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Performance of Transverse Post-Tensioned Joints Subjected to Negative Bending and Shear Stresses on Full Scale, Full Depth, Precast Concrete Bridge Deck System

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PERFORMANCE OF TRANSVERSE POST-TENSIONED JOINTS SUBJECTED TO NEGATIVE BENDING AND SHEAR STRESSES ON FULL SCALE, FULL DEPTH, PRECAST CONCRETE BRIDGE DECK SYSTEMS

by

Kayde Steven Roberts

A thesis submitted in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

in

Civil and Environmental Engineering

Approved:

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Dr. Marvin Halling                          Dr. Paul Barr
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Dr. Joseph Caliendo                        Dr. Byron Burnham
Committee Member                           Dean of Graduate Studies

UTAH STATE UNIVERSITY
Logan, Utah
2011
ABSTRACT

Performance of Transverse Post-Tensioned Joints Subjected to Negative Bending and Shear Stresses on Full Scale, Full Depth, Precast Concrete Bridge Deck Systems

by

Kayde Steven Roberts, Master of Science
Utah State University, 2011

Major Professor: Dr. Marvin Halling
Department: Civil and Environmental Engineering

Accelerated bridge construction has quickly become the preferred method for the Utah Department of Transportation (UDOT) as well as many other DOT’s across the United States. This type of construction requires the use of full depth precast panels for the construction of the bridge deck. The segmented deck panels produce transverse joints between panels and have come to be known as the weakest portion of the deck. Cracking often occurs at these joints and is reflected through the deck overlay where water accesses and begins corrosion of the reinforcement and superstructure below. For this reason post-tensioning of the deck panels is becoming a regular practice to ensure that the deck behaves more monolithically, limiting cracking.

The current post-tensioning used by UDOT inhibits future replacement of single deck panels and requires that all panels be replaced once one panel is deemed defective. The new curved bolt connection provides the necessary compressive stresses across the
transverse joints but makes future replacement of a single deck panel possible without replacing the entire bridge deck.

To better understand the behavior of the new curved bolt connection under loadings, laboratory testing was undertaken on both the curved bolt and the current post-tensioning used by UDOT. The testing specimens included full-scale, full-depth, precast panels that were connected using both system. The testing induced typical stresses on the panels and connections, subjecting them to negative bending and shear.

The overall performance of the curved bolt proved satisfactory. The moment capacity of both connections surpassed all theoretical calculations. The yield and plastic moments were 17% and 16% lower, respectively, than the UDOT post-tension system while at those moments deflection was relatively the same. Due to the anchorage location of the curved bolts, the reinforcement around the transverse joint received up to 5 times the strain of that of the post-tension connections. Although both systems performed well when subjected to shear forces and as compared to the theoretical capacities, the post-tension connection greatly surpassed the curved bolt in shear capacity.

(192 pages)
I would like to thank my graduate committee for their continued support and direction that were given to me throughout this research project, specifically my major professor, Dr. Marvin Halling, for giving me this opportunity to undertake this research task. I also want to thank Dr. Paul Barr for his much needed optimism and Dr. Joseph Caliendo for his great humor. I greatly enjoyed my graduate experience at Utah State University, which was due mostly to you three.

I also would like to thank Ken Jewkes for the use of his lab and equipment and most importantly his extended amount of time helping me with construction of various items used and tested in this research. I must also mention a good friend, Zane Wells, who spent countless hours with me in the laboratory and office, designing, constructing, and testing the deck specimens. I would also like to thank Matt Laurendeau, Wes Cook, Aaron Jensen, Sandeep Reddy, and Kayla Arrington for their periodic help over the research process. I also would like to recognize and thank UDOT for funding this research.

I would like to give special thanks to those who indirectly effected this research. First, my father and mother, Steve and Ruth Roberts, for their support over these last few years. Thank you Mom and Dad for instilling in me the importance of hard work, dedication, and determination, which has opened many opportunities in my life. I hope I can instill in my children the same qualities. Lastly, to my wonderful wife, Anne, thank you for your dedication to me and my schooling. Thank you for raising our son Drake and daughter Davi while I’ve been gone due to school and this research. I look forward
to the birth of our new daughter and hope she is as beautiful as you. You are the love of
my life.

Kayde Roberts
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LIST OF SYMBOLS

\( f'_c \) Compressive strength of concrete
\( f_r \) Rupture stress
\( \text{psi} \) Pounds per square inch
\( \text{lb} \) Pounds
\( \text{ksi} \) Kips per square inch
\( E \) Modulus of elasticity of steel
\( f_y \) Yield strength
\( E_c \) Modulus of elasticity of concrete
\( f'_t \) Tensile cracking stress
\( \text{kip-ft} \) Kip feet
\( I \) Moment of inertia
\( \Delta \) Deflection
\( \Delta_1 \) Deflection between supports
\( \Delta_2 \) Deflection at cantilevered end
\( P_p \) Ultimate load
\( P_c \) Cracking load
\( P_y \) Yield load
\( L \) Total length
\( \text{ENA} \) Elastic neutral axis
\( \text{PNA} \) Plastic neutral axis
\( M_{\max} \) Maximum moment
$M_{cr}$  Cracking moment

$M_p$  Plastic moment

$M_y$  Yield moment

$A$  Area

d  Distance

$y$  Distance to centroid

$C$  Compressive force

$T$  Tensile force

$C_s$  Clear spacing

$t_d$  Deck thickness

$D_T$  Diameter of transverse rebar

$D_L$  Diameter of longitudinal rebar

$V_c$  Shear capacity

$N_u$  Axial compression stress

$A_g$  Gross shearing area

$b_w$  Length of shearing area

$M_u$  Induced moment from shear
CHAPTER I
INTRODUCTION

Throughout the United States and specifically in Utah the implementation and use of Accelerated Bridge Construction (ABC) has quickly become the standard for bridge construction. The ABC process enables faster placement of concrete bridge decks and significantly shortens bridge construction time. The construction of an ABC deck is often achieved in a controlled environment and then brought onsite and quickly installed. The deck is constructed in manageable sections and installed one after another along the length of the bridge. These decks sections are connected to steel or concrete girders with the use of shear studs and high strength grout. The deck sections are connected transversely with a variety of post-tensioned, steel and grout connection methods.

The transverse joint connections are often the weakest portion of the bridge due to inefficiencies in load transfers from deck to deck. These joints often crack and begin to allow water to penetrate into the connections and eventually flow completely through to the girders below. Cracking significantly decreases the life of the bridge and specifically the deck and or the girders due to corrosion effects. Cracking also damages the asphalt and other overlays and can become quite costly. It has become the primary practice of Utah Department of Transportation (UDOT) to post-tension the deck systems longitudinally to limit transverse cracking. Previous studies have shown that the post-tensioned type connections are the most effective connection in strengthening the transverse joint and preventing cracking. UDOT has found that although this connection is the strongest it is also the most difficult to construct and maintain. Once this post-
tensioning system is installed it makes it very difficult to replace individual deck sections that begin to fail.

The creation and use of a new type of post-tensioned system named the curved bolt connection, designed by Utah State University, would provide the strength of a post-tension type of connection as well as making deck replacement quick, simple and ultimately possible. This paper gives an in depth report on the laboratory construction and testing of the current post-tension system being used by UDOT and the new curved bolt connection. All panels were constructed to full scale and tested in the negative bending and shear. Deflections versus load, cracking moment, ultimate loads and cracking were all recorded in the testing.

The primary purposes of this research project are as follows:

1. Investigate the construction process of the curved bolt connection.
2. Determine cracking moments for each deck connection.
3. Investigate and compare cracking failure for both connection types under negative bending.
4. Investigate resulting reinforcement strains due to the different types of post-tensioning under shear and bending stresses.
5. Determine ultimate moments for each connection in the.
6. Determine the ultimate shear strength of each connection on full scale test specimens.
7. Investigate cracking failure of each connection specimen under a shear failure.
CHAPTER II
LITERATURE REVIEW

Issa et al. (1995a) recognized a dramatic increase in the last 30 years of the use of full depth precast panels on not only new construction but also rehabilitation of older bridges. Understanding there were a variety of methods used to replace or construct bridge decks with the use of precast panels, Issa et al. (1995a) sent a questionnaire survey to the DOT’s throughout the United States as well as one providence in Canada. The survey included questions about types of construction, deck/panel dimensions, reinforcement, bonding material to fill joints and the type of connection system between panels. From this survey it was quickly seen the many different types of panels, joints, post-tension compressive forces and construction practices were being utilized. A need for deck joint research was needed.

Issa et al. (1995b) performed field evaluations on selected bridges throughout the United States. Bridges in over eleven states were visually inspected. The inspections included visual searches for any current and future problems associated with joints between precast panels, the connection between the deck and supporting system as well as the overlay system. The field inspection also included discussions with state engineers, discussing the construction, design and performance of each bridge inspected. The Bayview Bridge over the Mississippi River in Quincy utilized the use of a butt joint between panels with post-tension spaced at 7 inches. The initial tension stress of the post-tension bars was 105 ksi. Due to the compressive forces the joints behaved satisfactorily with only minor leaking. The 03200 Waterbury Bridge in Connecticut also
utilized post-tensioning but the amount of compressive force over the joint was varied depending type of span. A portion of the bridge was simple spans and a compressive force of 150 psi was used. The other portion included a three-span continuous section that utilized a compressive force of 300 psi. Although a variety of compressive forces over the joints were used, all transverse joints were performing satisfactory. Issa et al. (1995b) observed that bridges without post-tensioning performed poorly and show significant signs of cracking, debonding as well as leaking and rusting away the superstructure below. Some of these bridges that proved to be performing unsatisfactory in the transverse joint area were the High Street Overhead Separation Bridge, of California which utilized the use of a welded stud connection across the transverse joint and the Chulitna River Bridge in Alaska which used a female-to-female connection. Issa et al. (1995b) concluded that post-tensioning significantly increases the performance and life of a bridge deck and the superstructure below.

Research on the behavior and capacities of a variety of joint connections has been performed at Utah State University. Recent research performed by Porter (2010) and Julander (2010), included the testing of post-tension joint connections of which included two different types of curved bolts. The two types of curved bolts differed in length and diameter. The first of the two types of curved bolts was a 1 inch diameter curved bolt that had a linear distance from anchor point to anchor point of 24 inches. The second was a 7/8 inch diameter bolt with a linear distance from anchor point to anchor point of 36 inches. The radiuses of the 24 inch and 36 inch curved bolts were 18-7/16-inches and 39 inches, respectively. Both curved bolts were tested using deck panel specimens with a full depth thickness of 8-3/4 inches, a length of 3 feet and a transverse width of 18 inches.
Two panels were jointed together forming a female-to-female joint that was grouted and post-tensioned using the curved bolt connections. The test specimen then underwent a positive bending test through the use of a four point test apparatus setup. Porter (2010) and Julander (2010) concluded that the curved bolts failed at higher flexural loads than non-post-tension connections and could be used a viable type of post-tensioning but also stated that the geometry curved bolt plays a specific role in its behavior. It was also noted that the longer curved bolt, the 36 inch anchor point to anchor point proved to perform better, resisting higher loads than the 24 inch curved bolt.

Research at Virginia Polytechnic Institute has focused extensively on transverse joints. Swenty (2009) modeled the layouts of the transverse joints to mirror a bridge located in the Southwest Virginia. This bridge contained transverse joints that were located directly above the piers of the structure. The piers produce a large negative bending behavior of the deck panels to occur. The negative bending results in tensile stresses to form in the deck panels and is largest at the transverse joint due to the location of the joints. In this research three different joints were designed and tested. These joints included an Embedded Reinforcing Bar joint, Looped Reinforcing Bar joint and a Post-Tensioned joint. The Post-Tensioned joint was used on four joint tests. Two Post-Tension joints included non shrink grout as the joint material but one was post-tensioned to a compressive force of 167 psi across the joint while the other was post-tensioned to a compressive force of 340 psi across the joint. Of the other two joints, one contained pea gravel instead of grout and post-tensioned to a compressive strength of 167 psi across the transverse joint and the other was epoxied and post-tensioned to a compressive joint stress of 340 psi.
The negative bending specimens contained three panels that were connected using two joints. The testing apparatus was designed to provide maximum stress caused by an HS-20 vehicle. It was set up with a midpoint reaction and one end of the specimen tied down to the strong floor, while the other end received the load. The maximum moment was produced at the midpoint with moments at the two transverse joints equal to each other. Each test specimen was exposed to 1,000,000 cycles of truckload, which was an estimated 75 year design life of the bridge in southwest Virginia.

Several conclusions were made. Some of which included the following: first, the concrete-grout interface is the weakest point in all of the joints. Tensile strengths of the joint is weaker than the tensile strength of concrete as well as the tensile strength of grout. Second, shrinkage cracking will occur between the concrete and the grout interface if no surface preparation technique is used. Imperfection due to tie rods or formwork will cause cracking in the grout. Forth, in the post-tension transverse joints, cracking was first observed at a net stress level of 0.13 ksi. Lastly, the 340 psi joints performed the best based on cracking, deformation, ponding and strain distribution measurements. Swenty (2009) suggested that the tensile strength of the joint be reduced to 1.5 multiplied by the square root of the weakest compressive strength of the concrete and grout.

Transverse joints have been specifically tested to withstand shearing forces and effectively transfer loads from panel to panel. Most shear tests have been performed on smaller specimens as researched by Porter (2010) and Kim, Shin, Park (2003) have done. Full scale full depth testing has been performed by Robert (2007). This research included the testing of four different type of transverse joint configurations with both post-
tensioning and non-tensioned applications. The four joints tested included the butt joint, shear key, angular corrugated and round corrugated. Each joint that received post-tensioning was tensioned to a resulting compression force of 400 psi across the joint. The specimens had a full depth of 8-1/2-inches with a longitudinal length of 16 inches per panel and a transverse width of 48 inches. The full depth specimens were tested for shearing ultimate capacities. This was accomplished through the use of two roller type reactions that were located 1 inch from the joint and 16 inches apart, running parallel to the joint and spanned the transverse length. The load was applied through a 10 X 10 X 2 inch plate, placed 2 inches away from the center of the joint and positioned at the center of the joint transversely. The butt joint, angular corrugated and round corrugated utilized the use of SBA joint material which is specifically used when panels are required to slide into place and have small tolerances. The shear key specimen use a grout for the joint material and upon testing experienced debonding between the panels and the keyway grout when post-tensioning wasn’t provided. Of the four joints tested the shear key resulted in the lowest ultimate capacity due to the debonding of the grout from the panels. It was found that post-tensioning the joints greatly increased the shear capacities of the transverse joints. In three of the four different joints the shear capacities of the non-tension joints resulted in more than half the ultimate capacities of the post-tension specimen ultimate capacities. The only specimen that hardly surpassed their identical non-tensioned specimen was the butt joint but upon further examination it was found that reinforcement of the panel had moved during concrete pouring and had directly affected the results. It was very evident through this research post-tensioning significantly increases the shear strength of a joint.
CHAPTER III
FLEXURE TESTING

Deck Specimen Details

All test specimens constructed at Utah State University utilized a female-to-female connection for the keyway portions of the transverse joints. The female-to-female joint has proved to be an effective keyway joint (Issa et al., 2003). It provides an easy to access area for grouting as well an efficient shear link between panels to effectively transfer loads. Figure 1 shows the details of the female-to-female connection used by Utah State University in their test specimens of post-tensioned deck panels.

Transverse female-to-female joints work ideally under the circumstances that they are prepared properly grouted. Details for preparing and applying grout are specified by Departments of Transportation (DOT) as well as the manufacturer of the grout. Using both sources grouting is accomplished by cleaning the surface thoroughly removing any...
debris and other contaminates the joint may have come in contact with. The joint must then be saturated surface dry (SSD), which is applying water to the surface of the joint to prevent the concrete from the panel from pulling the water quickly from the grout after it is applied. After which, grout can be mixed using the manufactures specification and applied. The grout chosen for the deck joints, shear pockets and haunches for the test specimens at Utah State University was a BASF flow-able grout with a product name Masterflow 928. The American Association of State Highway and Transportation Officials (AASHTO) require, in section 5.14.4.3.2 and 4.14.4.3.3d, a compressive strength of 5 ksi after a 24 hour period (AASHTO, 2007). Although the Masterflow 928 Non-Shrink grout product lists its 24 hour compressive strength to be 4.5 ksi, this value seemed to be conservative for the mixture and actually produced compressive strengths much higher than 5 ksi after a 24 hour period. This grout is commonly used by Utah Department of Transportation (UDOT) and other DOT’s and has proved to be satisfactory in application, bonding and strength. All mixing and application was verified with a BASF representative and final product inspected. Grout and concrete information and application techniques are found in Appendix A. Although some bridges through the United States use only the bonding and shear key, the grout provides to link individual deck panels together, cracking has often resulted in such connections from stresses caused in transferring loads and expanding and contracting due to adverse climate conditions. Crack prevention is being sought out more fervently by DOT’s because of the quick deterioration of the bridge system and costly repairs incurred due to cracking. Longitudinal post-tensioning provides reinforcement necessary to prevent such problems.
Longitudinal post-tensioning is preferred by many DOT’s because it behaves more monolithically and requires less maintenance (Sullivan, 2003). UDOT currently provides longitudinal post-tensioning to multi-span bridges or bridges with a life expectancy of greater than 15 years. Bridges constructed without post-tensioning are projected to have a lifespan less than 10 years (UDOT, 2008b). Because post-tensioning of bridge decks has become more common, code limiting minimum spacing and stress across a transverse joint have been written. AASHTO section 5.14.3.3c requires a stress along the transverse joint due to post-tensioning of 250 psi after all the losses occur (AASHTO, 2007). Currently UDOT requires that all post-tensioning provide a stress of 300 psi along the length of the transverse joint to ensure that the AASHTO requirements are met once losses occur. Longitudinal post-tensioning strengthens transverse joints preventing cracking which ultimately can damage the bridge deck reinforcement in the joint, the girders and entire superstructure below.

