# U-BAR DETAILS WITH A SMALL RADIUS BEND FOR LONGITUDINAL AND TRANSVERSE CONNECTIONS

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## ABSTRACT

*With the majority of the bridges in our transportation infrastructure approaching their service lives, the need for accelerated bridge construction has become critical. Bridge construction using precast concrete components has been proven to reduce construction time and delay seen by the public. Therefore, the use of precast deck systems, such as deck bulb tees and full depth deck panels, will reduce construction time as well. In order for precast deck systems to be more widely used, design guidelines and specifications that produce full strength deck joints must be created.*

*The following paper presents the ongoing research being preformed on longitudinal and transverse joints for use in precast bridge deck systems. Three joint details are under investigation, two joint details consist of tightly bent U-bar reinforcement and the other detail consists of headed reinforcement. The joints will be subjected to simple static tests and the results will be used to create design guidelines and specifications for the development of full strength joints for use in precast bridge deck systems.*

**Keywords:** Precast Deck Systems, U-bar, Headed bar, Accelerated Bridge Construction

## Forward

The research presented in this paper has been performed under the ongoing National Highway Cooperative Research Program (NCHRP) 10-71 project, “Cast-in-Place Reinforced Concrete Connections for Precast Deck Systems”. Other research team members include R. Eriksson, C. Prussack, A. Schultz, S. Serguirant, and C. Shield. Ross Prestressed Concrete provided concrete and much needed assistance in pouring the specimens. Steve Abbott of Gerdau Ameristeel donated all conventional rebar used in the project and valuable manufacturing knowledge. Also, Gena Peters of Salit Specialty Rebar help acquire the stainless steel rebar used in the project and Engineered Wire Products donated all welded wire reinforcement. Publication of this paper does not necessarily indicate acceptance by the Academy, the Federal Highway Administration or by AASHTO.

## Introduction

The interstate system of the United States is one of our nation’s greatest achievements. It allows people and goods to be transported at anytime to any where in our nation. This system allows the American public access to fast and safe transportation at their convenience.

Due to the majority of the interstate system being constructed in the 1950’s, our interstate system is aging and is in need of repair and in some cases expansion. The majority of the bridges that were constructed during this boom of interstate construction were designed with an intended service life of fifty years. Therefore many of the bridges in our vital transportation system are approaching or have past their service lives and are in critical need of repair or replacement.

At the time when the interstate system was being constructed public inconvenience was of little concern in the construction process because new roads and bridges were being constructed. Now as the interstate system is being repaired and renovated, public inconvenience must be taken into account because the American public has grown accustomed to using the interstate system. The delay caused to the public by renovation and repair must be minimized to better serve those who use it. The engineering and construction workforces are now left with the task of finding rapid, practical, and cost effective ways to update and expand our interstate system and the bridges that connect it.

One of the major problem areas in bridge repair and renovation is the deck. Bridge decks are directly exposed to loading, the harmful effects of weather and the corrosive properties of deicing salts, so this portion of a bridge deteriorates faster then other portions of the same bridge. The deck has traditionally been constructed using cast in place methods which require time to form, place reinforcement, pour concrete, and cure the concrete. The previously mentioned process for the cast in place construction of bridge decks is a very time consuming process. The time required for deck construction needs to be reduced, so the delay and inconvenience seen by the public will decrease as well.

Precast deck panels have been used to decrease construction time. Also prestressed precast decked bulb tees have also been used so the construction time of the entire bridge superstructure can be drastically reduced. One common disadvantage in the use of both of these rapid construction methods is the lack of a continuous joint detail that can transmit both shear forces and moments. The joints previously used for precast bridge deck panels and prestressed precast decked bulb tees have been able to only transmit shear forces or they have been post-tensioned, which is also a time consuming process. Therefore, the need for continuous joint details in precast bridge decks is obvious. Continuous joint details will lead to better joints and a wider use of precast deck technologies to accelerate construction.

The use of full depth precast panels and deck bulb tees are one of the major areas of interest in accelerated bridge construction. One of the hurtles that must be overcome in order to use this technology is the lack of design guidelines and specifications for the development of longitudinal and transverse panel connections. These guidelines and specifications must produce a deck system that behaves similar to traditional cast in place designs and allows for rapid construction of the deck. The connections must be capable of transferring both shear and moment without the use of a post tensioning system and using the smallest joint width possible. The ability to use a precast decking system with small connection widths and no post tensioning will greatly increase the speed of bridge construction.

Connection details similar to those described above have already been developed and are used in practice in Japan and France (Ralls et al 2005). Both countries use connection details that do not use post tensioning and that produces a deck system that is durable, low maintenance, and behaves similar to that of a cast in place deck. The connection detail that is utilized in France uses a cast in place joint that is reinforced with looped bars projecting from full depth precast panels. Additional reinforcement is then threaded through the loop bars to complete the reinforcement installation. The Japanese use a similar system for both longitudinal connections between the top flanges of box girders and transverse connections in full depth precast panels. Connections similar to those used in France and Japan have also been used in the United States without the presence of design guidelines and specifications. The replacement of the 145$^{th}$ street Bridge in Manhattan, New York and the replacement of the Cooper River Bridges in South Carolina both used looped bar details for the connection between precast deck panels. The purpose of the following research is to test similar connection details, which would enable the development of design guidelines and specifications for transverse and longitudinal connections that could be used for full depth panel connections and connections between the top flanges of decked bulb tees.

Longitudinal (parallel to the direction of traffic) connections and transverse (perpendicular to the direction of traffic) connections were developed for the experimental program. Two connection details were developed for each connection type. A looped bar or u-bar connection detail, similar to those used in Japan and France was developed as well as a headed bar connection detail (Li et al 2008). Both details would produce shorter development lengths when compared to straight bars and thus would reduce joint widths, which would in turn reduce construction time. The u-bar detail will be tested using two different materials, stainless steel and welded wire reinforcement. The headed bar detail will only tested using conventional reinforcement.

The u-bar detail was designed to utilize an extremely tight bend. The inside bend diameter that was used in the u-bar design was three times the diameter of the bar

($3d_b$).The use of this extremely tight bend produced thinner deck sections that would reduce the weight of the deck and also aid in constructability. Although the tight bend of the u-bar would produce thinner decks it would also have to be designed using materials other then conventional rebar. This is due to the fact that the bend radius for conventional reinforcement is limited. According to ACI 318-05 (ACI 2005), the minimum bend diameter that can be used for the u-bar is six times the diameter of the bar ($6d_b$) and the minimum inside diameter of bend for a stirrup, which is made of a bar size of #5 or smaller, shall be four times the diameter of the bar ($4d_b$). Using the information provided the fact that the U-bar detail violates the minimum bend diameters required by ACI 318-05; Article 7.2 can be clearly seen.

