

To the Graduate Council:

I am submitting herewith a thesis written by Samuel Lewis entitled “Experimental Investigation of Precast Bridge Deck Joints with U-bar and Headed Bar Joint Details” I have examined the final electronic copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

Zhongguo Ma  
Major Professor

We have read this thesis  
and recommend its acceptance:

Edwin Burdette

Qihong Zhao

Accepted for the Council:

Carolyn R. Hodges

Vice Provost and  
Dean of the Graduate School

Experimental Investigation of Precast Bridge Deck Joints with U-bar and Headed Bar Joint  
Details

A Thesis  
Presented for the  
Master of Science  
Degree  
The University of Tennessee, Knoxville

Samuel Lewis  
May 2009

## **Acknowledgements**

First, I would like to thank my family for their support and encouragement throughout my engineering studies. Without my parents as examples, I would not have seen the work ethic and discipline required to accomplish long term goals. I would like to thank Dr. Ma for the opportunity to pursue a Master degree and for guiding me through the world of academic research. I am also grateful for the help given by the TBRL research group, especially Wayne Gibbs, Jayaprakash Vadivelu, Lungui Li and David Ritter. I would also like to acknowledge Larry Roberts and Ken Thomas, the civil engineering shop technicians, without whose help the transition from shop drawings to reality would be nearly impossible. I would also like to express my gratitude to Rick Merritt of Ross Prestressed Concrete, Inc, Steve Abbott of Gerdau Ameristeel, Ryan Pelter of Engineered Wire products and Gena Peters of Salit Specialty Rebar Inc. for donating their time and materials for the progress of this project. I would also like to thank Dr. Qihong Zhao and Dr. Edwin G. Burdette for serving on my committee. Dr. Edwin G. Burdette deserves special thanks for his counsel and his assistance in the weld tests. The research reported in this thesis is a part of the NCHRP 10-71 project “Cast-in-Place Concrete Connections for Precast Deck Systems”, sponsored by the National Cooperative Highway Research Program.

## **Abstract**

This thesis presents the experimental investigation of two joint details for use in precast bridge deck systems. U-bar and headed bar joint details were developed for use in accelerated construction applications. Both details, in practice, would consist of staggered protruding reinforcement that would allow for the anchorage of the precast deck component into the joint. Six specimens containing the joint details were constructed and tested. Three specimens were tested in flexure to simulate the forces that would be experienced in a longitudinal deck joint, and three specimens were tested in tension to simulate the forces that would be experienced in a transverse joint over an interior pier. The three specimens of each test type consisted of one specimen containing the headed bar detail and two specimens containing the u-bar detail. The u-bar detail was tested utilizing two materials, welded wire reinforcement and stainless steel reinforcement. Welded wire reinforcement and stainless steel reinforcement were used for the u-bar detail due to their ductility which was needed to fabricate the tight bend ( $3d_b$ ) used in the detail. The tight bend was used to minimize the thickness of the deck. The main objective of the testing was to determine if the joint details could create a precast deck system that could emulate the monolithic behavior of the predominately used cast-in-place deck systems. To achieve monolithic behavior in a precast deck system the joints must be able to transfer shear and tension forces as well as moments. The second objective of this investigation was to determine the best performing detail for further investigation. The additional investigation of the best performing joint detail would then be the first step in creating standard design guidelines and details to ease the future implementation of joints for precast bridge deck systems.