Post Tension Connection

Two types of transverse post-tension connections were constructed at Utah State University. The first longitudinal post-tension connection chosen for construction and testing is currently being used by UDOT and is considered the standard for post-tension bridges in Utah. This system consists of multiple lengths of high strength thread rod with a minimum diameter of 1-3/8 inch and that has an ultimate strength of 150 ksi. Each tension rod is anchored by a 1-3/4 inch steel plate that provides at least 50 square-inches of anchor area (UDOT, 2008c). That is, 50 square-inches that are in contact with the concrete deck, where the surface of the concrete is in direct opposition with the tension
forces. This specification is detailed by UDOT for the purpose of ensuring proper
reaction forces from the concrete without causing the concrete to crush or cracking to
occur once tensioned. Figure 2 shows the detail of a common post-tension plate currently
being used by UDOT.

As seen from Figure 2 the post-tension anchor plate is equipped with shear studs
that are required by UDOT to have a minimum size of 4 inch length with a \( \frac{1}{2} \) inch
diameter. The shear studs provide a positive connection to the concrete. Therefore,
anchor plates are placed in the concrete during the precast deck panel construction.
Figure 3 shows the detail of the anchor plate in the concrete with threaded rod. The post-
tensioned conduit runs the length of the panel and terminates at the anchor plate. The
conduit used for 1 3/8 inch diameter thread rod is a 2 inch diameter corrugated steel
tubing. The application of this connection is specified by UDOT to be space no more
than 6 feet from adjacent strands and a maximum distance of 3 feet from the edge of the
panel (UDOT, 2008a).

![Diagram of UDOT post-tension anchor plate detail](image)

Figure 2. UDOT post-tension anchor plate detail.
The longitudinal post-tensioning runs in pairs. While one rod links a deck panel to an adjacent panel the other rod links the deck panel to an opposite adjacent deck panel. This staggering of the rods benefit construction feasibility because lengths are minimized.

Figure 4 is the configuration of the post-tension system currently in use by UDOT and other DOT’s.

Figure 3. Post-tension anchor plate application detail.

Figure 4. Post-tension panel link detail (UDOT, 2008a).
The post-tension system currently in use and as shown in Figure 4 makes individual deck panel replacement very difficult without removing adjacent panels. Due to the staggering of the rods all panels are linked. The full scale specimens constructed at Utah State University utilized three of the post-tension connections across a 12 foot transverse joint maintaining the maximum distance of 6 feet but resulting in a center-to-center spacing of 4 feet, as seen in Figure 5. The 3 foot maximum distance from rod to edge was also maintained and resulted in a 2 foot spacing.

**Curved Bolt Connection**

The second post-tension system constructed and tested was the curved bolt connection. The curved bolt’s purpose is to provide post-tensioning over the critical portions of a bridge deck by only post-tensioning across the transverse. This allows linking of the deck panels to be focused on individual panels than the entire deck and

![Figure 5. Post-tension test deck.](image-url)
provides a means of single deck panel replacement. The proposed post-tensioning was designed and tested by Utah State University graduate students Scott Porter and Logan Julander (Porter, 2010 and Julander, 2010). From the research and testing it was found that the joint capacity was a function of the post-tension diameter, ultimate strength, length and bearing area. The 36 inch curved bolt with a diameter of 7/8 inches was selected because of its performance from the preceding research at Utah State University but the bearing area from the prior research had to be modified due to the full scale deck panel and the required tributary area of each post-tension rod. The 36 inch curved bolt is detailed in Figure 6.

Spacing of the curved bolt was calculated using the properties of the threaded rod and the required stress of 300 psi over the transverse joint. The curved bolts were designed as grade B7 thread rod having an ultimate yield stress of 120 ksi. Two bolts were designed to run parallel with each other to reduce the number of post-tension anchor areas, with the purpose of making construction quicker. The tributary length and spacing

![Figure 6. 36" Curved bolt connection](image)
associated with these two curved bolts was determined from Figure 7 and Equation 1.

\[ L = \frac{\sigma_s A_s N}{\sigma_P D} \text{ [in]} \]  

Where \( L \) is the length that experiences stress due to the post-tensioning, \( \sigma_s \) the yield stress of the curved bolts, \( A_s \) is the area of the curved bolt, \( N \) is the number of bolts linking the two panels, \( \sigma_P \) is the required stress across the joint and \( D \) is the depth of the joint that will experience the post-tension forces. Once these parameters are put into Equation 1 the resultant length experiencing a stress of 300 psi is calculated to be 56.6 inches. It was determined that the spacing of the post-tension curved bolts be reduced to 48 inches which reduced the stress in the bolts to 85% of their yield stress.
The curved bolts were spaced in pairs every 4 feet, matching the configuration of the post-tension specimens. Each tributary area of a pair of curved bolts with dimensions of 48 inches by 8.5 inches multiplied by the required compressive force of 300 psi results in a force of 122.4 kips. This force transferred directly to the curved bolts and in particular to the anchor plate. With this understanding the anchor plates were designed. Assuming a concrete compressive strength of 4.5 ksi and 122.4 kip force the required area for the anchor plate was calculated to be 27 square inches. The depth of the rebar inside the precast panel limited the width of the anchor plate in order to maintain the same cover of that of the anchor plate currently used by UDOT. The plate width was selected to be 3 inches leaving a calculated length of 9 inches. To ensure that cracking wouldn’t occur the plate was lengthened to 11 inches rather than the 9 inches calculated. This increase in length, provided a safety factor 1.2 times the required 27 square-inches. The thickness was also determined to be very important factor in order to uniformly distribute the load placed on the anchor plate. The thickness of the anchor plate was chosen to be 2 inches. Figure 8 is a detail of the designed plate.

![Figure 8. Curved bolt anchor plate Details](image-url)
The arc of the curved bolt was first designed with a radius of 39 from prior researchers at Utah State University (Porter, 2010 and Julander, 2010). With full-scale specimens the radius had to be adjusted to sink the ends of the bolts deeper into the precast panel in order to fit the newly designed anchor plate. The arc of the curved bolts were as a result designed with a radius of 60 inches. The 3 foot linear distance from bearing center of anchor plate to anchor plate was maintained in this change to ensure comparable data to previous research.

The full scale flexure deck specimens were designed to undergo negative bending testing and therefore required to be attached to a girder system. The attachment to the girders, required the use of shear stud block-outs and nelson shear studs to join compositely the girders to the deck. The design and details of the shear stud block-outs and studs used on the flexure specimens was designed to comply with UDOT’s standard details of Accelerated Bridge Construction. Figure 9 shows the details of the chosen block-outs and nelson studs.

Figure 9. Shear block-out and stud details.
The details of the shear block-outs and nelson studs in Figure 9 were used on a bridge on 800 North running over I-15 in Salt Lake city Utah. UDOT has set limits on the size of the block-outs limiting the top of the block-out to 17 inches maximum in length and the bottom to 16 inches maximum length. The width is required by UDOT to maintain a top dimension of 5 inches and a bottom dimension of 4 inches. The size, number and spacing of the shear studs doesn’t have as many required standards and is mostly left up the design engineer to determine. There is a minimum requirement on the location of the shear studs closest to the beam edge. The must maintain a minimum distance of 2 inches from the top edge of the block-out to center of the stud to ensure possible application in the field using a welding shear stud gun. The spacing of the of the shear block-outs was also required by UDOT to have a maximum spacing of 4 feet center to center as well as a suggested distance from panel edge to center of block-out of 15 ½ inches. Due to the geometry and placement of post-tension strands combined with symmetry it was determined that the pockets be spaced 3 feet on center and 15.5 inches from the panel edge as seen Figure 10, of the flexure specimen, of the post-tensioned standard deck.

Figure 10 also shows the standard deck reinforcement required by UDOT. All reinforcing bars are #6 rebar with a yield stress of 60 ksi. The spacing of the longitudinal reinforcing is required to be 12 inches apart on center except for where block-outs don’t permit. The transverse rebar is 2 ½ inches from the transverse joints then four bars spaced every 3 inches center to center followed by a spacing of 6 inches center to center spacing. The full-scale test specimens details are shown in Figure 11.
Figure 10. Deck panel reinforcing and block-out spacing.

Deck System Construction

The site for construction of the test deck specimens was located near Utah State University’s main campus in their System Materials And Structural Health (SMASH) laboratory. The precast panels were formed and constructed using typical precast practices. The formwork was placed on the floor of the SMASH lab and separated from the floor with plastic sheeting. The post-tension anchor plate was placed in position prior to any other construction because it sits lower in the precast panel than the lowest mat of rebar. The #6 rebar was cut and placed according to previous design.
Figure 11. Flexure specimen final details.

The transverse rebar was placed first and was separated by the plastic sheeting through the use of 1 inch continuous plastic supports. The longitudinal rebar was then placed on top of the transverse and tied using typical tie wire forming the lower mat of each precast panel.

Once the lower mats were constructed and in place the post-tensioning conduit was placed. The post-tension duct was a 2 inch galvanized corrugated pipe that was supported from formwork to anchor plate. Each corrugated post-tension conduit was
equipped with a 1 inch grout saddle, tube and valve located 6 inches from each end.

Figure 12 shows the anchor plate as well as a view of the corrugated conduit and the first mat of rebar in the formwork of the UDOT post-tension precast panel. Once the first mat was constructed and situated in each specimen, the block-outs which were cut from rigid insulation blocks then wrapped with plastic sheeting for ease of removal were then placed in their designed position. Strain gauges were applied to the longitudinal rebar on the lower mat.

The strain gauges chosen for the rebar application came by recommendation from a Micro-Measurements manager. The gauge chosen was designed specifically for civil engineering use. It was a 120 ohm resister gauge with 23 feet of preinstalled three conductor, vinyl insulated cable. Each rebar receiving a gauge was buffed down to a smooth surface using a 50 grit grinding disk. Then the surface was degreased, sanded and cleaned using the proper steps and procedures outlined by Vishay Micro-Measurements. Once the surface was prepared and cleaned the gauge was applied to each selected bar, receiving epoxy and clamped as seen in Figure 13 (a) and (b).

![Figure 12. Post-tension (a) Anchor plate (b) Conduit and grout tubing.](image-url)
After each gauge reached the proper curing time the clamps were removed and MCoat F was applied to protect the gauges from any water that they would possibly come in contact with once the concrete was applied. The MCoat F is also seen in Figure 13 (c) and (d). More information on the strain gauges used and proper application is shown in Appendix B. The vinyl insulated cable was protected through the use of half inch plastic tubing that was slotted and attached to a transverse rebar near the gauge area. The tubing combined a number of gauge wires and exited through the form work at mid thickness. The tubing’s purpose was to protect the cable from any other construction and concrete applications. The rebar instrumentation is shown in Figure 14.
Figure 14. Location of strain gauges.

The gauges were installed on top and bottom mats in the very same location as seen in Figure 14. On the full-size flexure panels a total of 20 gauges were placed in each panel. Twelve (six on each mat layer) of those gauges were applied to an area 10 inches from the transverse joint edge. Eight gauges (four on each layer) were placed in the mid-span of each panel.

After the lower mat strain gauges were applied and post-tension duct was in place, the second mat’s longitudinal rebar were placed on 4 ¾ inch plastic rebar chairs that were symmetrically placed every 3 feet. The chairs support the longitudinal rebar at the desired distance off the floor then the transverse rebar was place on top at the detailed distances. The distance from the top of the second mats transverse rebar to the top of the precast panel provides clear cover of 2 ¼ inches. Once the second or upper mat was
placed and tied the block-outs were anchored to the top rebar using tie wire to ensure they remained stationary during the placing of the concrete.

The curved bolt conduit was placed after the second mat was in place. The conduit that was chosen was a 1 ½ inch flexible plastic conduit. It was cut to length and glued to a rigid insulation triangular block-out. The conduit provided an area for the curved bolts to slide into and the block-out provided an area for the anchor plate to sit as well as an area for tightening down the bolts. The conduit and block-outs were held in place using a cantilevered arm that extended from the formwork and attached to the block-out. The desired radius was checked on the conduit and held in place with tie wire. The conduit was looped by a #4 rebar stirrup that attached to the lower mat. Figure 15 shows the application of this stirrup. Lifting hanger were also placed in the formwork at this time. They consisted of four threaded rods 20 inches in length and placed on the longitudinal sides of the panel, through the formwork, 18 inches from the end. The lifting hangars were designed to attach to hooks that would enable lifting each panel with ease. All eight test deck panels were constructed in the same fashion as previously described.

Figure 15. Curved bolt stirrup.
After the reinforcement, strain gauges and connections were in place, concrete was placed. The concrete provided was a AA/AE state mix with a specified compressive strength of 4000 psi. The slump of the concrete 4.5 inches and percent air was 6.5 percent. The post-tension connection flexure panels were poured first. The curved bolt flexure panels and the four shear panels were poured together with a second truck weeks later. Concrete test cylinders were prepared on site for a 28 day test and a time of experiment test. The concrete cured inside the lab for 28 days. After 28 days, cylinders were tested to ensure the panels desired compressive strength was reached and then moving the panels onto the girder system began.

The beams chosen for girders were purchased in two 40 foot lengths and cut to 20 foot lengths, resulting in four separate girders two for each flexure specimen. The beams were purchase separately through two companies, Bowman and Kemp Steel and Supply Co. in Ogden, Utah and PDM Steel Service Center, Inc. in Spanish Fork, Utah. The beams were prepared with three stiffeners on each. The purpose of the stiffeners was to prevent web buckling that could possibly occur due to large loads and a deep slender web. The stiffeners were located at the mid-span of each beam and 7 ½ feet from mid-span in each direction. The stiffeners were only located on one side of each beam. The beams were spaced 6 feet apart center to center as designed and shimmed off the floor with wood at ends and mid-span to be able to apply lifting straps around the structure once panels and grouting had been completed. On normal ABC projects precast panels are equipped with leveling bolts that are located periodically at each girder to align panels and account for elevation changes. The space left between the panel and the girder (called the haunch) would eventually be grouted and the deck forces transferred to the
grout. With the beams situated with their strong axis parallel to the level floor spacers were welded on top of the beam to replace the leveling bolts. This was only performed under the assumption that the deck spacing from girder to girder would be equal on all four girders. The spacers were 1 ½ inch solid steel rod cut at 1 ½ inches and placed equal distances down the beam totaling 6 spacers per beam. Figure 16 (a) is a view of the girder system, stiffeners and haunch spacers prior to deck application.

With the girders in place the cured precast panels were place symmetrically on the girder system. The transverse joints were carefully measured to ensure proper distances were achieved and edges aligned. Figure 16 (b) shows the precast panels being positioned onto the steel girders. To further ensure proper alignment the post-tensioning rod and curved bolts were placed in their desired conduits. Figure 17 (a) and (b) shows both systems inserted into their individual conduits.

Figure 16. (a) Girder system assembly (b) Placement of the deck panels.
The curved bolts and post-tension rods were purchased from Williams Form Engineering Corp. All rods were cut to length prior to Utah State receiving them. The curved bolts were rolled by Ipaco in Logan, Utah, to receive the desired diameter. All three post tension rods on the post-tension system that extended through the transverse joint were equipped with the same strain gauges as those installed on the rebar inside the deck panels. Three out of the six curved bolts also received strain gauges. Figure 18 (a) and (b) show the strain gauges applied to the post-tension bars. The purpose of the strain gauges were to measure the tensile force in the bolts during post-tensioning and use that force to calculate the stress applied over the transverse joint. The gauges were located in the transverse joints and the wire exited the transverse joint vertically. With the panels aligned, nelson shear studs were welded to the girders below. The shear studs and the stud welding gun and machine were rented from Western Stud Welding in Sandy, Utah. The operation of the stud gun and machine were overseen by a manager of Western Stud Welding. Periodically tests were performed on the studs by bending them over 30 degrees to ensure the weld penetrated the girder thoroughly.
All studs were applied to the girders were 6 inches long and had a diameter of 7/8 inches. Figure 19 is a view of the stud welding being performed as well as a completed shear stud area. After the welding of the shear studs were completed the ceramic ferrules at the base of the studs, that were used to contain the weld, were broken away and the surface of the girders were cleaned to remove any debris that might cause irregularities in the pouring of the non-shrink grout throughout the haunch.

Figure 18. Strain gauges (a) Curved bolts (b) Post-tensioning.