**Specimen Design**

As stated previously, two joint types were investigated. The first connection type that was designed was the longitudinal (parallel to the direction of traffic) connection type. This type of joint would be used for the connection of adjacent decked bulb tees and full depth deck panels. The second type of connection that was designed was the transverse (perpendicular to the direction of traffic) connection type. This type of connection detail would be used to connect full depth deck panels.

The bridge cross section and longitudinal section that were used for the deck design and thus the connection design were taken from two design examples that were found in the PCI bridge design manual. The results from the deck design were used to obtain an accurate estimate of the amount of steel needed in a bridge deck. Although a complete deck design was performed for the interior region of the deck and the negative moment region over an interior pier, the results obtained were not used absolutely. Differing amounts of reinforcing were obtained for the different connection details, so in order to obtain a specimen design for each joint type that could be easily compared to its competitors, adjustments were made to the amount of reinforcing steel used in each specimen and joint type.

Two PCI design examples were used to get a sense of the amount of reinforcement that would be needed in the deck between interior girders (longitudinal connection) and over an interior pier for a continuous span bridge system (transverse connection). The design example found in Section 9.8 of the PCI bridge design manual (BDM 1997) was used to design the deck between interior girders and Section 9.6 of the PCI bridge design manual was used to design the deck over interior piers in a continuous span bridge system. Both examples used the same bridge cross section and longitudinal section. The bridge cross section consisted of 4 BT-72 girders that were spaced at 12’ on center. The longitudinal section of the bridge consisted of a center span of 120’ and two side spans of 110’

An 8” thick deck was first designed for the interior portion of the deck. This design was carried out to obtain an estimate of the amount of steel used in decks with a conventional thickness. Due to the investigation of both longitudinal and transverse connection details two other deck thicknesses were also investigated. A 6.25” thick deck was designed for use in the development of the longitudinal connection details and a 7.25” thick deck was design for use in the development of the transverse connection details. The larger thickness of the deck used for the development of the transverse joint was due to the connection details used and the additional room needed to keep the transverse rebar outside of the longitudinal rebar. The transverse rebar in a deck are the main flexural reinforcement, so in order to obtain the most efficient design, the transverse rebar were given the largest moment arms possible without violating cover requirements. The thicknesses that were used were the smallest thicknesses possible, so the deck would be light weight. This was a concern of constructability and efficient material use.

The approximate strip method was the method used for the design of all thicknesses. Although the approximate strip method has been known to deliver overly conservative designs, it was the only method that could be used for design in this case. The empirical method was not used for design, because one of the design requirements states that the deck must be cast in place, which is not the case in this situation.

The first deck design that was completed was the design of the interior portion of deck with an 8" thickness. Two different yield strengths were used in the design, because both conventional rebar with yield strength of 60 ksi as well as stainless steel and welded wire reinforcement with a yield strength of 75 ksi, will be used in the specimens. Two different values of the exposure factor were also used in the design to show were the service limit state controlled the design. The values that were used for the exposure factor were the values corresponding to exposure classes one and two, which are 1 and .75, respectively. The exposure factor is directly related to allowable crack widths. So, the larger the value of exposure factor the larger the crack widths that will be seen at service loads. The following tables show the 8" thick deck design using steel yield strengths of 60 ksi and 75 ksi as well as changing the exposure factor ( $\gamma_e$ ) .

Table 1: Reinforcement Required for 8" Thick Deck

| | | fy = 60 ksi | | | | fy = 75 ksi | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | γe = .75 | | γe = 1 | | γe = .75 | | γe = 1 | |
| | | Bar Size | Spacing (in) | Bar Size | Spacing (in) | Bar Size | Spacing (in) | Bar Size | Spacing (in) |
| M+ (Bottom) | Transverse | # 5 | **8** | # 5 | **8** | # 5 | **8** | # 5 | **9.5** |
| | Longitudinal | # 5 | **11.5** | # 5 | **11.5** | # 5 | **11.5** | # 5 | **11.5** |
| M- (Top) | Transverse | # 5 | **7** | # 5 | **8.5** | # 5 | **7** | # 5 | **8.5** |
| | Longitudinal | # 4 | **18** | # 4 | **18** | # 4 | **18** | # 4 | **18** |

From Table 1, it can be seen that the service limit state controls the design for both the top and bottom transverse reinforcement when using a yield strength of 75 ksi and only controls the design for the top layer of transverse reinforcement when using a yield strength of 60 ksi. This can be observed because as the exposure class is changed the allowable spacing increases.

The interior portion of deck was then design for a thickness of 6.25". The results of this design were used to develop the specimen reinforcement and the longitudinal connection detail. The 6.25" deck design was completed for yield strengths of 60 ksi and 75 ksi. Also, exposure classes one and two were used in the design to show were the service limit state controlled the design. Again, 6.25" was used as the deck thickness to reduce the weight of the deck. Tables 2 and 3 show the results of the design for both the u-bar detail and the headed bar detail in all the previously described cases.

Table 2: Reinforcement Required for u-bar Details in a 6.25” Thick Deck

| | | fy = 75 ksi | | | |
|---|---|---|---|---|---|
| | | γe = .75 | | γe = 1 | |
| | | Bars Size | Spacing (in) | Bars Size | Spacing (in) |
| M+ (Bottom) | Transverse | # 5 | **4.5** | # 5 | **4.5** |
| | Longitudinal | # 5 | **6.5** | # 5 | **6.5** |
| M- (Top) | Transverse | # 5 | **4** | # 5 | **4.5** |
| | Longitudinal | # 4 | **12** | # 4 | **12** |

Table 3: Reinforcement Required for Headed Bar Detail in a 6.25” Thick Deck

| | | fy = 60 ksi | | | |
|---|---|---|---|---|---|
| | | γe = .75 | | γe = 1 | |
| | | Bars Size | Spacing (in) | Bars Size | Spacing (in) |
| M+ (Bottom) | Transverse | # 5 | **4** | # 5 | **4** |
| | Longitudinal | # 5 | **5.5** | # 5 | **5.5** |
| M- (Top) | Transverse | # 5 | **4** | # 5 | **4** |
| | Longitudinal | # 4 | **12** | # 4 | **12** |

Tables 2 and 3 show that the service limit state had very little effect on the design of the 6.25” thick deck. The only case that the service limit state was seen to govern was in the top layer of transverse reinforcement when the yield strength of 75 ksi was used. By comparing the required reinforcement tables provided for the 8” thick deck and the 6.25”, it can be seen that the required reinforcement drastically increases between the 8” and 6.25” thick deck designs. Although, more steel is required for the 6.25” thick deck, the decrease in weight provided the incentive to use the slimmer deck thickness even though a larger amount of reinforcing is required.