## Table of Contents

|                                                                                 |           |
|---------------------------------------------------------------------------------|-----------|
| <b>Chapter 1 Introduction .....</b>                                             | <b>1</b>  |
| Joint Details .....                                                             | 3         |
| <b>Chapter 2 Experimental Program.....</b>                                      | <b>7</b>  |
| Specimen Design .....                                                           | 7         |
| Experimental Set-up.....                                                        | 21        |
| Instrumentation .....                                                           | 25        |
| <b>Chapter 3 Specimen Construction, Reinforcement Cost and Fabrication.....</b> | <b>29</b> |
| Specimen Construction .....                                                     | 29        |
| Reinforcement Cost and Fabrication .....                                        | 35        |
| <b>Chapter 4 Constitute Testing.....</b>                                        | <b>37</b> |
| Concrete Testing .....                                                          | 37        |
| Reinforcing Materials Testing .....                                             | 38        |
| Weld Testing.....                                                               | 41        |
| <b>Chapter 5 Results and Discussion .....</b>                                   | <b>47</b> |
| Moment Capacity.....                                                            | 47        |
| Flexural Specimen Behavior.....                                                 | 55        |
| Flexural Crack Widths at Service Level Loading.....                             | 58        |
| Tensile Capacity.....                                                           | 59        |
| Tension Specimen Behavior .....                                                 | 62        |
| Tensile Crack Widths at Service Level Loading .....                             | 65        |
| Strain Gage Data .....                                                          | 66        |
| <b>Chapter 6 Conclusions and Recommended Future Research .....</b>              | <b>86</b> |
| <b>List of References.....</b>                                                  | <b>89</b> |
| <b>Vita .....</b>                                                               | <b>92</b> |

## List of Tables

|                                                                                           |    |
|-------------------------------------------------------------------------------------------|----|
| Table 1: Reinforcement Required for 203.2 mm (8") Deck ( $f_y = 420$ MPa) .....           | 12 |
| Table 2: Reinforcement Required for 203.2 mm (8") Deck ( $f_y = 520$ MPa) .....           | 12 |
| Table 3: Reinforcement Required for the U-bar Detail in a 158.8 mm (6.25") Deck .....     | 13 |
| Table 4: Reinforcement Required for the Headed Bar Detail in a 158.8 mm (6.25") Deck..... | 13 |
| Table 5: Negative Moment Longitudinal Reinforcement .....                                 | 15 |
| Table 6: Concrete Compressive Strengths, U-bar Specimens .....                            | 38 |
| Table 7: Concrete Compressive Strengths, Headed bar Specimens .....                       | 38 |
| Table 8: Weld Test Results, One Pass Welds .....                                          | 43 |
| Table 9: Weld Test Results, Beveled Welds.....                                            | 45 |
| Table 10: Weld Test Results, Beveled Welds and 110 ksi Welding Stick .....                | 45 |
| Table 11: Required Service and Strength 1 Limit State Unit Moments .....                  | 47 |
| Table 12: Modified Specimen Moments, Specimens Containing the U-bar Detail .....          | 48 |
| Table 13: Modified Specimen Moments, Specimens Containing the Headed Bar Detail.....      | 48 |
| Table 12: Theoretical Moments and Curvature .....                                         | 53 |

## List of Figures

|                                                                                               |    |
|-----------------------------------------------------------------------------------------------|----|
| Figure 1: Joint Directions and Representative Specimen Orientations .....                     | 6  |
| Figure 2: Bridge Cross-Section.....                                                           | 9  |
| Figure 3: Bridge Longitudinal Section .....                                                   | 9  |
| Figure 4: Composite Section Used for Negative Moment Longitudinal Reinforcement Design .      | 14 |
| Figure 5: U-bar Longitudinal Joint Specimen .....                                             | 19 |
| Figure 6: Headed Bar Longitudinal Joint Specimen.....                                         | 19 |
| Figure 7: U-bar Transverse Joint Specimen.....                                                | 20 |
| Figure 8: Headed Bar Transverse Joint Specimen.....                                           | 20 |
| Figure 9: Flexural Test Set-Up (Longitudinal Joint Test) .....                                | 22 |
| Figure 10: Tension Test Set-Up (Transverse Joint).....                                        | 24 |
| Figure 11: U-bar Joint Detail Strain Gage Configuration.....                                  | 26 |
| Figure 12: Headed Bar Joint Detail Strain Gage Configuration .....                            | 27 |
| Figure 13: Lacer Bar Strain Gage Configuration.....                                           | 28 |
| Figure 14: Specimen Construction.....                                                         | 32 |
| (a) Reinforcement in the Forms .....                                                          | 32 |
| Figure 15: Determination of Modulus of Elasticity.....                                        | 40 |
| Figure 16: Average Stress Verses Strain Curves .....                                          | 40 |
| Figure 17: Connection Detail, Conceptual Drawing.....                                         | 41 |
| Figure 18: Photo of the Top Connection Detail.....                                            | 42 |
| Figure 19: Weld Test Set-Up .....                                                             | 42 |
| Figure 20: One Pass Weld Failure .....                                                        | 44 |
| Figure 21: Moment Verses Deflection Curves .....                                              | 50 |
| Figure 22: Moment Verses Curvature Curves .....                                               | 50 |
| Figure 23: Cross Section used for Theoretical Calculations of the U-bar Specimens.....        | 52 |
| Figure 24: Cross Section Used for the Theoretical Calculations of the Headed Bar Specimen ... | 53 |
| Figure 25: Actual and Theoretical Moment Verses Curvature Curves for the U-bar Details .....  | 54 |
| Figure 26: Actual and Theoretical Moment Verses Curvature Curves for Headed Bar Details...    | 55 |
| Figure 27: Flexural Crack Patterns at Failure .....                                           | 56 |
| Figure 28: Total Applied Force Verses Deflection Curves .....                                 | 61 |
| Figure 29: Tension Crack Patterns at Failure .....                                            | 63 |
| Figure 30: Total Force Verses Rebar Strain Figures for Specimen WT-1 .....                    | 67 |
| Figure 31: Total Force Verses Rebar Strain for Specimen HT-1 .....                            | 71 |
| Figure 32: Moment Verses Rebar Strain Curves of Specimen SB-1 .....                           | 74 |
| Figure 33: Moment Verses Rebar Strain Curves of Specimen WB-1 .....                           | 78 |
| Figure 34: Moment Verses Rebar Strain Curves of Specimen HB-1 .....                           | 82 |