Figure 19. (a) Shear stud application (b) Finished shear stud pocket.
Cleaning the surfaces where grout was to be applied proves to be very important as Issa et al. (2003) have shown through research. The surface of the transverse joint was pressure washed thoroughly to remove the finish layer and expose the aggregate. Once clean and a bituminous surface exposed, water was placed on the joint periodically for a few hours before applying grout to prevent the concrete from absorbing the water in the grout. At the base of each joint a 1 inch foam backer rod was placed extending the length of the joint. Although in field installation, this backer rod is bonded to the base of the joint using a silicone caulking adhesive, formwork was applied below the backer rod to ensure that it would remain in place during grouting. The backer rod was important to keep in the joint because it takes the place of grout in the bottom of the joint reducing the jointed area which could ultimately take away from the capacity of the structure.

Grouting, using a non-shrink grout provided by BASF Chemical Company with the product name Masterflow 928, was performed to UDOT’s standards and mixed according to BASF’s instructions. The Masterflow 928 grout is a flow-able non-shrink grout with an expected one day compressive strength of 4.5 ksi. It was mixed and applied generously to the wetted and prepared joints. Prior to grouting the haunches and shear pockets, UDOT requires that post-tensioning of the deck panels. The purpose for tensioning the rods prior to grouting to the girders stems from the assumption that as the panels are compressed together there is possible movement between the panels and girders below. If the haunch and shear pockets were grouted prior to post-tensioning cracking and debonding is likely to occur between the panels and girders.

Strain gauges were monitored on the data acquisition system as tightening of the post-tension connections were performed. Tightening of the curved bolts and post-
tension rods were performed in a systematic manner to ensure stresses were uniform over the joint. As the curved bolt specimens were tightened all strain gauges went off scale prior to the required tensioning. It was observed that as the bolts were tightened the strain gauge wires were shorted due to pressure between the bolt threads and the conduit. Without the strain gauge reading the post-tensioning had to be performed using a calibrated torque wrench, multiplier and socket that is commonly used in post-tensioning field applications. It was rented from Williams Form Engineering Corp, which also supplied a torque curve to be able to determine the torque necessary to provide the present bolt type with the desired load. The curved bolts received a torque of 980 foot-pounds while the 1-3/8 inch diameter post-tension rod on the post-tension specimens received a torque of 2,300 foot-pounds to ensure a compressive stress across the joint of 300 psi. Figure 20 shows the process of applying torque to the post-tension rods. While applying torque to the curved bolts a crack formed that ran from the center of the anchor plate across the joint to the opposite anchor plate. The desired torque needed to obtain a compressive stress of 300 psi across the transverse joint was 960 foot-pounds. The crack formed prior to the desired torque, occurring around 850 to 890 foot-pounds. The resulting compressive stress across the joint was between 265 psi to 278 psi. The obtained stress, although smaller than the post-tension specimen, maintained the minimum required stress as outlined by AASHTO.

Grouting was once again performed on the deck systems. First, the post-tension specimens reached the desired torque, then non-shrink grout was pumped into the grout tubes that were attached to the conduit.
This process is performed on post-tension systems to protect the post-tension rod from corroding due to possible water collection in the conduit. Second, formwork was placed around the haunch area and grout was poured into the shear block-outs filling the haunch and the shear pockets. Block-outs on both systems where the anchor plates are located were filled last. The grout cured and after it reached the desired compressive strength of 5 ksi, the deck systems were maneuvered into place for the negative bending test. Figure 21 shows one of the finished flexure deck test specimen being moved into the testing apparatus.

**Negative Moment Apparatus Setup**

Testing of the full-scale flexure specimens were performed at Utah State University at the location of their construction, the SMASH lab. The test apparatus was designed using spherical bearings as reaction points for the test specimens. These reactions were designed perform like simply supported reactions. Figure 22 is an elevation view of the testing apparatus for the flexure deck systems.
Figure 21. Placement of the flexural specimens into the testing apparatus.

Figure 22. Designed flexure test apparatus.
Spherical bearings were placed at mid-span and a ½ foot from the edge of the concrete deck, of the deck system. The bridge system was tied directly over the end reaction/bearing and through the strong floor with four 2 in threaded rod. The threaded rod was attached to the top flange of a W27 X 161 that spanned the width of the bridge system. Tying the bridge system to the strong floor directly over the bearing reaction created a pin type support while at mid-span the bearing reaction was a roller type support. These reactions created a cantilevered end that was positioned under a reaction frame where two hydraulic, 250 ton capacity at 10,000 psi rams were positioned directly above the girders. Applying the load at the end of the bridge system created negative bending with the greatest moment occurring at the mid-span of the bridge deck system. The center of the rams were located at the opposite end of the far reaction but also maintained a distance of ½ a foot from the end of the deck panel. Figure 23 is an actual view of the curved bolt flexure specimen in the negative bending apparatus.

Figure 23. Actual flexure test apparatus
Instrumentation of the flexure test was performed with the use of 7 potentiometers (string pots), 2 load cells, 3 girder strain gauges and 40 deck panel reinforcement strain gauges. Load cells with a capacity of 400 kips each were placed directly below the hydraulic rams, 6 inches from the end of the concrete deck and position directly over each girder. All seven potentiometers had a range of 5 inches to measure deflection/movement. Five of the seven potentiometers were positioned to measure vertical displacement. Of the vertical potentiometers, two were positioned under the hydraulic rams, below the girders. Two others were located mid-distance between the two reaction points and below the girders. One potentiometer was attached to the concrete deck, six inches from the edge, located directly below the W27 X 161 and centered between the girders. The purpose of these five potentiometers were to compare the actual deflections to the theoretical that were shown in Figure 26 and calculated in Table 1. The potentiometer under the spreader beam was placed to measure the vertical deflection that occurs from uplift off the reaction due to significant loading over the cantilevered end. Two potentiometers were placed horizontally, extending over the transverse joint, 6 inches from the edge of the deck. They were placed with the front of the potentiometer casing 1 foot from the face of a steel anchor, to capture the total amount of cracking over the joint. Figure 24 displays the profile and elevation locations of each instrument excluding the strain gauges located in the concrete deck. Locations for strain gauges on the reinforcement of the concrete deck can be seen in Figure 14. Stain gauges were applied to the girders near the mid-span to map the neutral axis as load is applied. Due to the stiffeners that were added for extra stability to ensure web buckling wouldn’t occur, the strain gauges could not be placed directly under the joint.
The gauges were placed 3 inches from the center of the span with the top and bottom strain gauges placed 3 inches below and above the flanges on the web. The final gauge is located directly in the middle of the web between the top and bottom gauges. Figure 24 also shows the location of the strain gauges that were applied to the girders.

Once the flexural decks were instrumented and all sensors connected to the data acquisition system, loads were applied to the cantilevered end simultaneously in predefined increments. Initially increments of 20 kips per ram (40 kips total) were chosen but with inspection these increments changed per specimen in order to more fully capture the responses of the system. Every load was applied twice in a testing series to ensure consistency with the data. The testing was monitored by a number of students and professors as well as digitally recorded for further evaluation of the system.

**Deck System Theoretical Behavior**

The transverse post-tension connections previously mentioned were applied to full scale specimens. Deck panels on a bridge system undergo a variety of loads, namely shear, positive bending and negative bending. Most deck systems have more than one span or length of girders. The longitudinal transition from girder to girder is usually supported in a pin or roller type of reaction through the use of a pier. Because the bridge girders are not continuous over the pier, girders undergo mainly positive bending. However, the concrete deck that spans the girders is continuous over the entire length of the bridge and therefore the deck undergoes a large negative moment over the piers. The negative moment on the bridge system applies a tension force at the top of the concrete.
Figure 24. Instrumentation location (a) Profile view (b) Elevation view.
If the tensile forces are large enough, cracking can occur in the concrete and the system will allow water to penetrate through and deteriorate the quality of the bridge members.

The following is a condensed summary of the calculations used in designing the test specimens to ensure proper testing and reliable results. Theoretical calculations were needed to determine proper beam sizes, testing procedures, expected loads and deflections. The first calculations are entirely dedicated to the full scale testing of the deck post-tension connections undergoing negative bending.

The deck specimens undergoing bending required a girder system in order to properly simulate a negative bending application on a bridge system. The deck system was designed to sit on two girders for stability. The girders would need to provide the proper stress area to ensure the elastic neutral axis was below the deck. Requiring the neutral axis to be below the deck was a result of two conclusions: first, the deck specifically, must be entirely in tension once load is applied to ensure unwanted compressive stresses wouldn’t interfere with solid results. Second most multi-span bridges throughout the United States have relatively deep girders or large pre-stressed concrete girders and due to the large areas of both girder types, the elastic neutral axis is well below the deck system. The process of choosing girder size required that the Elastic Neutral Axis (ENA) be determined iteratively using different beam sized joined to the precast deck panels and including the post-tension reinforcing. In order to be able to accurately calculate the elastic neutral axis, the deck system was transposed into a homogenous system by use of a scaling factor \(n\). The factor \(n\) was calculated by dividing the Modulus of Elasticity of the steel by the Modulus of Elasticity of the concrete. The Modulus of Elasticity of concrete was found by using Equation 2.
\[ E_C = 57000 \sqrt{f'^c} \text{ [psi]} \]  
(2)

Where the compressive strength of the concrete \((f'^c)\) is 4.5 ksi and therefore the Modulus of Elasticity of concrete is 3,800 ksi. The iterative process was then accomplished through the use of Equation 3.

\[ ENA = \frac{\Sigma A y}{\Sigma A} \text{ [in]} \]  
(3)

where \(A\) is multiple simplified areas and \(y\) is the distance from the top of the concrete deck to each of those areas. The girders chosen for the negative bending test specimens were a W21 X 122. With this girder size the elastic neutral axis was calculated to be around 9.3 inches below the datum (datum located at the top of the deck). Figure 25 displays the geometry and details of the cross section of the curved bolt flexure specimen with the neutral axis also shown.

The precast concrete deck panel was designed using UDOT’s standards for precast concrete deck panels. As seen in Figure 25 the width of the deck was designed.

Figure 25. Deck transverse details and ENA.
using a standard one lane width of 12 feet with a deck thickness of 8 ⅜ inches. Using the cross section displayed in Figure 25, the moment of inertia was determined. Using the factor (n) the moment of inertia was calculated using the derivation of the parallel-axis theorem shown in Equation 3.

\[ I = I + Ad^2 \ \text{[in}^4\text{]} \]  

(4)

where the moment of inertia (I) is equal to individual moments of inertia summed to the multiple of their Area (A) and squared distance (d) to the neutral axis. The point at which a load becomes large enough to cause the concrete to crack due to bending is known as the rupture stress (\( f_r \)), shown in Equation 5.

\[ f_r = 7.5 \sqrt{f'c} \ \text{[psi]} \]  

(5)

As a result Equation 5 results in the rupture stress being equal to 503 psi with the design concrete compressive strength of 4.5 ksi.

Knowing the moment of inertia, elastic neutral axis and rupture stress for each test specimen the cracking moment was determined. Due to the beam still being within the elastic range the basic stress definition (Equation 6) was applied and rearranged to find the cracking moment.

\[ \sigma = \frac{My}{I} \Rightarrow f_r = \frac{M_{cr}y}{I} \therefore M_{cr} = \frac{\sigma I}{y} \]  

(6)

The flexure test deck specimens once attached to the W21 X 122 girders were assumed to be completely composite. This assumption greatly simplifies future calculations, namely: plastic neutral axis, plastic moment, maximum moment and
deflections. To create a negative moment region over the transverse joint a simply supports beam with an overhang as shown in Figure 26 was selected for further calculations.

Assuming the entire structure acts compositely a cantilevered beam was used to determine the maximum moment and deflections. Maximum moment was computed using Equation 7 as well as deflections between the supports (Equation 8.1) and at the cantilevered end (Equation 8.2)

\[
M_{max} = \frac{PL}{2} \text{ [kip-ft]} 
\]

\[
\Delta_{1 max} = \frac{PL}{48EI} \left( L^2 - \left( \frac{1}{4} L \right)^2 \right) \text{ [in]} \tag{8.1} 
\]

\[
\Delta_{2 max} = \frac{P(L/2)^2}{3EI} (L) \text{ [in]} \tag{8.2} 
\]

Equations 7, 8.1 and 8.2 are reliant on two unknowns, namely: load(P) and \( M_{max} \) for Equation 7, load(P) and \( \Delta_{2 max} \) for Equation 8.1 and load(P) and \( \Delta_{1 max} \) for Equation 8.2. Solving for any of these unknowns leads to the solving of all three equations.

Figure 26. Simplified deck system analysis.
Replacing $M_{\text{max}}$ with $M_{cr}$ leads to the required load to cause cracking of the structures deck as well as the related deflections at that moment before the polar moment of inertia and neutral axis significantly change. The cracking moment calculations assume that the concrete deck is continuous without any joints and doesn’t take into account the compressive forces provided by the post-tensioning. Therefore, the theoretical calculations could differ from the actual behavior of the deck system. These calculated theoretical values are reported below in Table 1.

As the specimen passes its defined rupture stress the bridge system still has a lot of load capacity remaining. After the concrete has cracked it is assumed that the concrete can no longer carry any more tensile loads and therefore, all forces are taken up in the post-tensioning and the steel girders. The structure is modeled at this point without the concrete portion of the structure. The structure yields when the point at which the composite cross section of only the steel reaches its yield point.

Table 1. Theoretical Properties of the Concrete Composite System

<table>
<thead>
<tr>
<th>Property</th>
<th>Post-Tensioned</th>
<th>Curved Bolt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic Neutral Axis (ENA)</td>
<td>9.28 in</td>
<td>9.31 in</td>
</tr>
<tr>
<td>Moment of Inertia ($I$)</td>
<td>158,900 in$^4$</td>
<td>158,700 in$^4$</td>
</tr>
<tr>
<td>Cracking Moment ($M_{cr}$)</td>
<td>718 kip-ft</td>
<td>715 kip-ft</td>
</tr>
<tr>
<td>Deflection 1 ($\Delta_{1,\text{max}}$) @ Cracking</td>
<td>0.019 in</td>
<td>0.018 in</td>
</tr>
<tr>
<td>Deflection 2 ($\Delta_{2,\text{max}}$) @ Cracking</td>
<td>0.077 in</td>
<td>0.076 in</td>
</tr>
<tr>
<td>Cracking Load ($P_c$)</td>
<td>95.7 kips</td>
<td>95.3 kips</td>
</tr>
</tbody>
</table>
Yield properties are computed using Equations 3, 4, 6, 7, 8.1 and 8.2. These equations result in the ENA, yielding moment, deflection and resulting yield load of the cracked deck specimen. The resulting values are recorded in Table 2.

Once the yielding has occurred the steel becomes plastic. In order to find the plastic moment, the Plastic Neutral Axis (PNA) was required and was found using simple statics. By setting compression forces equal to tension forces of the cross section PNA was solved for by knowing the calculating the distance from a datum to the change in force (compression to tension). Once the plastic neutral axis was located the plastic moment is easily calculated. Solving for the plastic moment required the summing of moments about the Plastic Neutral Axis. Equation 9 is the summing of moments equation required to produce a value for the plastic moment.

\[ M_p = \sum T_i d_i + \sum C_i d_i \]  
\[ \text{[kip-ft]} \]  

where \( M_p \) is the plastic moment of the composite section excluding the concrete. \( T_i \) and \( C_i \) are the tension and compression forces respectively, of individual areas \( (i) \) and \( d \) is the distance from the PNA to the centroid of each of the individual areas.

Table 2. Theoretical Yield Properties of the Steel Portion of the Specimens

<table>
<thead>
<tr>
<th>Property</th>
<th>Post-Tension</th>
<th>Curved Bolt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic Neutral Axis (ENA) @ Yield</td>
<td>11.83 in from top</td>
<td>11.65 in from top</td>
</tr>
<tr>
<td>Yield Moment, ( M_y )</td>
<td>2496.09 kip-ft</td>
<td>2458.98 kip-in</td>
</tr>
<tr>
<td>Deflection 1 ( (\Delta_1)_{\text{max}} ) @ Yield</td>
<td>8.02 x 10^{-4} in</td>
<td>8.45 x 10^{-4} in</td>
</tr>
<tr>
<td>Deflection 2 ( (\Delta_2)_{\text{max}} ) @ Yield</td>
<td>0.75 in</td>
<td>0.79 in</td>
</tr>
<tr>
<td>Yield Load ( (P_y) )</td>
<td>331.81 kips</td>
<td>327.86 kips</td>
</tr>
</tbody>
</table>
Knowing the plastic moment of each system makes it possible to also solve for the total load at the plastic point by once again replacing $M_{\text{max}}$ with $M_p$ and solving for an ultimate load. The resulting values are listed below in Table 3.

With the theoretical data listed in Table 1 and Table 3 and understanding of the flexure test behavior is now available and able to compare to the actual data obtained from testing. Figure 27 is graph of the theoretical load versus deflection for both systems.