The next step that was completed was the design of the deck over an interior pier in a continuous span bridge system. If the girder and the deck acted compositely in this situation, the negative moment that would be experienced by the composite section would create tension in the deck. Therefore additional longitudinal reinforcement would be needed in that location. This design was carried out by using a deck thickness of 7.25” and using the bridge cross section provided in the design example found in Section 9.6 of the PCI bridge design manual. The negative moment that was used to calculate the required additional reinforcement was taken for the same design example. The value of the negative moment used was 4837.2 ft-kips. The composite section used for the reinforcement design is shown in Figure 1.

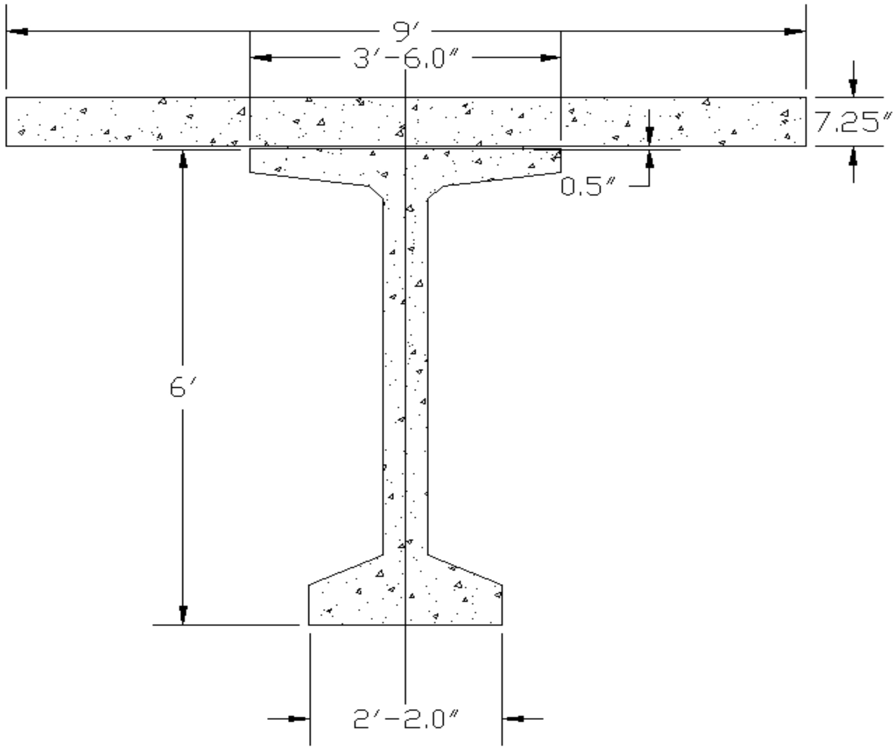



Figure 1: Section Used for Negative Moment Longitudinal Reinforcement Calculation

The amount of longitudinal deck reinforcement was determined by a conventional flexural design using the composite section shown above. The centriod of the reinforcing was assumed to be at mid height of the deck and the amount of reinforcing was required for both 60 ksi and 75 ksi yield strengths. Due to the use of the u-bar detail the spacing of the top and bottom layers of reinforcement had to be the same. Table 4 contains the results of the designs.

Table 4: Longitudinal Reinforcement

| Longitudinal Reinforcement ($Mu^-$ region) | | |
|---|---|---|
| fy (ksi) | rebar size | spacing (in) |
| 60 | #5 | 4.5 |
| 75 | #5 | 5.5 |

After considering the results from each type of deck design the specimens and connections were developed. The lap splice length of each joint detail (headed and u-bar) was set at 6”. This distance was measured between the bearing surfaces of adjacent headed bars in the case of the headed bar connection detail and measured between the inside bearing surfaces of adjacent u-bars. Lacer bars were added where possible for additional confining reinforcement. The bar spacings were developed based on the deck designs previously mentioned.

The specimen and connections designed to test the longitudinal joint (flexural test) were developed based on the spacings calculated during the interior deck design.

The longitudinal reinforcement in the specimen, which corresponds to transverse reinforcement in the bridge deck was decided to be a #5 bar at a 4.5” spacing. Although, a 4” spacing was required for the reinforcement with a yield strength of 60 ksi, the longitudinal spacing of 4.5” was used so that all the specimens in the first phase of testing would have the same spacing for the longitudinal reinforcing. The top layer of transverse reinforcement in the specimens, which corresponds to the top layer of longitudinal reinforcement in the deck, was decided to be a #4 bar at 12”, which was what was calculated during the design. The bottom layer of transverse reinforcement in the specimen, which corresponded to the top layer of longitudinal reinforcement in the bridge deck, was decided to be a #5 bar at a 6” spacing. The 6” spacing was the average between the spacing required for the 60 ksi and the 75 ksi design. Figures 2 and 3 show the specimen and connection designs that will be used for the flexural testing of the longitudinal joint details.

