use in ABC projects through mechanical splicing of the integral diaphragm and the pile cap. To complete this task, two ABC details were evaluated in the laboratory based on constructability, strength and durability. The construction process used to fabricate and erect the specimen was documented and is presented in this report, as this criteria often governs the design of ABC details. The specimen were tested for strength and durability by simulating thermal loads and live loads. Strain gages placed on the concrete and reinforcing steel captured the strain developed in the testing to

evaluate strength. Displacement transducers placed across the precast joint measured the crack width that developed under loading in order to assess durability. The ABC details investigated are the grouted rebar coupler detail and the pile coupler detail. In order to establish baseline performances for an integral abutment, a typical cast-in-place detail was also constructed and tested.

In the grouted rebar coupler detail, a plywood template was used to “match cast” the pile cap and the integral diaphragm. The template marked the locations of the spliced reinforcing steel and served as the base for the formwork in the integral diaphragm, holding the grouted couplers in position. The template proved to be simple to construct and resulted in the successful alignment of seventeen spliced steel bars and grouted couplers over the length of an eight foot specimen. A grout bed was pumped into the precast joint on the specimen, unfortunately grout leaked past two of the grouted coupler seals and obstructed the grouting of two couplers. Even with the two un-grouted rebar couplers, there was more than adequate strength created by the connection and the crack width that developed at the precast joint was comparable to that of the cast-in-place specimen.

The pile coupler detail was developed to facilitate the use of a slide in bridge with integral abutments. The pile coupler reduced the number of spliced connections between the pile cap and integral diaphragm significantly in order to facilitate adequate construction tolerances. The splicing system worked well during construction; however the detail’s performance in terms of strength and durability was less than ideal. If there is a demand for the benefits of the pile coupler detail in terms of constructability, the detail should be further investigated as several lessons were learned from the testing which could improve the structural performance of the detail.

CHAPTER 1. INTRODUCTION

1.1 Background

In Accelerated Bridge Construction (ABC) projects, bridge components or entire superstructures are prefabricated and then lifted or slid into place. Engineers and contractors work to design and build bridges in this manner so that the majority of construction can occur outside of the right-of-way, reducing road closure times and impact to the traveling public. Using ABC techniques, road closure times due to bridge construction have been reduced anywhere from months to weeks and sometimes to even days. These ABC techniques are relatively new to most agencies and currently require significant increases in cost and planning, as ABC bridges do not follow typical designs nor construction methods. The benefits of ABC projects are economically realized when factors such as traffic disruption, environmental impacts, and improved highway work zone safety are given monetary values. As ABC has gained popularity in the bridge community, knowledge has been gained, expanded, and shared, significantly increasing the quality of the ABC product and the acceptance of this method of bridge construction. Since ABC is still relatively new, there are types and aspects of bridges whose benefits have not yet been, or are rarely, utilized in the ABC movement.

The integral abutment bridge has seen limited use in ABC practice today, but has distinct characteristics and advantages which can benefit the long term viability of ABC. The integral abutment was originally developed to eliminate or move the expansion joint off of the bridge superstructure. Expansion joints are fragile and if not designed, constructed and maintained properly, will allow chlorides and debris to penetrate the deck joint and cause corrosion to critical substructure elements. This elimination of the expansion joint has seen widespread usage as it often leads to reduced maintenance costs. The so called joint-less bridge is also faster and less expensive to construct because the integral abutment is simple in geometry, has only one row of foundation piling, and eliminates the use of beam bearings. While there are many benefits to the integral abutment bridge there are also some drawbacks to their use which typically stem from the complex soil-structure interaction. Since there is no expansion joint, the entire bridge expands and contracts as one and thus the maximum length and skew are typically limited on integral abutment bridges.

Integral abutments are often large and heavily reinforced to transfer and distribute load between the superstructure and substructure, which makes it difficult to use this bridge type in ABC projects. This typically results from two reasons, the first being that mechanical splicing of the abutment is difficult due to construction tolerances and the second being that transportation of the abutment as a whole is difficult due to weight issues. To overcome these two design complications, the integral abutment bridges that have been constructed in ABC practice have relied on cast-in-place closure pours to create part of or the entire integral diaphragm. These closure pours alleviate construction tolerances and create an ABC integral abutment detail that is contractor friendly. However, the downside to cast-in-place closure pours is in the high performance concrete (HPC) or ultra high performance concrete (UHPC) used in the pour. These materials add significant cost to the project, as the material must achieve a high early strength so the bridge may be quickly opened to the traveling public.

1.2 Research Scope, Objectives and Tasks

The goal of this research is to provide information that will aid in the planning, design, and construction for ABC projects utilizing integral abutment designs. Engineers with the Bridge Engineering Center (BEC) and the technical advisory committee (TAC) discussed many possible details for integral abutments in the ABC application, of which, the most promising were selected for full scale laboratory investigation. The laboratory specimens were evaluated on three criteria: constructability, strength, and durability.

The following five tasks were completed to meet the objectives of the project:

1. Conduct a literature review examining ABC projects and integral abutments
2. Develop and design details for an integral abutment using ABC methods.
3. Fabricate the most promising designs for testing in the laboratory and document the construction and erection process
4. Test the designs in the laboratory, measuring performance of the detail regarding durability and strength
5. Present the results of this study in a final report discussing the findings of the research for future use of integral abutment bridges with accelerated bridge construction.

CHAPTER 2. LITERATURE REVIEW

2.1 Connection Details for Prefabricated Bridge Elements and Systems (2009)

The integral abutment bridges previously built using ABC techniques commonly utilize precast pile caps and girders and rely on cast-in-place closure pours to form the integral connection (Fig. 1) [3]. The disadvantage to cast-in-place closure pours is that they use rapid curing high performance concretes which add significant cost to the project. One means of eliminating the closure pour is through the use of grouted rebar splice couplers (Fig. 2). Grouted rebar couplers function by inserting steel reinforcing bars into a sleeve which is then grouted shut. The splice is capable of developing the full strength of the steel reinforcing over a short distance and has been around for several decades. Grouted rebar couplers often create tight construction tolerances when large amounts of splices are present, due to this integral abutments have seen rare used with the technology.

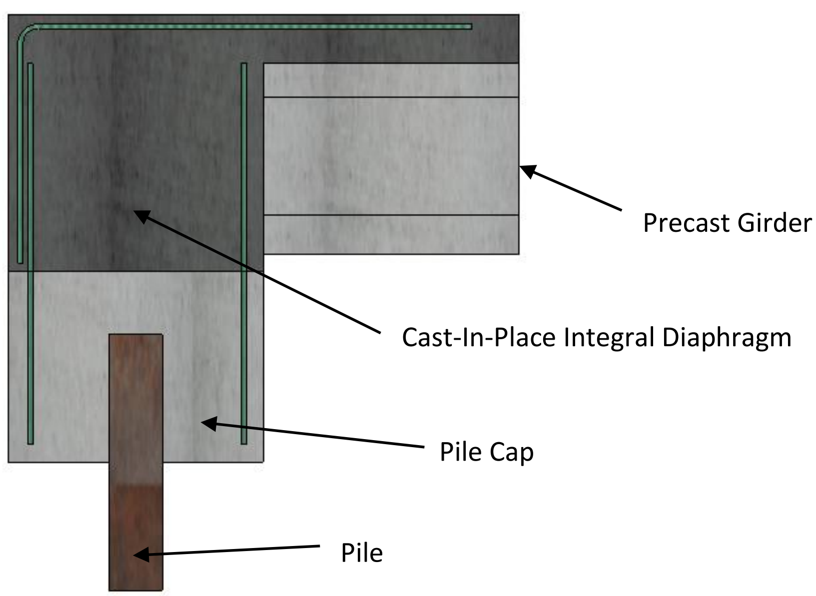

Figure 1. Integral abutment with closure pour

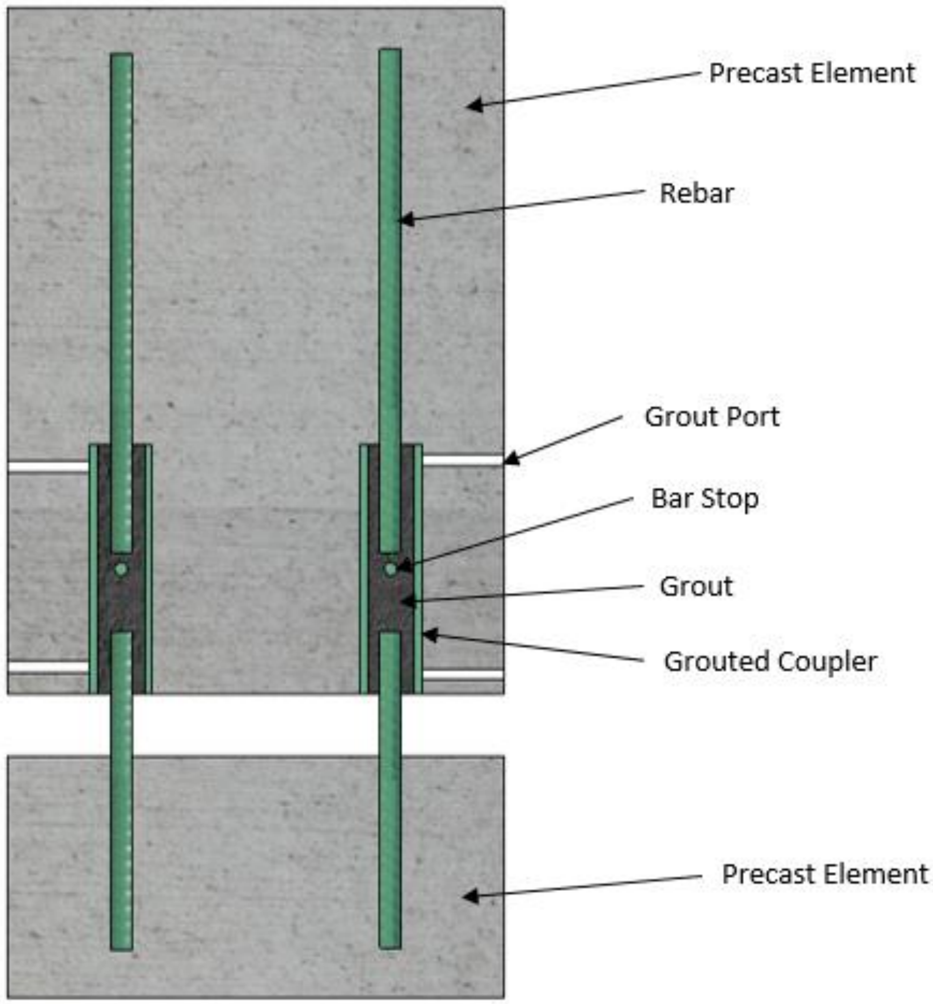

Figure 2. Grouted coupler conceptual drawing

Though not widely utilized, grouted couplers are gaining in popularity and have been successfully used in ABC projects like the Mill Street Bridge in Epping, New Hampshire [3]. This bridge is unique in the fact it was constructed entirely from modular precast elements which utilized grouted couplers to make all of the precast connections. The dimensional tolerances in precast elements when using grouted couplers are a major concern for contractors. Techniques like match casting and measuring couplers and reinforcing locations from a single point are used to minimize construction errors and ensure field alignment. To increase constructability for the precast elements in the Epping Bridge, the design team oversized the grouted reinforcing splice couplers by two sizes, which is acceptable for some types/brands of rebar couplers. Utilizing these strategies, the Mill Street Bridge was successfully erected in eight days.

2.2 Accelerated Bridge Construction Manual (2011)

ABC projects that use an integral abutment design typically utilize a cast-in-place closure pour to form the integral connection between the superstructure and the substructure [2] (Fig. 3). The abutments are often prefabricated and post tensioned transversely, or connected using grouted shear keys. To reduce the size of the integral diaphragm closure pour, the option of a prefabricated backwall can be used. The alternative to cast-in-place closure pours are mechanical splice connections such as grouted rebar couplers. Couplers are attractive because the mechanical connection is fast and strength is achieved rapidly. The limiting factor when using grouted couplers is the dimensional tolerances associated with aligning the steel reinforcing and grouted couplers.

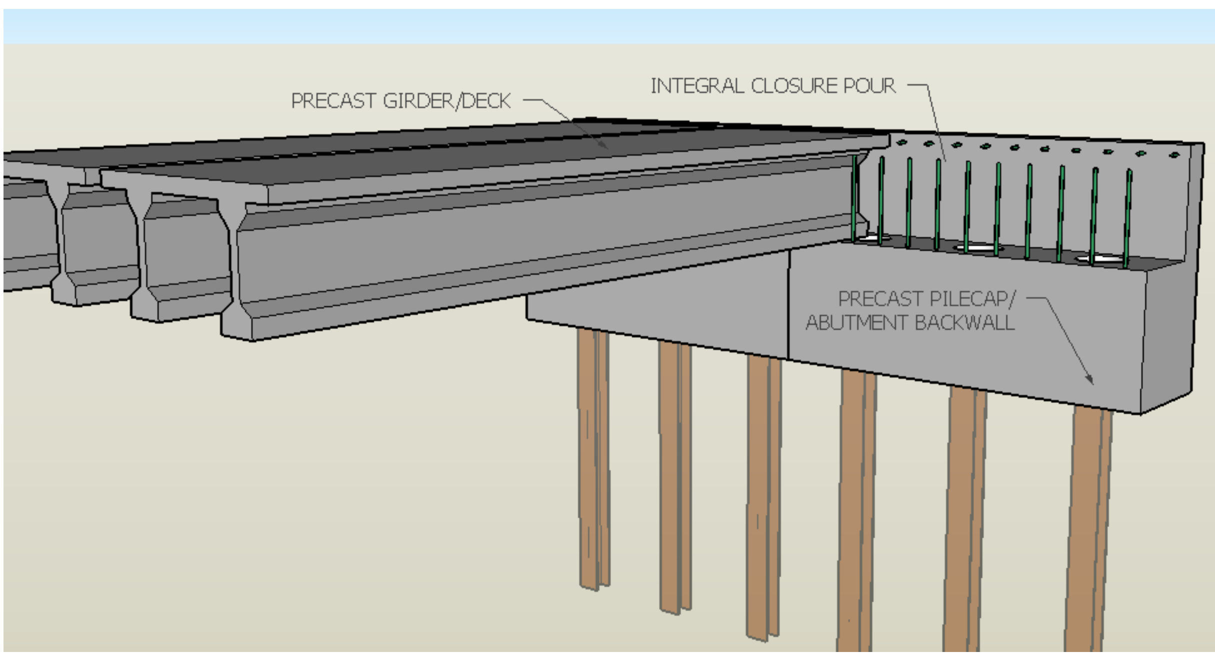

Figure 3. 3D Integral abutment with closure pour

2.3 Innovative Bridge Designs for Rapid Renewal ABC Toolkit (2014)

The substructure designs for ABC projects described in this toolkit are based on the assumption that pile driving will occur within +/- 3 inches of the specified plan locations [13]. Integral abutments are desirable for use in ABC projects as they offer a variety of benefits including faster initial construction speed, enhanced service life and lower lifetime maintenance costs. Integral abutments typically have a single row of abutment piling which saves construction time and material costs. The long term durability is improved as there is no expansion joint or beam bearings that require maintenance and/or replacement. The use of the integral abutment bridge is also advantageous for use in seismic areas were a common problem is the unseating of beams after an extreme event.

The elimination of beam bearings in an integral abutment bridge also improves the tolerance issues associated with erecting precast beam elements if a cast in place integral diaphragm is used. The cast in place integral diaphragm is easy and fast for contractors as there is limited formwork required to place this concrete. On the other hand, the use of fully precast elements is desirable to maximize erection speed. In Fig. 4 a full precast integral abutment system is shown where the pile cap and integral abutment are connected using steel dowels and grouted voids to allow for generous construction tolerances. In scenarios when the precast elements are too heavy, such as in a heavy abutment system, voids should be placed inside of the elements. Once the elements are in place the voids are to be filled with self-consolidating concrete to complete the element.

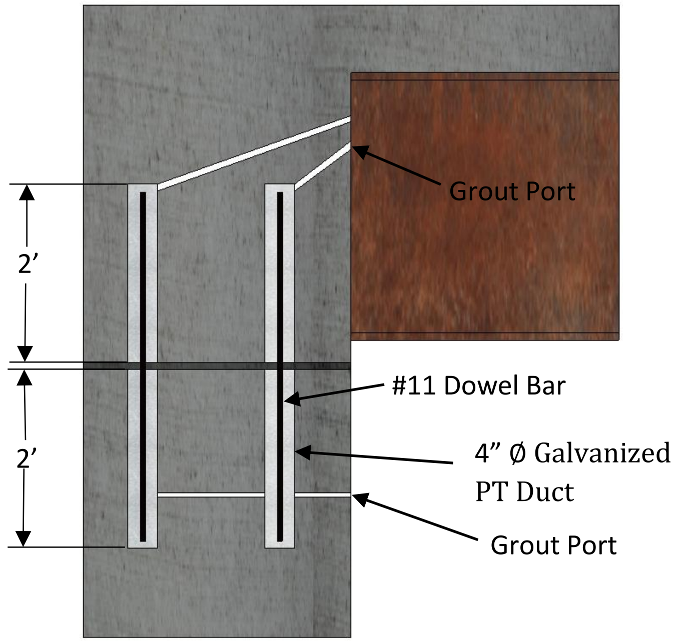

Figure 4. Integral Abutment with bar dowels

The report also recommends two specifications for the contract documents which facilitate the use of grouted rebar couplers. The first specification is the requirement of a template to place the grouted rebar couplers and steel reinforcing bars in the field. The second specification is that the precast elements should be “dry fit” to check for proper alignment before leaving the fabrication yard. These two practices will ensure that the elements were fabricated properly and facilitate erection of the bridge. Additional recommendations for the design of elements using grouted rebar couplers are they are placed on the bottom side of the precast elements so debris will not fall into the couplers. The reinforcing bars located on the top of the precast elements also facilitate the storage and transportation as the bars are less likely to be bent out of position.

2.4 Laboratory and Field Testing of and Accelerated Bridge Construction Demonstration (2013)

The Keg Creek Bridge, near Massena, Iowa was built entirely using modular precast elements [11]. The Keg Creek Bridge is a three span, two lane, semi-integral abutment bridge, which is a common layout that could serve as a template for thousands of future ABC projects. The substructure of the Keg Creek Bridge utilized grouted rebar couplers which spliced the reinforcing bars between the precast footings, columns and pier caps. The use of grouted rebar couplers and precast elements allowed the substructure to be erected in a few days, where months of work would have been required to create similar cast-in-place components. The downside to using grouted rebar couplers is that construction tolerances are often tight between precast elements. In order to ensure alignment between precast elements, a template was used to tie the rebar cage and to hold the grouted couplers. The template was seen as critical to the success of the system and is promising for use in future projects [9].

The erection of the superstructure was accelerated through the use of modular elements comprised of steel beams, a precast concrete deck and precast semi-integral abutments with an overhanging backwall. The longitudinal and transverse deck joints, along with semi-integral abutments allowed for adequate construction tolerances when placing the superstructure. The modular deck elements had reinforcing that protruded into the longitudinal and transverse deck joints, which were filled with ultra-high performance concrete (UHPC) to create moment resisting connections. During a bridge inspection post construction, it was noticed that efflorescence appeared on the underside of the longitudinal joints, indicating that chlorides had penetrated the deck joint from the top of the bridge deck. The use of joints in

ABC is critical and information regarding long performance should be monitored. Overall, the Keg Creek Bridge demonstrated that the use of precast elements can be successfully used to erect a three span bridge in two weeks.

2.5 Plastic Energy Absorption Capacities of #18 Reinforcing Bar Splices under Monotonic Loading (1994)

In the AASHTO code, reinforcing steel bar splices are required to develop a minimum of 125% Fy of the reinforcing bar. The only splice allowed in plastic hinge zones is the full penetration weld which is undesirable from a constructability standpoint [11]. In order to investigate the use of other bar splices, the research team investigated the ductility of the full penetration weld, grouted rebar coupler and other splicing technologies. In some scenarios the splice a splice may not developing the minimum yield strain 0.00207 before the connected bars fail. A need exists to establish a requirement for ductility of splices that will allow for the dissipation of energy in a seismic event. In all but one of the NMB grouted splice sleeve tested in this study, the bar fractured outside of the coupler in monotonic loading after bar the bar yielded. One coupler violently ruptured, after the minimum yield strain was developed, it was later determined that the coupler failed due to a manufacturing defect. Further investigations should investigate the rotational capacity of the hinge created by the yielding reinforcing bars when spliced with grouted rebar couplers in full structural concrete members.

2.6 Evaluation of Grout-Filled Mechanical Splices for Precast Concrete Construction (2008)

The Michigan DOT performed laboratory testing on the NMB Splice Sleeve and the Lenton Interlok grouted rebar couplers [8]. These couplers are capable of simulating traditional cast-in-place construction by providing continuity between the reinforcing steel bars in precast elements. The need for rapid erection of bridges has led to an increased demand and use of the grouted coupling technology. The prefabrication of integral abutments is desirable for use with grouted couplers because of the fast field connections. This combination has been previously used with success on the Route 9N over Sucker Creek, in Hague, New York, 1992. This bridge used grouted rebar couplers to connect precast deck elements to the precast abutment wall stem (Fig. 5). The use of integral abutments is desirable for rapid replacement projects as the need for expansion joint and beam bearings is eliminated.