Table 3. Theoretical Properties of the Steel Composite System

<table>
<thead>
<tr>
<th>Property</th>
<th>Post-Tension</th>
<th>Curved Bolt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic Neutral Axis (ENA)</td>
<td>15.53 in</td>
<td>20.367 in</td>
</tr>
<tr>
<td>Moment of Inertia ($I$)</td>
<td>9,119 in$^4$</td>
<td>8,083 in$^4$</td>
</tr>
<tr>
<td>Plastic Moment ($M_p$)</td>
<td>3060 kip-ft</td>
<td>2,858 kip-ft</td>
</tr>
<tr>
<td>Ultimate Load ($P_p$)</td>
<td>408 kips</td>
<td>381 kips</td>
</tr>
</tbody>
</table>

Figure 27. Theoretical elastic behavior.
As seen from the graphs there are two elastic regions of the structure. The first is exhibited from the concrete composite structure and its elastic contribution is from the initial loading to the cracking moment. The second elastic region is formed due to the concrete cracking and forcing the composite section to reduce to just the post-tension steel and the concrete girders. This elastic region continues until the plastic moment is reached. Both elastic regions are important for understanding the behavior and added capacity and ultimately for comparing the two systems.
CHAPTER IV
FLEXURE RESULTS

Post-Tension Flexure Results

The first flexure test specimen that underwent a series of loads was the standard post-tensioning currently in use by UDOT. The loading was originally determined to be in 40 kip increments using five data readings per second. This incremental loading was adjusted at the beginning of the first set of tests. It was determined for the post-tension flexure test, to initially load the specimen at 120 kips and then adjust depending on the reaction and output of each subsequent test. The resulting loadings, maximum deflections at the cantilevered ends as well as laboratory notes are shown in Table 4. It was noted that while testing the flexure specimens uplift off the reactions located directly under the spreader beam occurred due to a combination of resulted bending in the spreader beam and the transitional steel between the spreader beam and the strong floor.

Table 4. Post-Tension Maximum Loads, Deflections, and Laboratory Notes

<table>
<thead>
<tr>
<th>Test #</th>
<th>Max. Load (kips)</th>
<th>Max. Deflection Cantilever End (in)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>120</td>
<td>0.316</td>
<td>Slight debonding and cracking along transverse joint.</td>
</tr>
<tr>
<td>2</td>
<td>140</td>
<td>0.355</td>
<td>Large noise. ¼ inch uplift on east reactions.</td>
</tr>
<tr>
<td>3</td>
<td>180</td>
<td>0.589</td>
<td>Multiple loud noises.</td>
</tr>
<tr>
<td>4</td>
<td>220</td>
<td>0.615</td>
<td>Very loud noise at 195 kips. Large debonding at joint.</td>
</tr>
<tr>
<td>5</td>
<td>280</td>
<td>0.800</td>
<td>Cracking propagating from pocket to pocket and transversly.</td>
</tr>
<tr>
<td>6</td>
<td>340</td>
<td>1.020</td>
<td>Loud noise at 245 kips. Crack between deck and haunch.</td>
</tr>
<tr>
<td>7</td>
<td>480</td>
<td>1.239</td>
<td>Loud noise at 340 kips. Major cracks through joint &amp; haunch.</td>
</tr>
<tr>
<td>8</td>
<td>558 (Failure)</td>
<td>4.183</td>
<td>Failure due to plastic hinge forming at center reaction.</td>
</tr>
</tbody>
</table>
This uplift was measured using a potentiometer that was located between the girders and directly below the tied down spreader beam as seen in Figure 24. The uplift was subtracted appropriately from the resulting deflections.

The concrete deck showed minor visible cracking at the joint along the bond between the concrete and the non-shrink grout around a moment of 600 kip-feet but still seemed to be in satisfactory shape. The debonding followed the joint around the post-tension block-outs rather than propagating a crack linearly across to the other side of the block-outs. Cracking of the block-outs and major debonding did not occur until loads that caused a moment of around 1500 kip-feet to 2000 kip-feet were applied. Between moments of 2250 kip-feet and 3000 kip-feet major cracking of the haunches and shear pockets occurred. The post-tensioning forces opposed the composite behavior of the bridge system and began to force the concrete deck to separate from the girders. This was evident when cracks propagated down the entire length of the haunches. Loud noises were heard at these large moments and it was concluded that possible shear stud failure and or concrete separation of the shear pockets was occurring. Total load compared to the deflection at the cantilevered end was plotted to better determine the cracking moment for the deck structure. The plot is displayed in Figure 28 and utilizes a variety of colors signifying each loading cycle. From the plot in Figure 28 the plastic moment of the composite structure is easily identified and is shown to occur around a total load of 550 kips. This loads equal a plastic moment of 4,125 kip-feet at the transverse joint. Identifying a cracking moment from the plot in Figure 28 proved to be difficult.
To better identify the cracking moment of the system, data from the potentiometers at locations placed across the joint as seen in Figure 24 were utilized to map the opening of the joint. The resulting total load versus the opening of the joint can be seen in Figure 29. From the plot in Figure 29, the cracking moment is more easily identified. It is seen to initially occur around a total load of 75 kips which results in a moment of 560 kip-feet. The curve in Figure 29 has a very similar shape to the curve in Figure 28 but differs somewhat because it focuses solely on the cracking of that joint. This cracking as well as the instrumentation can be seen in Figure 30. From the plot in Figure 29, the cracking moment is more easily identified. It is seen to initially occur at a total load of 75 kips which corresponds to a moment of 560 kip-feet.
Although the curve in Figure 29 has a very similar shape to the curve in Figure 28, it differs somewhat because it focuses solely on the cracking of that joint. This cracking as well as the instrumentation can be seen in Figure 30. The joint was completely debonded by failure as seen in Figure 30 but instead of continuing to debond around the post-tension block-outs a large crack propagated through the midsection of the block-out as seen in Figure 30 (c). Figure 30 (a) shows some cracking around the joint at the edge. Although the deck was tested entirely in tension, after each test the load was released causing the specimen to return to its original position but once higher loadings were obtained large debonding of the joint grout from the concrete occurred and parts of the non-shrink grout slipped down causing a pinching effect once load was released and cracked a portion of the concrete in the joint. Figure 30 (a) shows the joint debonded on
the right side but the concrete on the left uncracked while the same joint in Figure 30 (b) shows the joint at failure completely debonded in both sides and due to slippage and pinching crushing of the concrete occurred on the left side once the load was released.

The testing was stopped once plastic deformation began to occur for two purposes. First, the large compressive forces located at the bottom of each girder became substantial enough to cause yielding and began to form a plastic hinge directly over the center support as seen in Figure 31, (a) and (b).

![Figure 30](image-url)
The second reason for putting an end to the post-tension flexure testing was due to significant deflection caused by the forming of the plastic hinge. Room for further deflection was unavailable because the girders spanned two feet past the bridge deck which caused a greater deflection at the ends of the girders. While there was around 10 inches between the strong floor and the girders at the potentiometers location there was only a space of 1/2 inch at the ends of the girders left to deflect. This deflection also caused major compensation of the spherical bearings which were approaching a

Figure 31. Post-tension flexure specimen girder failure.
dangerous angle. Therefore, for safety reasons as well absent deflection room without major adjustments of the test apparatus, the testing was terminated.

Strains on the reinforcement were of value to understand the stress transfer from the post-tensioning and distributing it to the deck concrete and specifically the reinforcement. Four, strain gauges from each deck panel were selected for comparison in this report. The location of these strain gauges are taken from the top mat of each panel at the locations shown in Figure 32. These strain gauges were chosen because they represent a good average of the stresses throughout the deck. Their results are best compared using plots of micro-strain versus total load. The summary of plots are shown in Figure 33 through Figure 36. Appendix D and E contain plots of all strain gauge data for both the post-tension and curved bolt flexure specimens, respectively.

Figure 32. Post-tension flexure test results strain gauge locations.
Figure 33. Post-tension flexure strain gauge #1 (a) East panel (b) West panel.

Figure 34. Post-tension flexure strain gauge #2 (a) East panel (b) West panel.
Figure 35. Post-tension flexure strain gauge #3 (a) East panel (b) West panel.

Figure 36. Post-tension flexure strain gauge #4 (a) East panel (b) West panel.

The first strain gauges resulted in very similar data with the maximum micro-strain occurring around 220 and 230 at the maximum load. The strain gauges in the second position act relatively the same for both panels as well, showing a maximum
micro-strain of 300 at the max applied load. There is an interesting curve in both plots of the second strain gauge, the last maximum loadings of 480 kips and to failure produced a nearly vertical line on the plots. Load increased from 150 to 250 kips while the micro-strain was unchanged. This could possibly be due to the previous loading where large sounds were heard and assumed to be the shear studs separating from the girder or the concrete causing the system to lose its composite nature. The third strain gauge seem to perform very differently but with a closer inspection it is seen that they behave almost identically for both panels before the loading series leading to failure. Once they reach the maximum load and then the load is released the east panels reinforcements strain remains constant while the west panels reinforcement continues to strain. It is unknown why this occurred but the vital portion of the behavior of the decks was very similar. The last strain gauge located mid-deck panel seemed to behave the most different from east panel to west. The maximum strain on the reinforcement on the east panel was 450 for the max load while the west panel produced a strain of 850 at that same load. This is strain difference suggests that the composite nature of the system was compromised before the loading to failure because where shear pockets and studs were separating strain transfer to the reinforcement was lost. This was very evident to extreme cracking from pocket to pocket and extreme haunch cracking.

Curved Bolt Flexure Results

The second flexure test specimen that underwent a series of loads was the newly proposed post-tension joint system named the curved bolt. The testing sequence was
adjusted from the earlier test to provide similar load patterns as well as reduce the amount of tests performed in order to reduce the length of testing. The prior test spanned a period of two days which resulted in significant down time before the final testing which caused the data acquisition to be shut down then restarted which zeroed out all the instruments. Zeroing of the instruments required extra calculating to link the two sets of data together for final analysis. The resulting loadings, maximum deflections at the cantilevered ends as well as laboratory notes are shown in Table 5. It was noted that while testing the flexure specimens uplift off the reactions located directly under the spreader beam occurred due to a combination of resulted bending in the spreader beam and the transitional steel between the spreader beam and the strong floor. This uplift was measured using a potentiometer that was located between the girders and directly below the tied down spreader beam as seen in Figure 24. The uplift was subtracted appropriately from the resulting deflections.

The concrete deck showed minor visible cracking at the joint along the bond between the concrete and the non-shrink grout.

Table 5. Curved Bolt Maximum Loads, Deflections, and Laboratory Notes

<table>
<thead>
<tr>
<th>Test #</th>
<th>Max. Load (kips)</th>
<th>Max. Deflection (in)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100</td>
<td>0.285</td>
<td>Slight debonding and cracking along transverse joint.</td>
</tr>
<tr>
<td>2</td>
<td>200</td>
<td>0.624</td>
<td>Minor cracking evident around curved bolts</td>
</tr>
<tr>
<td>3</td>
<td>300</td>
<td>0.945</td>
<td>Large noise at 248 kips and excessive cracking around bolts.</td>
</tr>
<tr>
<td>4</td>
<td>400</td>
<td>1.298</td>
<td>Cracking spanning entire length, wraps around curved bolts</td>
</tr>
<tr>
<td>5</td>
<td>470 (Failure)</td>
<td>4.195</td>
<td>Failure due to plastic hinge forming at center reaction.</td>
</tr>
</tbody>
</table>
This occurred around a moment of 580 kip-feet but still seemed to be in satisfactory shape. The debonding followed the joint the entire length but seemed to be minimal at this point. Cracking eventually started from the joint propagating around each curved bolt and occurring around 1500 kip-feet. At a moment of 1860 kip-feet excessive cracking around the curved bolts occurred but was highly exaggerated around the curved bolts closest to the edges of the panel. The cracking continued to exaggerate prior to failure but shear pockets and haunches remained uncracked. Failure of the specimen was finally incurred when a plastic hinge formed over the mid-reaction. Total load compared to the deflection at the cantilevered end was plotted to better determine the cracking moment for the deck structure. The plot is displayed in Figure 36 and utilizes a variety of colors signifying each loading cycle.

![Figure 37. Curved bolt flexure specimen moment and load vs. deflection.](image-url)
From the plot in Figure 37 the plastic moment of the composite structure is easily identified and is shown to occur at a total load of 460 kips. This load corresponds to a plastic moment of 3450 kip-feet at the transverse joint. Identifying a cracking moment from the plot in Figure 37 also proved to be difficult, therefore; to better identify the cracking moment of the system, data from the potentiometers at locations placed across the joint as seen in Figure 24 were utilized to map the opening of the joint. The resulting total load versus the opening of the joint can be seen in Figure 38. From the plot in Figure 38, the cracking moment is more easily identified. It is seen to initially occur around a total load of 40 kips which corresponds to a moment of 300 kip-feet.

Figure 38. Curved bolt flexure test moment and load vs. joint opening.
The curve in Figure 38 has a very similar shape to the curve in Figure 37 but has some very visible differences. The joint opening versus deflection plot shows that with every load cycle the joints opening gets substantially larger. Although this is most likely the case for the curved bolt specimen it is noted that after 240 kips significant cracking occurred around the curved bolts and the sensors were actually placed on this crack exaggerating the actual joint opening. Figure 39 shows the cracking that caused possible exaggeration of the curved bolt opening data.

The joint experiences significant debonding on both sides of the joint between the non-shrink grout and the concrete. There was also large cracking that occurred around the curved bolts. The cracking can be seen in Figure 40.

Figure 39. Cracking interference of the curved bolt joint opening
The geometry of the cracking seem to exhibit similar characteristics to typical spalling but very extreme. The crack formed at the surface near at the corner of the anchor plate and the depth of the crack followed the conduit to the midpoint of the transverse joint. The length of the crack propagated towards the edge and/or the nearest curved bolt. At around a load of 250 kips the cracks became apparent and by failure the concrete had completely separated from the panels. The behavior of the curved bolts resulted in both anchor plates lifting up out of the concrete but it seemed that one side of each curved bolt exhibited a larger uplift than the other. There was no pattern to which end lifted higher but it should be noted that both sides had complete concrete separation. Figure 41 shows the anchor plate of a curved bolt connection lifting out of the concrete. Figure 41 also shows cracking from the anchor plate that propagates away from the joint but end at the edge of the panel. This type of cracking was visible at all of the connections next to the edge and occurred at the same moment of the extreme cracking, around a load of 250 kips.
These cracks seemed to stall from further separation at higher loads due to the extreme cracking near the joint and around the curved bolts.

To better understand the stress in the concrete and reinforcement the strain gauges located in the concrete on the longitudinal reinforcing were plotted. From the twenty strain gauges in every panel four gauges were selected as a good average of representing the stress in the bridge. The locations of these four gauges were chosen to be similar to the post-tension specimen strain gauges for further analysis and therefore, the locations are seen in Figure 32. The following strain gauge data for gauges one through four for the top mat of each panel are plotted in Figure 42 through Figure 45.

The first strain gauge had large differences in strain from panel-to-panel during the minor loading cycles but at the largest loading both panels exhibited a strain of 300 to 400 micro-strain. Strain gauge #2 differed greatly as well from panel to panel.
Figure 42. Curved bolt strain gauge #1 (a) East panel (b) West panel.

Figure 43. Curved bolt strain gauge #2 (a) East panel (b) West panel.
The east panel experienced large jumps in strain up to the point of noticeable significant cracking, around 200 to 300 kips but then strain settled for the remainder of the tests and reached a maximum strain of around 800 micro-strain. The first strain
gauge had large differences in strain from panel-to-panel during the minor loading cycles but at the largest loading both panels exhibited a strain of 300 to 400 micro-strain. Strain gauge #2 differed greatly as well from panel to panel. The east panel experienced large jumps in strain up to the point of noticeable significant cracking, around 200 to 300 kips but then strain settled for the remainder of the tests and reached a maximum strain of around 800 micro-strain. The second strain gauge on the west panel never really underwent a major amount of strain and reached a maximum strain of around 150 micro-strain at the greatest load cycle. Strain gauge #3 had a maximum strain of 1,000 micro-strain at the largest load in both panels. The behavior from panel to panel was relatively the same for both strains. Strain gauge #4 had a very similar pattern from panel to panel with a similar jump in strain around the initial load of 150 to 175 kips. It seems as though from 100 to 200 kips the curved bolts acted satisfactory as linking the two panels by straining the longitudinal rebar at these loads. It is unclear why the differences in strain from panel to panel but the best explanation is most likely found in the cracking behavior of the curve bolts. As previously stated, significant stresses caused the curved bolts to crack significantly on one pane rather than the other. There was no pattern and seemed to be fairly random. Strains in the reinforcement were directly affected by the curved bolt forces and when placed in tension, they felt the same tensile forces that were placed on the concrete. One of the biggest similarities of all four strain gauges in the panels is during the maximum load cycles the strain in the east panel gauges decreased significantly while the strain in the west panels followed a similar decreasing curve as the mid and lower loading cycles.
The strain data presented by both system imply possible differences in each system caused by the geometry and type of post-tensioning. The location of cracking and type of cracking suggest that more strain was placed on the post-tension decks to separate from the attached girders. To understand the composite behavior of the bridge specimens, the elastic response of each system was compared to the theoretical calculated responses found in Figure 27. The theoretical behavior of the system measuring the deflection against the load is compared to the actual behavior of the system by plotting only the elastic backbone portion of the curves from Figure 28 and Figure 37 and is displayed in Figure 46 and Figure 47.

Figure 46. Post-tension flexure behavior, theoretical vs. experimental.
Figure 47. Curved bolt flexure behavior, theoretical vs. experimental.