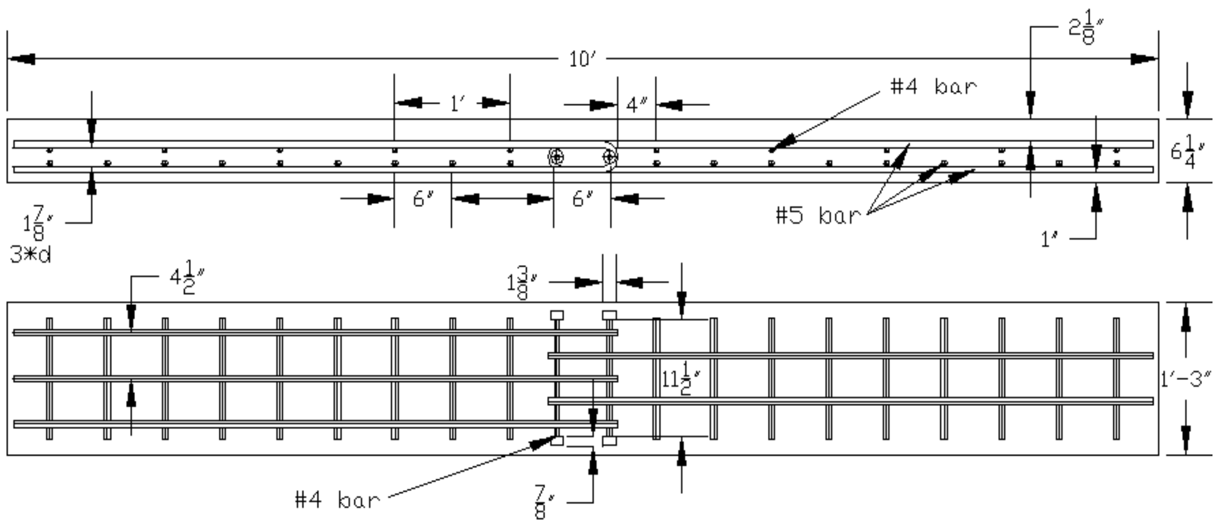



Figure 2: U-bar Longitudinal Joint Specimen

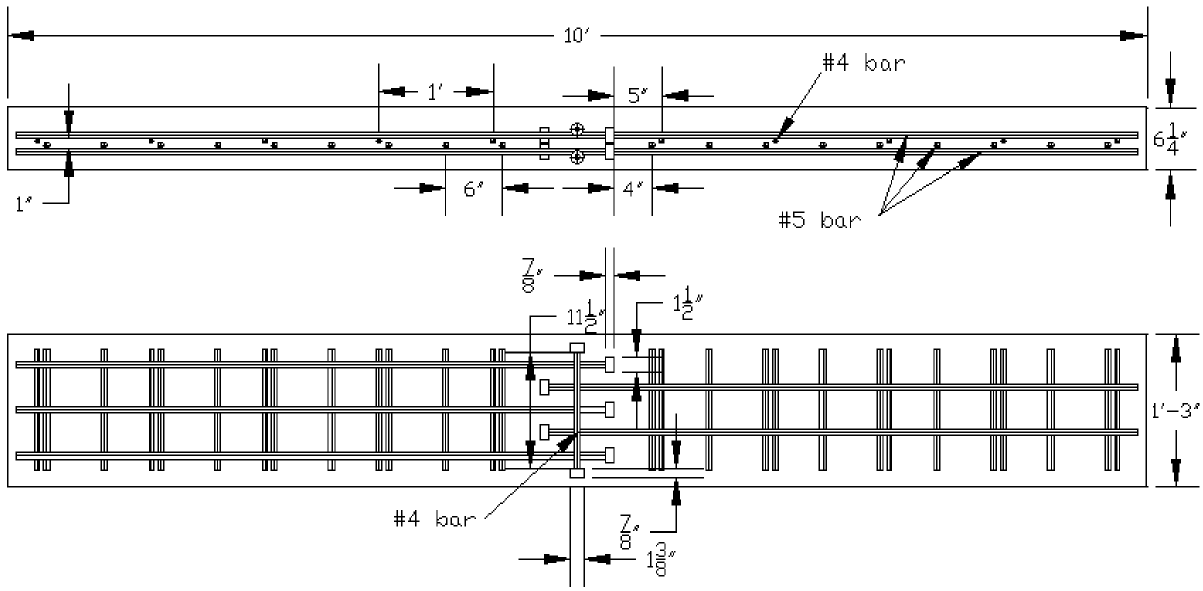



Figure 3: Headed bar Longitudinal Joint Specimen

The specimen and connections designed to test the transverse joint (tension test) were developed based on the design of the interior deck and also the design for the deck over an interior pier of a continuous bridge system. The top and bottom layers of longitudinal reinforcement in the specimen, which now corresponds to the longitudinal reinforcement of the bridge deck, were decided to consist of #5 bars spaced at 4.5”. This was the required spacing for 60 ksi yield strength steel in the deck over an interior pier. The top and bottom layers of transverse reinforcement, which correspond to transverse reinforcement in the bridge deck, were decided to be the same as the transverse reinforcement in the flexural specimens. Although, the transverse reinforcement in the flexural specimen corresponds to longitudinal reinforcement in the bridge deck and the opposite for the tension specimens, the same transverse specimen reinforcement was used in the both specimen types. The same transverse reinforcement will ease in construction of both specimen types. Figures 4 and 5 show the specimen and connection designs that will be used for the tension testing of the transverse joints details.