## **Chapter 1 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 anywhere in our nation. This system allows the American public access to fast and safe transportation at their convenience.

Due to the most of the interstate system being constructed in the 1950's and 1960's, our transportation infrastructure is aging and is in need of repair, replacement and in some cases expansion. The majority of the bridges that were built 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 passed their service lives and are in critical need of repair or replacement.

At the time when the bridges in the interstate system were first being constructed, public delay was of little concern in the construction process because the interstate was new construction. Now, as the bridges in our transportation system are being repaired or replaced, public delay must be considered in the construction process because the motoring public has become reliant on them. The delay caused to the public by bridge renovation and replacement must be minimized to better serve those who use them. The engineering and construction workforces are now left with the task of finding rapid, practical, and cost effective ways to update and expand the bridges that connect our vital interstate system.

One of the major problem areas in aging bridges is the deck. Bridge decks are directly exposed to loading, the harmful effects of weather, and the corrosive properties of deicing salts, so this component of the bridge deteriorates faster than other components of that bridge. The

deck has traditionally been constructed using cast-in-place methods, which requires time to form, place reinforcement, cast concrete, and for the concrete to cure. The previously mentioned process for the construction of cast-in-place decks is very time consuming. Therefore, deck construction and replacement are ideal situations where construction time and public delay can be reduced.

Full depth precast deck panels and prestressed precast decked bulb tee (DBT) girders have been used successfully to reduce the time required for bridge deck construction and renovation. One of the hurdles that must be overcome in order to enable a wider use of this technology is the development of design guidelines and standard details for the joints used in these systems. The design guidelines and standard details for the joints of precast deck systems must produce full strength joints, but still allow for accelerated construction.

This paper presents the experimental testing results of two precast deck joint details. Two joint details were investigated to observe their behavior, ensure that they are full strength joints, and to select the best performing detail for further testing. Both joint details were designed for use in accelerated construction applications. To allow for accelerated construction, the details were developed to minimize deck thickness and joint width, and to provide for easy placement of precast concrete components. Additional testing of the best performing detail would be the next step in developing design guidelines and standard details for the joints of various precast deck systems.