The theoretical curve of each post-tension type has a significant shift in deflection around a total load of 100 kips. This shift is caused from the assumption in the calculations that there would be a cracking moment due to a continuous deck of concrete. Once the deck cracks, the system moves to another elastic portion of only the steel. This includes not only the girders but also the post-tensioning. The deck is not a continuous single slab due to the a transverse joint and no significant cracking occurred around 100 kips. Debonding of the joint was the most noticeable cracking at 100 kips but from the graphs in Figure 46 and Figure 47 the debonding had no effect in the change of the elastic portion of the load versus deflection. Instead both systems acted as though the concrete deck wasn’t there and the elastic behavior relied only on the post-tensioning and girders. The curves do show the composite behavior of both systems. Theoretically the ideal
results would show a linear line of the elastic portion of the graphs if the composite nature between the decks and the girders was perfect throughout the test. Both specimens showed a non linear curve at some point during, although complete composite nature was not lost at these points, the results suggest some minimal separation did, in fact, occur.

The post-tensioning of UDOT’s standard system produces a very evident linear curve at the lower loads of testing. Its slope is nearly identical to the slope of the elastic portion of the theoretical system after the calculated cracking moment. Around a load of 150 kips and a moment of 1,125 kip-feet, the post-tension specimen, load versus deflection began slope away from the theoretical elastic line. It is recorded in the laboratory notes listed in Table 4, that multiple large noises were heard at this loading cycle and was also noted that possible reasons were separation or failure of shear studs. It is most probable that this was the case and can be concluded that at a moment of 1,125 kip-feet the axial forces provided by the post-tensioning began to force separation of the deck from the steel girders. This is also evident, where inspection of the haunches and shear pockets showed significant cracking.

The curved bolt system performed similarly to the post tension system up to a load of 100 kips and a moment of 750 kip-feet. After this moment the curved bolt doesn’t follow the trend followed by the post-tension system by following the theoretical slope of the elastic portion after cracking. Instead it begins to slowly curve away from the theoretical. Although this separation suggests the decks were beginning to have minimal separation from the girders like the post-tension system, however, the separation of the curved bolt decks differed greatly to that of post-tension decks. Unlike the post-tensioning forcing the deck panels to pull away from the girders, the deck panels pulled
away from the post-tensioning causing the cracking around the curved bolts. The deck panels remained composite with the girders but the link between panels was lost and therefore the composite behavior as a whole system was lost as well. Once the cracking occurred around the curved bolts the axial forces were lost and the bridge system acted as though it consisted of only girders.

Table 6 shows a comparison of the strains, maximum moments, and maximum deflection of both systems. The results summarized in Table 6, show that the capacity of the post-tension system currently being used by UDOT is larger. However, the purpose of this research and testing was to understand if the curved bolt system would be a viable option for post-tensioning joints, ensuring the strength of a post-tension connection without limiting future deck replacement.

Table 6. Flexure System Comparison

<table>
<thead>
<tr>
<th>Property</th>
<th>Post-Tension (P)</th>
<th>Curved Bolt (C)</th>
<th>Ratio (C/P)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking Moment, M&lt;sub&gt;cr&lt;/sub&gt;</td>
<td>560 kip-ft</td>
<td>300 kip-ft</td>
<td>0.53</td>
</tr>
<tr>
<td>Yield Moment, M&lt;sub&gt;y&lt;/sub&gt;</td>
<td>3,600 kip-ft</td>
<td>3,000</td>
<td>0.83</td>
</tr>
<tr>
<td>Plastic Moment, M&lt;sub&gt;p&lt;/sub&gt;</td>
<td>4,125 kip-ft</td>
<td>3,450 kip-ft</td>
<td>0.84</td>
</tr>
<tr>
<td>Deflection @ Yield, Δ&lt;sub&gt;y&lt;/sub&gt;</td>
<td>1.35 in</td>
<td>1.34 in</td>
<td>0.99</td>
</tr>
<tr>
<td>Deflection @ Plastic, Δ&lt;sub&gt;p&lt;/sub&gt;</td>
<td>1.90 in</td>
<td>1.82 in</td>
<td>0.96</td>
</tr>
<tr>
<td>Max. Strain @ Gauge #1</td>
<td>230, 200 με</td>
<td>1200, 500 με</td>
<td>5.22, 2.50</td>
</tr>
<tr>
<td>Max. Strain @ Gauge #2</td>
<td>300, 320 με</td>
<td>800, 200 με</td>
<td>2.67, 0.63</td>
</tr>
<tr>
<td>Max. Strain @ Gauge #3</td>
<td>200, 720 με</td>
<td>1000, 1050 με</td>
<td>5.00, 1.46</td>
</tr>
<tr>
<td>Max. Strain @ Gauge #4</td>
<td>500, 1400 με</td>
<td>1600, 1700 με</td>
<td>3.20, 1.21</td>
</tr>
<tr>
<td>Moment @ Initial Composite Losses</td>
<td>1,125 kip-ft</td>
<td>750 kip-ft</td>
<td>0.67</td>
</tr>
</tbody>
</table>
Analysis of the data shows that the curved bolt system performed satisfactory when comparing the yield and plastic moments as well as producing similar deflections at both moment points. There is a significant difference in the cracking moment of each joint but this property was difficult to determine due to both the systems hardly exhibiting a cracking curve behavior as seen in Figure 46 and Figure 47. The behavior of cracking was disrupted due to the concrete not being completely continuous longitudinally because of the transverse joint. Debonding as well as the post-tensioning provided little evidence of a cracking moment. The largest difference seen in Table 6 is the strains on the reinforcement. The strains on the curved bolt panel reinforcement were 1.2 to 5.2 times larger than the strains on the post-tension panel reinforcement. The probable cause is due to the curved bolt being anchored 1-1/2 feet from the joint and therefore all resisting forces apply significant stress to the reinforcement very near the joint. The post-tension system stresses the full longitudinal length of the precast deck distributing the stresses across a larger area. The last comparison includes the moment at which the deck system loses its composite nature. Although both systems lost composite behavior the type of loss was significantly different. The post-tensioning forced separation of the deck from the girders losing the added strength of the post-tensioning to the system. The curved bolt system separated from the deck while the deck stayed composite with the girders and lost added strength from post-tensioning to the system.
Joint and post-tensioning details from the flexure test decks were repeated for a shear deck test specimens. The female-to-female joint dimensions were completely identical to the flexure specimens. Joint preparation complied with UDOT’s and the grout manufacturers specification. The shear deck panels were decreased in length in the longitudinal direction for ease of construction, moving as well as most the longitudinal length had no significant role in testing. The shear test specimens were designed to maintain a full-scale transverse joint and to provide the necessary 300 psi over the joint. Therefore, the longitudinal length needed was only to accommodate the post-tension systems as well as provide an effective shear area. The shortening of the longitudinal length of each panel also remove unnecessary dead weight that would cause flexural stresses during testing. The primary concern for longitudinal length per panel was ultimately the needed length for the curved bolt. The curved bolt was designed to have an anchor point to anchor point spacing of 36 inches. The transverse joint spanned 1 inch of the curved bolt directly in the center. Therefore, 18 inches was the length per panel but subtracting ½ inch for the transverse joint, 17-1/2 inches became the minimum length required for the longitudinal direction for each panel. For construction ease as well as uniformity the specimens were increased to 24 inches per panel resulting in a total longitudinal length for the entire test specimen once joined and grouted together of 4 feet 1 inch. This and other details are shown in Figure 48 of the plan view of each shear deck.
specimen test. As seen from Figure 48, the spacing of the curved bolts and UDOT standard post-tensioning remains the same as the flexure systems. The anchor plates, diameters, bolts and radius of the curved bolts also remained exactly the same as the flexure test specimens. The shear panels would not be placed on top of girders for testing because shearing across the joint can be accomplished without such support. Therefore the shear test panels were not equipped with block-outs for shear stud areas for future beam application. The purpose of the shear panels is to provide useful data and information on joint failure when undergoing significant shear forces. Any supports (beams/girders) would alter desired results.

Figure 48. Shear specimen details (plan view).
Deck System Construction

Shear deck panels were constructed in a similar manner as the flexure specimens. First formwork was placed on the floor of the Utah State SMASH laboratory. Between the formwork and the floor of the SMASH laboratory plastic sheets were placed to separate the deck panels from the floor below. Inside the formwork rebar was placed and mirrored the flexural specimens rebar spacing, maintaining the specs for UDOT reinforcement requirements. The reinforcement was limited to a 2 foot section due to the shorter panel length. The reinforcement was also #6 rebar with a yield stress of 60 ksi. The first mat was constructed first with the transverse rebar laid first on the plastic spacers. The longitudinal rebar was then placed on top of the transverse rebar and tied forming the decks lower mat. Figure 49 is a detail of the post-tension shear test deck showing the reinforcement location as well as strain gauge placement.

The strain gauges used on the reinforcement in the shear panels were also purchased from Vishay Micro-Measurements and were the same strain gauges as used on the flexure specimens.

![Figure 49. Shear test panel reinforcement with strain gauge locations.](image)
The gauges were placed in the same location as the flexure specimens. Strain
gauges were applied to top and bottom mats resulting in a total of 12 gauges per panel.
Mcoat F was placed over each gauge to protect the gauge from water when coming in
contact with concrete. The longitudinal rebar was spaced every foot and the transverse
rebar was spaced 2-1/2 inches from the transverse joint then spaced every 3 inches for 9
total inches, followed by a spacing of 6 inches and 3-1/2 inches.

After the first reinforcement mat and strain gauges were positioned and installed
the post-tensioning conduit for the UDOT specimen were placed spanning from the
formwork to the anchor plate. Following the conduit placement was the second mat of
rebar, with the longitudinal rebar below then the transverse reinforcement being placed
on top and tied. Figure 50 show the formwork, reinforcement and strain gauges of the
shear test panels.

![Image of shear test panels](image)

Figure 50. Shear panel formwork, reinforcement and strain gauges.
Each panel was equipped with an 18 inch 1 inch diameter threaded rod that was positioned 14 inches from the transverse joint. After the 4000 psi AA/AE concrete mix was poured and vibrated into each panel they were allowed to cure for 28 days before moving. Once ready, forms were stripped and each panel was pulled out and placed at the proper distance from its mirror panel forming the required distances for the proper female-to-female transverse joint. The transverse joint were pressure washed thoroughly, exposing the bituminous material. Once the surface of the joint was saturated for a few hours the 1 inch backer rod was placed in the bottom of the joint and the Masterfow 928 grout was poured into each joint. Figure 51 is an elevation view of the grout filled transverse joint on the curved bolt specimen. Once the grout reached a compressive strength of 5 ksi, the post tensioning underwent torque to apply the required compressive stress of 300 psi over the joint.

Figure 51. Grouted joint.
Once the required torque for each specimen was reached the block-outs for each bolt area on each shear test deck were filled with the non-shrink grout as well as the post-tension conduit. With the shear test decks completely grouted and post-tensioned torqued, each one was moved under the reaction frame and into the testing apparatus for the designed shear test.

Shear Test Apparatus Setup

The shear test apparatus and test specimens were both uniquely designed to provide uniform shearing across the joint with negligible bending stresses as well as a uniform distributed load. The design of the apparatus was designed after a shear test performed at the University of New Hampshire in which they also successfully tested full-scale bridge deck joints in shear. The testing apparatus placed the shear test deck specimens on top of 2 inch tall by 2 inch wide steel reactions that ran parallel to the transverse joint. One reaction was spaced a distance of \( d/2 \) from the edge of the reaction to the center of the transverse joint. The value \( d \) is the distance from the top to the extreme compressive area to the centroid of the tensile reinforcement. For the test panels, the distance \( d \) was measured to the longitudinal rebar on the lower mat. This distance was calculated from Equation 10.

\[
d = t_d - C_S - D_T - \frac{1}{2} D_L \ [\text{in}]
\]

Where \( t_d \) is the thickness of the deck, \( C_S \) is the required clear spacing of 1 inch, \( D_T \) and \( D_L \) are the diameters of the #6 transverse and longitudinal rebar, respectively. As a result the distance \( d \) is 6.625 inches. Therefore, the distance \( d/2 \) from the centroid of the joint to
the edge of the reaction was 3 5/16 inches. The second reaction was placed with its back edge plum against the back edge of the panel opposite of the panel resting on the first reaction. The details of the reactions and distributed load are shown in Figure 52. The load was placed a distance $d/2$ from the centroid but on the opposite side of the joint from the first reaction. With both load and reaction being space a distance $d/2$ from the centroid of the joint, the total distance between them becomes the distance $d$. This distance was chosen to provide a shearing effect to start outside the joint and run on a 45 degree angle through the center of the joint and out to the reaction.

The load was designed to be a distribute load that ran parallel to the join. This was accomplished through the use of a 2 inch by 2 inch steel bar that spanned the length of the deck with a W27 X 161 spreader beam centered on top of the 2 X 2 inch steel bar. Prior to loading safety measures were taken to ensure the spreader beams stability.

![Diagram](image)

Figure 52. Shear test apparatus details.
This was accomplished by using wood shims to support the overhanging edges as well as chains wrapping around the ends of the spreader beam and hooked to cranes above. The load was applied using a ram with a 10,000 psi pressure force resulting in 600 ton force. Figure 53 shows the final set up and placement of the testing apparatus as well as the spreader beam and ram.

Instrumentation included the 12 rebar reinforcement strain gauges per panel totaling 24 per test specimen. Plus, two potentiometers were applied to the bottom of the spreader beam at each overhanging end and resting on the floor measuring the deflection of the test decks during loading. The load cell was placed between the ram and the spreader beam measuring the applied load.

Figure 53. Final shear test setup.
These were all monitored and recorded using the Vishay Data Acquisition System and the software Strain Smart. With the test deck in place and the testing apparatus set up loading commenced. The loading initially was performed in 32 kip intervals, loading and unloading twice for every desired load but was adjusted later for individual circumstance and results for each specimen.

**Deck System Theoretical Behavior**

Another critical failure that a transverse joint could likely experience is shear. Because the entire deck of a bridge is in individual segments the transverse joints is the location of shear forces transferring from one panel to another. If the shear forces are larger than the designed shear capacity cracking will occur and water is able to enter into the deck system and corrode reinforcement and super-structure elements below. American Concrete Institute (ACI) code has several equations for calculating the shear capacity of a bridge deck. Equation 10 is the simplified equation for shear and is listed in ACI as Equation 11-3.

\[ V_c = 2\sqrt{f'_c b_w d} \]  

(11)

where \( f'_c \) is the strength of the concrete is psi, \( b_w \) is the width of the concrete sections and \( d \) is the depth from the top of the section to the tensile reinforcement. The definition of \( d \) is the distance from extreme compression fiber to centroid of tensile reinforcement. For UDOT’s standard specs \( d \) is measured to be 6.625 inches. This equation only accounts for the area of the concrete being sheared and therefore the post-tensioning is not a factor.
in this calculation. For a more relevant calculation, Equation 2 (ACI 11.3.1.2) is used to calculate the shear capacity of members that are in axial compression.

\[ V_c = 2 \left( 1 + \frac{N_u}{2000A_g} \right) \sqrt{f_c'} b_w d \]  

(12)

where \( N_u \) is the axial compression pounds per square inch and \( A_g \) is the gross area of the concrete on the shearing plane. This equation provides more capacity due to the multiplication factor of the axial compression. Equation 13 is an equation from the ACI code in section 11.4.1 for shear capacity of concrete for pre-stressed members.

\[ V_c = \left( 0.6\sqrt{f_c'} + 700\frac{V_{ud}}{M_u} \right) b_w d \]  

(13)

This equation accounts for the location of the shear and the corresponding moment to increase the shear capacity. \( V_{ud} \) is the shear at the location of the shearing load and \( M_u \) is the corresponding moment at this location. Table 7 yields the results of the three ACI shear capacity equations. Post-tension geometries are not included in any of the geometries and therefore the resulting values are the same for both connection types.

Table 7. Theoretical Shear Capacities

<table>
<thead>
<tr>
<th>Shear Capacity (Vc) per ACI Equation</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Eq. 11 (Simplified)</td>
<td>128.00 kips</td>
</tr>
<tr>
<td>Eq. 12 (Axial Force)</td>
<td>142.80 kips</td>
</tr>
<tr>
<td>Eq. 13 (Pre-stressed Force)</td>
<td>372.47 kips</td>
</tr>
</tbody>
</table>
Post-Tension Shear Results

The shear panel with the post-tensioning was tested first. The loading was spread across the length of the transverse joint to produce a uniform loading. Strain gauges were used to understand the stress distribution through the panels as the shearing load was applied. From the results three specific strain gauges were used for analysis to understand the stresses at a variety of locations. The locations of the three were chosen for their uniqueness and difference from each other. The location of these strain gauges are shown in Figure 54.

Testing on full-scale specimens in this manner has limited prior research to understand what to expect for each loading. Therefore, the loading cycles were determined merely from expected theoretical outcomes. The post-tension shear test was expected to fail much earlier than what actually happened causing a few extra cycles than what was initially expected. The resulting maximum loading per cycle is shown in Table 8 as well as the max deflections and strains per each loading. The strains are listed with a “S” or “N” noting the panel where each strain gauge is located, either south or north, respectively. The numbers listed with the north and south letters are for identifying the location of the strain gauges within the panel and can be found in Figure 54. All strain gauges listed are results from the lower mat locations. Appendix E and F show a more complete listing of strain gauge data for both the post-tension and curved bolt shear test, respectively.
Figure 54. Location of shear specimen analyzed strain gauges.