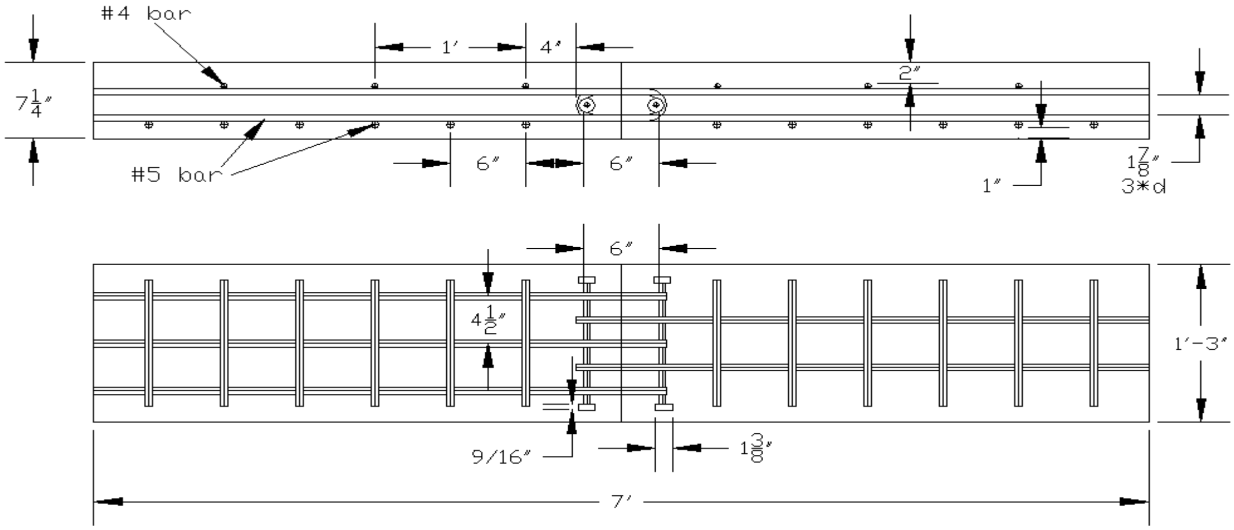



Figure 4: U-bar Transverse Joint Specimen

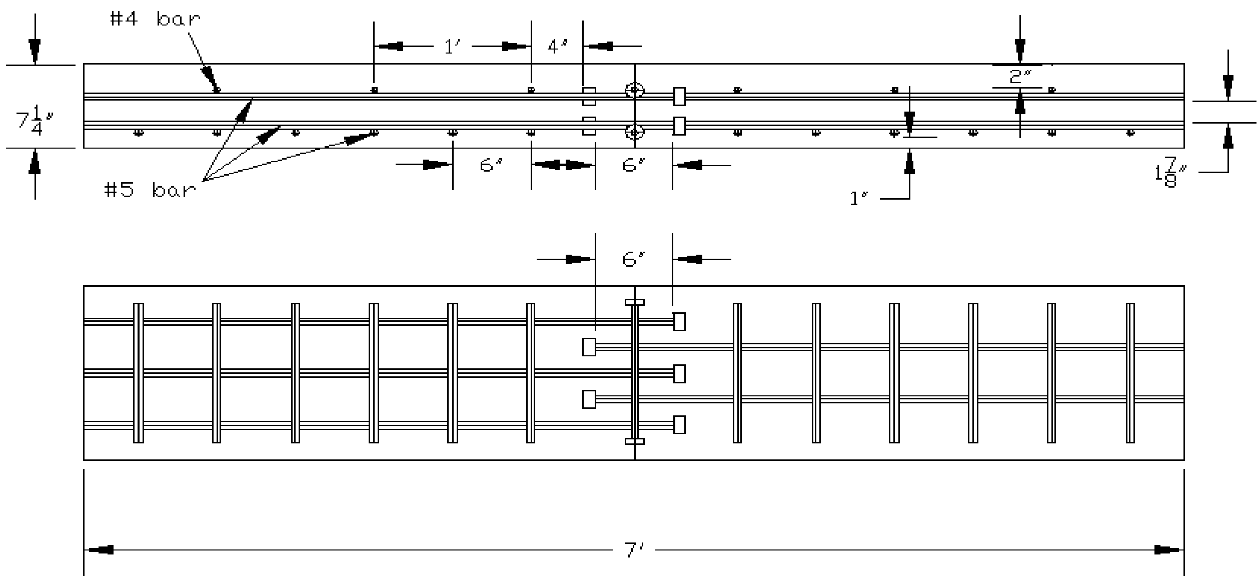



Figure 5: Headed bar Transverse Joint Specimen

**Experimental Setup**

Simple static tests will be performed for both the longitudinal connections and transverse connections. The connection details designed for use in longitudinal joints were tested using an altered version of the four point bending test as shown in Figure 8. The actuators that will be used for loading the specimens will be located on the ends of the specimens and the supports will be located inside the actuators. This experimental set up produces an upward deflection, which produces the tension region in the top of the specimen. The tension region being located in the top portion of the specimen produces cracks in the top of the specimens, which produces safer conditions for observing the cracking and crack propagation in the specimens. This test was designed to simulate the forces that would be experience by the connection if it was used to connect the top flanges of two adjacent deck bulbed tees, for example.

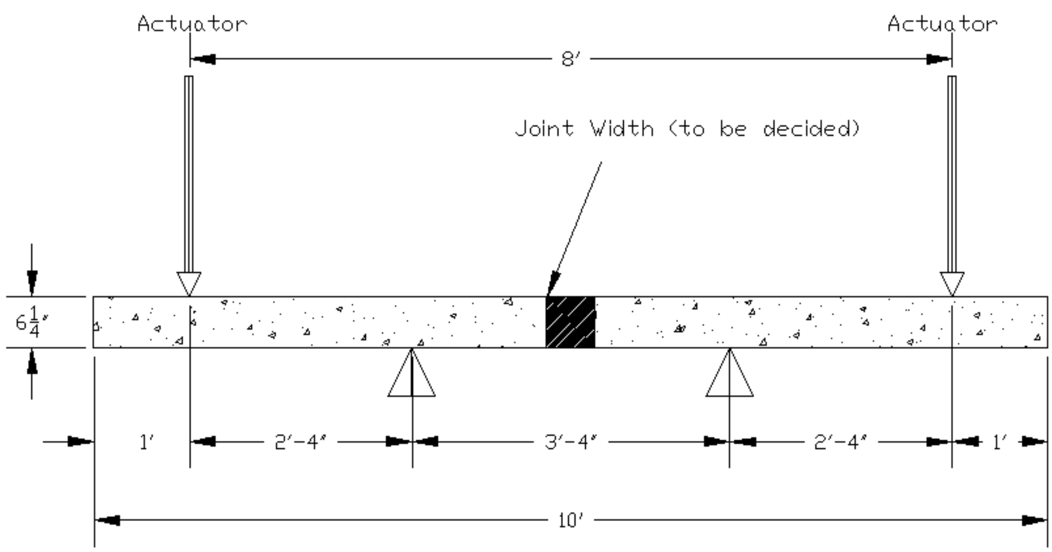
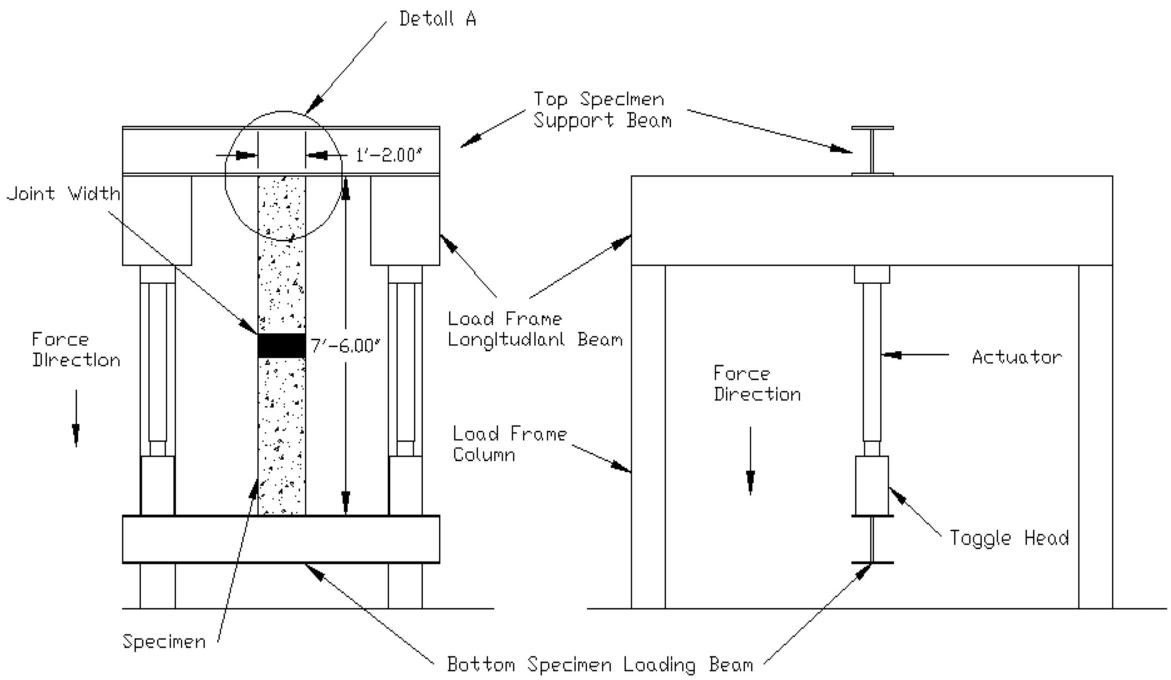



Figure 6: Testing Setup for Longitudinal Joint

The transverse joints will be tested in pure tension. These connections will be tested in pure tension, because the worst loading case that could be seen by a transverse joint would be produced in the negative moment region over an interior pier of a continuous span bridge system. In this case, if the deck was compositely connected to the girder, the entire deck would be in tension. This fact was the reason the transverse details will be tested in pure tension. Figure 7 illustrates the experimental set up used in the testing of the transverse connection details.