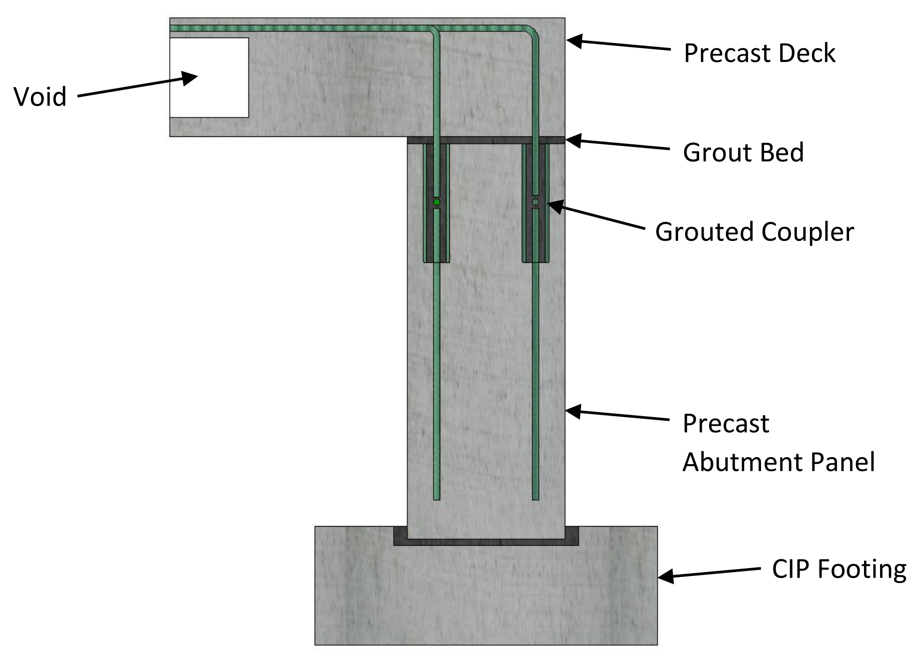

Figure 5. Grouted rebar coupler joining precast deck and abutment

Prior to using this technology in departmental projects, the Michigan DOT desired a better understanding of how these couplers perform in: strength, fatigue, slip, and creep with respect to the AASHTO LRFD requirements. After testing, the couplers met the requirements in pre and post-fatigue slip testing, having less than 0.010 in of displacement. The couplers met the requirement that 125 percent of F_y must be developed in the steel reinforcing bars prior to failure of the system. Creep testing demonstrated that the splices were not vulnerable to displacement under a sustained load in magnitude of 40 percent F_y of the steel reinforcing. Lastly, none of the systems failed after one million cycles in a fatigue test where stress ranged between 6ksi and 26ksi. Subsequently, the research team recommended that the grouted rebar couplers be approved for use in Michigan DOT projects. Further research is suggested by the research team in the investigation of the effects of misalignment in the bar splices. This effect is desirable to understand, as perfect alignment may not be the case in construction projects.

2.7 High Strain-Rate Testing of Mechanical Couplers (2009)

Due to the often congested areas in concrete construction encountered when lapping steel reinforcing bars, mechanical splices has become popular to alleviate the congestion [12]. Mechanical splices have studied and proven to be an effective and simple means to splice steel reinforcing; however, there have been little studies that investigated the performance of these splices under high strain rates such as blast loading. One type of splice investigated which relates to this research is the grouted rebar coupler, where two bar ends are grouted into the coupler. There were six grouted couplers tested in pure tension, two in each of the following categories: slow, intermediate and high strain rates. The dynamic tensile strength of these spliced connections had a good performance in all three strain rates. The ductility of the bars achieved in this loading condition was poor in comparison to the control bars tested. Additional tests should be performed in order to evaluate the performance of the grouted rebar couplers when used in structural concrete.

2.8 Precast Column-Footing Connections for ABC in Seismic Zones (2013)

The use of grouted couplers has increased as the need and demand for ABC projects has increased [5]. The ability of grouted couplers to splice rebar between precast elements to simulate cast-in-place construction has made them a popular choice for bridge designers. Currently, the use of grouted couplers with ABC in seismic zones has been limited because of the performance uncertainties relating to the new technology. Concern exists for the use of grouted couplers between column and footing connections, where energy must be dissipated in seismic events through nonlinear deformations. The goal of the research was to investigate the use of grouted couplers and headed couplers for ABC connections in moderate to high seismic zones. The researchers constructed five, half scale, column to foot connections that included: a cast-in-place typical detail, two headed coupler and two grouted rebar coupler details. Performances of the headed coupler and grouted coupler details were similar to that of the cast-in-place detail with regards to energy dissipation, force-displacement ratios and damage progression. After testing the headed reinforcement coupler connections and grouted rebar couplers were removed and inspected for damage. Consistent through all models, the splicing was undamaged while the longitudinal bars experienced failure. The headed reinforcement connections had a marginally better performance with respect to the cast-in-place model; however, this method of splicing featured tighter construction tolerances and was more time consuming to connect. Due to the performances of the analytical and experimental models created in this research report, the researchers suggested the removal of the restrictions placed on grouted rebar couplers by AASHTO in seismic zones.

2.9 Laboratory Connection Details for Grouted Coupler Connection Details for ABC Projects (2015)

With the increase in demand for precast bridge elements, often times new technologies have been used before major advancements in empirical and theoretical relationships exist [6]. The grouted rebar coupler that is often used to connect precast elements falls into this category. The majority of research on this technology has focused on a direct tension test that may not accurately represent conditions met in the field. In order to investigate the grouted rebar coupler in a realistic application, a precast element system was fabricated for testing in the laboratory (Fig. 6). The system tested #14 epoxy coated reinforcing bars that are spliced by epoxy coated grouted rebar couplers manufactured by Dayton Superior. The precast joint for the first five specimens utilized W. R. Meadows 588-10k grout for the bedding material. The ability of the grouted rebar coupler to develop flexural capacity between elements was investigated in three loading cases. The loading cases for the specimen were pure bending, axial load plus bending and a cyclical test of the system in pure bending. Overall, the static testing demonstrated that the empirical calculations utilized in the design of the specimen were accurate.

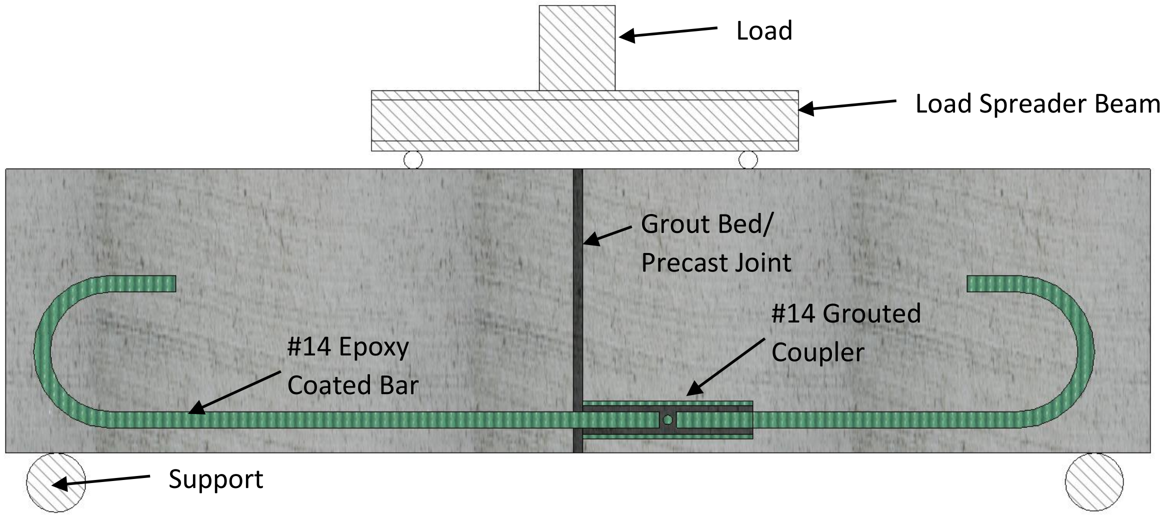

Figure 6. Grouted rebar coupler precast laboratory specimen

The crack located at the precast joint in this case opened almost immediately under load and the application of axial load to the specimen had little effect. The last specimen was fabricated was unique in the fact it used a UHPC grout bed, which marginally increased the load required to crack the joint. The specimen subjected to one million cycles of fatigue stress placed the reinforcing splice at 18ksi of stress in accordance with AASHTO LRFD design specification. The total deflection observed in this test remained constant through the million cycles and the crack width at the precast joint did not exceed 0.02 inches.

Additional specimens were created to measure the susceptibility of the spliced connection to chlorides which is of concern to bridges where de-icing salts is present. These specimens consisted of #14 epoxy coated steel bars spliced with a grouted coupler and placed in the center of an 8" diameter concrete cylinder. The joint at the specimen was un-cracked and this specimen was soaked in a 3 percent chloride solution for six months. Periodic readings were taken and no evidence of corrosion was seen.

CHAPTER 3. ABC INTEGRAL ABUTMENT DETAILS

The primary objective of this research was to investigate integral abutment details for ABC through laboratory testing. Since integral abutments with integral diaphragm closure pours have already been used successfully in the ABC community, this research focused on eliminating the closure pour through the use of a precast pile cap and integral diaphragm. The following sections discuss the design philosophy for integral abutments, the development of ABC details chosen for this investigation, and potential applications of the ABC details.

Each detail was evaluated on constructability, insuring contractor friendly practices can be used to construct and connect the precast pile cap and integral diaphragm successfully in the field. In addition to this, it was desired that the connection be comparable in strength and durability to a cast-in-place integral abutment, giving agencies confidence in the use of the new technology. To test the details, loads were simulated through the use of actuators, load frames and a reaction floor to evaluate the strength and durability of the precast connection between the pile cap and integral diaphragm. The strength criteria evaluated the shear and flexural capacity of the precast connection, while the durability criteria examined the crack width that developed at the precast and or cold joint, in addition to monitoring additional cracking that may develop at other location. Crack widths were measured in order to provide information on the design’s vulnerability to water and chlorides that could infiltrate the construction/precast joint and corrode the reinforcing.

3.1 Cast-in-Place (CIP)

In order to evaluate the new ABC details in this investigation, the research team constructed a traditional cast-in-place integral abutment to establish baseline performances in constructability, strength, and durability. In general, an integral abutment is designed so that the superstructure and the substructure are rigidly connected, creating continuity, and a joint-less bridge. During thermal expansion and contraction of the superstructure, translation with small rotations of the pile cap is desired by engineers. To achieve this, engineers design the foundation piling below the pile cap to be relatively flexible, allowing the entire abutment to translate and rotate without inducing extreme forces in the foundation and superstructure. To design the connection between the integral diaphragm and the pile cap, vertical reinforcing steel is placed across the cold joint so that the connection is capable of developing the sum of all the plastic moment capacities of the foundation piles [7]. Engineers also turn this vertical reinforcing steel along the back face of the diaphragm into the deck, providing additional flexural strength for the negative moment region that exists in the girder and deck at this location.

The standard integral abutment detail from the Iowa DOT was chosen to serve as the cast-in-place specimen in this laboratory investigation. The detail is shown in plan view in Fig. 7, and again in a section view taken through the girder in Fig. 8. This standard detail is similar in design to those used by other agencies, and involves a cold joint with compression and tensile reinforcement to rigidly connect the pile cap and integral diaphragm. To provide better carry over and correlation with the study of the ABC details developed for this research, the width of the Iowa standard abutment was increased from 3 to 4 feet for the laboratory investigation.

This change was made so that the cast-in-place specimen would share the same width as the ABC details investigated in order to make valid comparisons of constructability, strength and durability. The resulting cast-in-place laboratory specimen is shown in plan view in Fig. 7, and again in section view through the girder in Fig. 10. It should be noted that the laboratory detail was constructed without the foundation pile in the pile cap. This was left out in order to simplify the test configuration.

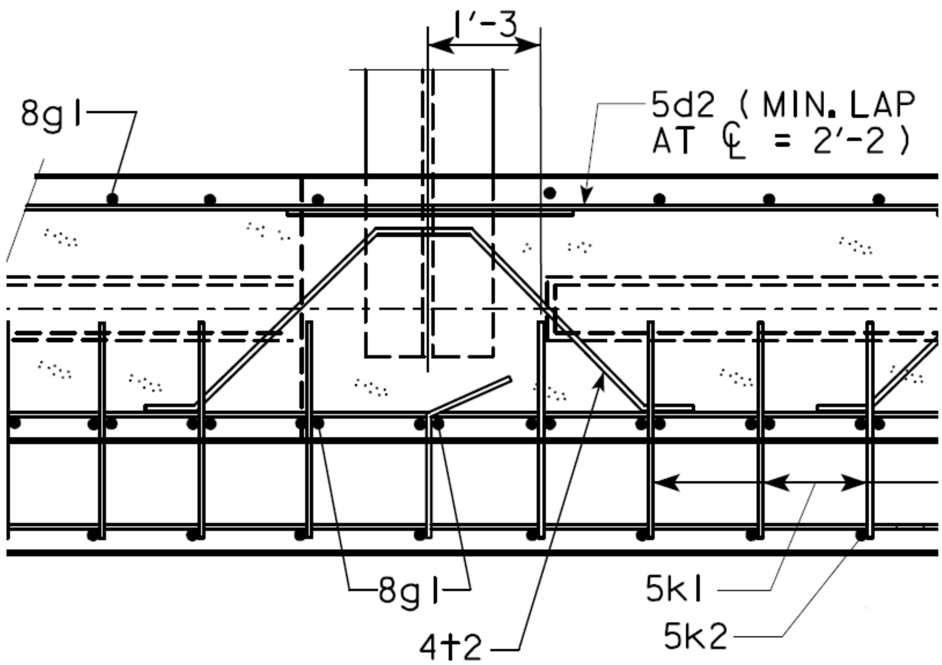

Figure 7. Integral abutment plan view from Iowa DOT

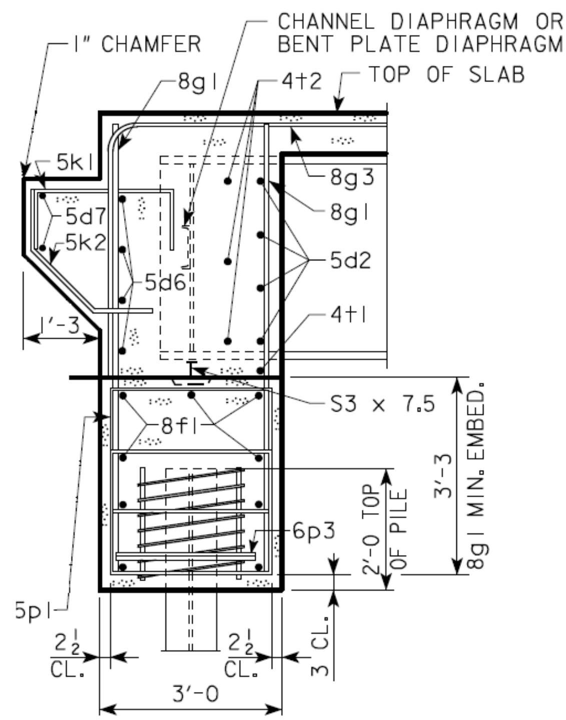

Figure 8. Integral abutment section view from Iowa DOT

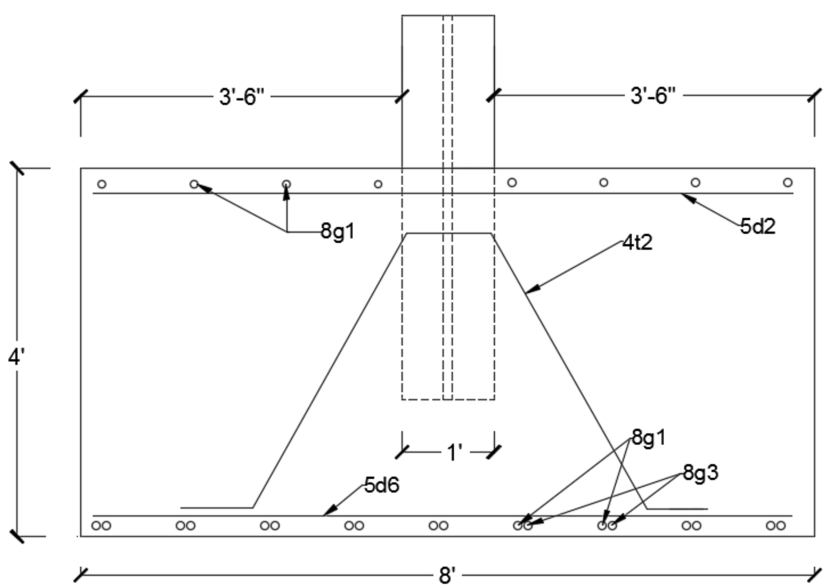

Figure 9. Cast-in-place specimen plan view

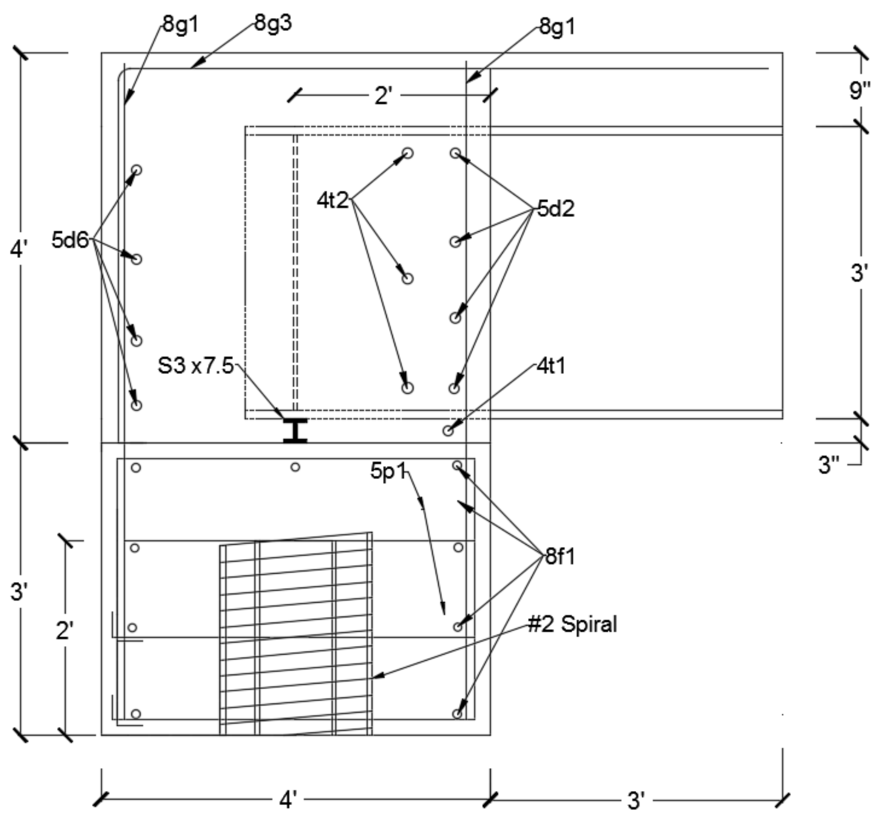

Figure 10. Cast-in-place specimen section view

3.2 Grouted Rebar Coupler

The first ABC integral abutment detail selected for laboratory investigation is one that splices the pile cap and the integral diaphragm using grouted rebar splice couplers, and is referred to here as the grouted rebar coupler detail. The detail conceptually applies itself well to a precast element system such as the one shown in Fig. 11. The chances of success for the grouted rebar coupler detail are maximized in this scenario, when a longitudinal (along the length of the bridge) and a transverse (across the length of the bridge) closure pour are utilized. These closure pours minimize the number of grouted couplers that require alignment per precast element connection, and also eliminates a precast element that requires alignment at both ends. The reinforcing steel bars protruding from the pile cap add complications to the constructability aspect of the bridge if slide-in-bridge construction was attempted using this detail. For a slide in bridge, the superstructure would require jacking, sliding in an elevated state, and simultaneous lowering and alignment of a large quantity of grouted couplers.

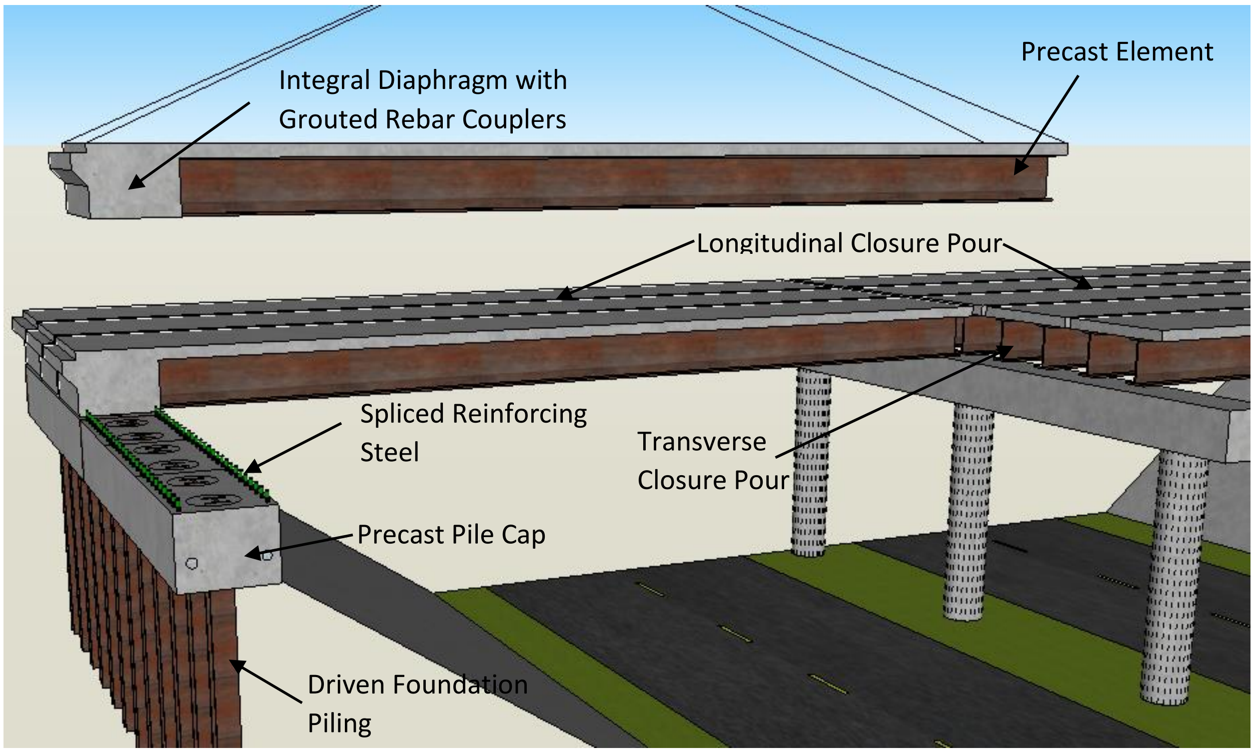

Figure 11. PBES using integral abutment with grouted rebar couplers

As previously mentioned, the width of the standard cast-in-place Iowa pile cap was increased from three to four feet to suit the ABC application. This modification in width allows for a precast pile cap, shown conceptually in Fig. 12, to be cast with CMP pocket voids that fit over top of the driven foundation piling. The pile cap is then connected to the foundation piles by filling the CMP voids with a specially designed concrete chip mix which provides a strong pile-to-pile cap connection [4].