The testing of the post-tensioning panel proved to be very durable. Around 45 kips a crack was seen to form from the loading edge and propagate toward the joint. The crack continued through the joint and reached the bottom of the panel exiting near the support around 180 kips. After the cracking propagated through the entire deck deflection continued without any significant failures. Due to the post-tensioning ultimate failure of the joint was never reached. With the deck completely cracked the post-tensioning took the entire vertical load and did not allow for the deck to come apart. The load was finally limited to 750 kips because the joint had significantly deflected and further deflection was limited due to space between the deck panels and the floor. The final crack that propagated through the joint is shown in Figure 55.

Table 8. Post-tension Shear Test Max Loadings, Deflections, and Strains

<table>
<thead>
<tr>
<th>Test #</th>
<th>Max. Load (kips)</th>
<th>Max. Deflection (in)</th>
<th>Max. Strains (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>S1</td>
</tr>
<tr>
<td>1</td>
<td>70</td>
<td>0.087</td>
<td>9.7</td>
</tr>
<tr>
<td>2</td>
<td>130</td>
<td>0.121</td>
<td>20.8</td>
</tr>
<tr>
<td>3</td>
<td>196</td>
<td>0.142</td>
<td>27.7</td>
</tr>
<tr>
<td>4</td>
<td>264</td>
<td>0.161</td>
<td>28.0</td>
</tr>
<tr>
<td>5</td>
<td>400</td>
<td>0.238</td>
<td>26.6</td>
</tr>
<tr>
<td>6</td>
<td>750</td>
<td>0.698</td>
<td>91.2</td>
</tr>
</tbody>
</table>
Figure 55. Final shear failure of post-tension system.

For further analysis the strains of the gauges listed in Table 8 and shown in Figure 54 were plotted. Understanding the behavior of the strains for each gauge should give greater incite to the strength the different post-tension system add to their respective specimens. The plots of each strain gauge are shown in Figure 56 through Figure 58, with two plots per figure, one for each panel at the location specified. As stated prior, all gauges that are shown were located in the lower mat.

The strain gauges located at position #1 showed a small strain in the south panel and initially never surpassed a micro-strain of 30 until the final loading was completed and released the strain increased to 90 micro-strain.
The north panel performed differently, after each loading cycle the strain increased segmentally until a maximum strain at 70 micro-strain. The second strain gauge performed similarly in both the north and south panels, reaching a maximum micro-strain of 60. The biggest difference is the largest loading cycle. Instead of a growing tensile strain the north panel exhibited some compressive strains at the larger loads. This was also a pattern of gauge #1 of the north panel.
Figure 58. Post-tension, strain gauge #3 (a) South panel (b) North panel.

The third strain gauges also showed similar patterns in the lower loading cycles, with strains around 40 to 50 micro-strains per panel. With the larger loading cycle the gauge showed a significant increase in strain in the north panel and remained relatively normal in the south panel.

The most evident conclusion from the analysis of these six strain gauges is that the strains remained relatively the same at load below 400 kips and after which they varied extremely from panel to panel due to the forces concentrating almost entirely on the post-tension reinforcement. It is also noted that strains were relatively low in the bottom mat of reinforcement.

**Curved Bolt Shear Results**

The loading sequence in the curved bolt shear specimen was initially adjusted because of the maximum loads obtained on the post-tension system. They were further adjusted due to small indirect loading cause from a data acquisition error. The resulting maximum loads with deflections and strains are listed in Table 9. The strain gauge letter
and number sequence as well as location is entirely the same as the post-tension shear analysis for more effectively comparing these two systems.

The initial cracking load wasn’t determined as well as the post-tension specimen due to data recording mishaps of the early testing. From further evaluation cracking was estimated to begin at a load of 40 kips. Although the initial cracking was not mapped and recorded during testing the remainder of the cracking was successfully mapped. The curved bolt initially caused a crack to propagate from the distributed load toward the joint. Once the crack reached the joint, significant loads caused the curved bolt anchor plates to be pulled into the panel due to the concrete crushing against the anchor plate. Figure 59 show the anchor plate being pulled into the panel and the separation of the concrete due to the excessive forces.

The anchor areas on the same panel as the distributed load were where this crushing took place. Once this crushing began, the cracking at the joint changed from what was expected. The cracking turned into a debonding failure of the joint due to the lack of compressive forces that were lost once the anchor area was compromise. The denbonding continued down the joint rather than propagating through the joint to the support on the opposite panel.

Table 9. Curved Bolt Maximum Loadings, Deflections, and Strains

<table>
<thead>
<tr>
<th>Test #</th>
<th>Max. Load (kips)</th>
<th>Max. Deflection (in)</th>
<th>Max. Strains (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>S1</td>
<td>S2</td>
<td>S3</td>
</tr>
<tr>
<td>1</td>
<td>107</td>
<td>0.087</td>
<td>-15.1</td>
</tr>
<tr>
<td>2</td>
<td>204</td>
<td>0.177</td>
<td>-17.9</td>
</tr>
<tr>
<td>3</td>
<td>219 (Failure)</td>
<td>0.222</td>
<td>-17.9</td>
</tr>
</tbody>
</table>

26.5 | 32.7 | 40.1 | 44.3 | 58.0 | 47.5 |
The debonding and failed anchor areas caused the specimen to deflect faster and ultimately fail at a lower load than was expected and complete failure occurred around a loading of 219 kips. The final crack and debonding is shown in Figure 60.

To better understand the behavior of the stress issued to the reinforcement because of the curved bolt post-tensioning the previously stated strain gauges were plotted using micro-strain versus the total load. The resulting plots are shown in Figure 61 through Figure 63.

The first strain gauge is very different from north to south panel. From further investigation, three out of the four mats experienced compressive forces at the edge locations. The one mat that didn’t experience the compressive forces was the north panel lower mat.
Figure 60. Curved bolt final crack and debonding.

Figure 61. Curved bolt, strain gauge #1 (a) South panel (b) North panel.
The reason for this difference in strains is most likely due to the failure behavior. Strain gauge #2 exhibited the similar behavior at the lowest load cycle but at the large cycles the strains in the north panel exhibited larger strains. The strains for the south and north panels at the maximum loading resulted in 15 and 60 micro-strain, respectively. The third strain gauge exhibited a similar pattern to that of the second strain gauge. The
small loading cycles resulted in similar strains but at larger loads the strains increased dramatically on the north panel. The resulting strains were around 20 and 50 micro-strain for the south and north panels, respectively.

The most obvious similarity is the north panel on all the strain gauges exhibited larger tensile strains than the south panel. The north panel was the location of the distributed load and also the location at which the anchor plates were pulled into the deck panel. The edges of the reinforcement of the south panel as well as the top mat of the north panel exhibited small compressive forces.

Comparing the strains of the post-tension shear system with the curved bolt shear system proved to be promising. Both specimens performed very similarly at lower loads. Comparing the north panels of each system showed that strains were relatively the same up to a total load of 220 kips. Since the post-tension system held a significant amount of load after cracking larger strains were produce. In both specimens the north panels exhibited higher strain levels than the south. The underlying difference was the compressive forces seen in the edges of the curved bolt specimen decks. Possible reasons for this difference is due to the failure behavior of the curved bolt specimen. To better understand the performance of each system the total load versus deflection of both post-tension specimens were plotted together, as seen in Figure 64.

The resulting plot showing the load versus deflection shows the added strength of the UDOT type post-tensioning. The curved bolt seems to perform better than the post-tension system up to a load around 150 kips. After the 150 kips the strength added from the post-tension system surpasses the strength of the curved bolt and shortly after, around 220 kips the curved bolt completely cracks and major deflection occurs.
Therefore, the curved bolt performs satisfactory when considered at lower loads, which is more likely to occur on a bridge system. When compared to the calculated shear values in Table 7, the curved bolt exceeds the calculated values of Equation 1 and 2 but was significantly less than the prestress equation (Equation 13). The shear equations are specified for beams that do not have transverse joints and are considered completely continuous and are therefore somewhat conservative. The post-tension system surpassed all three calculated values but significant cracking occurred through the entire deck at shearing force of 180 kips across the transverse length of the deck. The curved bolt performed satisfactory in the shear testing.
CHAPTER VII

DISCUSSION

The post-tension systems currently used by UDOT and the newly proposed curved bolt systems both performed satisfactory in laboratory testing in flexure and shear. The length, orientation and location of the post-tension as well as the anchor type, area and location are the main determining factors of whether the system will perform satisfactorily. The post-tension system provides significant support and axial forces throughout the deck and across the joint. The curved bolt system provides focused post-tensioning across the weakest portion of the bridge deck, the transverse joint. This research purpose was to determine if the newly proposed curved bolt system would provide the needed post-tensioning across the transverse joint while maintaining the possibilities of single deck replacement which the current post-tension system provided by UDOT does not meet. The newly proposed system, the curved bolt performed satisfactory in both the flexure and shear test, never surpassing the capacities of the post-tension system but providing capacities that are of a reasonable percentage of the post-tension system.

In flexure the yielding moment for both systems were very evident. The yielding moment capacity of the curved bolt system performed up to 83% of the yield capacity of the post-tension system. Similarly the plastic moment of the curved bolt system performed up to 84% of the plastic moment capacity of the post-tension system. The deflections at the cantilevered end at these capacities were also very similar only varying by 1 to 4% for the yield and plastic moment capacities, respectively. The flexure analysis
also included the understanding and comparing of the strains on the deck reinforcement caused by the different post-tension systems. The results listed in the flexure section of this paper show an obvious trend. The curved bolts strain the reinforcement significantly more at the regions closest to the transverse joint than the post-tension system. It is concluded that this is due to the anchor locations of each system. The curved bolt anchor area is less than 1-1/2 feet away from the transverse joint causing the tensile forces to accumulate greatly in the joint area when undergoing negative bending. The post-tension system undergoes the same tensile forces but because the anchor areas are located at the end of the deck the entire deck spreads those same forces along the longitudinal length. When plotting the load versus deflection for the entire test, up until failure of the girder system, reveals a very important characteristic of the behavior and added capacity to the system from the type of post-tensioning. If the composite nature of the system is mostly preserved, longitudinal post-tensioning that runs through the entire length of the panels as does post-tension system, the capacity of the system as a whole is increased.

The curved bolt systems were found to fail before the entire system failed and therefore added no compressive forces to the composite structure and the girders failed as if there was not a concrete deck. The post-tension system forced a non-composite failure due to the shear studs failing as was evident to the significant cracking along the haunch system. It should be noted that all results are relative to the girder system used for analysis. Basic conclusions about the deck and post-tension system should be made with this understanding.
The shear testing revealed not only the capacities of the different systems but also presented very essential information on failure types. When the capacities are compared using a load versus deflection curve, conclusions can be made that the curved bolt system seemed to deflect less than the post-tension system at lower loads. However, the post-tensioning of the post-tension system provided significant strength to the panels increasing the capacity after a total load of 150 kips was applied.

The shear forces produced a variety of strains throughout each system. When comparing the strains in each system it was noted that between 0 and 200 kips the strains from panel to panel compared to their relative locations were very similar. Therefore, at lower loads prior to and during cracking strains on both systems regardless of the type of post-tension performed very similarly. The largest difference in strains was seen in the largest loading of the post-tension system. Because the curved bolt failed prior to the larger loading, strain gauge data could not be compared. One also necessary observation from the strain analysis was that the north panels, or the panels with the distributed load recorded having higher strains than the south panels on both specimen. This was most likely due to the location of the longitudinal reinforcement with respect to the occurring load or reaction.

The failure type of the shear specimens is arguably the most important comparison. Both specimens showed signs of cracking around 40 to 45 kips distributed along the transverse length and complete cracking throughout the depth of the deck was seen to occur around the distributed load of 180 for the post-tension system and 150 for the curved bolt system. Both values are reasonable when considering required shear. The failure of the curved bolt differed greatly from the post-tension specimen. The
curved bolts initially cracked similar to the post-tension system but instead of propagating through the joint it debonded around the joint because a loss of axial load due to the curved bolts being pulled into the panel on one side. Because the curved bolt is post-tensioned from the deck surface very near the joint capacities of that system are less than if connected from end to end.
CHAPTER VIII

CONCLUSION

The final results revealed that the curved bolt performed 16% less in yield capacity than the post-tension connection and 17% less in the plastic capacity when tested in negative bending. The strains on the reinforcement nearest the joint were much larger on the curved bolt specimens due to the closeness of the anchor area near the joint. In flexure the post-tension system exhibited cracking along the haunches while the curved bolt experienced cracking around each connection. Both system experienced debonding between the non-shrink grout material and the concrete panel.

Testing the specimens in shear produced a larger range of results. Cracking occurred at nearly the same load for both specimens. The ultimate capacity of the curved bolt system was 70% less than the post-tension system. However, the curved bolt system did surpass 2 out of the 3 calculated shear capacities while the post-tension system surpassed all three. The post-tension specimen never experienced major cracking while the curved bolt connection did experience large cracking. Although the behavior of the curved bolt system was less as impressive of the post-tension system, it surpassed calculated shear capacities of a continuous prestressed member which is ideal for a transverse joint.

Future research should be performed on the curved bolts because of its promising results provided by this research. The curved bolts main disadvantages, as found from testing, is the type of failure. Capacities and failure types can be increased and changed with modification of a few properties. Research could be performed using longer curved
bolts to ensure a larger amount of compressive area to distribute the stress throughout that larger area. The anchor plate would possibly benefit if positive connection between the plate and the concrete was performed through the use of shear stud or wedging to the reinforcement. This would hopefully prevent uplift of the anchor when in negative bending. The anchor system should also be well below the concrete which would lessen the radius of the curved bolt limiting the vertical forces the bolt puts on the concrete above. Spacing of the curved bolts could be adjusted by increasing the number of curved bolts per length of joint, which would lessen the tensile forces per bolt which would prevent concrete crushing issues when significant shear forces are applied.
REFERENCES


The concrete compressive strengths were experimentally determined on the same day as the testing to ensure an accurate value for the strength of the concrete. They were found from test cylinders that were poured at the same time as their corresponding decks were poured. The pouring and testing of these cylinders complied with American Society for Testing and Materials (ASTM) C 39 standards. The resulting compressive strengths are found in Table 10.

Table 10. Tested Compressive and Tensile Strengths of Concrete

<table>
<thead>
<tr>
<th>Panel Type</th>
<th>Test #</th>
<th>$f'_c$, psi (28-Day)</th>
<th>$f'_c$, psi (Test-Day)</th>
<th>$f'_t$, psi (28-Day)</th>
<th>$f'_t$, psi (Test-Day)</th>
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<tr>
<td>Post-Tension Flexure</td>
<td>1</td>
<td>4444</td>
<td>5387</td>
<td>282.9</td>
<td>319</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>4286</td>
<td>5566</td>
<td>404.1</td>
<td>548</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>4232</td>
<td>5765</td>
<td>373.6</td>
<td>548</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>4321</td>
<td>5572</td>
<td>353.5</td>
<td>472</td>
</tr>
<tr>
<td>Curved Bolt Flexure</td>
<td>1</td>
<td>3900</td>
<td>5035</td>
<td>329</td>
<td>319</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td></td>
<td>5162</td>
<td>268</td>
<td>458</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>5311</td>
<td>289</td>
<td>464</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>5170</td>
<td>295</td>
<td>414</td>
<td></td>
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<tr>
<td>Post-Tension Shear</td>
<td>1</td>
<td>3900</td>
<td>4919</td>
<td>329</td>
<td>513</td>
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<td></td>
<td>2</td>
<td>5615</td>
<td>268</td>
<td>380</td>
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<td></td>
<td>3</td>
<td>5702</td>
<td>289</td>
<td>557</td>
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<td></td>
<td>Average</td>
<td>5412</td>
<td>295</td>
<td>483</td>
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<tr>
<td>Curved Bolt Shear</td>
<td>1</td>
<td>3900</td>
<td>4919</td>
<td>329</td>
<td>513</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>5615</td>
<td>268</td>
<td>380</td>
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<td></td>
<td>3</td>
<td>5702</td>
<td>289</td>
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<td></td>
<td>Average</td>
<td>5412</td>
<td>295</td>
<td>483</td>
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</tr>
</tbody>
</table>
## MASTERFLOW® 928
High-precision mineral-aggregate grout with extended working time

### Description
Masterbatch® 928 grout is a hydraulic cement-based mineral-aggregate grout with an extended working time. It is ideally suited for grouting machines or plates required to maintain load-bearing support. It can be placed from fluid to slump pack over a temperature range of 45 to 90°F (7 to 32°C). Masterflow® 928 grout meets the requirements of ASTM C 1107, Grades B and C, the Army Corps of Engineers’ CECOM G-19, Grades B and C, and a fluid consistency over a 30-minute working time and ANSI/NSF 61 approved. Suitable for use with potable water.