Detail A

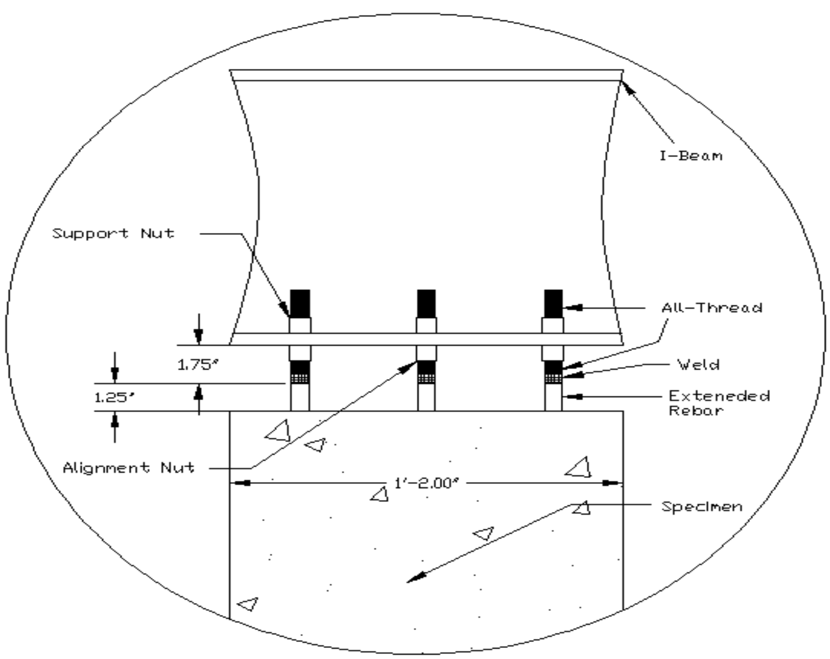



Figure 7: Testing Setup for Transverse Joint

The specimens will be attached to the support and loading beams by welding large pieces of all thread to the longitudinal rebar that was extended out of the concrete. The all thread will then be secured to the loading and support beams by nuts. This can be seen in detail A.

After the preliminary longitudinal and transverse connection details are tested, the connection details that show superior qualities for both connection directions will be tested further. Three additional specimens will be tested using the superior connection detail for both connection directions. The connections will be altered by changing the lap splice length, spacing and adding additional lacer bars. These alterations will be made in the joint detail to observe how changes in each criterion affect the behavior of the joint. The data gathered from the second phase of testing will enable guidelines and specifications to be made to help future design of such joints.

**Instrumentation**

To get a better understanding of the behavior of the u-bar and the headed bar joint details, strain gages were installed on the reinforcement in each joint. The use of strain gages gives direct readings of the strain experienced by the reinforcement in the joint, which can then be used to calculate the forces experienced by the reinforcement. The strain gage configuration in both types of joints was determined based on development length calculations for each type of joint detail. The development lengths that were calculated for each joint type were only used as a datum or reference point. The strain gages were installed around the calculated development length, so that the actual development length of that joint detail could be found.

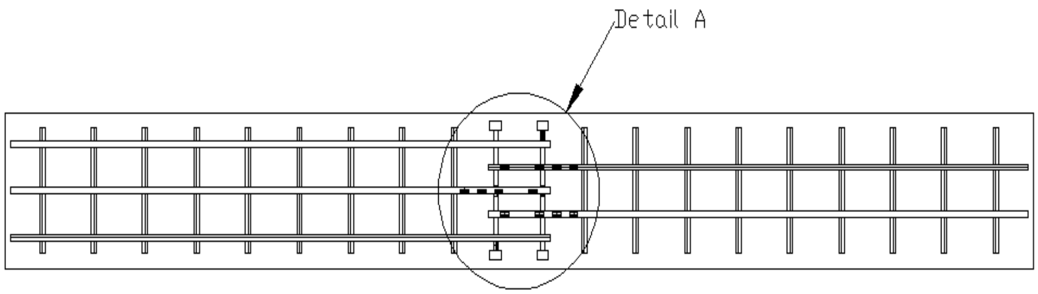
It was assumed that the development length for the u-bars can be calculated based on equations found in ACI 318-05 for hook bars. The development length equation for a standard hook in tension was used. The following equation shows the ACI development length equation for a standard hook in tension.

$$l_{dh} = \left[ \frac{.02\Psi_e \lambda f_y}{\sqrt{f'_c}} \right] \text{ (ACI 12.5.2)}$$

In using the previous equation $\psi_e$ and $\lambda$ were both set equal to one, because the rebar being used was not epoxy coated and the concrete being used was not lightweight concrete. The concrete compressive strength and the steel yield strength that were used in the calculation were 7 ksi and 75 ksi, respectively. Also, the development length modification factor of .7 was used, because the specimen met the bar cover perimeters of having not less then 2.5” of side cover and not less then 2” of cover beyond the extension of the bar. The development length of a standard hook bar in tension for this situation was calculated to be 7.84”.

As stated previously, the calculated development length was only used as a reference point, due to the fact that the equation that was used did not accurately represent the u-bar joint. The equation used represented the development length of an ACI standard hook in tension. The u-bars used in the specimen do not meet the dimensional requirements of an ACI standard hook. ACI states that the minimum inside diameter of a standard hook has to be greater then or equal to $6d_b$. The u-bars that were used in the specimens all had bends that were equal to or less then $3d_b$, which is clearly a deviation from the dimensional restrictions set forth in ACI 318-05. Also, even though the u-bars have a 180 degree bend like that of a 180 degree hook the u-bars used in the specimens were not hooks at all, due to both ends of the u-bars continuing into the specimen as compared to a 180 degree hook were one end of the reinforcing bar terminates shortly after the hook.

The strain gage configuration used, allowed for strain readings to be obtained at various distances along the u-bars within the specimens. Gages were installed on both sides of the u-bar at distances of 2”, 6”, 8” and 10” from the face of the bend. An additional strain gage was installed on the outside face in the center of the bend in all gauged u-bars. Figure 8 shows the u-bar specimen and a detail view of the joint region so the strain gage configuration can be clearly seen.