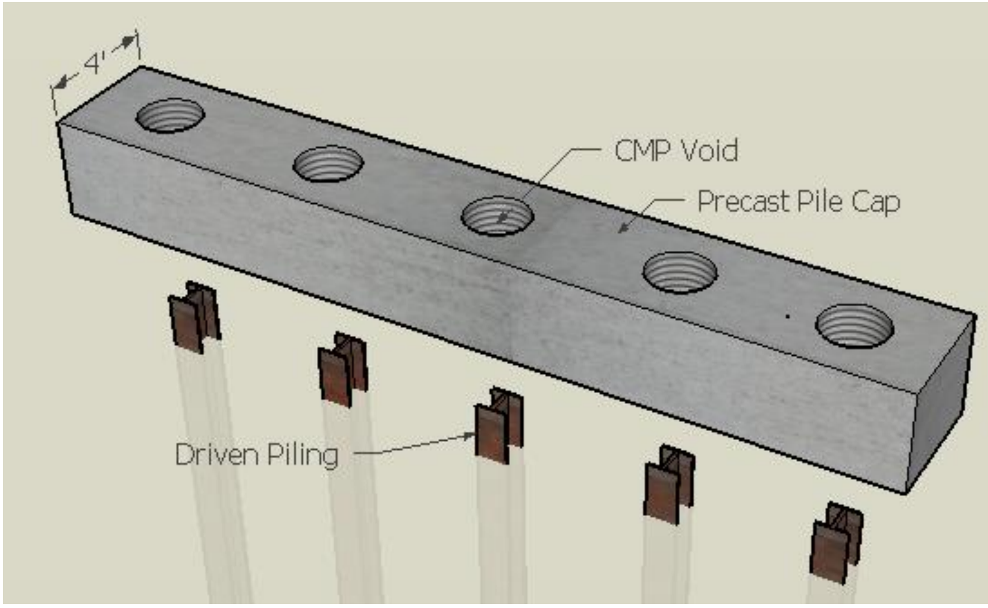

Figure 12. Precast pile cap

To test an integral abutment detail that utilizes grouted couplers, the standard Iowa cast-in-place abutment detail was modified to include the use of Dayton Superior's sleeve-lock grouted rebar splicing system. To modify the design of this detail, the typical cold joint was replaced with a precast joint and the reinforcing steel across the cold joint are spliced using grouted couplers. In plan view the specimen looks the same as the cast-in-place specimen, which is eight feet in width, and required splicing and coupling of seventeen reinforcing bars spaced at one foot intervals. The width of the laboratory specimen is similar in width to a precast element that might be used in the field as eight and a half feet is the maximum transportation width. Since these two are relatively the same size, information on constructability from the laboratory would apply well to an element system created at a precasting plant. Should the elements be precast on site however, these dimensions can/will change depending on the contractors, and their equipment's capabilities.

The resulting laboratory detail utilizing grouted rebar couplers is shown in a section view taken through the grouted rebar couplers in Fig. 13, and again in a section view taken through the girder in Fig. 14. It should be noted that the laboratory specimen was constructed as appears in the two section views except that the foundation pile and CMP were not included to simplify the test setup. To create a flat surface for the precast elements a ¾” grout bed was detailed at the precast joint. The vertical bars passing through the joint, marked 8g1 in Figs. 13 and 14, were spliced using grouted couplers. Since the grouted couplers are larger in diameter than the reinforcing steel, additional concrete cover is provided by moving the vertical bars closer to the center of the section. This design modification slightly reduced the moment arm between the effective internal tension and compression force couple that resists moment within the section. Since the precast elements are lifted and moved into position, additional reinforcing is provided to resist flexure and shear forces that develop in the elements. These bars consist of longitudinal bars marked 8f3, as well as stirrups marked 8p3, 5p2 and 4p1, shown in Figs. 13 and 14.

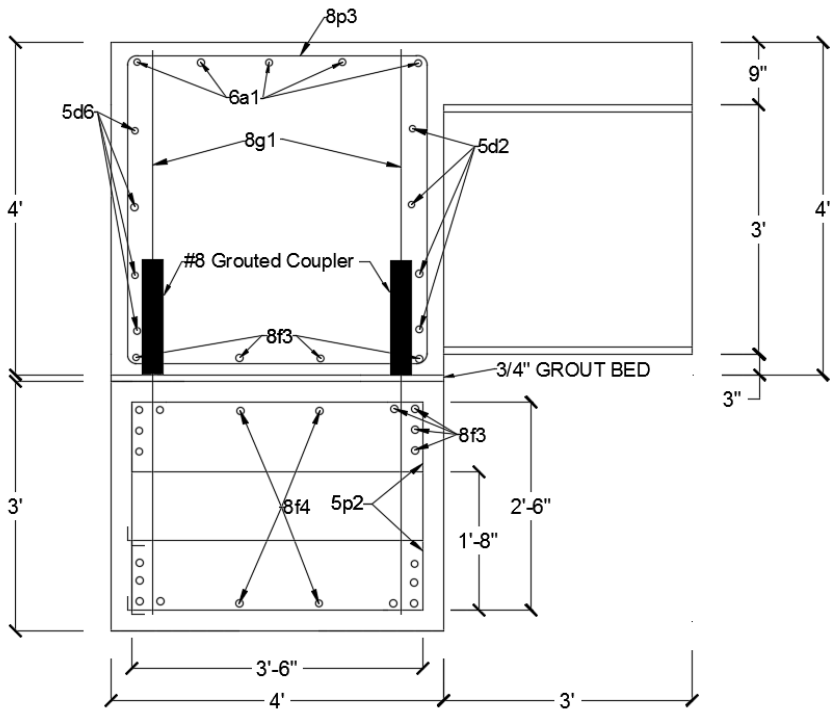

Figure 13. Grouted coupler section view through couplers

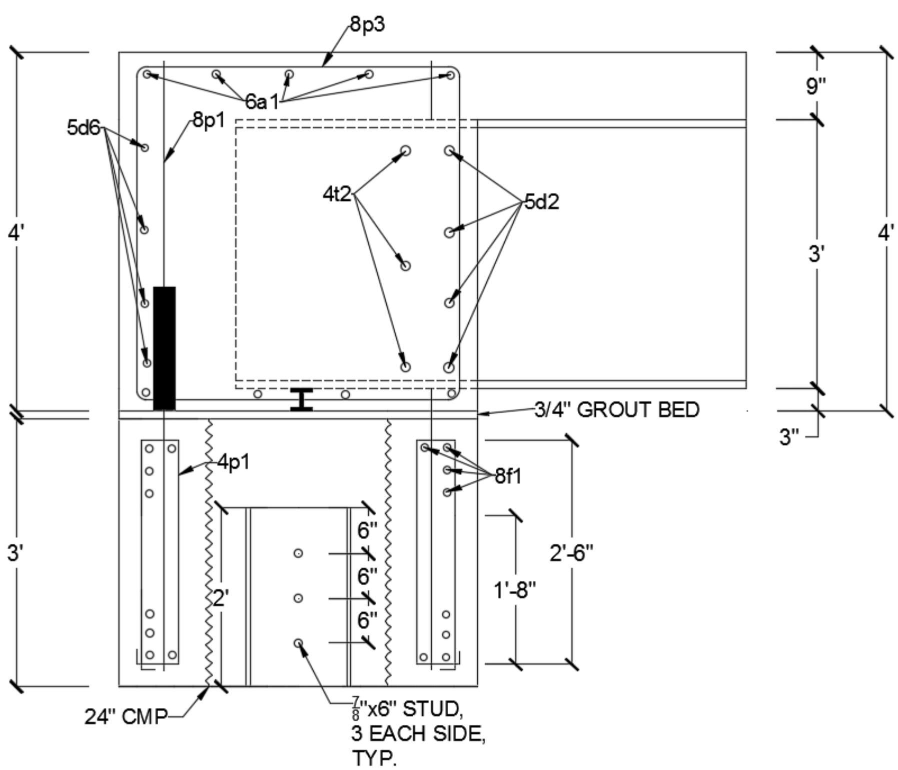

Figure 14. Grouted coupler section view through girder

3.3 Pile Coupler

The pile coupler detail was developed and designed to facilitate the use of integral abutments with a slide in bridge application, and is shown conceptually in Fig. 15. The pile coupler design aims to minimize the number of mechanical connections between the integral diaphragm and pile cap to facilitate the use of slide-in-bridge construction. The pile coupler design uses a two foot length of HP section and a 24” diameter CMP to essentially create a large grouted coupler that splices the integral diaphragm and pile cap. The resulting detail and dimensions of the laboratory specimen are illustrated in plan view in Fig. 16 and in subsequent section views in Figs. 17, 18 and 19. The philosophy of this design is that the connection between the pile cap and integral diaphragm is designed to develop the sum of the plastic moment capacities of the foundation piling; so the same section used for the foundation piling could also be used to splice and couple the pile cap and integral diaphragm. The key for success lies in the ability of this detail to develop the strength of the relatively short HP section within the grouted connection.

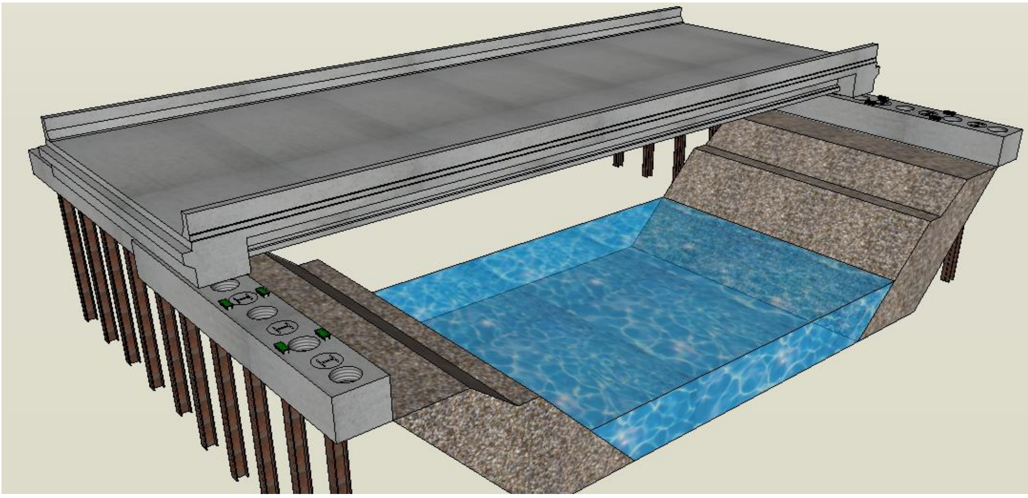

Figure 15. Slide in bridge using pile couplers

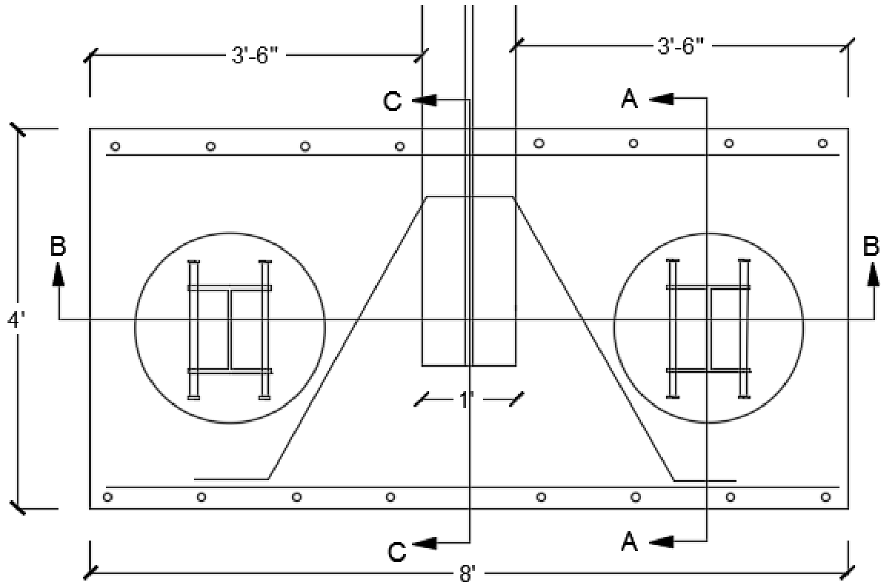

Figure 16. Plan view pile coupler

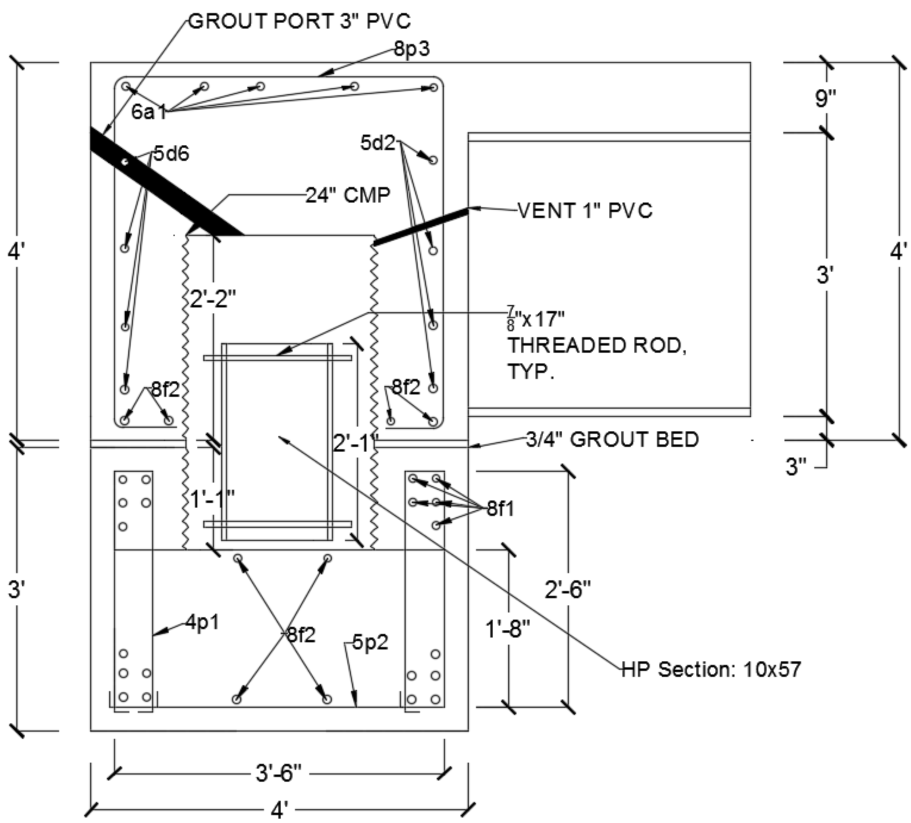

Figure 17. Section view A - pile coupler

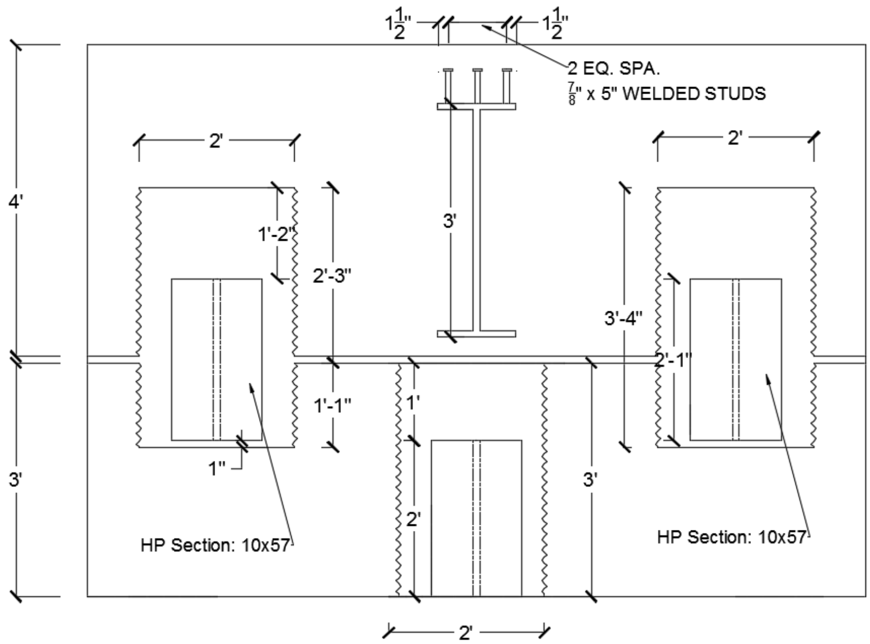

Figure 18. Section view B - pile coupler

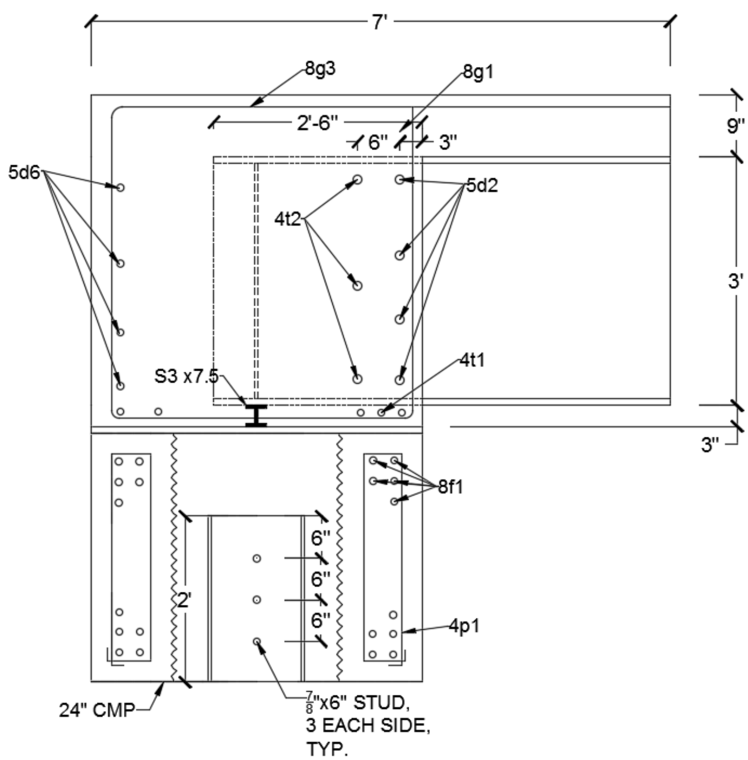

Figure 19. Section view C - pile coupler

The system facilitates the horizontal sliding of a full superstructure by suspending and containing the entire two foot HP section inside of the superstructure until the slide is complete. This eliminates the need to jack the sliding superstructure to pass over protruding reinforcing from the pile cap, such as is required in the grouted coupler detail. Once the superstructure is in position, the HP section is lowered into the CMP void present on the pile cap so that precast joint bisects the final resting position of the two foot long HP section (Fig. 17). The longest possible HP section is desirable to couple the pile cap to the integral diaphragm so the strength of the HP section can adequately be developed. However, the length of the HP section was limited to two feet, as the containment within the superstructure was seen as critical to the success of this detail for constructability reasons. To increase connectivity between the grout and the HP section, threaded rods are detailed for use as shown previously in Fig. 17.

Prior to grouting and casting the integral diaphragm a steel cable is attached to the HP section and strung through a hook on the CMP lid and out of the 1” vent. This allows workers to suspend and lower the HP section within the CMP void in the abutment. In order to guide and prevent the HP sections from rotating out of strong axis bending, reinforcing steel is welded to the lid of the CMP and fits in the 4 corners of the web and flanges of the HP section. In order to grout the CMP void once the HP section is lowered into place, a 3” diameter PVC pipe is cast into the diaphragm at an angle so that grout can be gravity fed (Fig. 17). A 1” PVC pipe is also cast into the diaphragm and doubles as an air vent and as a way to suspend and lower the pile (Fig. 17). The vent pipe is tilted upwards slightly so the CMP void fills entirely with grout, pushing out all the air inside, before the grout begins to exit out of the PVC vent. At this time, the vent is plugged and grout was poured until the 3” PVC vent was completely filled.

During the design of this detail there was talk amongst the research team and technical advisory committee of extending the CMP void to the top of the concrete deck. This would eliminate the suspension of the HP section and allow for the use of longer HP sections, as they could be placed into the voids once the bridge superstructure was slid in place. The subsequent grouting process would also be easier, as access to the voids would be on top of the bridge deck. Despite these advantages, this route was ultimately not chosen in order to avoid the resulting construction joint on the bridge deck where the use of de-icing salts is heavy during the winter months. The infiltration of chlorides at construction joints on ABC projects has been observed and the resulting effect on long term durability is unknown. Subsequently, partial depth voids within the integral diaphragm and pile cap were selected to avoid a construction joint on the bridge deck, which was seen as critical to the long term success of this detail.

CHAPTER 4. CONSTRUCTION

4.1 Cast in Place Specimen

Construction for the cast-in-place specimen began with the pile cap. Reinforcing steel was tied, formwork was placed and the concrete was placed and broom finished to create a good bonding surface to the integral diaphragm (Fig. 20). To construct the integral diaphragm and deck, formwork was attached directly to the pile cap, reinforcing steel was tied (Fig. 21), and a W36x150 girder was placed on the pile cap. Epoxy coated bars were used for the vertical bars that connect the pile cap to the diaphragm. Black bar was used in the rest of the abutment because the slip between the concrete and reinforcing steel was not seen as critical to the evaluation and performance of the detail in this study. The diaphragm and three feet of deck were cast monolithically, which is consistent with construction practices in the field. Fig. 22 shows the completed cast-in-place specimen in addition to the reaction blocks used for rigidly connecting the specimen to the floor.