### Features
- **Extended working time**
- **Can be mixed at a wide range of consistencies**
- **Freeze/thaw resistant**
- **Hardens free of bleeding, segregation, or settlement shrinkage**
- **Contains high-quality, well-graded quartz aggregate**
- **Sulfate resistant**
- **ANSI/NSF 61 approved**

### Benefits
- Ensures sufficient time for placement
- Ensures proper placement under a variety of conditions
- Suitable for exterior applications
- Provides maximum effective bearing area for optimum load transfer
- Provides optimum strength and workability
- Suitable for use with potable water

### Where to Use
- **Industries**
  - Power generation
  - Pulp and paper mills
  - Steel and cement mills
  - Stamping and machining
  - Water and waste treatment
  - General construction

### Applications
- Where a nonshrink grout is required for maximum effective bearing area for optimum load transfer
- Where high one-day and later-age compressive strengths are required
- Applications requiring a nonshrink grout
- Compressors and generators
- Pump bases and drive motors
- Tank bases
- Conveyors
- Grouting anchor bolts, retainer and dewatering systems
- Nonshrink grouting of precast masonry panels, beams, columns, curtain walls, concrete columns and other structural and non-structural building components
- Repairing concrete, including existing voids and cracks, and potholes

### Locations
- Interior or exterior
- Marine applications
- Freeze/thaw environments

### How to Apply
#### Surface Preparation
1. Steel surfaces must be free of dirt, oil, grease, or other contaminants.
2. The surface to be grouted must be clean, SDG, and roughened to a CSP of 5 – 9 following CRI Guideline 03/22 to permit proper bond. For freshly placed concrete, consider using Liquid Surface Cleaner (see Form No. 16857/08) to achieve the required surface profile.
3. When concrete, sheath or tendons are anticipated, concrete surfaces should be chipped with a “cold-chip” hammer, to a roughness of plus or minus 3/8” (10 mm). Verify the absence of moisture following CRI Guideline 03/22.
4. Concrete surfaces should be saturated (pumped) with clean water for 24 hours just before grouting.
5. All standing water must be removed from the foundation and removed before grouting.

### Packaging
- 55 lb (25 kg) bulk bags
- 3,900 lb (1,700 kg) bulk bags

### Shelf Life
1 year when properly stored

### Storage
Store in unopened bags in clean, dry conditions.
Technical Data

Composition
MuellerFlow™ 926 is a hydraulic cement-based material-vaporaincrete.

Compliances
- ASTM C 1107, Grades B and C, and G40 021, Grades B and C, requirements at a fluid consistency over a temperature range of 40 to 90°F (4 to 32°C).
- City of Los Angeles Research Report Number RR 23137
- ANSI / NSF 61, for use with potable water.

Test Data

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<tr>
<th>PROPERTY</th>
<th>RESULTS</th>
<th>1502 ENTRIES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (psi)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plastic</td>
<td>4,200 (15)</td>
<td>4,200 (18)</td>
</tr>
<tr>
<td>3 days</td>
<td>4,200 (15)</td>
<td>4,200 (18)</td>
</tr>
<tr>
<td>7 days</td>
<td>4,200 (15)</td>
<td>4,200 (18)</td>
</tr>
<tr>
<td>28 days</td>
<td>4,200 (15)</td>
<td>4,200 (18)</td>
</tr>
<tr>
<td>Flocculent</td>
<td>2,500 (15)</td>
<td>2,500 (18)</td>
</tr>
<tr>
<td>3 days</td>
<td>2,500 (15)</td>
<td>2,500 (18)</td>
</tr>
<tr>
<td>7 days</td>
<td>2,500 (15)</td>
<td>2,500 (18)</td>
</tr>
<tr>
<td>28 days</td>
<td>2,500 (15)</td>
<td>2,500 (18)</td>
</tr>
</tbody>
</table>

| Volume change (%)          |         |              |
| 1 day                      | > 6     | 0.0 – 0.26   |
| 2 days                     | 9.04    | 0.0 – 0.26   |
| 3 days                     | 8.06    | 0.0 – 0.26   |
| 7 days                     | 9.06    | 0.0 – 0.26   |

| Setting time, h: min       |         |              |
| Initial set                | 2:30    | 3:00         |
| Final set                  | 4:30    | 5:00         |

| Flexural strength (psi)    |         |              |
| 3 days                     | 1,099 (5.9) | 1,099 (5.9)  |
| 7 days                     | 1,069 (7.2) | 1,069 (7.2)  |
| 28 days                    | 1,159 (7.9) | 1,159 (7.9)  |

| Modulus of elasticity (psi) |         |              |
| 2 days                      | 2,66 ± 16 (1.54 ± 16) |
| 7 days                      | 3,02 ± 16 (2.68 ± 16) |
| 28 days                     | 3,24 ± 16 (2.32 ± 16) |

| Coefficient of thermal expansion | |              |
| Initial: 0.0 (mm/m°C) | 0.5 x 10^6 (13.7 x 10^6) | 0.5 x 10^6 (13.5 x 10^6) |

| Split tensile and tensile strength (psi) | |              |
| Spalling Tensile | 575 (6.3) | 575 (6.3) |
| 3 days           | 575 (6.3) | 575 (6.3) |
| 7 days           | 618 (6.3) | 618 (6.3) |
| 28 days          | 657 (6.3) | 657 (6.3) |

| Punching shear strength (psi) | |              |
| 3 days                       | 2,269 (18.3) | 2,269 (18.3) |
| 7 days                       | 2,269 (18.3) | 2,269 (18.3) |

| Resistance to rapid freezing and thawing | |              |
| 36% Cycles RFT 450% | 470 (5.6) | 470 (5.6) |

<table>
<thead>
<tr>
<th>ASTM C 693, Procedure A</th>
</tr>
</thead>
<tbody>
<tr>
<td>1003 – 0.25% flexural load per ASTM C 230</td>
</tr>
<tr>
<td>12% – 0.25% flexural load per ASTM C 230</td>
</tr>
<tr>
<td>25% – 0.25% flexural load per ASTM C 230</td>
</tr>
</tbody>
</table>

*Test results are averages obtained under laboratory conditions. Expect reasonable variations.
Test Data, continued

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>RESULTS</th>
<th>TEST METHODS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate tensile strength and bond stress</td>
<td>ASTM E 480, test*</td>
<td></td>
</tr>
<tr>
<td>Diameter</td>
<td>Depth</td>
<td>Tensile strength/hardness</td>
</tr>
<tr>
<td>(in mm)</td>
<td>(in mm)</td>
<td>(ksi MPa)</td>
</tr>
<tr>
<td>1 1/4 (32)</td>
<td>1 1/4 (32)</td>
<td>21,500 (150-50)</td>
</tr>
<tr>
<td>1 1/8 (28)</td>
<td>1 1/8 (28)</td>
<td>31,500 (213-50)</td>
</tr>
<tr>
<td>1 3/8 (35)</td>
<td>1 3/8 (35)</td>
<td>19,500 (135-50)</td>
</tr>
</tbody>
</table>

Notes:
1. Core was machine to a flat consistency.
2. Recommended design stress is 2.25 ksi (15.7 MPa).
3. Refer to the "Adhesive and Ductile Failure Capacity Design Guidelines" for more detailed information.
4. Tensile tests with coated basalt cores were governed by concrete failure.

Jobaito Testing

If strength tests must be made at the job site, use 1-1/4" (32 mm) metal rod molds as specified by ASTM C 504 and ASME C 114. Do not use concrete molds. Control field and laboratory tests on the basis of desired placement consistency rather than yield on water content.

6. Anchor bolt holes must be grouted and sufficiently set before the major portion of the grout is placed.

7. Shave the foundation from sun light 24 hours before and 24 hours after grouting.

Forming

1. Forms should be liquid tight and nonabsorbent. Seal forms with putty, sealant, caulk, polyurethane foam.

2. Modernly cast equipment should utilize a head form stipple at 45 degrees to enhance the bond. A reversible head box may provide additional head of minimum cost.

3. Side and end forms should be a minimum of 1" (25 mm) distant horizontally from the object grouted to permit evaporation of air and any remaining saturation water as the grout is placed.

4. Leave a minimum of 2" between the bearing plate and the form to allow for ease of placement.

5. Use sufficient bracing to prevent the form from buckling or moving.

6. Eliminate large, unsupported grout areas whenever possible.

7. End form a minimum of 1" (25 mm) higher than the bottom of the equipment being grouted.

8. Extension points may be necessary for both indoor and outdoor installation. Consult your local BASF representative for suggestions and recommendations.

Temperature

1. For precision grouting, store and mix grout to produce the desired mixed-grout temperature. If bags of material are hot, store cold water; if bags of material are cold, store warm water to achieve a mixed-product temperature as close to 70°F (21°C) as possible.

Recommended Temperature Guidelines for Precision Grouting

<table>
<thead>
<tr>
<th>MORTAR</th>
<th>MIXED</th>
<th>WATER</th>
</tr>
</thead>
<tbody>
<tr>
<td>(°F)</td>
<td>(°F)</td>
<td>(°F)</td>
</tr>
<tr>
<td>0° - 8°</td>
<td>40° - 80</td>
<td>15° - 33</td>
</tr>
</tbody>
</table>

2. If temperature extremes are anticipated or special placement procedures are planned, contact your local BASF representative for assistance.

3. When grouting at minimum temperature, use float, grout plate, and grout temperatures do not fall below 40°F (4.4°C) until after final set. Protect the grout from freezing (0°F or -18°C) until it has attained a compressive strength of 3,000 psi (21 MPa).

Mixing

1. Place estimated water (use potable water only) into the mixer, then slowly add the grout. For a final consistency, add with 3 lbs. (1.4 kg) of water to 14 lbs. (6.5 kg) per lb. bag.

2. The water demand will depend on mixing efficiency, material, and ambient temperature conditions. Adjust the water to achieve the desired mix. Recommended flow in 25 - 30 seconds using the ASTM C 595 Flow Cone Method. Use the minimum amount of water required to achieve the necessary placement consistency.

3. Moderately sized batches of grout are best mixed in one or more clean mortar pans. For large batches, use ready-mix truck and 3,000 lb. (1,500 kg) bags for maximum efficiency and economy.

4. Mix grout a minimum of 5 minutes after all material and water is in the mixer. Use mechanical mixing only.

5. Do not mix more grout than can be placed in approximately 30 minutes.

6. Transport by wheelbarrow or buckets to the equipment being grouted. Minimize the transporting distance.

7. Do not temper grout by adding water and mixing after it has set.

8. DO NOT VIBRATE GROUT TO FACILITATE PLACEMENT.

9. For aggregate extension guidelines, refer to Appendix MB-10: Guide to Grouting.
Application
1. Always place grout from only one side of the equipment to prevent air or water entrapment beneath the equipment. Place Mastertop® 968 in a continuous pour. Extend grout that becomes unworkable. Make sure that the material fills the entire space being grouted and that it remains in contact with pipes throughout the grouting process.
2. Immerse all internal surfaces within a bowl and coat the exposed grout with clean wet towels or bar gauges. Keep grout moist until grout surface is ready for finishing or until it sets.
3. The grout should offer significant resistance to penetration of a pointed metal tool before the grout begins to develop or harden or a grout pipe is cut back of the grout to allow for the development of a valve. The grout should be spread on a long surface with a recommended topping compound or grout with ASTM C 309 or preferably ASTM C 1315.
4. Do not bivert grout. Use two sheet metal inserts under the pipe to help contain the grout.
5. Consult your BASF representative before placing lifts more than 8' (2.4 m) in depth.

Curing
Care all exposed grout with an approved curing compound in accordance with ASTM C 309 or preferably ASTM C 1315. Apply curing compound immediately after the net set is reached to minimize potential moisture loss.

For Best Performance
- For guidelines on specific anchor bolt applications, contact BASF Technical Service.
- Do not add plastizers, accelerators, retarders, or other admixtures not licensed in writing by BASF Technical Service.
- The water requirement may vary with mixing efficiency, temperature, and other variables.
- Hold a pre-job conference with your local representative to plan the installation. Hold conferences as early as possible before the installation of equipment, sinks, pilings, or mill work.

Health and Safety
MASTERFLOW® 968
WARNING!
Masterflow® 968 contains silica, crystalline quartz, portland cement, limestone, calcium oxide, gypsum, silica, ammonium, magnesium oxide.

Risks
Product is alkaline on contact with water and may cause masonry to shrink or loosen. Ignition or inhalation of dust may cause irritation. Contains small amount of free respirable quartz which has been listed as a suspected human carcinogen by the International Agency for Research on Cancer. Reported or prolonged exposure to free respirable quartz may cause silicosis or other serious and delayed lung injury.

Precautions
Avoid contact with skin, eyes, and clothing. Prevent inhalation of dust. Wash thoroughly after handling. Keep container closed when not in use. Do not take internally. Use only with adequate ventilation. Use impervious gloves, eye protection, and an HEPA hooded or fitted respirator in accordance with applicable Federal, state, and local regulations.

First Aid
In case of eye contact, flush thoroughly with water for at least 15 minutes. In case of skin contact, wash thoroughly with soap and water. If irritation persists, SEEK MEDICAL ATTENTION. Remove and wash contaminated clothing. If inhalation causes physical discomfort, remove from work area. If discomfort persists or any breathing difficulties occur or are unusual, SEEK IMMEDIATE MEDICAL ATTENTION.

Waste Disposal Method
This product is not classified or disposed of in accordance with federal or state regulations. Dispose of in a landfill in accordance with local regulations.

For additional information on personal protective equipment, first aid, and emergency procedures, refer to the product Material Safety Data Sheet (MSDS) on the job site or contact the company at the address or phone number given below.

Preparation 65
This product contains silica listed by the State of California as known to cause cancer, birth defects or other reproductive harm.

VOC Content
0.0 g/l or 0.0 lacquer less water and exempt solvents.

For medical emergencies only, call ChemTree (1-800-424-9300).
APPENDIX B – STRAIN GAUGE DETAILS AND INSTALLATION PROCEDURES

* THESE MATRIX RELATED DIMENSIONS ARE ±0.005
Strain Gage Installations for Concrete Structures

Introduction

The installation of strain gages for concrete structures presents several unique challenges to the installer, whether measurements are made on the concrete surface or within the concrete, or on reinforcement bars within the structure. For example, special preparation is required to ensure that strains on the irregular surface of the concrete are fully transmitted to the strain gage, and when gages are bonded to reinforcement bars, provisions are necessary to protect the installation from mechanical damage during fabrication and from the hostile environment of the concrete itself. This Application Note outlines recommendations for gage, leadwire, and protective coating selections and installations under these conditions. The surface preparation materials and installation accessories referenced throughout are described in detail in the Vishay Micro-Measurements Strain Gage Accessories Data Book.

Installing Strain Gages on Concrete and Other Irregular Surfaces

Strain gages can be satisfactorily bonded to almost any solid material— including concrete—if the surface is properly prepared. For smooth surfaces on nonporous materials, only the basic operations of solvent degreasing, abrading, application of layout lines, conditioning and neutralizing are required. For concrete and other materials with an uneven, rough and porous surface, an extra operation must be added to fill the voids and seal the surface with a suitable precoating before the gage is bonded.

Degreasing

Use a stiff-bristled brush and a mild detergent (Figure 1) to remove any loose soil or plant growth. Rinse with clean water. A degreaser such as CSM-2 may be needed if oils and greases are present. Remove surface irregularities with the wire brush, a disc sander, or grit blaster. Blow or brush all loose dust from the surface.

Conditioning

Gently apply M-Prep Conditioner A, a mildly acidic solution, to the surface in and around the gaging area. Scrub with a stiff-bristled brush. Hot contaminated Conditioner A with gauze sponges. Rinse the area thoroughly with clean water. Reduce the surface acidity by scrubbing with M-Prep Neutralizer 5A. Blot with gauze sponges and rinse with water. Dry the surface thoroughly. Warming the surface gently with a propane torch or heat gun will hasten evaporation.

Filling

Application of a 100%-solids adhesive to the gaging area (Figure 2) will provide a suitable gage-bonding surface. For test temperatures up to +200°F (+93°C), M-Bond AE-10 is normally used. At higher temperatures, M-Bond GA-61 is recommended. In applying the adhesive as a sealer to the surface, work the adhesive into any voids, and level to form a smooth surface. After the adhesive is cured, it should be abraded with 120-grit abrasive paper until the base material is exposed. (If a thin adhesive, like M-Bond 200, will be used to bond the gage, the base material should not be exposed.)

Layout Lines

Using a ballpoint pen or round-pointed metal rod, burnish layout lines. Scrub them with Conditioner A, apply Neutralizer 5A, and dry as before. Supplemental layout lines may be drawn with ink on the concrete outside the gaging area.

Gage Bonding

Normal procedures should be followed for bonding the gage to the prepared gaging surface. Special notice should be paid to several points, however. First, the gage length of strain gages used on concrete should be at least 5 times...
Strain Gage Installations for Concrete Structures

the diameter of the largest aggregate in the concrete. This often results in the use of patterns with gage lengths of 1 in (25 mm) or more. N2A-Series or encapsulated EA-Series gages, which tend to be flatter during handling, are highly recommended for their ease of installation under these circumstances. Further, bonding with a quick-curing adhesive, like M-Bond 200, is not recommended, even when test conditions may warrant its use. Accurate gage alignment and an even application of pressure as the adhesive is cured are more difficult when bonding longer gages. A slower curing adhesive, like M-Bond AE-10 shown in Figures 3 and 4, will allow time for realigning the gage, if necessary. It will also enable the use of a suitable pressure pad and clamping fixture as outlined in Vishay Micro-Measurements Application Note TT-610.