Detail A

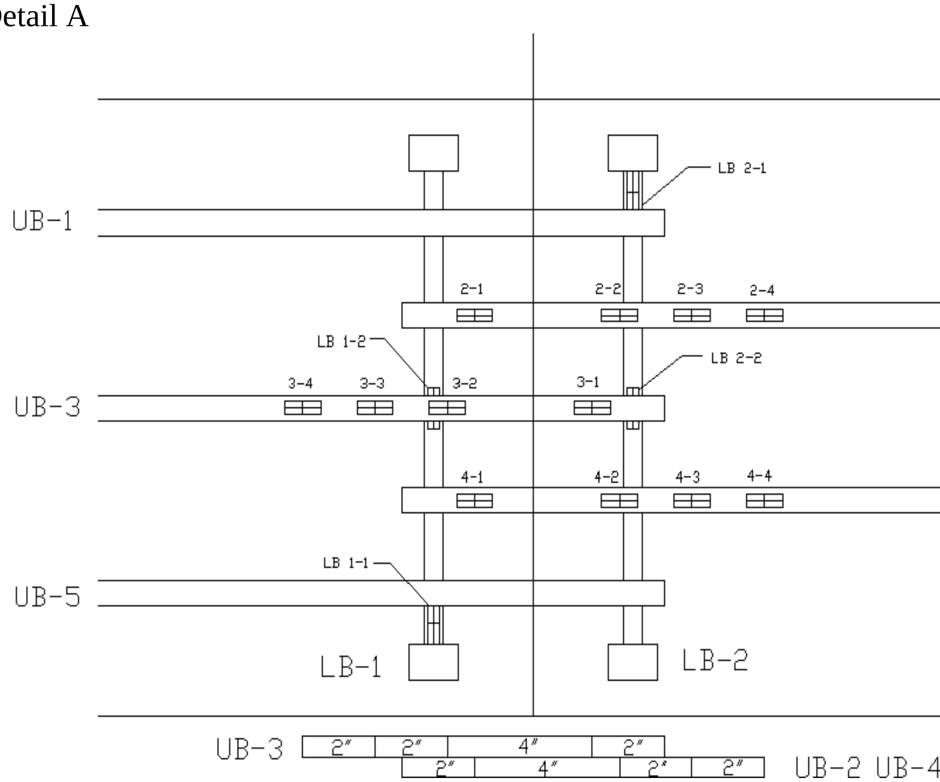


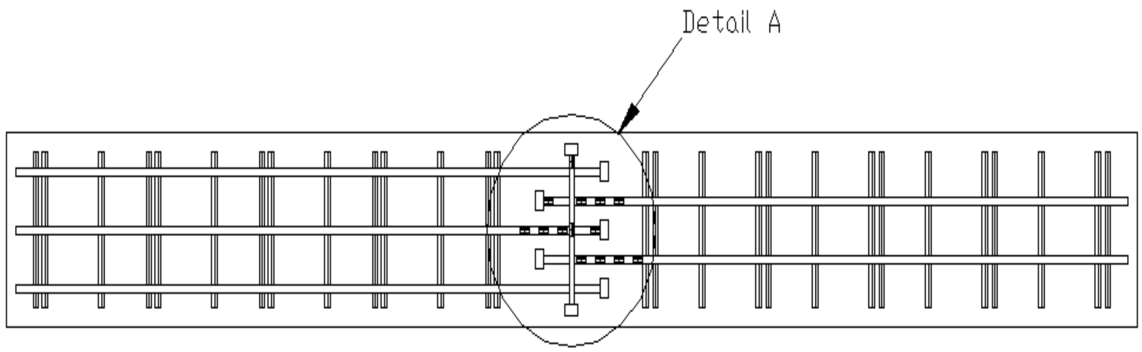

Figure 8: Strain Gage Configuration in u-bar Detail
(All distances are measured from the center of bend of the u-bar to the center of the corresponding strain gage)

Research from the University of Texas at Austin (UTA) (Thompson et al 2006) was used to determine the development length of the headed bars used in the specimens. This research discussed the anchorage length of headed reinforcement as well as the lap splice length of headed bar reinforcement. The anchorage length is the distance from the bearing surface of the head to the point of maximum bar stress, also known as the development length. The development of the reinforcement can be taken as the stress contributed by the mechanical anchorage of the head and the bond along the anchorage length to the point of maximum reinforcement stress. The lap splice length is the distance between the bearing surfaces of two adjacent spliced headed bars. The lap splice length was determined using the anchorage length and an appropriate strut and tie model, which used the recommended strut angle of 55 degrees.

The stress caused by the mechanical anchorage of the head was calculated separately for the top and the bottom layers of reinforcement, due to different cover dimensions. The stress calculated for the top and bottom layers of reinforcement were found to be 84.2 ksi and 67.8 ksi, which are both larger then the 60 ksi yield strength of the reinforcement. This indicates that the stress provided by the mechanical anchorage of the head is sufficient to yield the reinforcement. The minimum anchorage length, recommended to be “$6d_b$” by UTA, was used to calculate the anchorage length for both layers of reinforcement. The stress provided by the bond along the reinforcement need not be checked, due to the high stresses provided by the bearing of heads. The anchorage lengths that were calculated for the top and bottom layer of reinforcement were both determined to be 3.75”. This indicates that the reinforcing bar should yield around 4” away form the bearing surface of the head.

The lap splice length was also calculated using the method produced by the UTA. The anchorage lengths that were previously calculated and the appropriate strut and tie model were used to calculate the splice length. The splice length that was calculated was found to be 5.02” for both layers. The splice length that was provided in the specimens was 6”. So, adequate splice length was provided for the development of the headed bars according to the research conducted at UTA

Using the anchorage lengths and splice lengths that were previously determined, the strain gage configuration was designed. Strain gages were placed as close as possible to the head and then gages were placed at 4”, 6”, 8” and 10” away from the bearing surfaces of the heads. Strain gages were installed on both layers of reinforcing bars. Figure 9 shows the specimen and a detail view of the joint region so the strain gage configuration can be clearly seen.