Figure 20. CIP pile cap

Figure 21. CIP integral diaphragm

Figure 22. CIP integral abutment specimen

4.2 Grouted Rebar Coupler Specimen

To construct the grouted rebar coupler specimen, the pile cap was fabricated very similarly to the cast in place specimen, taking extra time to precisely place the vertical bars that connect the pile cap to the diaphragm. Since these bars are being spliced with grouted rebar couplers it was important that these bars be plumb and in the correct location, facilitating a proper fit later on. However, maintaining exact placement of the vertical bars is impractical, as the bars continually shifted while tying the reinforcing steel cage and during the concrete pour (Fig. 23). After casting, all of the vertical reinforcing steel was within ½" of the planned locations and relatively plumb (Fig. 24). Using a cheater bar, the reinforcing steel that had shifted during the pour were bent to the vertical position. To ensure the couplers would be properly aligned with the protruding reinforcing steel in the pile cap after casting the diaphragm, a template was created to 'match cast' the specimens. The template (Fig. 25) was a 4'x8' sheet of plywood that was laid over top of the pile cap reinforcing steel, so that the exact locations could be marked and then drilled into the template. Form plugs (Fig. 26) were then installed into the holes on the template and tightened to hold the grouted coupler tight to the template. With the template complete and the grouted couplers in place, the template served as the base for the formwork and the rest of the reinforcing steel was tied and the steel girder was moved into place (Fig. 27). The remaining formwork was erected and the integral diaphragm was cast separate from the pile cap (Fig. 28).

Figure 23. Pile cap rebar, formwork and pour

Figure 24. Grouted coupler pile cap

Figure 25. Grouted coupler template

Figure 26. Form Plug

Figure 27. Grouted coupler and rebar

Figure 28. Integral diaphragm and deck, formwork and rebar

With construction of the integral diaphragm complete (Fig. 29), a trial fit of the pile cap and diaphragm was made to insure that the 17 reinforcing steel bars and grouted couplers aligned. With proper alignment confirmed, ½" steel shims were placed on the pile cap to support the integral diaphragm during placement of the grout bed. To ensure that the bedding grout doesn't infiltrate the rebar couplers, seal plugs are placed on the protruding reinforcing steel bars, see Fig. 30 and 31. Next, the surfaces of the precast joint were wetted to the saturated surface dry condition (Fig. 32), and formwork was installed to cover and seal the precast joint (Fig. 33) in preparation for pumping the grout bed.

Figure 29. Integral diaphragm

Figure 30. Seal plug

Figure 31. Neoprene disk, seal plug and shim

Figure 32. Integral diaphragm placement

Figure 33. Grout bed formwork

Three holes were cut on the front and back of the grout bed formwork, as well as one hole on either side, so that grout could be pumped into the precast joint from multiple locations. Starting at one corner of the specimen, grout was pumped via a hand pump (Fig. 34) until clean grout started coming out of the hole on the opposite side of the specimen. This process was repeated, alternating back and forth from the front to back side of the specimen until no more grout could be pumped into any of the holes, plugging individual holes once it appeared that area was adequately filled with grout. Removal of the formwork and inspection of the perimeter of the joint indicated that the grout had adequately filled the bedding joint (Fig. 35). However, without opening up the joint completely it is not possible to know the adequacy of the grout coverage across the entire bedding area. Rough calculations were performed to determine the amount of grout required to fill the joint; this number closely matched the quantity of grout pumped into the joint, giving confidence that the entire joint or a large amount of the joint, had been filled.

Figure 34. Grout hand pump

Figure 35. Completed grout bed and coupler grouting

In preparation for grouting the rebar couplers, air was blown into each grouted coupler to clean the coupler of any dust and check that the top and bottom ports were unobstructed. Two of the grouted couplers did not pass the air test as grout from the bed seeped past the seal plug and partially filled the couplers blocking fill from the lower port. The grouted couplers labeled 2 and 17 in Fig. 36 were blocked. To grout the functioning couplers, grout was mixed one bag at a time, according to manufacturer recommendations, and poured into the hand pump. The nozzle of the hand pump was then placed into the bottom port of a grouted coupler and grout was pumped until clean grout flowed out of the top port. The top port was then immediately plugged and care was taken to quickly remove the nozzle at the bottom port and plug the port as quickly as possible. This is the process outlined and recommended by the rebar coupler manufacturer.

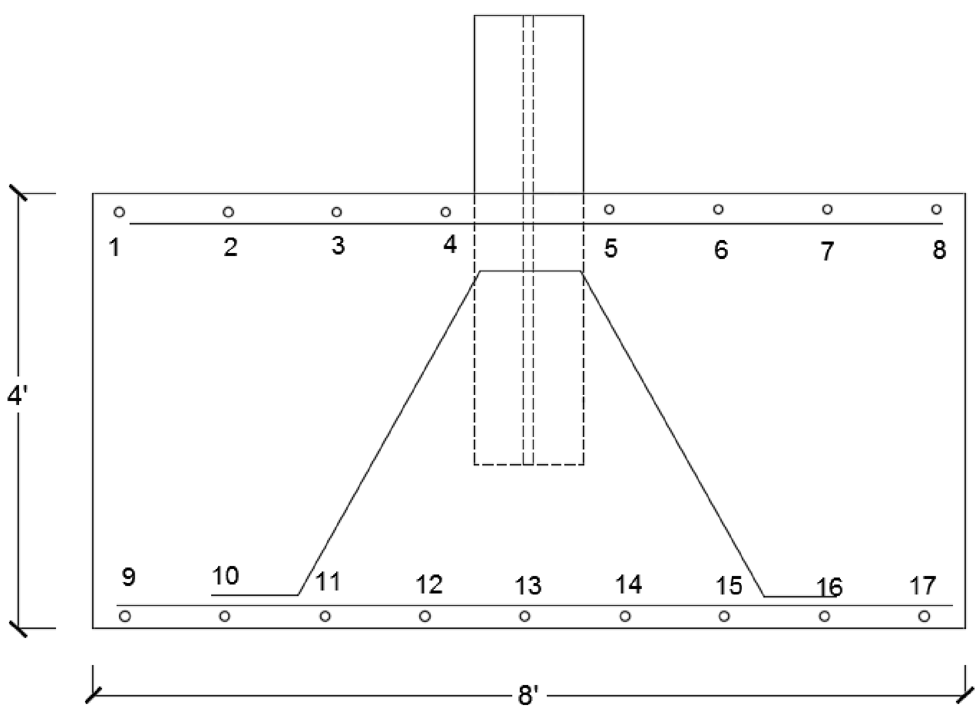

Figure 36. Grouted coupler layout

Overall, the grouted coupler specimen took more time and effort to construct than the traditional cast-in-place specimen. Most of the extra time was in the alignment of the spliced vertical bars in the pile cap. Fortunately, the template for the grouted couplers worked well and facilitated a successful fit for the precast element connection. Even though the precast elements were more challenging to construct, the crucial aspects for ABC projects is the erection time. Placing the precast element, grout bed formwork, grouting the bed and finally the couplers was a fast process and could be replicated in the field facilitating a quick erection of the bridge. A contractor should pay special attention to sealing the bottom of the grouted couplers during placement of the grout bed, so grout does not leak into the couplers and block the grouting operation as it did in this investigation.

4.3 Pile Coupler Specimen

To construct the pile coupler specimen, the reinforcing steel cage for the pile cap was tied with 3” PVC pipes fitted at the bottom of the cage, which are used to post tension the pile cap to the floor for testing (Fig. 37). Formwork was erected around the cage and a CMP was used to create a void in the pile cap for the HP section (Fig. 38). In order to seal the bottom of the CMP and create the void, plywood was cut into two half circles and placed in the bottom of the CMP which facilitated easy removal of the plywood after concrete placement (Fig. 39). To create the void in the integral diaphragm, a circular piece of sheet metal, 3/16” thick, was fabricated and used as a lid for the CMP. The thickness of the lid was chosen so that minimal deflection, less than a half inch, would occur from concrete pressure present on the lid during the pour. A U-shaped anchor bolt was installed in the center of the CMP lid which in combination with a steel cable, functioned as a pulley for suspending and lowering the HP section inside of the integral diaphragm. Holes were drilled in the steel lid and #4 reinforcing steel bars were welded in the holes, creating a guide system for lowering the HP section (Fig. 40). To attach the lid to the CMP, three holes 1/8" in diameter were drilled and tie wire was used to secure the two together (Fig. 41). Holes were drilled in the HP section and threaded rods and nuts were installed in lieu of shear studs (Fig. 42).

Figure 37. Pile cap rebar cage

Figure 38. Pile cap rebar, formwork and CMP

Figure 39. CMP void in pile cap

Figure 40. CMP lid with rebar guides and U-bolt

Figure 41. CMP with lid

Figure 42. HP section with threaded rods

Once the integral diaphragm reinforcing steel bars were tied, the CMP voids were placed inside the cage (Fig. 43). A 3” PVC duct with a flange was attached to the CMP lid and ran to the back side of the abutment formwork. The flange on the 3” PVC allows for the CMP void to be filled all the way to the top, before grout starts filling the 3” PVC pipe. A 1” hole was also drilled in the CMP for a ¾” inch PVC pipe that functioned as a vent and gave access to the pulley system (Fig. 44). A Steel cable was attached to the HP section and run through the U-bolt on the CMP lid and through the ¾” PVC pipe, where it remained during the casting of the integral diaphragm, and was retrieved after the formwork was removed. With the HP section and CMP void in place, the integral diaphragm was ready to be cast (Fig. 45). Once the concrete had been placed, finished, and cured, the formwork was removed and the steel cables connected to the HP sections were pulled tight. This suspended the HP sections, for transportation of the integral diaphragm (Figs. 46 and 47). Washer plates and clamps were used to hold the cable tight during the transportation and placement of the superstructure.

Figure 43. Integral diaphragm rebar cage

Figure 44. Side view of CMP void

Figure 45. Integral diaphragm deck rebar

Figure 46. Integral diaphragm with suspended HP sections

Figure 47. Suspended HP sections

Prior to placement of the diaphragm on top of the pier cap, 1.5" foam backer rod was placed around the CMP on the pile cap (Fig. 48) to create a dam around the CMP void. This was principally done in order to prevent grout bed material from filling the CMP void. The backer rod also alleviated inaccuracies in placement of the CMP voids, as the top and bottom CMP were not perfectly aligned. The surface of the precast joint was then wetted to the saturated surface dry condition before the diaphragm was lowered onto the pile cap. Once the integral diaphragm was in place, the metal clamps were taken off of the steel cable holding up the HP sections, and the piles were lowered. In the laboratory, several checks were made prior to the final install to insure and confirm the proper function of the pile lowering system. To place the grout bed, formwork was installed over the precast joint and the grout bed was pumped into place using the technique utilized in the construction of the grouted coupler specimen (Figs. 49 and 50). Following completion of the grout bed, grout for the CMP void was transported and poured using a barrel with a closable valve (Fig. 51). The system poured grout into a 3” PVC 90 degree elbow (Fig. 52), which funneled grout into the CMP from the 3” PVC tubing cast into the diaphragm. The CMP was filled with grout until the grout flowed out of the vent on the front side of the abutment, filling paused temporarily so the vent could be plugged, and the 3” PVC pipe was then filled to the top with grout. There was some settling of the grout inside of the PVC pipe, however the amount was less than two inches. Bleed water leaked through the grout bed in several locations, indicating a relatively high porosity in some locations in the grout bed (Fig. 53).

Figure 48. Pile cap with backer rod seal

Figure 49. Grout bed formwork back

Figure 50. Grout bed formwork front

Figure 51. Grout funnel system

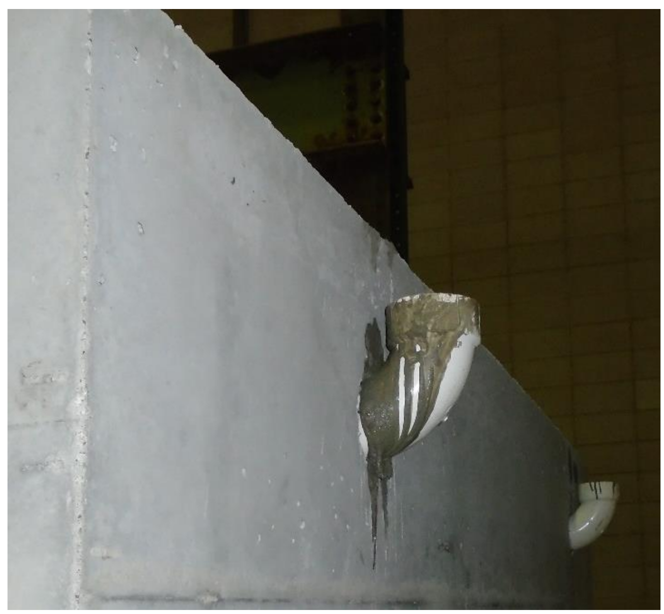

Figure 52. PVC pipe for receiving grout

Figure 53. Bleed water passing through the grout bed

CHAPTER 5. LABORATORY TESTING

5.1 Methodology

In order to determine the strength and durability of the integral abutment specimens, a reaction block and post tensioning system was designed to affix the specimen to the laboratory strong floor (Figs. 54-56). Using the reaction blocks, actuators, and load frames in the lab, forces were applied to the specimen simulating live loads and thermal loads. These loads tested the integral abutment laboratory specimen for strength and durability of the cold joint and precast joints, as well as the overall design of the surrounding integral diaphragm, pile cap and concrete deck. While testing durability by means of cyclical testing was not possible given the available resources in this study, information on durability by means of measuring crack widths present under load can be used to examine the risk of exposing the precast joint to water, chlorides, and debris.

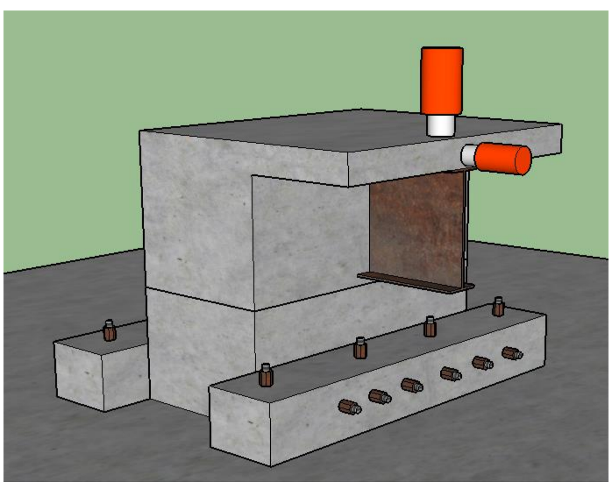

Figure 54. Lab test setup front 3D

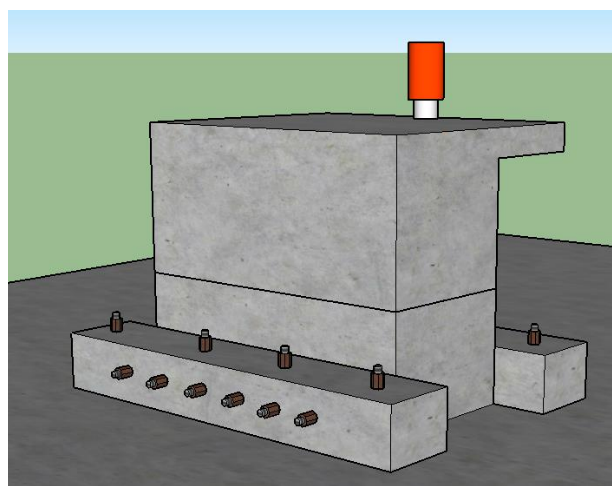

Figure 55. Lab test setup rear 3D

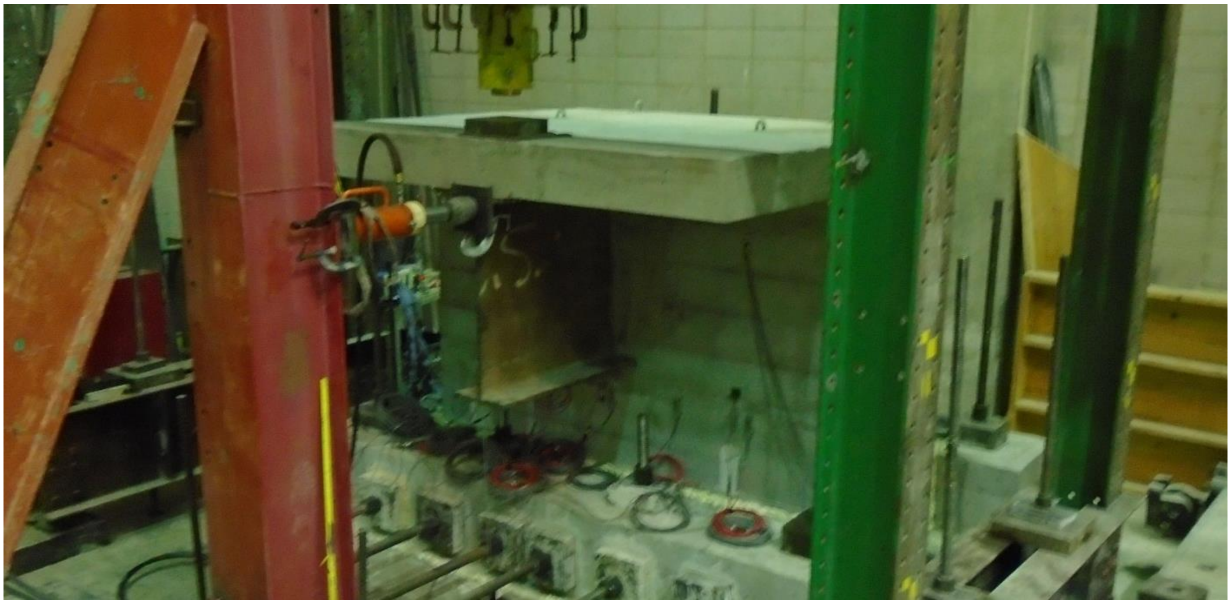

Figure 56. Laboratory loading, horizontal and vertical actuators

The reaction block and post tensioning system used in the lab is different from how the abutment reacts and behaves in the field, where translation and rotations of the abutment occur due to flexibility of the piles and girders. By not including piles in the laboratory specimen, which provide flexibility in an integral abutment bridge, a worst case loading scenario is possible.

The first load applied in the laboratory was the horizontal load, which developed tensile stresses in the front face of the abutment. This type of loading, according to the free body diagram in Fig. 57, simulates stresses that a full integral abutment bridge would experience during thermal contraction. The horizontal load chosen to be applied in the laboratory was 100 kips. This load was chosen after examining the thermal forces that could be resisted by the stiffness of the foundation piling and surrounding soil. The intent of the horizontal load is not to fail the specimen, but rather obtain expectations in performance of the abutment under service loading.

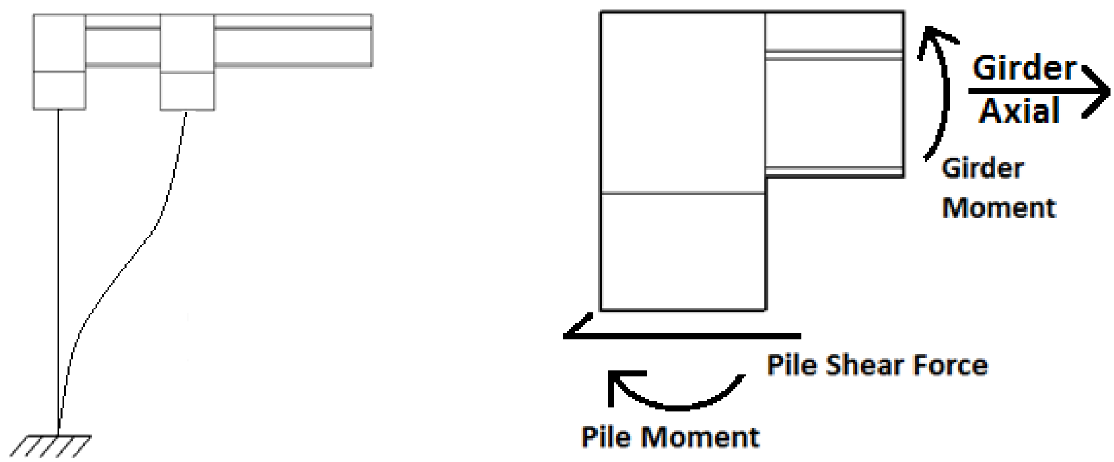

Figure 57. Thermal contraction and free body diagram

The second load applied in the laboratory, was the vertical load. This type of loading developed tension in the back face of the abutment, which according to the free body diagrams in Figs. 58 and 59, is the same type of stress developed in a full integral abutment bridge during live load and thermal expansion. The intent of this loading is to first examine the durability of the precast/cold joint through measuring the crack width that develops under the service loading condition. The vertical load in the lab was capable of applying a force of 400 kips, which is an applied moment of 2000 kip-ft, measured from the load to the center of the abutment. This load significantly exceeds stresses that are expected to be developed in the service loading condition and also the maximum possible stresses that can be developed given the relative strength of the foundation piling. In addition, this loading scenario was utilized in the prediction of the failure mechanism of the integral abutment detail. The resulting information can then be used to determine an accurate factor of safety in the details, as well as a range in the types of foundation piling that can be used in conjunction with the detail.