Soldering
Concrete and adhesive fillers are relatively poor heat conductors. Accordingly, care should be taken when soldering leads directly to the strain gage. Excessive heating of the tabs can be eliminated by using gages with Option W (integral printed circuit terminal), or Option P (preattached leadwires), which is shown in Figure 5.

Attention to these procedures will help ensure successful installations of strain gages on the surface of concrete and other similar solids. If you have any questions about your particular application, contact our Applications Engineering Department for recommendations.

Strain Measurement Within Concrete Structures

Vishay Micro-Measurements EGF-Series Embedment Strain Gages (Figure 6) are specially designed for measurement of mechanical strains within concrete structures. The sensing grid has an active gage length of 4 in (100 mm) to average strains in aggregate materials, and is fully encapsulated in a polymer concrete material to closely match the mechanical properties of typical structural concrete, guard against mechanical damage, and to protect against moisture and corrosive attack. EGF-Series Gages incorporate a 10-ft (3-m), jacketed, three-conductor cable for ease of use in field installations, and are compatible with conventional strain measurement instrumentation.

Gage Installation
No preparation of the gage itself is required; however, as with bonded or welded strain gages, EGF-Series Gages must be accurately aligned along the intended strain measurement direction during the installation process. Care should be taken to secure the gage in the desired location and orientation, and to tie the leadwire cable to any available support, before the concrete is poured. While the Embedment Gage must be completely encapsulated.
Strain Gage Installations for Concrete Structures

in concrete to ensure complete strain transfer from the structure, normal pouring techniques are usually all that are required.

Cable Splices

EGP-Series Gages are provided with a 10-ft (3-m) cable to allow for making cable splices outside the concrete structure. When splices are required, all connections should be soldered, and they should be protected from moisture and other contamination with a suitable cable splice sealant.

Strain Gage Installation

On Concrete Reinforcement

Strain gage installation on reinforcing rods follows the same general procedure recommended for most steel components. These procedures are, however, subjected to mechanical abrasion and a moist, corrosive environment. Accordingly, the following special attention is required:

Surface Preparation

1. degrease with a degreaser (CSM-2) over at least a 6-in (150-mm) length of the bar at the proposed gage location.

2. descale and smooth the rebars around its circumference with a grinder wheel. (Aluminum oxide or silicon carbide abrasive of approximately 50 mesh is preferred.) A 3-in (75-mm) length generally provides a sufficiently large descaled area for gage and protective coating installations. Surface finish after this operation should be about 180 microinches (5 μm) rms.

3. Wet abrade with Conditioner A and 220-grit silicon carbide wet-on-dry paper (SCP-1). Use sufficient Conditioner A to prevent material from drying on the rebars surface while abrading.

4. Wipe dry with a clean gauze sponge (GSP-1), then repeat Step 3 with 320-grit paper and dry again.

5. Surface finish should be 63 to 125 microinches (16 to 32 μm) rms at the completion of the second wet-abrading operation.

6. Lay out the gage locations.

7. Scrub the installation area with Conditioner A and a cotton applicator (CSP-1). Wipe dry with gauze sponges.

8. Scrub the area thoroughly with Neutralizer 5A and a cotton applicator and wipe dry with a gauze sponge as previously noted. This step must be accomplished thoroughly to neutralize all traces of Conditioner A used in Steps 3 through 7.

9. Mask an area with PCT-2M gage installation tape (or M10-2 M-Inch) tape at the gage location to minimize flow-out of adhesive for subsequent protective coating application.

Adhesive Selection

M-Bond AE-18 adhesive is a good selection when a room-temperature cure of a field application is required. This adhesive will cure in 6 hours at +75°F (+24°C). Other adhesives that may be used, depending upon the test environment, are: M-Bond AE-15, M-Bond 600/800, or GA-6 adhesive. Application of the adhesive should follow the specific instructions accompanying it.

Gage Selection

CEA-Series gages are the most popular choice when the cross section of the bar is 3/8 in (3mm) or larger in diameter. Where very stable installations are required (e.g., for tests in excess of one year) on 1/4 in (6 mm) or smaller diameter rebar rod, WK-Series gages are recommended. When conditions are not favorable for bonding gages, CEA- and LWK-Series Weldable Gages (Figure 7) may be used.

Leadwire Considerations

When utilizing one active strain gage (quarter-bridge configuration), it is good practice to use a three-leadwire system. Vishay Micro-Measurements EA- and CEA-
Strain Gage Installations for Concrete Structures

Series strain gages can be supplied with a preattached three-leadwire cable (Options P and P3, respectively) to eliminate the need for attaching leadwires at the job site and to reduce installation time.

Alternately, leadwires may be soldered to the strain gage tabs after gage bonding. If a parallel or twisted cable is used, separate the individual (leadwire) conductors for a distance of about 1 in (25 mm) from the cable end and, if Teflon®-insulated cable is used, etch the insulation with Tetra-Etch compound; if vinyl-insulated cable is used, prime the insulation with thinned M-Coat B. These materials should not be allowed to flow onto the bare strands of the conductors.

After allowing the M-Coat B to air dry for at least two hours at room temperature (about +75°F (+24°C)), thermally strip the leadwire ends and tin and solder the wires to the strain gage tabs. For most rebar installations, 361A-20K solder will give excellent results. Carefully remove all rosin flux from the soldered connections using rosin solvent (RSK®) before applying the protective coating.

Environmental Protection

Apply M-Coat 3 to the gage installation carefully following the procedures outlined in Vishay Micro-Measurements Instruction Bulletin B-147. The coating should be built up to provide approximately 1/4 in (6 mm) thickness completely surrounding the rebar (Figure 8) at the gage location, and should be carried back far enough to cover the leadwire area previously primed with M-Coat B. Allow this coating to cure 24 hours at +75°F (+24°C), or 4 hours at +125°F (+40°C).

As a final step, the instrumentation leads extending from the gage, out through the concrete, should be placed in conduit to prevent mechanical damage to the leadwire system. Of course, the complete installations should be thoroughly checked with a Model 1300 Gage Installation Tester before and after the concrete is poured. A properly installed and protected strain gage is capable of many years of service on embedded reinforcing bars, providing data about load effects throughout the life of a concrete structure — from initial construction forces to unexpected severe loading conditions.
M-Coat F
Vishay Micro-Measurements

Protective Coating

FEATURES
- Excellent for outdoor applications
- No cure required
- Versatile

DESCRIPTION
Kit of selected materials easily applied in various combinations. Provides environmental and mechanical protection. Particularly well suited to field applications where conditions are not ideal. Typical applications include pipelines, tunnels, bridges, reinforcement bars in concrete structures, heavy machinery, ships, aircraft, motor vehicles, and pressure vessels.

CHARACTERISTICS
Cure Requirements:
No mixing or curing required.

Operating Temperature Range:
Short Term: -70°F to +200°F [-50°C to +100°C]  
Long Term: -20°F to +175°F [-30°C to +80°C]

Shelf Life:
1 year at 45°F [+24°C]

PACKAGING OPTIONS
Kit:
12 pieces (4-1/2in x 3-3/4in x 1/8in [115.9 x 95.3 x 3.2mm]) each
- M-Coat FB Butyl Rubber Sealant
- M-Coat FN Neoprene Rubber Sheets
1 roll (0.003 in x 2 in x 20 ft [0.008 mm x 50 mm x 6m])
M-Coat FA Aluminum Foil Tape
2 brush cap bottles (1 oz [30 ml] ea)
- M-Coat B Air-Drying Nitrile Rubber Coating
- M-Coat FT Teflon® Tape

Bulk:
- M-Coat FB-2 Butyl Rubber Sealant — 25 pieces
- M-Coat FN-2 Neoprene Rubber Sheets — 25 pieces
- M-Coat FA-2 Aluminum Foil Tape — 20 ft [6m] roll
- M-Coat FT Teflon® Tape
1 x 20 x 0.003 in [50 x 0.008 mm] — 10 pieces

Teflon is a Registered Trademark of DuPont.
APPENDIX C – POST-TENSION FLEXURE TEST STRAIN GAUGE DATA

Figure 65. Post-tension, flexure, east panel, top mat, position #1.

Figure 66. Post-tension, flexure, east panel, top mat, position #2.
Figure 67. Post-tension, flexure, east panel, top mat, position #3.

Figure 68. Post-tension, flexure, east panel, top mat, position #4.
Figure 69. Post-tension, flexure, east panel, top mat, position #5.

Figure 70. Post-tension, flexure, east panel, top mat, position #6.
Figure 71. Post-tension, flexure, east panel, top mat, position #7.

Figure 72. Post-tension, flexure, east panel, top mat, position #8.
Figure 73. Post-tension, flexure, east panel, top mat, position #9.

Figure 74. Post-tension, flexure, east panel, top mat, position #10.
Figure 75. Post-tension, flexure, east panel, bottom mat, position #1.

Figure 76. Post-tension, flexure, east panel, bottom mat, position #2.
Figure 77. Post-tension, flexure, east panel, bottom mat, position #3.

Figure 78. Post-tension, flexure, east panel, bottom mat, position #4.
Figure 79. Post-tension, flexure, east panel, bottom mat, position #5.

Figure 80. Post-tension, flexure, east panel, bottom mat, position #6.
Figure 81. Post-tension, flexure, east panel, bottom mat, position #7.

Figure 82. Post-tension, flexure, east panel, bottom mat, position #8.
Figure 83. Post-tension, flexure, east panel, bottom mat, position #9.

Figure 84. Post-tension, flexure, east panel, bottom mat, position #10.
Figure 85. Post-tension, flexure, west panel, top mat, position #1.

Figure 86. Post-tension, flexure, west panel, top mat, position #2.
Figure 87. Post-tension, Flexure, west panel, top mat, position #3.

Figure 88. Post-tension, Flexure, west panel, top mat, position #4.
Figure 89. Post-tension, Flexure, west panel, top mat, position #5.

Figure 90. Post-tension, Flexure, west panel, top mat, position #7.
Figure 91. Post-tension, Flexure, west panel, top mat, position #8.

Figure 92. Post-tension, Flexure, west panel, top mat, position #9.
Figure 93. Post-tension, Flexure, west panel, top mat, position #10.

Figure 94. Post-tension, Flexure, west panel, bottom mat, position #1.
Figure 95. Post-tension, Flexure, west panel, bottom mat, position #2.

Figure 96. Post-tension, Flexure, west panel, bottom mat, position #3.
Figure 97. Post-tension, Flexure, west panel, bottom mat, position #4.

Figure 98. Post-tension, Flexure, west panel, bottom mat, position #5.
Figure 99. Post-tension, Flexure, west panel, bottom mat, position #6.

Figure 100. Post-tension, Flexure, west panel, bottom mat, position #7.
Figure 101. Post-tension, Flexure, west panel, bottom mat, position #8.

Figure 102. Post-tension, Flexure, west panel, bottom mat, position #9.
Figure 103. Post-tension, Flexure, west panel, bottom mat, position #10.
APPENDIX D – CURVED BOLT FLEXURE TEST STRAIN GAUGE DATA

Figure 104. Curved bolt, flexure, east panel, top mat, position #1.

Figure 105. Curved bolt, flexure, east panel, top mat, position #2.
Figure 106. Curved bolt, flexure, east panel, top mat, position #3.

Figure 107. Curved bolt, flexure, east panel, top mat, position #4.
Figure 108. Curved bolt, flexure, east panel, top mat, position #5.

Figure 109. Curved bolt, flexure, east panel, top mat, position #6.
Figure 110. Curved bolt, flexure, east panel, top mat, position #7.

Figure 111. Curved bolt, flexure, east panel, top mat, position #8.
Figure 112. Curved bolt, flexure, east panel, top mat, position #9.

Figure 113. Curved bolt, flexure, east panel, top mat, position #10.
Figure 114. Curved bolt, flexure, east panel, bottom mat, position #1.

Figure 115. Curved bolt, flexure, east panel, bottom mat, position #2.
Figure 116. Curved bolt, flexure, east panel, bottom mat, position #3.

Figure 117. Curved bolt, flexure, east panel, bottom mat, position #4.
Figure 118. Curved bolt, flexure, east panel, bottom mat, position #5.

Figure 119. Curved bolt, flexure, east panel, bottom mat, position #6.
Figure 120. Curved bolt, flexure, east panel, bottom mat, position #7.

Figure 121. Curved bolt, flexure, east panel, bottom mat, position #8.
Figure 122. Curved bolt, flexure, east panel, bottom mat, position #9.

Figure 123. Curved bolt, flexure, east panel, bottom mat, position #10.
Figure 124. Curved bolt, flexure, west panel, top mat, position #1.

Figure 125. Curved bolt, flexure, west panel, top mat, position #2.
Figure 126. Curved bolt, flexure, west panel, top mat, position #3.

Figure 127. Curved bolt, flexure, west panel, top mat, position #4.
Figure 128. Curved bolt, flexure, west panel, top mat, position #5.

Figure 129. Curved bolt, flexure, west panel, top mat, position #6.
Figure 130. Curved bolt, flexure, west panel, top mat, position #7.

Figure 131. Curved bolt, flexure, west panel, top mat, position #8.
Figure 132. Curved bolt, flexure, west panel, top mat, position #9.

Figure 133. Curved bolt, flexure, west panel, top mat, position #10.
Figure 134. Curved bolt, flexure, west panel, bottom mat, position #1.

Figure 135. Curved bolt, flexure, west panel, bottom mat, position #2.
Figure 136. Curved bolt, flexure, west panel, bottom mat, position #3.

Figure 137. Curved bolt, flexure, west panel, bottom mat, position #4.
Figure 138. Curved bolt, flexure, west panel, bottom mat, position #5.

Figure 139. Curved bolt, flexure, west panel, bottom mat, position #6.
Figure 140. Curved bolt, flexure, west panel, bottom mat, position #7.

Figure 141. Curved bolt, flexure, west panel, bottom mat, position #8.
Figure 142. Curved bolt, flexure, west panel, bottom mat, position #9.

Figure 143. Curved bolt, flexure, west panel, bottom mat, position #10.
APPENDIX E – POST-TENSION SHEAR TEST STRAIN GAUGE DATA

Figure 144. Post-tension, shear, south panel, top mat, position #1.

Figure 145. Post-tension, shear, south panel, top mat, position #2.
Figure 146. Post-tension, shear, south panel, top mat, position #3.

Figure 147. Post-tension, shear, south panel, top mat, position #4.
Figure 148. Post-tension, shear, south panel, top mat, position #5.

Figure 149. Post-tension, shear, south panel, top mat, position #6.
Figure 150. Post-tension, shear, south panel, bottom mat, position #1.

Figure 151. Post-tension, shear, south panel, bottom mat, position #2.
Figure 152. Post-tension, shear, south panel, bottom mat, position #3.

Figure 153. Post-tension, shear, south panel, bottom mat, position #4.
Figure 154. Post-tension, shear, south panel, bottom mat, position #5.

Figure 155. Post-tension, shear, south panel, bottom mat, position #6.
Figure 156. Post-tension, shear, north panel, top mat, position #1.

Figure 157. Post-tension, shear, north panel, top mat, position #2.
Figure 158. Post-tension, shear, north panel, top mat, position #3.

Figure 159. Post-tension, shear, north panel, top mat, position #4.
Figure 160. Post-tension, shear, north panel, top mat, position #5.

Figure 161. Post-tension, shear, north panel, top mat, position #6.
Figure 162. Post-tension, shear, north panel, bottom mat, position #1.

Figure 163. Post-tension, shear, north panel, bottom mat, position #2.
Figure 164. Post-tension, shear, north panel, bottom mat, position #3.

Figure 165. Post-tension, shear, north panel, bottom mat, position #4.
Figure 166. Post-tension, shear, north panel, bottom mat, position #5.
APPENDIX F – CURVED BOLT SHEAR TEST STRAIN GAUGE DATA

Figure 167. Curved bolt, shear, south panel, top mat, position #1.

Figure 168. Curved bolt, shear, south panel, top mat, position #2.
Figure 169. Curved bolt, shear, south panel, top mat, position #3.

Figure 170. Curved bolt, shear, south panel, top mat, position #4.
Figure 171. Curved bolt, shear, south panel, top mat, position #5.

Figure 172. Curved bolt, shear, south panel, bottom mat, position #1.
Figure 173. Curved bolt, shear, south panel, bottom mat, position #2.

Figure 174. Curved bolt, shear, south panel, bottom mat, position #3.
Figure 175. Curved bolt, shear, south panel, bottom mat, position #4.

Figure 176. Curved bolt, shear, south panel, bottom mat, position #5.
Figure 177. Curved bolt, shear, south panel, bottom mat, position #6.

Figure 178. Curved bolt, shear, north panel, top mat, position #1.
Figure 179. Curved bolt, shear, north panel, top mat, position #2.

Figure 180. Curved bolt, shear, north panel, top mat, position #3.
Figure 181. Curved bolt, shear, north panel, top mat, position #4.

Figure 182. Curved bolt, shear, north panel, top mat, position #5.
Figure 183. Curved bolt, shear, north panel, top mat, position #6.

Figure 184. Curved bolt, shear, north panel, bottom mat, position #1.
Figure 185. Curved bolt, shear, north panel, bottom mat, position #3.

Figure 186. Curved bolt, shear, north panel, bottom mat, position #4.
Figure 187. Curved bolt, shear, north panel, bottom mat, position #5.

Figure 188. Curved bolt, shear, north panel, bottom mat, position #6.