Detail A

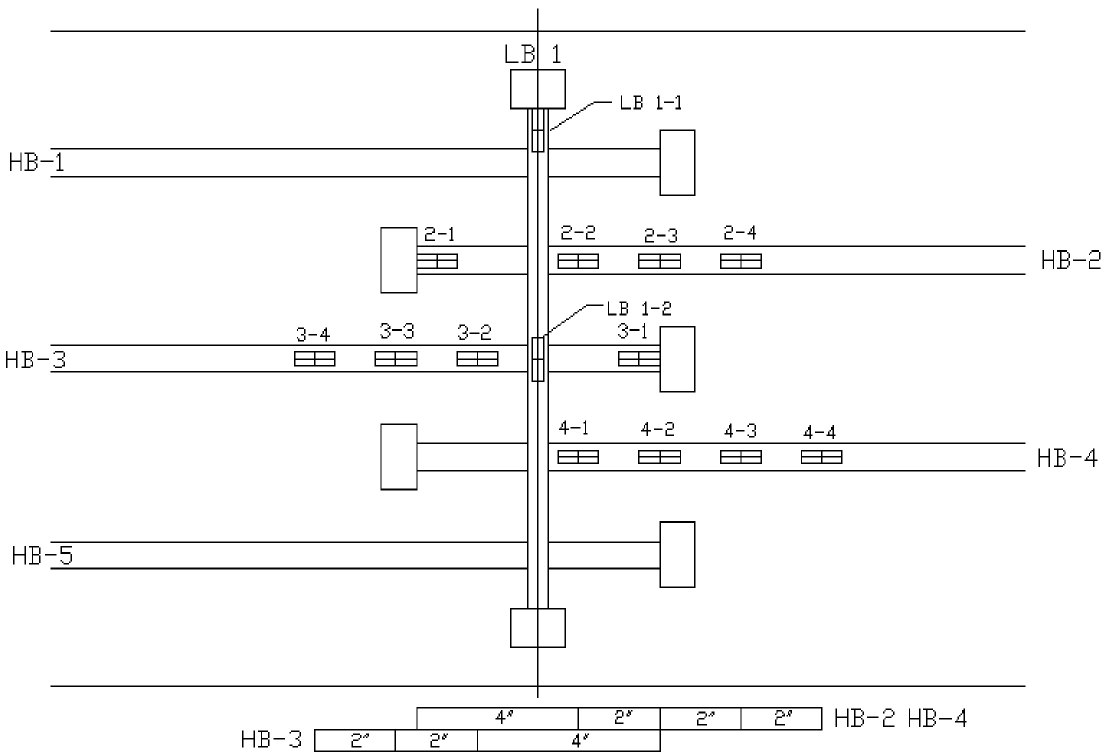



Figure 9: Strain Gage Configuration in Headed Bar Details

All distances are measured from the bearing surface of the head to the centerline of the corresponding strain gage. Strain gages 2-1 and 3-1 will be placed 1" from the bearing surface of the head.

Strain gages were also placed on the transverse lacer bars, so that the behavior of the lacer bars and the over all behavior of the joint details would be better understood. A strain gage was placed 1" from the bearing surface of the head and an additional gage was placed at the center of the lacer bars. Figure 10 shows the strain gage placement on the lacer bars.

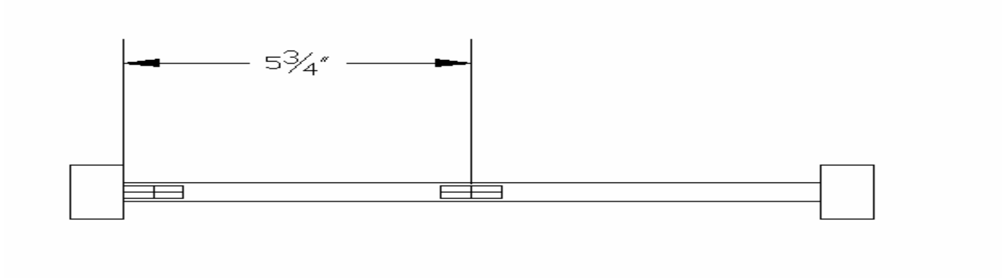



Figure 10: Strain Gage Placement on Lacer Bars

Linear variable differential transducers (LVDT) will be used to measure the displacement and determine the curvature. Also, DEMEC will be applied to the specimen to obtain the compressive and tensile strain at the surface of the concrete, as well as crack width growth. The DEMEC system will consist of metal discs embedded in the surface of the concrete, which act as gage points. These gage points will then be manually measured by a mechanical dial gage to obtain the concrete surface strain, crack width and crack width growth.

**Specimen Fabrication and Progress**

The first four U-bar specimens have been completed. Strain gage installation and rebar tying have been completed for the first two headed bar specimens. The two headed bar specimens are currently at Ross Prestressed Concrete in preparation for the concrete pour. Figures 11 – 14 show the reinforcement cages at the joints of the U-bar details and a completed flexural and tension specimen.

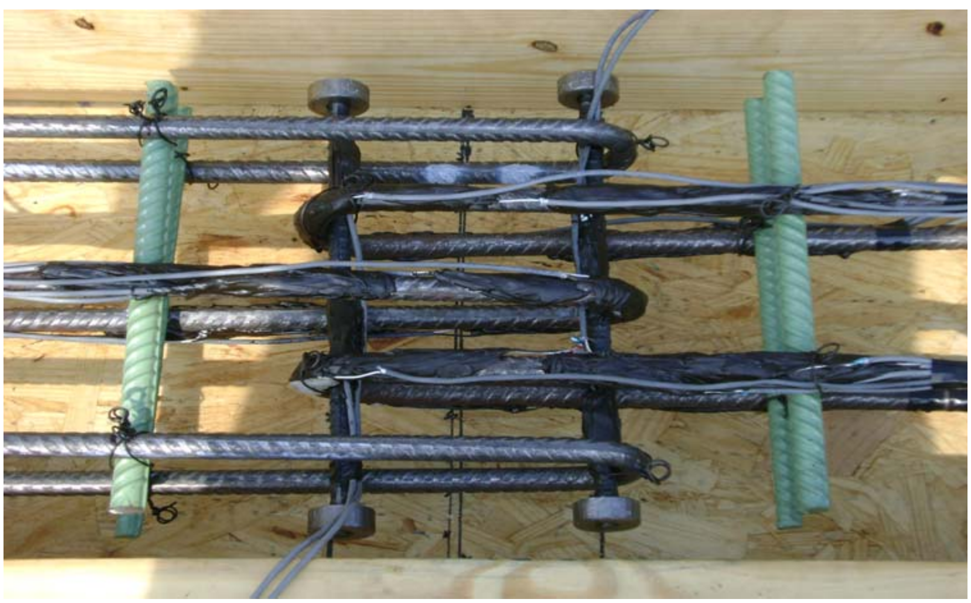


Figure 11: Welded Wire Reinforcement Joint Detail (U-bar)

Figure 12: Stainless Steel Joint Detail (U-bar)

Figure 13: Completed Tension Specimen

Figure 14: Completed Flexural Specimen

**Summary**

Longitudinal and transverse connections for precast deck systems were developed for the experimental program. Two connection details were developed for each connection type: a looped bar or u-bar connection detail as well as a headed bar connection detail. Both details would produce shorter development lengths when compared to straight bars and thus would reduce joint widths, which would in turn reduce construction time. The u-bar detail will be tested using two different materials, stainless steel and welded wire reinforcement. The headed bar detail will only tested using conventional reinforcement. The u-bar detail was also designed to utilize an extremely tight bend. The inside bend diameter that was used in the u-bar design was three times the diameter of the bar ($3d_b$). The use of this extremely tight bend produced thinner deck sections that would reduce the weight of the deck and also aid in constructability.

Testing of the first six specimens will begin by the last week of September and will be completed by the end of October.

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