### ***Joint Details***

A u-bar detail and a headed bar detail were the two joint details that were tested. The primary reinforcement of the u-bar detail are reinforcing bars that are bent 180 degrees at their center to a specified bend diameter and placed in the joint vertically, so that they provide the top and bottom reinforcement for the deck. The primary reinforcement in the headed bar detail are reinforcing bars that terminate in bearing heads. In practice, the u-bars and headed bars would extend out of precast deck components, which would allow for anchorage of that component into the joint. The extended reinforcement of both details would be staggered or out of phase when compared to the deck component adjacent to it. This placement would allow for space between each consecutive reinforcing bar in the joint, which would lead to easy placement in the field. The deck components would then be placed so that the rebar in the joint would have a specified overlap length and spacing. The overlap length is the distance between bearing surfaces of adjacent reinforcing bars, and the spacing is the center to center distance of adjacent bars. The joint would then be completed after the addition of transverse lacer bars and grout. The addition of transverse lacer bars adds confinement and continuity to the joint. The previously mentioned process will decrease construction time when compared to cast-in place deck systems.

Both details reduce joint width by the use of bearing. The bearing surface of the u-bar is the inside of the bend, and the bearing surface of the headed bar is the head. The bearing surfaces of both joint details reduce the development length or overlap length required, when compared to a straight bar, and therefore reduce the joint width required to produce yielding in the reinforcement. Yielding of the reinforcement in the joint will produce full strength joints and allow the precast deck system to emulate the behavior of a cast-in-place deck system.

The joint details were also designed to minimize deck thickness. This was accomplished by using small diameter Lenton Terminator<sup>®</sup> heads for the headed bar detail and a small bend diameter for the u-bar detail. Although the deck thickness was minimized, two inches of top cover and one inch of bottom cover was maintained.

The u-bar detail was designed to utilize an extremely tight bend. The inside bend diameter that was used was three times the diameter of the bar ( $3d_b$ ), thus with to #5 bars being used, the inside diameter of the bend was 47.63 mm (1 7/8"). ACI Committee 318-08 (2008) (ACI 318-08) has set minimum bend diameters for different rebar sizes and materials. For a #5 bar made of conventional steel, the minimum bend diameter, per ACI 318-08, is six times the diameter of the bar ( $6d_b$ ), and for D31 welded wire reinforcement the minimum bend diameter is four times the diameter of the bar ( $4d_b$ ) when used as stirrups or ties. Clearly the u-bar bend diameter that was used ( $3d_b$ ) violates the minimum allowable bend diameters established by ACI 318-05.

The minimum bend diameters are established primarily for two reasons: feasibility of bending the reinforcement without breaking it and possible crushing of the concrete within the tight bend. To ensure that the reinforcement would not be broken while bending, two ductile reinforcing materials were used: welded wire reinforcement and stainless steel reinforcement. Concrete crushing in the tight bend was closely observed in the experimental investigation to determine if it would occur

Both longitudinal joints (parallel to the traffic direction) and transverse joints (perpendicular to the traffic direction) were designed and tested utilizing each joint detail. Both joint directions were investigated so that the results of this experimental program could apply to

several precast deck systems. Figure 1 shows the two joint directions tested and the specimen orientations used to represent the joints.

The two joint directions would experience different forces and were therefore tested differently. The joint details and testing procedures are discussed in greater detail in the following sections of this paper.