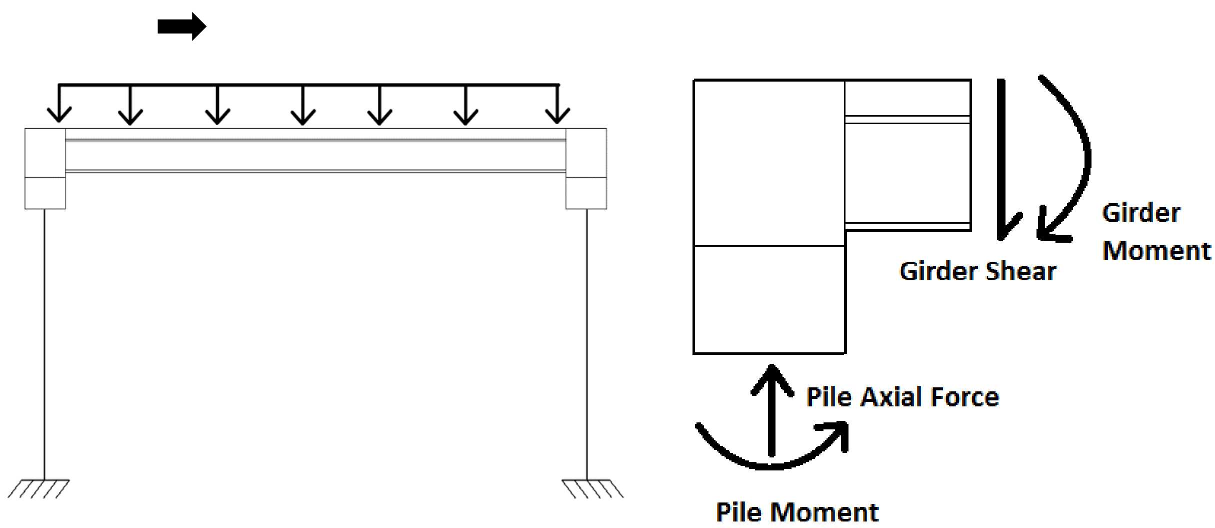

Figure 58. Live load and free body diagram

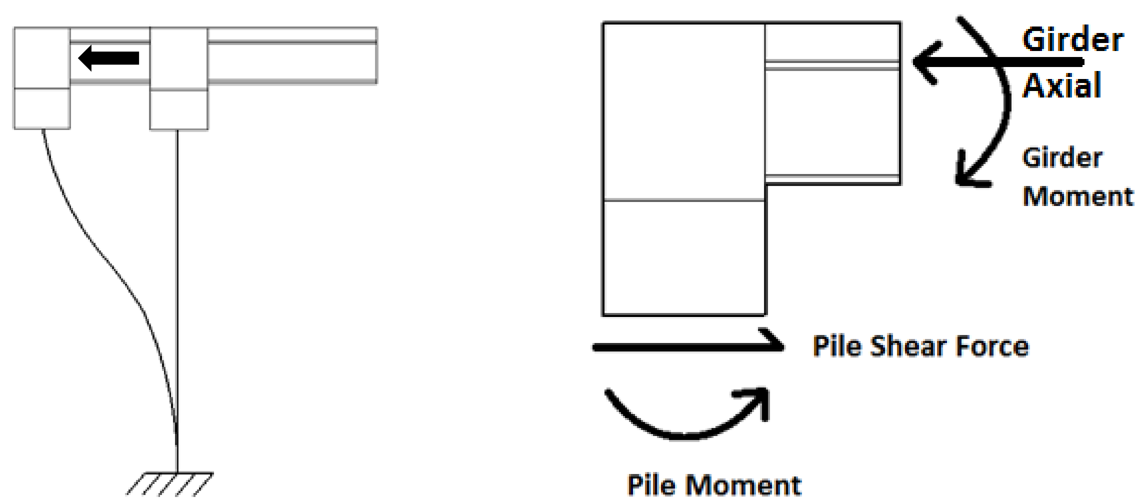

Figure 59. Thermal expansion and free body diagram

5.2 Instrumentation

To measure the durability of the specimens, the cold/precast joint between the pile cap and integral diaphragm was instrumented with displacement transducers. The displacement transducers were placed on the front side of the abutment during the horizontal loading, and on the back face of the abutment during the vertical loading. The transducers measured the width of the crack that developed on the tension face of the abutment in order to compare this information for the various specimens. Additional displacement transducers were placed on the rear side of the abutment during horizontal loading to measure horizontal slip between the pile cap and integral diaphragm. The locations of the vertical displacement transducers are represented by squares and the horizontal displacement transducers are represented by pentagons in Figs. 60, 61 and 62 for the different specimens and loading cases. Photographs of the typical instrumentation setup are shown for the horizontal loading test in Figs. 63 and 64.

To measure the strength of the specimens, strain transducers were placed on the tension and compression faces of the abutment two inches below the joint. The locations of these gauges on the specimen, and in the different loading cases, are illustrated in Figs. 60, 61 and 62. The gauges are also pictured in the horizontal loading test in Figs. 63 and 64. On the vertical "8g1" steel reinforcing bars which connects the pile cap to the integral diaphragm, sacrificial strain gauges were installed on the bars prior to casting the specimen. Two strain gauges were placed on each steel reinforcing bar, one 4 inches below the joint and the second 18 inches above the joint, which was directly above the grouted couplers. To measure the development in strength of the HP section used to splice the pile coupler specimen, three strain gauges were placed on each flange as shown in Fig. 65. The use of the concrete and reinforcing steel strain gauges allos study of the failure mechanisms of the abutments tested. This information allows for the comparison in strength of the ABC specimens to the standard integral abutment design, as well as comment on the relative strengh of the abutment in comparison to the foundation piling.

Additional displacements instrumentation using string pods to measure displacements between the specimen and the laboratory floor was used. This instrumentation was used to evaluate the tie down system used to restrain the specimen by measuring slip and calculating rotation of the specimen. After this information was examined, there were no significant rotations or slip that occurred during the testing of any of the three specimens.

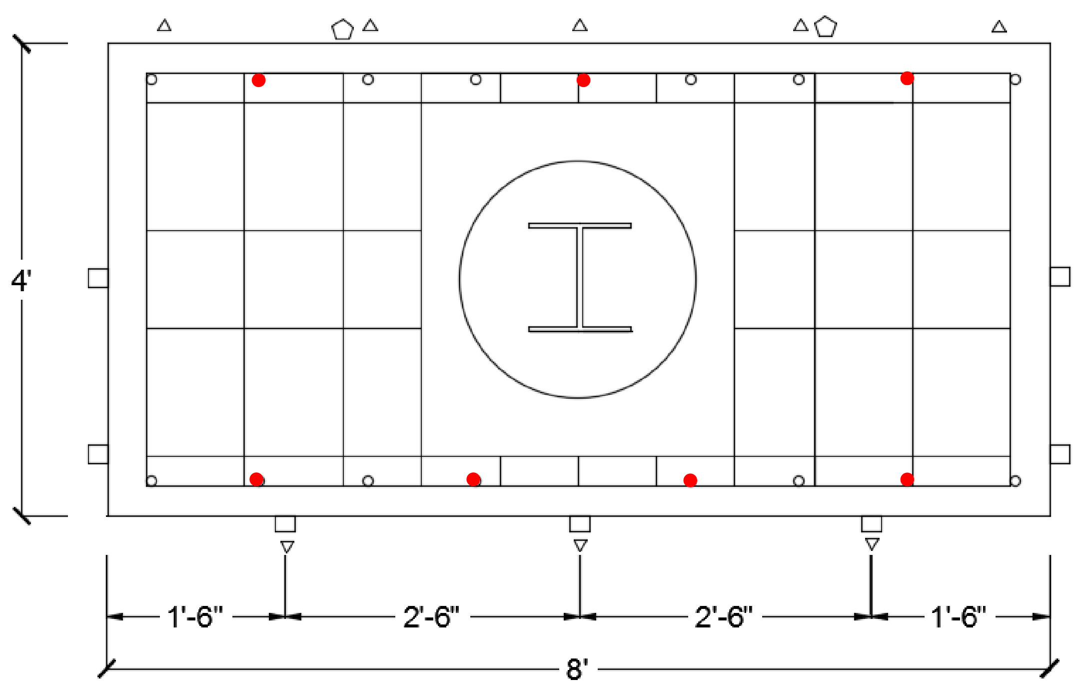

Figure 60. Plan view for horizontal loading, all specimens

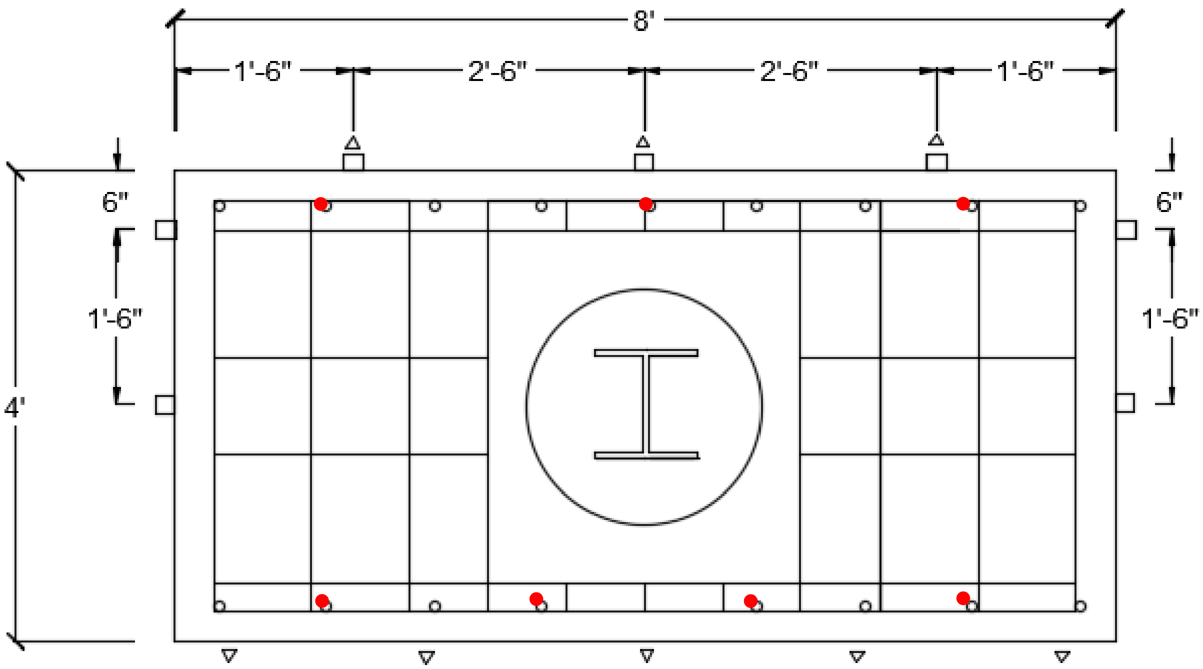

Figure 61. Grouted coupler and CIP instrumentation plan view for vertical loading

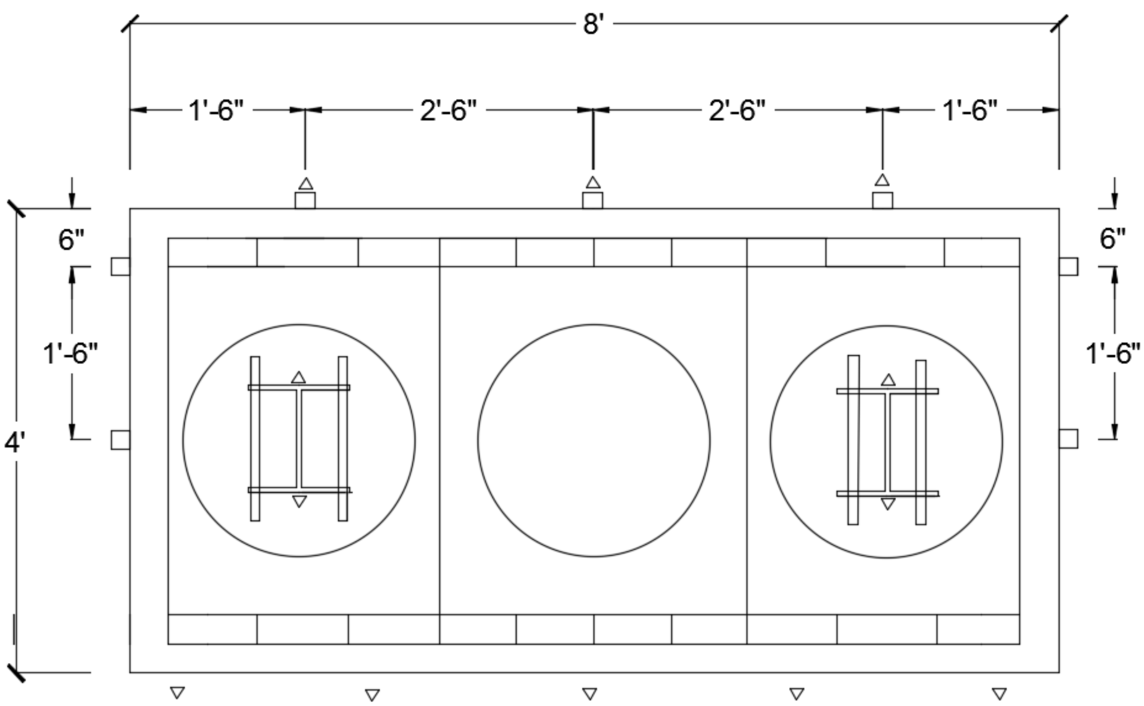

Figure 62. Pile coupler instrumentation plan view for vertical loading

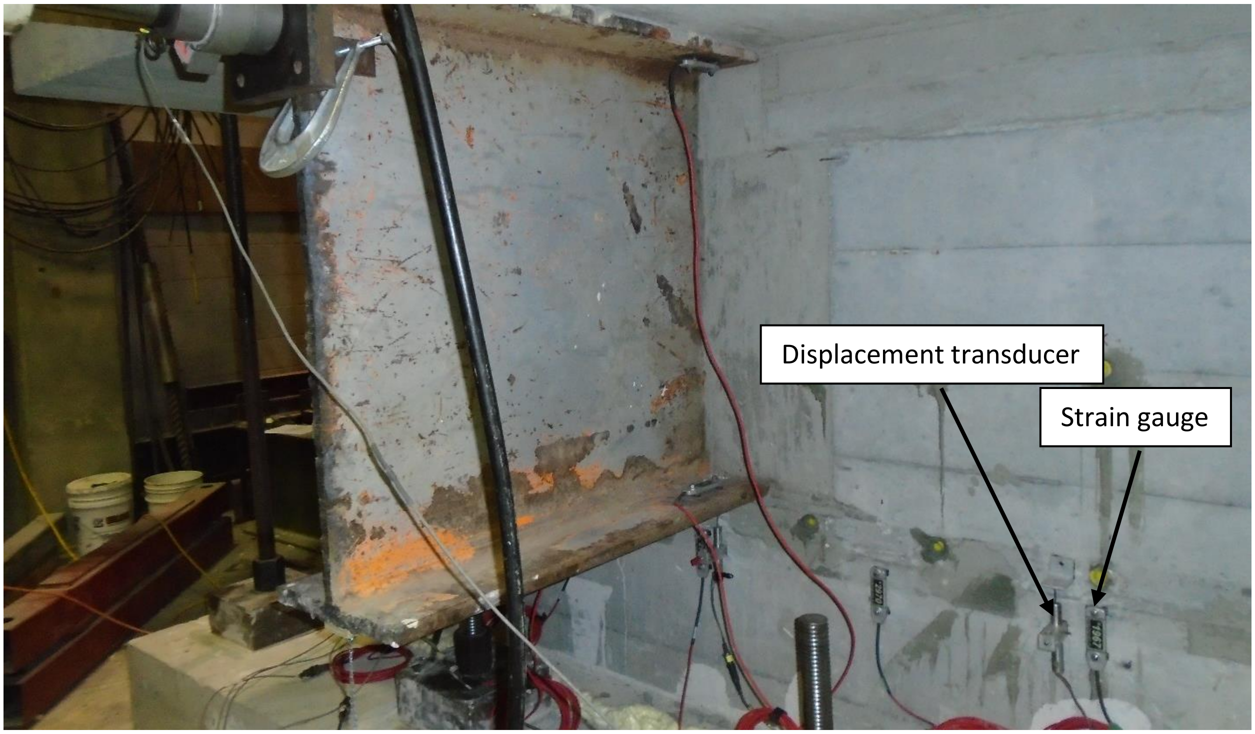

Figure 63. Front face of the abutment during horizontal loading

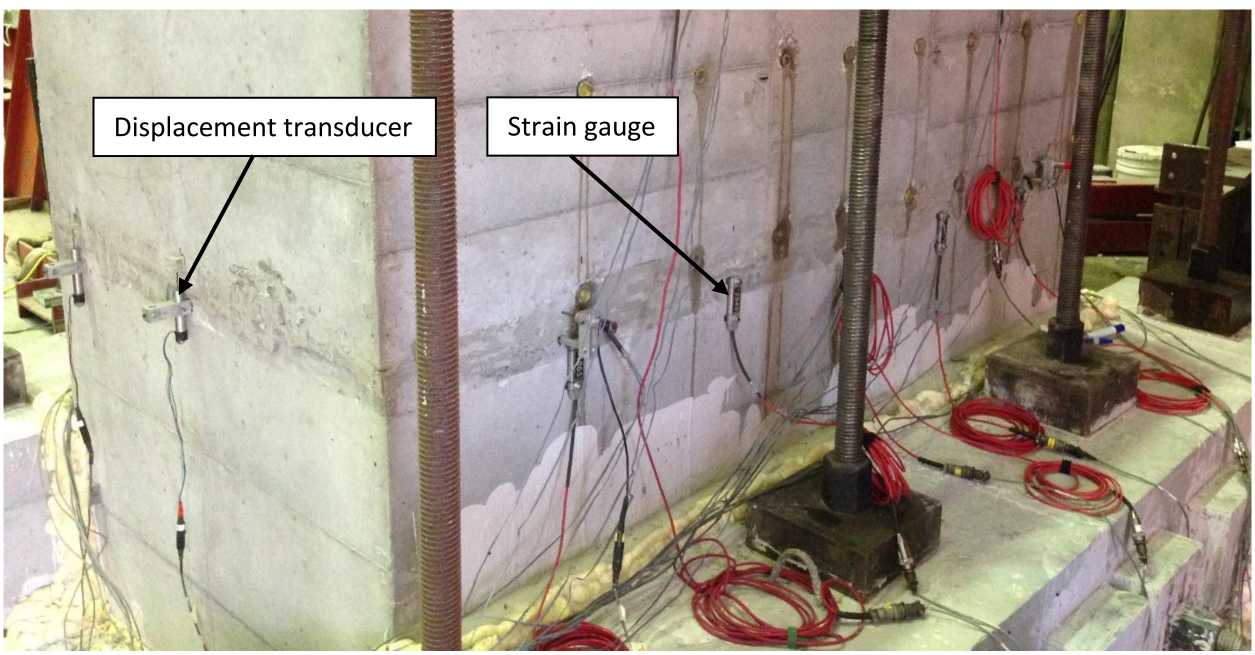

Figure 64. Rear face of abutment for horizontal loading

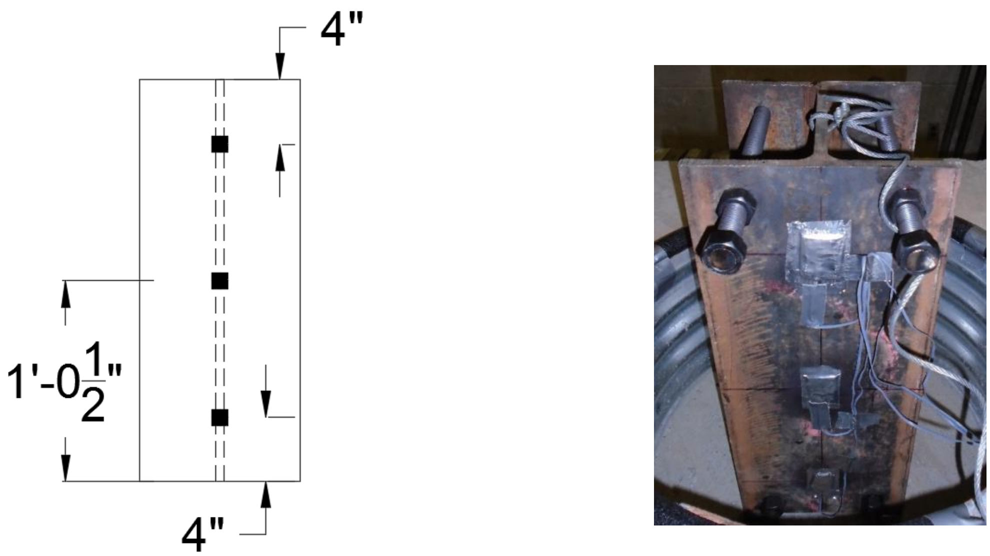

Figure 65. H pile instrumentation

5.3 Results

To summarize the results from the instrumentation placed on the laboratory specimen, a numbering system was created and is shown below in Fig. 66 and 67. The displacement transducers that measure the crack width were numbered, this numbering is used for both the vertical and horizontal testing, where the transducers are on the tension side of the abutment. In Fig. 67, the strain gauges present on the vertical reinforcing steel in the cast-in-place and grouted rebar coupler specimen have been labeled as well.