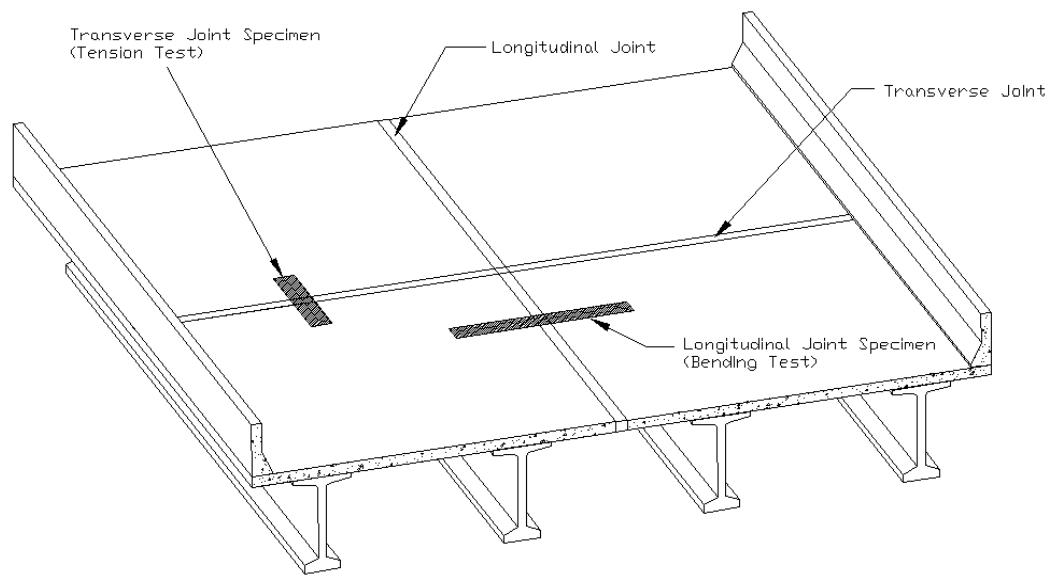


**Figure 1: Joint Directions and Representative Specimen Orientations**

## **Chapter 2 Experimental Program**

### ***Specimen Design***

Due to two joint directions being investigated, two design methods were used to determine a realistic spacing for the reinforcement of the joint details. The spacing of the rebar in the longitudinal joint was designed utilizing the AASHTO strip method of deck design (AASHTO LRFD 2007). This method takes into account the largest positive and negative moments that would be experienced by the longitudinal joint. However, to determine the controlling load case for a transverse joint, the transverse joint should be positioned over an interior support in a continuous span bridge system. In this case, if the deck was compositely connected to the girder, the deck would have to resist large tensile forces that would be produced by the negative moment developed there. So, the rebar spacing for the transverse joint was determined using flexural analysis of the deck-girder composite section, conservatively assuming that all the tension force created by the negative moment would be resisted by the deck. The spacing of the rebar in each joint direction was determined by its corresponding deck design.

Two PCI design examples were used to design the reinforcement spacings for the joint details. The longitudinal joint spacing was designed using design example 9.8 in the PCI Bridge Design Manual (PCI 2003). The spacing for the transverse joint reinforcement was designed using design example 9.6 in the PCI 2003. Both examples used the same bridge cross section and longitudinal section. The bridge cross section consisted of four BT-72 girders that were spaced at 3.66 m (12') on center. The longitudinal section of the bridge consisted of a center span of 36.76m (120') and two side spans of 33.53 m (110'). The PCI design examples were used, so

that the steel areas determined would represent the amount of steel in a typical bridge deck. The bridge cross section and longitudinal section that were used in the specimen and connection design can be seen in Figures 2 and 3.

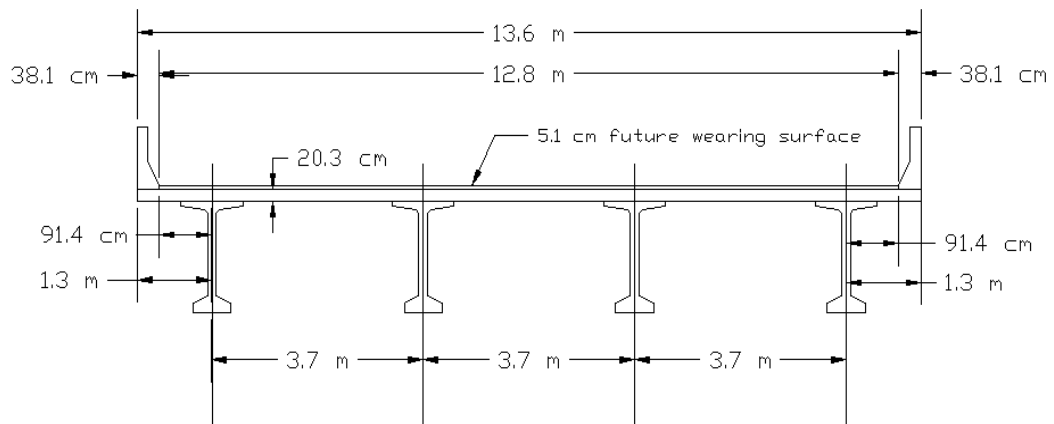


**Figure 2: Bridge Cross-Section**

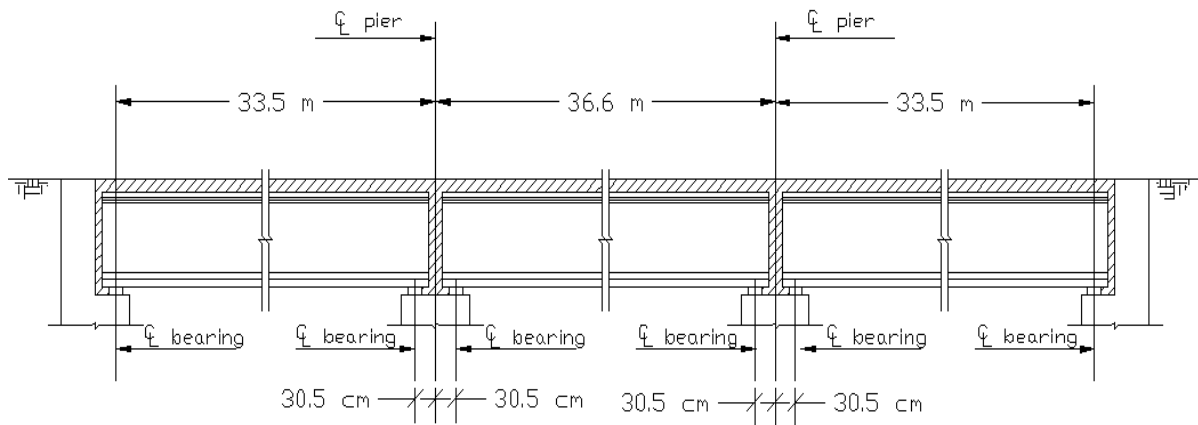


**Figure 3: Bridge Longitudinal Section**

A 203.2 mm (8") thick deck was first designed for the interior portion of the deck. This design was carried out to obtain an estimate of the area of steel used in decks with a conventional thickness. In order to minimize the deck thickness, two other thicknesses were also investigated. A 158.8 mm (6.25") deck was designed for the longitudinal joint direction utilizing both joint details, and a 184.2 mm (7.25") deck was designed for the transverse joint direction utilizing both joint details. The transverse joint required a larger deck thickness in order to keep the transverse deck rebar as the outer reinforcement layer. The transverse rebar in a deck is the main flexural reinforcement; thus in order to obtain an efficient design, the transverse rebar was given the largest moment arms possible without violating cover requirements. The 158.8 mm (6.25") and 184.2 mm (7.25") deck thicknesses were the thinnest deck sections possible when considering the joint details and cover requirements.

As stated previously the AASHTO strip method was used for the design of the rebar spacing for the longitudinal joint for both details. This method conservatively models the deck as a continuous 304.8 mm (12") wide beam running perpendicular to the bridge girders. The girders were modeled as rigid point supports for the continuous beam model of the deck. All dead load moments were conservatively found by averaging the maximum moment equations of a uniformly loaded simply supported beam and a uniformly loaded fixed beam. This averaged maximum moment equation was then used to calculate all moments that were due to dead load. Live load moments were determined utilizing Table A4-1 in the AASHTO Bridge Design Specification (AASHTO LRFD 2007). The maximum positive and negative live load moments found in table A4-1 were used for design of the entire strip of deck.

The top and bottom layers of transverse deck reinforcement were designed using flexural theory and the AASHTO strength and service limit states. The bottom layer of longitudinal

reinforcement or the secondary moment reinforcement was designed based on a percentage of the steel area determined for the bottom layer of transverse reinforcement. The percentage used to determine the bottom longitudinal steel area was based on the effective length of the deck between the girders. The top layer of longitudinal reinforcement or the temperature and shrinkage steel was determined based on the geometry of the entire deck or the strip of deck, whichever required the larger area of steel.

In the design of all deck thicknesses, two steel yield strengths were used. A yield strength ( $f_y$ ) of 420 MPa (60 ksi) and an  $f_y$  of 520 MPa (75 ksi) were used. The  $f_y$  of 420 MPa (60 ksi) corresponds to the conventional steel used for the headed bars, and the  $f_y$  of 520 MPa (75 ksi) corresponds to the welded wire reinforcement and stainless steel used for the u-bars. Two different values of the exposure factor ( $\gamma_e$ ) were also used in the designs to show where the service limit state controlled the design. The values that were used for the  $\gamma_e$  were the values corresponding to exposure class one and two; the  $\gamma_e$  values are 1 and .75, respectively. The exposure factors are directly proportional to the crack widths expected at service level loading. So, the larger the exposure factor the larger the expected crack widths.