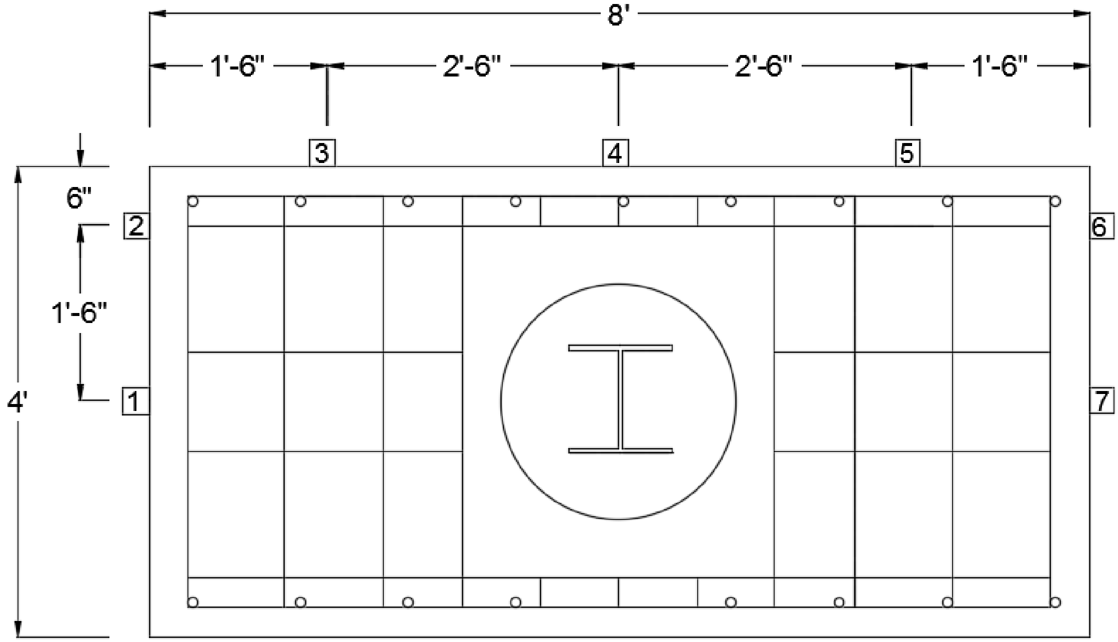

Figure 66. Displacement transducer numbering

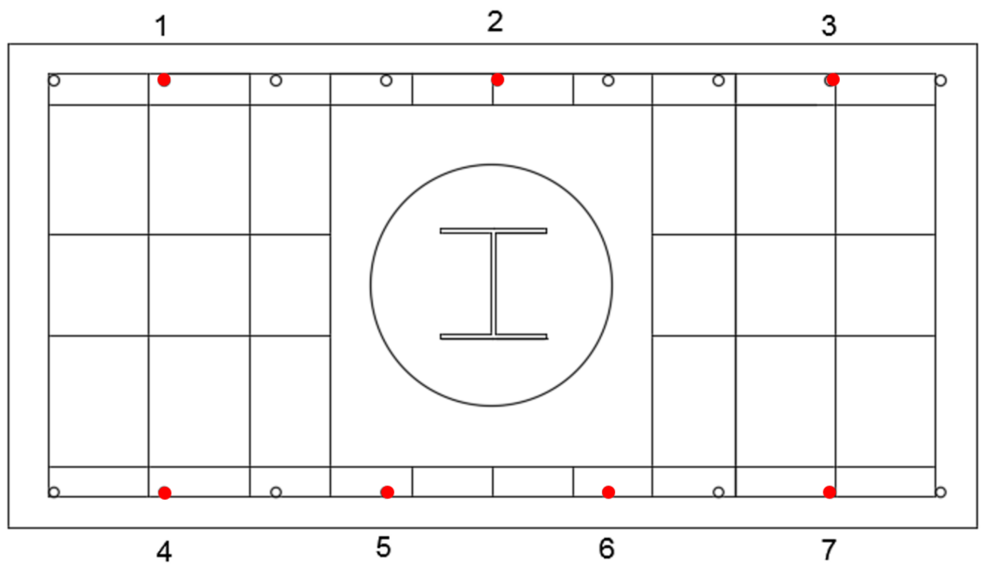

Figure 67. Rebar strain gauge numbering

5.3.1 Cast in Place

First, a horizontal load of 100 kips was applied to the cast-in-place specimen. This loading resulted in no significant signs of distress; only a minor crack that opened at the cold joint measuring 0.001 inch wide was observed. In Fig. 68 the width of the crack that developed between the pile cap and diaphragm at various locations is plotted against the applied moment. The applied moment is calculated by multiplying the load by the vertical distance from the load to the joint between the pile cap and diaphragm. The cast-in-place specimen had a maximum crack opening of 0.001 inches at an applied moment of approximately 310 ft-k and experienced no horizontal slip between the pile cap and integral diaphragm during the test.

Next, the vertical load was applied up to approximately 385 kips. The specimen showed no visible signs of distress at this point other than the crack that developed at the cold joint which reached a maximum width of 0.025 inches at a peak moment of approximately 1800 ft-k. The crack width vs applied moment is plotted below in Fig. 69, where the moment is calculated by multiplying the load by the horizontal distance to the center of the integral diaphragm/pile cap.

The maximum stress measured in the vertical reinforcing steel connecting the pile cap to the diaphragm was 42 ksi. The stress measured in the reinforcing bar is plotted against the calculated applied moment in Fig. 70. Since the specimen was still in the linear elastic range, the reinforcing steel would not have yielded until an applied vertical load of 550 kips, corresponding to an applied moment of 2590 ft-k. According to the strain measurements in the concrete, the failure would have been ductile in accordance with AASHTO Bridge Design Specifications [1]. The strains recorded in the strain gage located 18 inches above the construction joint were low, as the section remained un-cracked at this location throughout the testing.

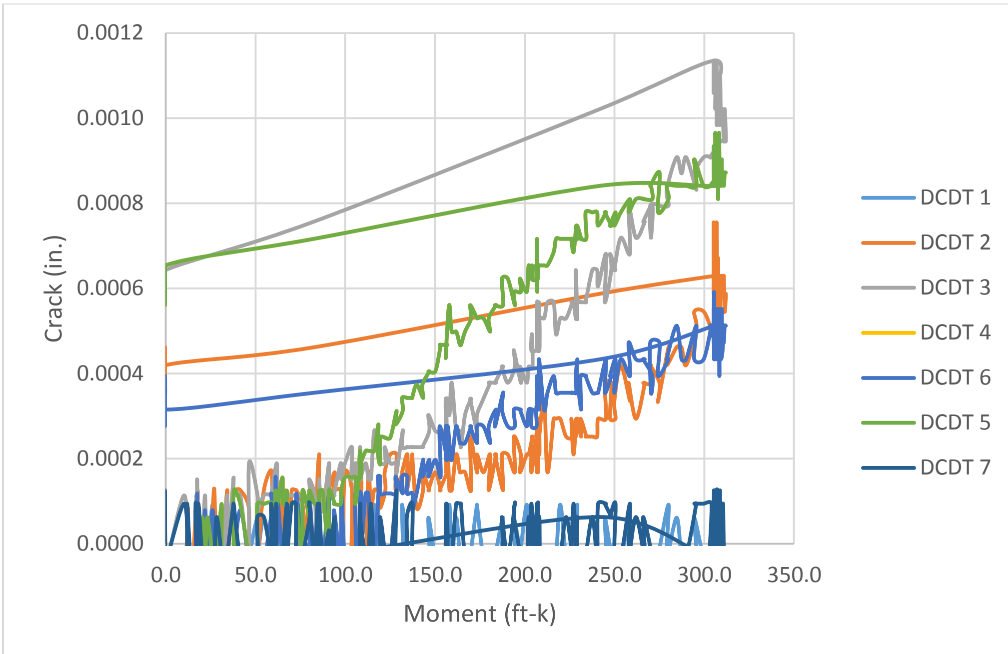

Figure 68. Crack vs. applied moment from horizontal load

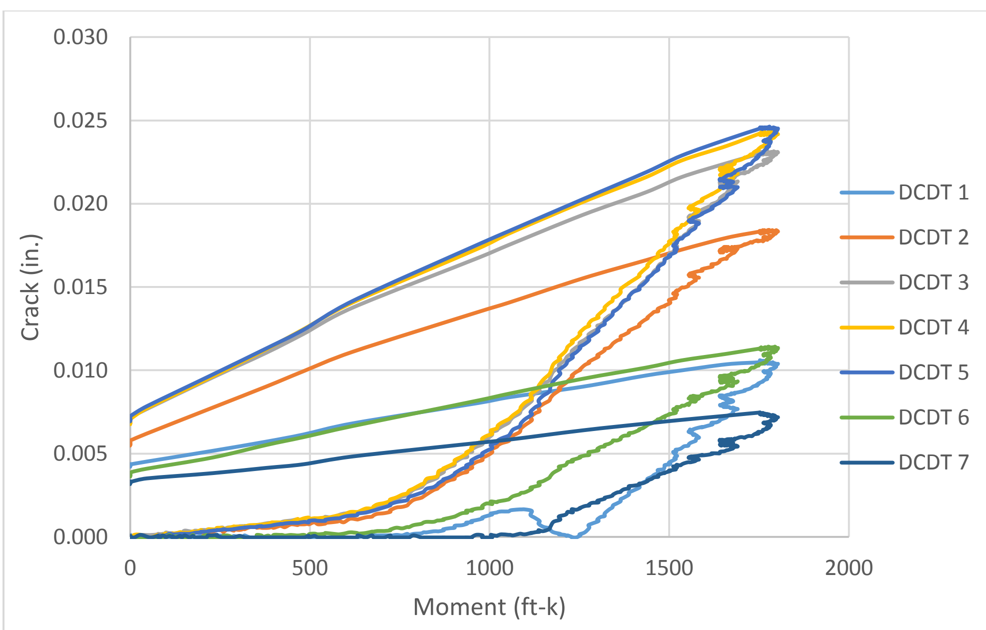

Figure 69. Crack width vs. moment from vertical load

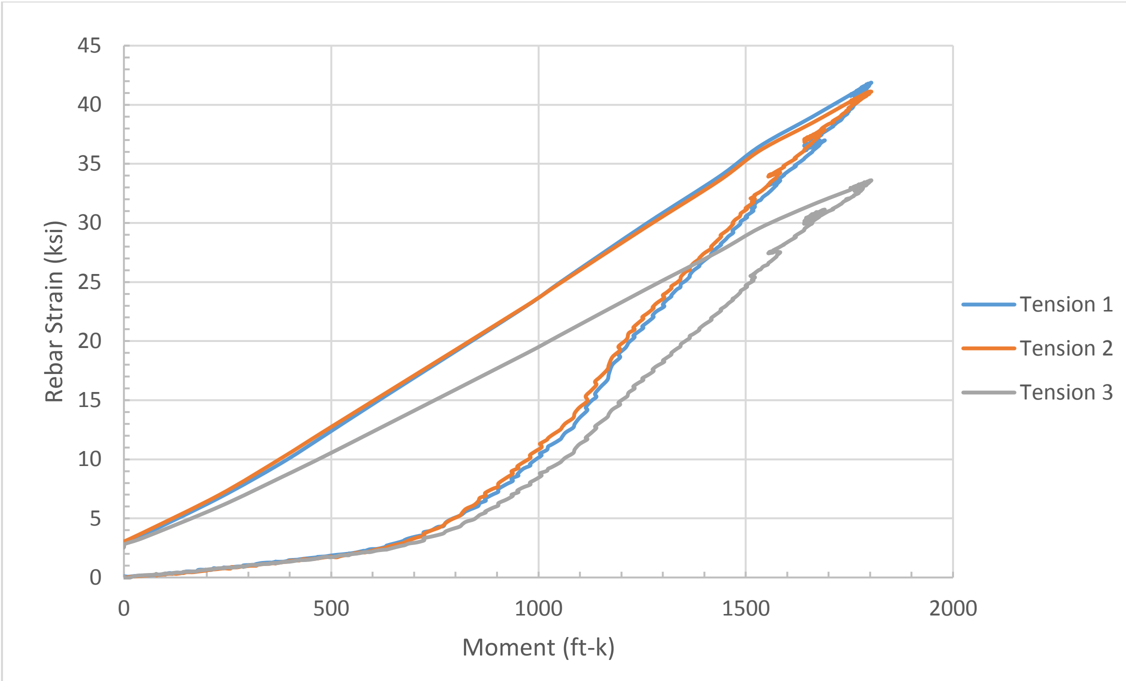

Figure 70. Tension rebar stress vs. moment from vertical load

5.3.2 Grouted Rebar Coupler

The grouted rebar coupler specimen was loaded horizontally up to 100 kips, at which point the maximum crack width on the front side of the specimen was 0.001 inches (Fig. 71), which exhibited a performance nearly identical to the cast-in-place specimen (0.001 inches). Again, no horizontal slip was measured between the pile cap and the integral diaphragm. The vertical load placed on the specimen peaked at 338 kip at which point the reaction frame used on this specimen unexpectedly reached capacity, resulting in a maximum applied moment of 1550 ft-k. The maximum crack width vs applied moment for the vertical loading is shown in Fig. 72, which had a maximum value of 0.035 inches and is pictured in Fig. 73. This value was larger than the crack that developed on the cast-in-place specimen having a value of 0.020 inches measured at the same applied moment of 1550 ft-k.

The tensile stress in the reinforcing steel and the applied moment are plotted in Fig. 74, as can be observed the measurement reached a maximum stress of 43 ksi. Extrapolating the data within the linear elastic range, the vertical reinforcing steel in the specimen would yield at an applied moment of 2180 ft-k, which is 17 percent less than the yield point in the cast-in-place specimen. The point at which cracking first occurred in the grouted coupler specimen was also earlier than the cast-in-place specimen, which occurred at an applied moment of approximately 180 ft-kip versus 700 ft-kip, respectively. The reduced strength of the grouted rebar coupler specimen is most likely due to several factors. The first factor is that two of the grouted rebar couplers experienced a grouting failure during construction of the abutment; thus, these bars were not contributing to the behavior. There was also a decrease in the distance between the compression and tension force couple within the section, as a result of the increased concrete cover demand for the grouted couplers. Lastly, the bond strength between the grout bed and the precast element was lower than the bond strength of the cold joint in the cast-in-place specimen.

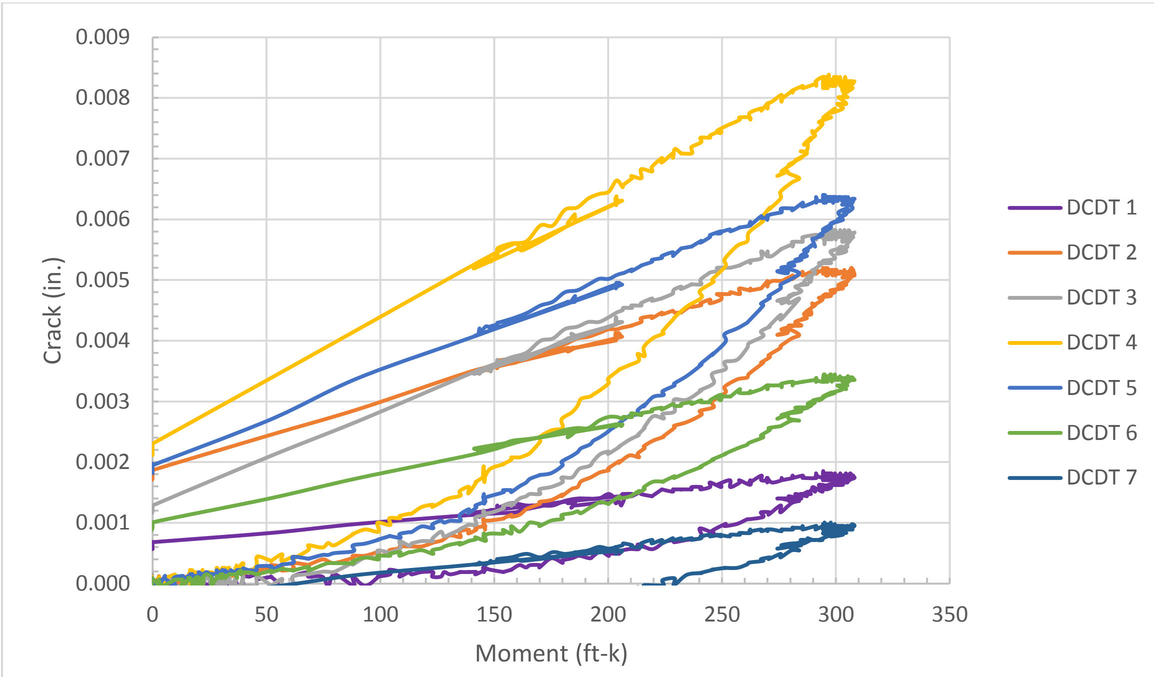

Figure 71. Crack width vs. moment from horizontal load

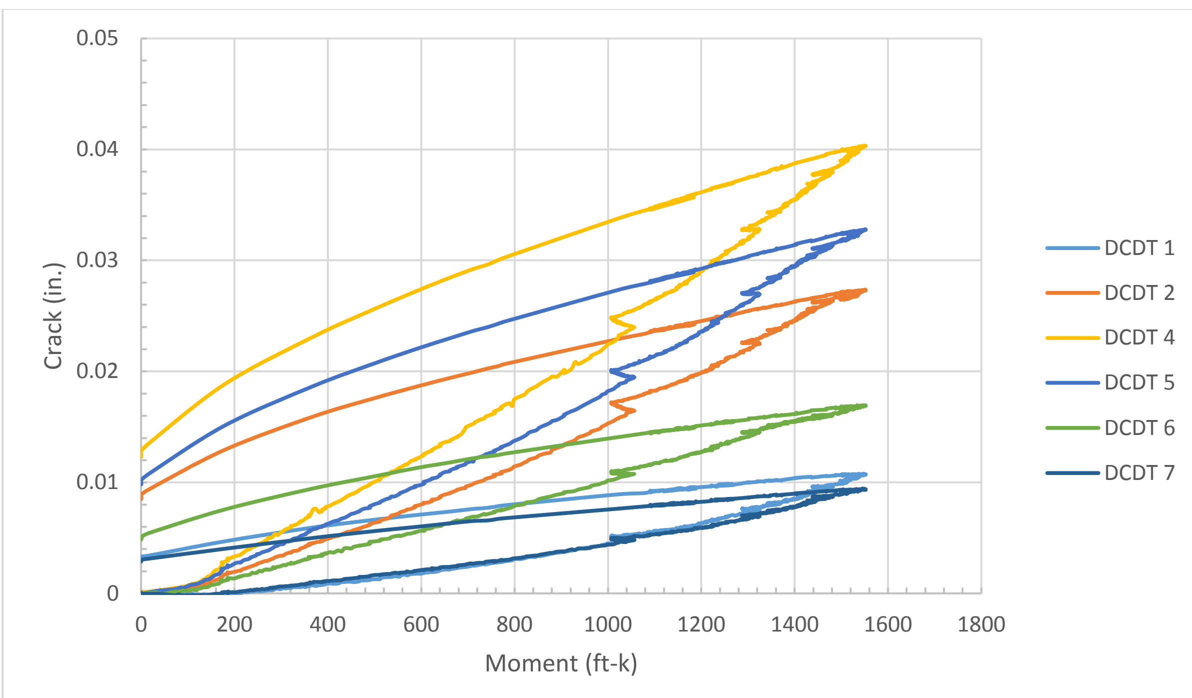

Figure 72. Crack width vs. moment from vertical load

Figure 73. Crack between grout bed and diaphragm

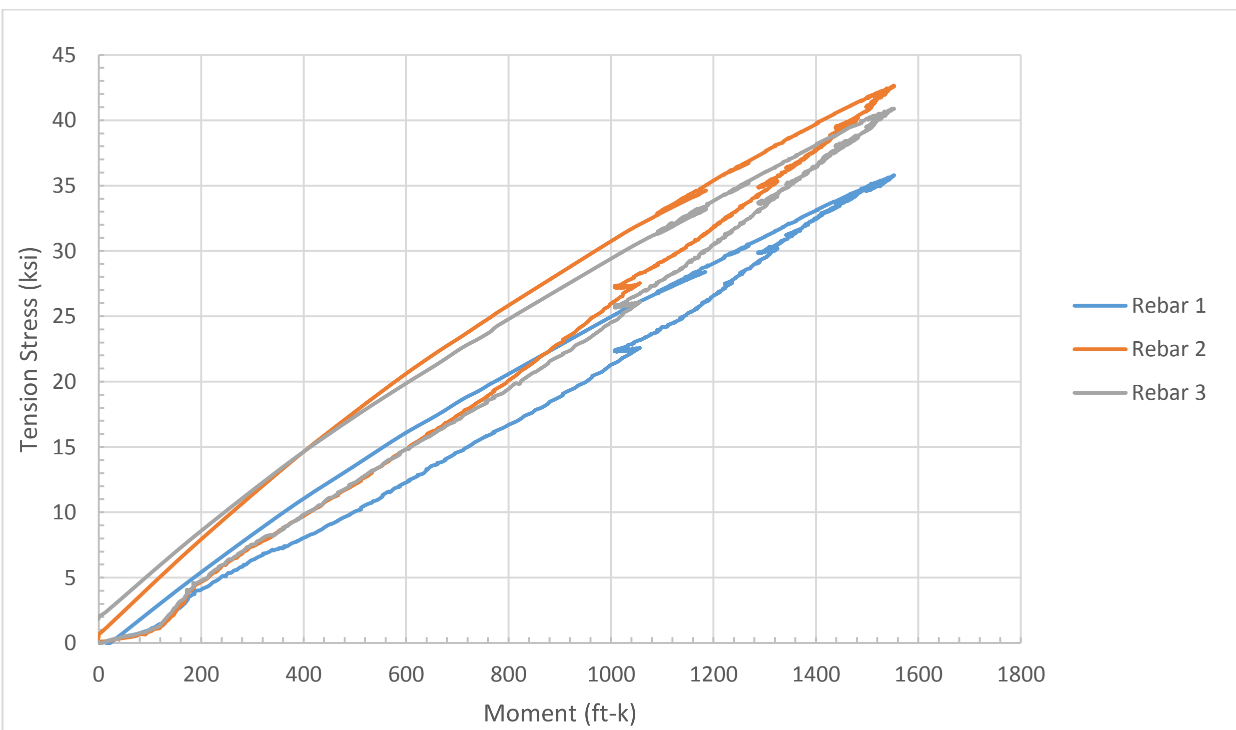

Figure 74. Tension rebar stress vs. moment from vertical load

5.3.3 Pile Coupler

The pile coupler specimen was also loaded to 100 kips in the horizontal load case and the maximum crack that occurred at the front of the precast joint was 0.050 inches (Fig. 75). This crack width is significantly greater than the crack width measured in the cast-in-place specimen of 0.0011 inches. The vertical loading of this specimen reached the ultimate strength of the detail at an applied moment of 1124 ft-k (Fig. 76). The joint opening on the rear face of the specimen became so large that the displacement transducers were out of range. At the ultimate load applied to the specimen, the crack width between the pile cap and integral diaphragm was measured to be 1.75 inches.

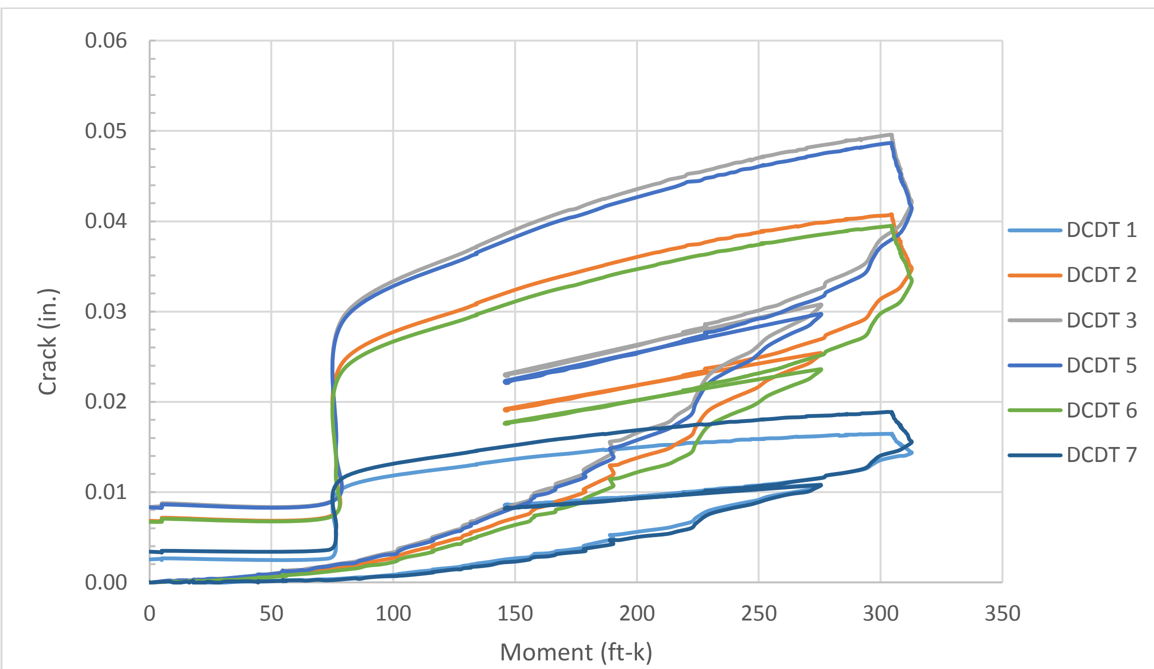

Figure 75. Crack vs. moment from horizontal load

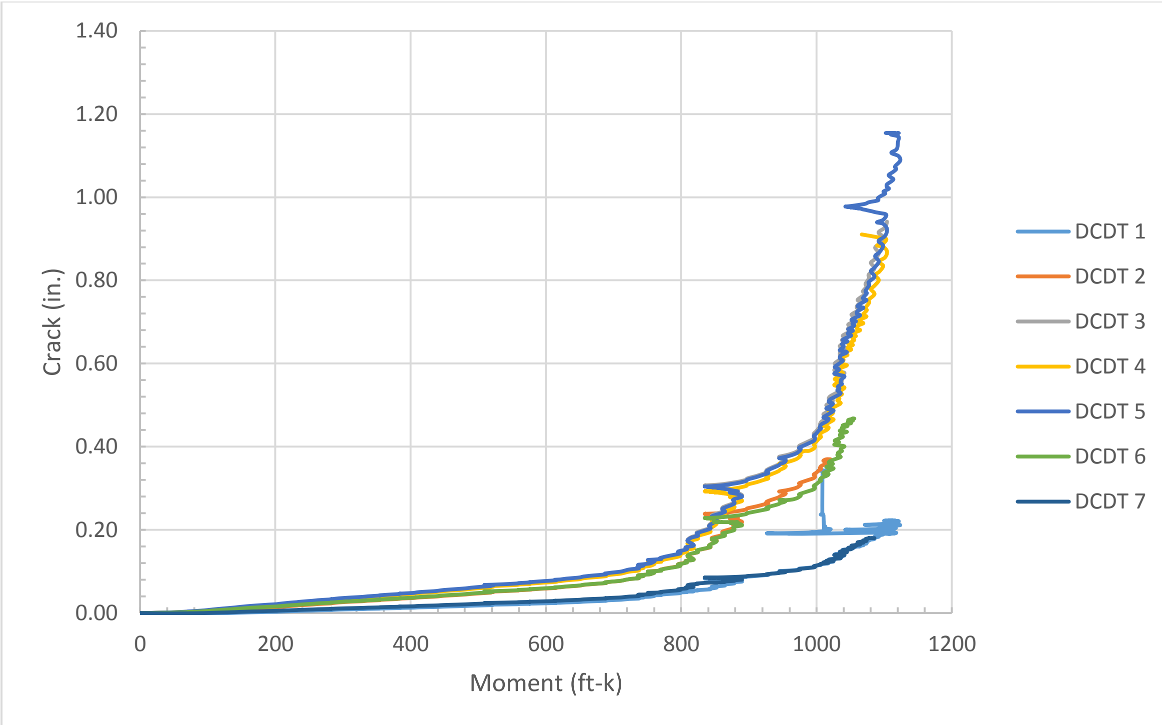

Figure 76. Crack width vs. moment from vertical load

Cracking in the integral diaphragm became prominent at an applied bending moment of approximately 800 ft-kips, and is shown at the maximum load applied in Figs. 77-78. The vertical crack on the integral diaphragm is at the centerline of the CMP and the horizontal cracks correspond to the top and bottom of the CMP used to create the pile coupler voids in the pile cap and diaphragm.

Figure 77. Pile coupler damaged west side

Figure 78. Pile coupler damaged East side

Most likely some amount of slip occurring between the HP section and the grout, as well as between the CMP and the grout/concrete, allowed for the initial crack between the pile cap and integral diaphragm to develop. Once the concrete in the integral diaphragm began to crack, which occurred at around 800 ft-k (Fig. 76), the opening at the precast joint began to significantly increase. At this point large amounts of rotation and cracking began to develop within the abutment until ultimate failure occurred.

The maximum stress captured by the gauges attached to the HP section was 26 ksi, indicating that yielding of the HP section was likely not a failure mechanism of the detail. The failure mechanism between the pile coupler/CMP/concrete within the detail is unknown. An attempt was made to jackhammer through the cracked concrete to investigate the failure mechanism; however, no definitive conclusions could be made. Jackhammering exposed the CMP (Fig. 79 and 80) and showed that slip had occurred between the CMP and the concrete in the diaphragm, as well as that the grout had failed in tension within in the CMP. Examining the photo in Fig. 78, deformation within the integral diaphragm section is noticeable as a result of the large crack that developed within the section. This reveals that a less than ideal amount or distribution of reinforcing steel was present within the abutment resist the tension stresses developed by the pile coupler mechanism.

Figure 79. Pile coupler deconstruction

Figure 80. Deconstruction up close

5.3.4 Foundation Pile Strength

To design an integral abutment, the connection between the pile cap and the integral diaphragm is designed to be stronger in shear and flexure than the driven foundation piling. Thus the plastic moment capacity of the foundation piling limits the flexural stresses that can be developed in the abutment. This laboratory investigation did not include foundation piles in the testing; however, it is important to understand the performance of the abutment relative to the system in which it will be used in the field. Below in Fig. 81 and Fig. 82, the plastic moment capacity of two foundation H-pile sections of various sizes are plotted along the x-axis which represents the applied moment. Along the y-axis in Fig. 81, and in the accompanying table, the crack width at the cold/precast joints in all three specimens were plotted to illustrate durability as a measurement of the joint opening. In Fig. 82 and in the accompanying table, the tensile stress in the center reinforcing steel is plotted along the y-axis to illustrate the strength of the detail. The plastic moment capacity for two piles is plotted because the lab specimen was eight feet in width and a pile spacing of four feet was chosen to make this comparison.

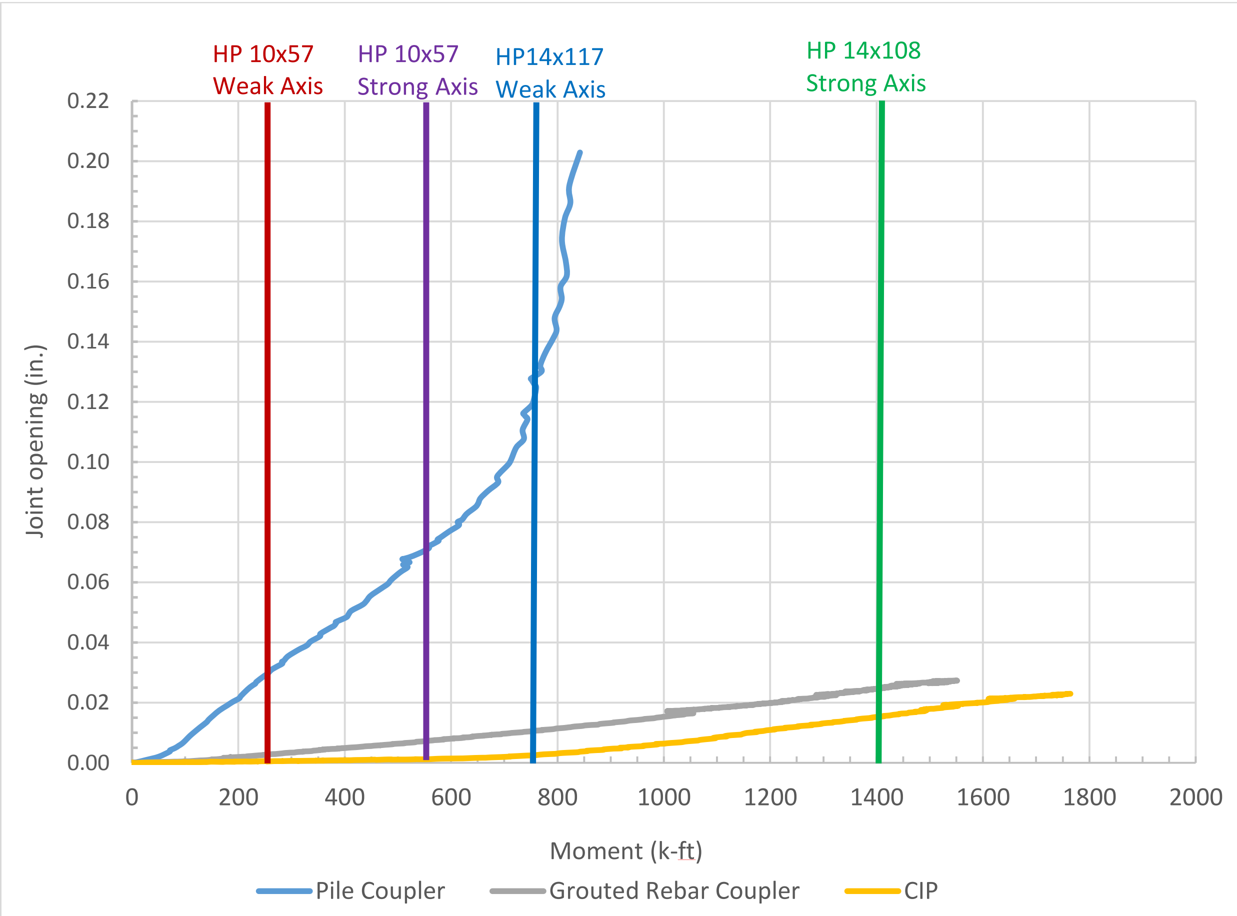

Figure 81. Foundation pile strength (2 piles) vs. abutment joint opening

Specimen	Joint opening at pile yield (Inch)			
	HP 10x57 Weak Axis (252 ft-k)	HP 10x57 Strong Axis (554 ft-k)	HP 14x117 Weak Axis (762 ft-k)	HP 14x102 Strong Axis (1408 ft-k)
Cast-in-place	0.000	0.001	0.003	0.015
Grouted Rebar Coupler	0.004	0.011	0.016	0.035
Pile Coupler	0.028	0.069	0.127	N/A

Table 1. Foundation pile strength (2 piles) vs. abutment joint opening

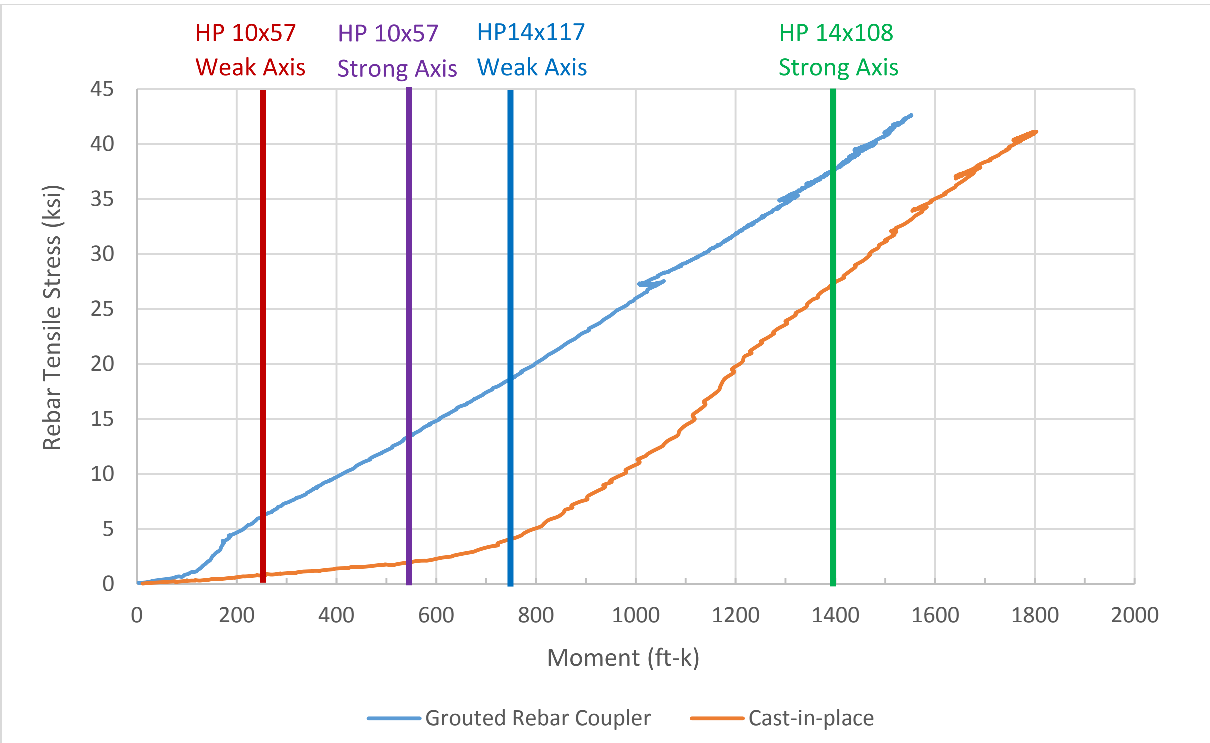

Figure 82. Foundation pile strength (2 piles) vs. abutment rebar stress

Specimen	Stress in rebar (ksi)			
	HP 10x57 Weak Axis (252 ft-k)	HP 10x57 Strong Axis (554 ft-k)	HP 14x117 Weak Axis (762 ft-k)	HP 14x102 Strong Axis (1408 ft-k)
Cast-in-place	0.1	2.1	4.4	27.9
Grouted Rebar Coupler	6.4	13.6	18.8	37.9

Table 2. Foundation pile strength (2 piles) vs. abutment rebar stress

The crack width of the cold/precast joints for the cast-in-place specimen and grouted coupler specimen were both relatively low in magnitude. The grouted rebar coupler detail experienced a wider crack width than the cast-in-place specimen due to several factors including: bond strength between the grout bed and precast element, incomplete grouting of two couplers, and increased concrete cover for the reinforcing steel. Engineering judgement is required to determine tolerable crack widths in concrete structures are based on many factors. These factors are climate, soil type, length of the bridge, type of foundation pile, which all vary greatly from agency to agency. Examining these factors will play a role in the engineering judgement used to select: joint protection, amount of concrete cover, and type of corrosion resistant bar, in order to establish a service life for the structure. Additionally, the crack widths presented in this report are after one loading of the abutment, these values are expected to increase many cycles of loading and as deterioration starts to occur.

While the strength of the grouted rebar coupler specimen was also slightly less than that of the cast-in-place specimen, it is clear that both details satisfy the design philosophies stated in this report. The relative strength of the cast-in-place and grouted rebar coupler detail in comparison to the foundation piling suggest that a smaller abutment could be used to satisfy the design. This reduction in weight or number of spliced connections is advantageous to the constructability of the detail, which is often a driving factor in ABC. Further investigations and calculations are needed to support this claim, which will likely vary greatly on the individual needs of a particular agency. However, a reduction in the overall strength of the design will likely increase the crack width between the pile cap and integral diaphragm, which may already be governing the design.

CHAPTER 6. FINITE ELEMENT MODELING OF THE ABUTMENT

To further study the integral abutment design evaluated in this research, a finite element model of the cast-in-place specimen was established using Ansys (2015). The purpose of the finite element modeling was to investigate several situations and scenarios that were not possible with the given constraints of the project and laboratory. To model the concrete in the integral abutment, a SOLID65 element was chosen which is a three dimensional brick with eight nodes, each having three degrees of translational freedom. The steel girder was modeled using a BEAM188 element for both of the flanges and a SHELL181 element for the web; both elements have 6 degrees of freedom at each node, three translational and three rotational. To model the reinforcing steel within the integral abutment a LINK180 element was chosen capable of tension and compression but not shear. Using these elements and linear elastic material models, the Ansys model was used to investigate different distributions of tensile reinforcement as well as the relative strength of the abutment in comparison to foundation piling. The models created in this research are all eight feet in width and match the size and dimensions of the laboratory specimen.

6.1 Distribution of Reinforcement

In the laboratory specimen design phase, the research team considered modifying the reinforcing bars that connect the pile cap to the integral diaphragm by reducing the number of reinforcing steel bars and using a larger size bar to maintain an equivalent area of steel. This modification is desirable for the use of precast elements because a smaller number of reinforcing steel bars that require splicing increases constructability. To investigate the effects of changing the distribution of reinforcement, three models were established, the first of which featured nine #8 reinforcing steel bars across the tension side of the abutment and served as a validation model. The results from this model were compared to the strains and crack widths measured in the cast-in-place laboratory specimen, showing that a reasonable degree of accuracy could be obtained through the modeling strategies used in this investigation. The second and third model featured six #10 and three #14 reinforcing steel bars across the tension face of the cold joint. Properties of the three models are listed in Table. 3 and it should be noted again that this describes an 8 foot transverse segment of an integral abutment. The table shows that for each of the three models, a similar area of steel was utilized, while the total number of reinforcing bars was reduced. The total number of reinforcing steel bars was calculated by assuming that there was always one more steel bars on the tensile side of the specimen than the compression side, the extra bar being located directly behind the girder.

Model	Rebar Size	A_{rebar} (in^2)	No. of Tension Rebar	Area Tension Steel (in^2)	Total No. of rebar to be spliced
1	#8	0.79	9	7.11	17
2	#10	1.27	6	7.62	11
3	#14	2.25	3	6.75	5

Table 3. Ansys model properties

The construction/precast joint between the pile cap and integral diaphragm has a low tensile strength and is where the concrete section first cracks. As loading continues this crack continues to open and the remainder of the concrete section remains uncracked, as demonstrated in the experimental study of the abutment. To model this behavior in Ansys using linear elastic material models, an iterative process was utilized to find and locate the neutral axis of the cracked concrete section. This process assumes that the bond between the two concrete pours has already been broken by the loading used in the model before the analysis is run. This is an accurate assumption, as the loads placed on the models were greater than the loads that had cracked the cold joint in the cast-in-place laboratory specimen. To model the cold joint in Ansys, two independent sets of nodes were created at the same locations at the interface between the pile cap and integral diaphragm. A first guess of the neutral axis location for the iterative process was taken to be at the center of the concrete section. The nodes on the compression side of the neutral axis were merged so that compression stresses could be transferred while tension stresses on the tension side would only develop in the reinforcing. After the model was run using this first assumption of the neutral axis location, any nodes on the compression side of the axis with a tension force were released and a second iteration was run. This process was repeated until the nodes on the compression side of the neutral axis were in compression. There is error that exists through modeling the cold joint using this process because the nodes had a spacing of 6 inches, so the placement of the neutral axis could not be exact. However, merging and unmerging single nodes instead of an entire row of nodes allowed for the average location of the N.A. to be within the 6 inch spacing.

The only reinforcing steel included in the model across the cold joint was the tensile reinforcing steel. The rest of the reinforcing steel was excluded because little, if any, cracking of the concrete occurred during the testing of the specimen and the reinforcing steel has a relatively small impact on the stiffness of an un-cracked section. To restrain the model, the nodes along the base of the model at the front and back of the pile cap were restrained in the global X, Y and Z directions. This assumption was made because little horizontal and vertical movement, 0.006 in. and 0.002 in. respectfully, was measured between the cast-in-place laboratory specimen and the strong floor. In addition to this, nodes were restrained in the model in the global Z and X directions 18 inches above the base of the pile cap which are located at the same location as the top of the reaction block used in the laboratory.

A wireframe of the three resulting Ansys models is shown below in Figs. 83, 84 and 85 along with the global coordinate axis. In Figs. 86 and 87 an element plot shows the meshing and size of the solid elements in an isometric view. The results of the cast-in-place laboratory specimen and each Ansys model are shown for comparison in Table 4. In the wireframe figures, the stresses in the bars marked with a red line are displayed in Table 4 for comparison, which mirror the locations of the reinforcing steel bars instrumented with strain gauges in the laboratory testing. To obtain the crack width at the cold joint, the difference in the total vertical displacement between two nodes at the center of the joint were taken.

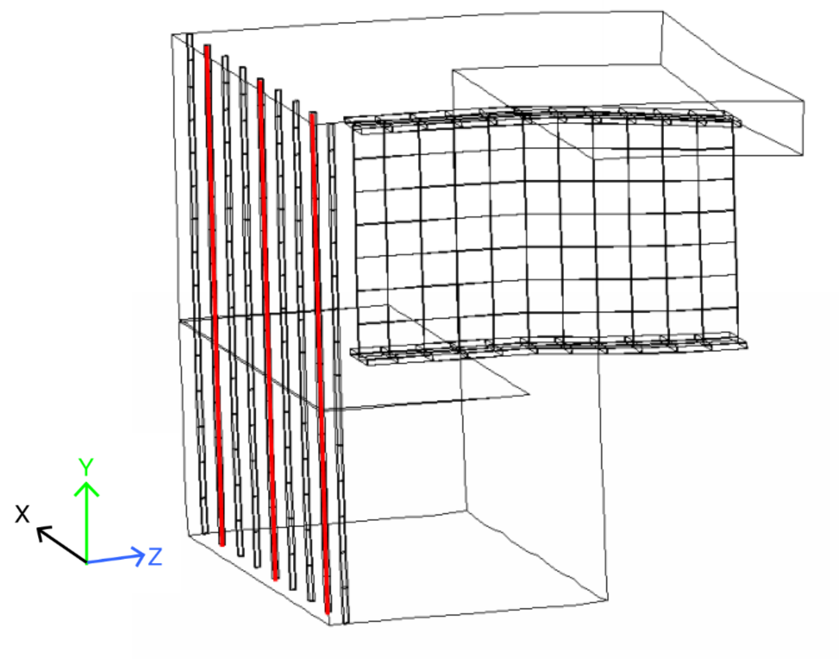

Figure 83. Model 1 with nine #8 bars

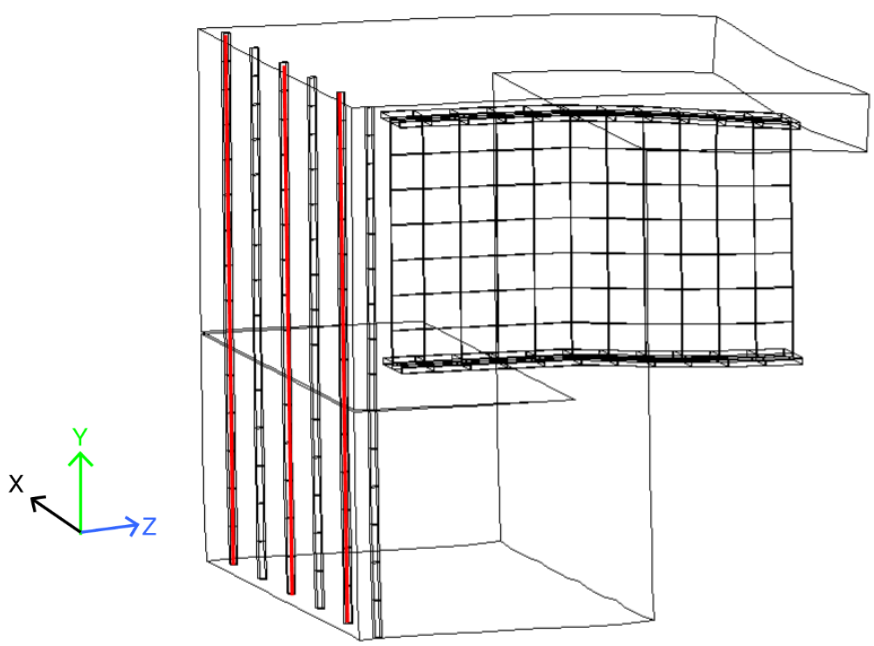

Figure 84. Model 2 with six #10 bars

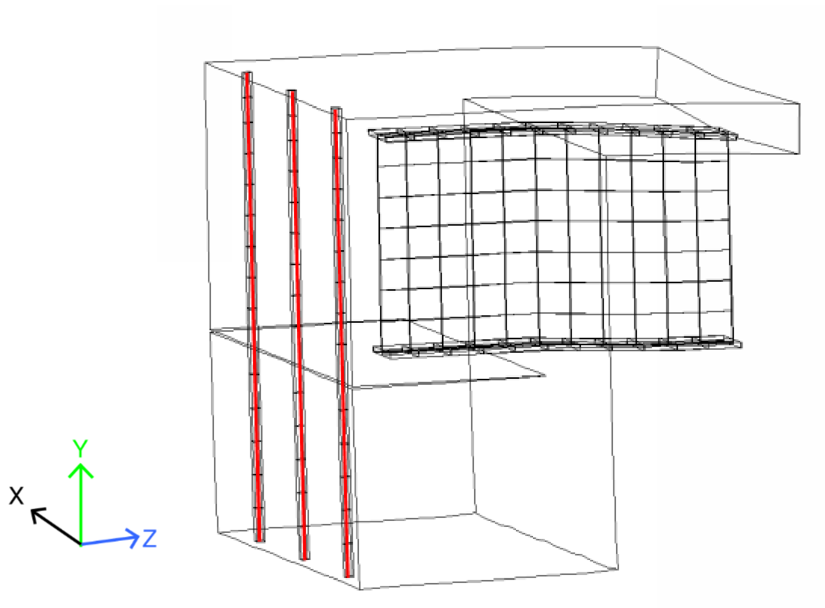

Figure 85. Model 3 with three #14 bars

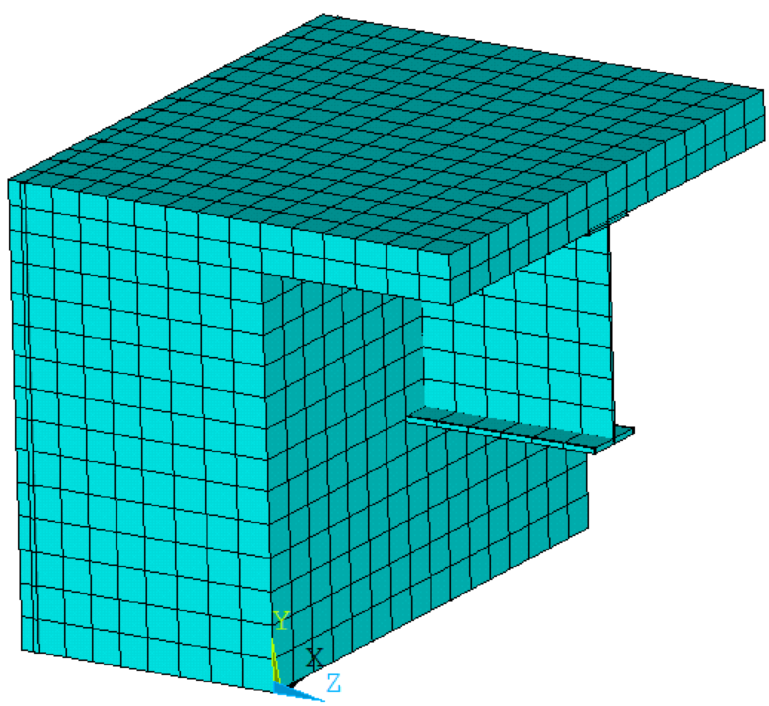

Figure 86. Element plot 1

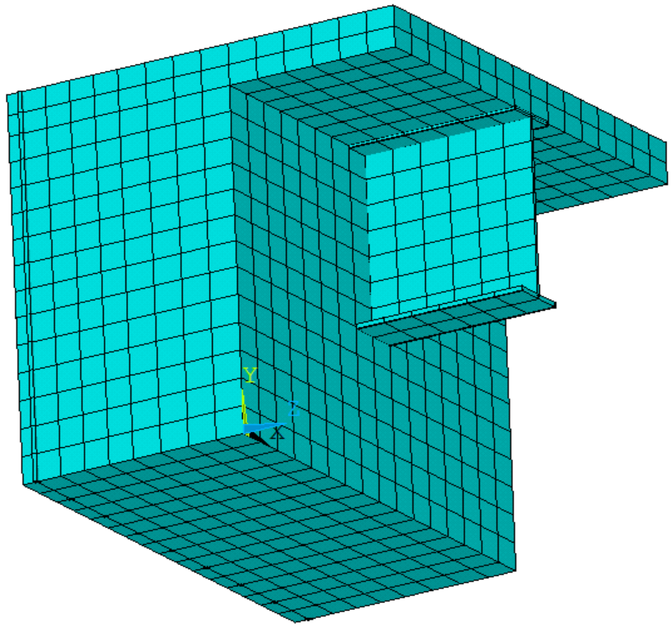

Figure 87. Element plot 2

Specimen/ Model	Load Applied (Kips)	Rebar Size	Center Rebar Stress (ksi)	Side Rebar Stress (ksi)	Side Rebar Stress (ksi)	Δ_{mid} joint (in.)
Cast-in-place	385	#8	41.86	41.11	33.6	0.0243
Model 1	385	#8	42.90	42.54	42.54	0.0205
Model 2	385	#10	39.96	40.57	39.4	0.0195
Model 3	385	#14	41.22	42.3	42.3	0.0212

Table 4. Specimen/ansys model result

The first model closely matches the results from testing the integral abutment cast-in-place laboratory specimen, demonstrating that a relatively accurate model could be produced using the modeling strategies used. The second and third models that vary the reinforcing in the abutment show that stress in the reinforcing steel and crack width at the construction joint may not be a concern if a detail like this were to be used in the field. However, there is the potential for undesirable cracking effects that may exist in the integral diaphragm from the increased stiffness at the locations of the larger reinforcing bars. Cracking of concrete is difficult to predict using finite element software and further studies, such as a more detailed model or laboratory investigations, should be performed before the use of #10 or #14 reinforcing steel bars are used in the field at a larger spacing. Furthermore, if a precast element were chosen that featured five grouted rebar connections, any one failure connection would produce a larger impact than if twenty connections were used. If one in five connections fail, 20% of the connection strength is missing while only 5% of the reinforcing is missing in the system of twenty connections. The system featuring more connections is robust to any one missed connection as demonstrated in the grouted rebar coupler specimen tested in this laboratory investigation.

6.2 Foundation Piling Model

Since the laboratory specimen did not include foundation piling, a finite element model was created that featured the laboratory specimen with two HP 10x57 foundation piles used in strong axis bending. To model the foundation piling, a BEAM188 element and an I-shape section was used. The goal of this model is to demonstrate the relative strength of the pile cap to integral diaphragm connection in comparison to the plastic moment capacity of the steel foundation piling. In the laboratory testing and previous modeling, flexural stresses developed in the abutment section were higher than possible in the field due to the fixity obtained through the tie. The Ansys model demonstrates the relative strength of the abutment in comparison to the foundation piles and suggests that the flexural strength of the pile cap to integral diaphragm connection is somewhat overdesigned. Subsequently, additional models were created that featured less #8 bars on the tension side of the abutment. A reduction in steel in the abutment can allow for either 1) fewer spliced connections to be used in a precast element or 2) a designated number of redundant connections should there be a grouting failure or misaligned bar.

To restrain the pile in the model, it is assumed that the base of the pile is fully restrained against rotation. This is a worst case scenario because it allows for the greatest stresses to develop in the abutment. To find the load at which the plastic moment capacity of the piles is developed, the load was incrementally applied in the model until the stress in the pile reached 50 ksi. The resulting model is illustrated in Fig. 88, the blue and red shading indicate 50 ksi of compression and tension respectively in the pile. In Table 5, the stress in the reinforcing steel in the various models was found using the iterative approach of locating the neutral axis.

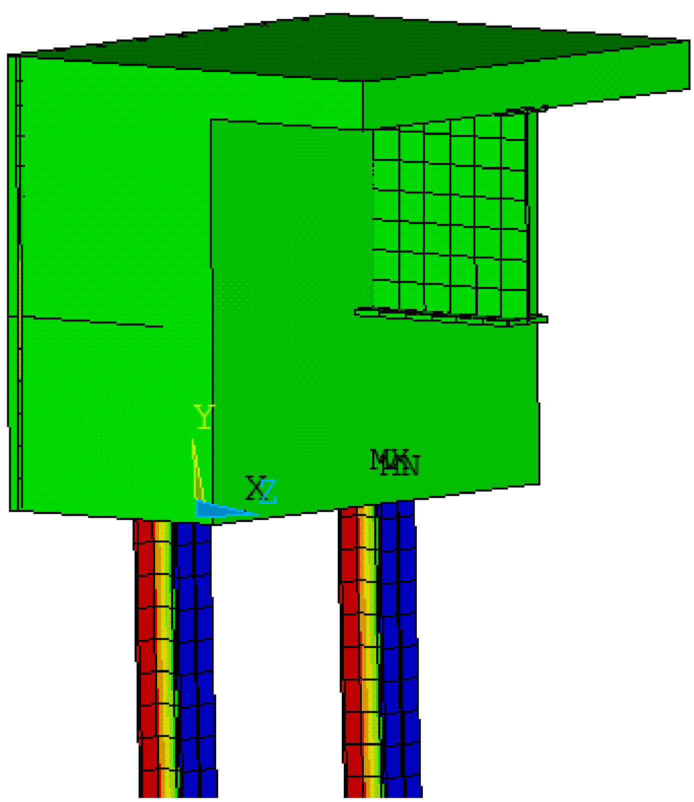

Figure 88. Model with two foundation piles

Specimen/ Model	Load Applied (Kips)	Rebar Size	No. of reinforcing bars (tension)	Max Rebar Stress (ksi)	Δ_{mid} joint (in.)
Model 1	99	#8	9	11.06	0.0053
Model 2	99	#8	7	14.41	0.0065
Model 3	99	#8	5	20.03	0.0097
Model 4	99	#8	3	31.65	0.0107

Table 5. Foundation pile model results

The results of this modeling are not surprising considering the strong performance of the integral abutment in the laboratory setting. While the strength of the reinforcing steel may be adequate if as few as three #8 bars were to be used on the tension side of the abutment, there may be undesirable cracking due to the sparse distribution of reinforcement that was not captured by this finite element modeling. Additional reinforcing may be required within the abutment to control cracking and distribute stresses evenly throughout the system. At the very least, this modeling suggests that the integral abutment connection is more than adequately designed. Either the size of the abutment or the area of steel could be reduced in order to facilitate a precast element connection.

CHAPTER 7. SUMMARY

Integral abutments have rarely been used in Accelerated Bridge Construction as they are often large, heavy and have complex reinforcing details. These aspects make the integral abutment difficult to precast because of weight and construction tolerances. As a result, integral abutments constructed in ABC projects have typically relied on cast in place closure pours. However, there are certain benefits to precasting the abutment, as material closure pours add significant cost to the project and add curing time to the project schedule. In order to investigate precast integral abutments, two details were designed, constructed and tested in the structures laboratory. The two ABC details investigated in this research spliced the integral abutment at the typical construction joint between the pile cap and integral diaphragm. The first detail, called the “grouted rebar coupler”, utilized grouted reinforcing splice couplers to splice the vertical reinforcing steel that passed through the precast joint. The second detail, called the “pile coupler”, utilized a two foot section of steel H-pile and a grouted void to create the spliced connection. In addition to these ABC details, a cast-in-place specimen was constructed and tested in order to establish baseline performances for the integral abutment design. The integral abutments were evaluated and compared on three criteria: constructability, strength, and durability, which were seen as critical to the needs and implementation of the details to ABC projects.

Prior to constructing the grouted rebar coupler specimen, previous research and also engineers on the TAC expressed concerns for the tight construction tolerances that arise when using grouted rebar couplers. The grouted coupler specimen in the laboratory was eight foot in width and had 17 reinforcing steel bars that required splicing in order to make the precast connection. Through the use of a plywood template, the locations of the 17 steel bars on the pile cap were marked and transferred to the base of the integral diaphragm formwork. Form plugs were installed into the template which held the grouted rebar couplers in place during the construction of the integral diaphragm, effectively “match casting” the two elements. This technique proved to be simple, cost effective and resulted in the successful alignment of 17 couplers and steel bars in the laboratory. Constructing and erecting a precast element system in the field that requires alignment on one end should not pose a challenge to a prefabricator/contractor team. Significant complications arise in constructability when a precast element requires alignment on two ends, placement of the pile caps and grouted couplers within the integral diaphragm would likely need to be exact.

The strength and durability of the grouted rebar coupler specimen is comparable to that of the cast-in-place specimen. The crack width that developed at the precast joint in the grouted rebar coupler was 0.035 inches, compared to 0.019 inches in the cast-in-place specimen. Additionally, the yield strength of the grouted rebar coupler detail was estimated to be 17% lower than the cast-in-place detail. These reductions in performance are likely due to three factors: grouting failures within the coupler, reduced internal moment arm, and a lower tensile strength between the grout bed and the concrete. Even with this reduction, the performance of the grouted rebar coupler should not be evaluated entirely based on the maximum load applied in the laboratory. Rather, the performance should be based on the maximum stresses that could be developed by the foundation piling used in a design. While the detail’s strength likely satisfies conditions met by even the stiffest of foundation piles, the durability or maximum tolerable crack width of the detail will likely control. While the crack widths developed under load will satisfy some, there is no definitive conclusion to be made as a variety of factors come into play here: climate, soil type, type of corrosion resistant reinforcing, joint protection, design service life and use of de-icing salts. For some scenarios the performance of the grouted coupler detail will be more than adequate. For these scenarios, further investigations in the reduction of weight or amount of spliced reinforcing should be made to facilitate the needs of ABC projects.

The pile coupler detail attempted to create an integral abutment that facilitates the construction and erection of a full slide in bridge. This detail aims to reduce the number of grouted connections between the pile cap and the integral diaphragm while also eliminating any protruding reinforcement in the pile cap that obstructs the sliding process. The grouted pile connections had a spacing of 4 feet in the spliced abutment, which drastically reduces the number of connections required compared to the grouted rebar coupler detail. Construction of the pile coupler proved to facilitate the needs of ABC, at least on a small scale, as the process of lowering two piles and grouting the connections was fast and simple. While the pile coupler detail is promising in terms of constructability, the performance in strength and durability was less than ideal. The ultimate strength and durability of this detail was significantly less than that of the cast-in-place and grouted rebar coupler details. The laboratory specimen experienced significant cracking within the integral diaphragm and pile cap, indicating a poor distribution of reinforcing steel to resist the stresses developed by the HP coupling system. The exact failure mechanism of this detail in the laboratory is unknown; however after the testing several improvements became apparent. These improvements are: using a longer length of pile, increasing the number of threaded rods/shear studs on the pile, increasing/modifying the amount of reinforcing steel in the abutment, and the use of two pile couplers acting as a force couple. In addition to these improvements, taking a different route and utilizing the pile coupler as a hinge may be a better design alternative. Instead of a pile, a thin rod could be used to allow the superstructure to rotate without inducing large stresses in the abutment, while the successful lowering and grouting mechanism is preserved.

REFERENCES

[1] AASHTO. 2012. *AASHTO LRFD Bridge Design Specifications, 6th ed.* Washington, DC, 2012.

[2] Culmo, Michael, P.E. "Accelerated Bridge Construction. Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems." *Federal Highway Administration*. Office of Bridge Technology, HIBT-10, Nov. 2011. Web. <https://www.fhwa.dot.gov/bridge/abc/docs/abcmanual.pdf>.

[3] Culmo, Michael, P.E. "Connection Details for Prefabricated Bridge Elements and Systems." *Federal Highway Administration*. Office of Bridge Technology, HIBT-10, Mar. 2009. Web. <https://www.fhwa.dot.gov/bridge/prefab/if09010/report.pdf>.

[4] Dahlberg, Justin, and Brent Phares, Dr. "Iowa ABC Connections." *The Bridge Engineering Center*. Iowa State University, June 2015. Web. <http://www.intrans.iastate.edu/research/documents/research-reports/Iowa_ABC_connections_w_cvr1.pdf>.

[5] Haber, Zachary. "Precast Column-Footing Connections For Accelerated Bridge Construction In Seismic Zones." *State of California Department of Transportation*. California Department of Transportation, May 2013. Web. <http://wolfweb.unr.edu/homepage/saiidi/caltrans/GroutedCouplers/PDFs/CCEER-13-08-July2-13.pdf>.

[6] Hosteng, Travis. "Laboratory Investigation of Grouted Coupler Connection Details for ABC Bridge Projects." *Bridge Engineering Center*. Iowa Department of Transportation, Federal Highway Administration, Aug. 2015. Web. <http://www.intrans.iastate.edu/research/documents/research-reports/grouted_coupler_connections_for_ABC_w_cvr.pdf>.

[7] INDOT. "Integral Abutment Design." *Load and Resistance Factor Design (LRFD) Examples*. Indiana Department of Transportation, 1998. Web. 2015. <http://www.in.gov/dot/div/contracts/design/lrfd/>.

[8] Jansson, Peter, P.E. "Evaluation of Grout-Filled Mechanical Splices for Precast Concrete Construction." *Www.michigan.gov*. Michigan Department of Transportation Construction and Technology Division, May 2008. Web.

[9] Nelson, James, P.E. "IOWA ACCELERATED BRIDGE CONSTRUCTION HISTORY." Iowa DOT. Precast Concrete Institute, 2014. Web. <http://www.iowadot.gov/bridge/abc/Presentations/2014%20PCI%20Iowa%20ABC%208%20-%20Final%20Revisions%20Accepted.pdf>.

[10] Noureddine, Issam. "Plastic Energy Absorption Capacity of #18 Reinforcing Bar Splices under Monotonic Loading." *State of California Department of Transportation*. N.p., 1996. Web. <http://www.dot.ca.gov/hq/esc/earthquake_engineering/Research_Reports/vendor/csus/Plastic%20Energy%20Absorption%20of%20%2318%20Bar%20Splices%201996.pdf>.

[11] Rouse, Jon, and Brent Phares, Dr. "Laboratory and Field Testing of an Accelerated Bridge Construction Demonstration Bridge: US Highway 6 Bridge over Keg Creek." *The Bridge Engineering Center*. Iowa State University, Apr. 2013. Web. <http://www.intrans.iastate.edu/research/documents/research-reports/keg_creek_bridge_w_cvr.pdf>.

[12] Rowell, Stephen. High Strain-Rate Testing of Mechanical Couplers. Rep. no. ERDC TR-09-8. N.p.: US Army Corps of Engineers, 2009. Print.

[13] Transportation Research Board. "Innovative Bridge Designs for Rapid Renewal." *Transportation Research Board*. Strategic Highway Research Program, Feb. 13. Web. <http://onlinepubs.trb.org/onlinepubs/shrp2/SHRP2_S2-R04-RR-2.pdf>.