The 203.2 mm (8") deck thickness was the first deck section that was designed. This deck thickness was not used for the specimens, but it was used to observe the increase in steel area required for the slimmer 158.8 mm (6.25") and 184.2 mm (7.25") deck sections. Tables 1 and 2 show the results of the interior deck design for the 203.2 mm (8") deck thickness with both 420 Mpa (60ksi) and 520 Mpa (75 ksi) steel yield strengths. The tables also show the design using both exposure factors ( $\gamma_e$ ).

**Table 1: Reinforcement Required for 203.2 mm (8") Deck ( $f_y = 420$  MPa)**

|             |              | $f_y = 420$ MPa (60 ksi) |              |                |              |
|-------------|--------------|--------------------------|--------------|----------------|--------------|
|             |              | $\gamma_e = .75$         |              | $\gamma_e = 1$ |              |
|             |              | Bars Size                | Spacing (mm) | Bars Size      | Spacing (mm) |
| M+ (Bottom) | Transverse   | # 5                      | <b>203.2</b> | # 5            | <b>203.2</b> |
|             | Longitudinal | # 5                      | <b>292.1</b> | # 5            | <b>292.1</b> |
| M- (Top)    | Transverse   | # 5                      | <b>177.8</b> | # 5            | <b>215.9</b> |
|             | Longitudinal | # 4                      | <b>457.2</b> | # 4            | <b>457.2</b> |

**Table 2: Reinforcement Required for 203.2 mm (8") Deck ( $f_y = 520$  MPa)**

|             |              | $f_y = 520$ MPa (75 ksi) |              |                |              |
|-------------|--------------|--------------------------|--------------|----------------|--------------|
|             |              | $\gamma_e = .75$         |              | $\gamma_e = 1$ |              |
|             |              | Bars Size                | Spacing (mm) | Bars Size      | Spacing (mm) |
| M+ (Bottom) | Transverse   | # 5                      | <b>203.2</b> | # 5            | <b>241.3</b> |
|             | Longitudinal | # 5                      | <b>292.1</b> | # 5            | <b>292.1</b> |
| M- (Top)    | Transverse   | # 5                      | <b>177.8</b> | # 5            | <b>215.9</b> |
|             | Longitudinal | # 4                      | <b>457.2</b> | # 4            | <b>457.2</b> |

From Table 2, it can be seen that the service limit state controls the design for both the top and bottom transverse reinforcement when using a steel yield strength of 520 MPa (75 ksi). The service limit state only governs the design for the top layer of transverse reinforcement when the 420 MPa (60 ksi) yield strength was used. The service limit state is shown to govern when the change in exposure factor changes the required rebar spacing.

The 158.8 mm (6.25") thick deck section was then designed. The results from this design were used for the design of the reinforcement for the longitudinal joint direction for both joint details. Again, both yield strengths and exposure classes were used. Table 3 shows the results of the design for the u-bar detail, and Table 4 shows the design results for the headed bar detail.

**Table 3: Reinforcement Required for the U-bar Detail in a 158.8 mm (6.25”) Deck**

|             |              | $f_y = 520 \text{ MPa (75 ksi)}$ |              |                |              |
|-------------|--------------|----------------------------------|--------------|----------------|--------------|
|             |              | $\gamma_e = .75$                 |              | $\gamma_e = 1$ |              |
|             |              | Bars Size                        | Spacing (mm) | Bars Size      | Spacing (mm) |
| M+ (Bottom) | Transverse   | # 5                              | <b>114.3</b> | # 5            | <b>114.3</b> |
|             | Longitudinal | # 5                              | <b>165.1</b> | # 5            | <b>165.1</b> |
| M- (Top)    | Transverse   | # 5                              | <b>101.6</b> | # 5            | <b>114.3</b> |
|             | Longitudinal | # 4                              | <b>304.8</b> | # 4            | <b>304.8</b> |

**Table 4: Reinforcement Required for the Headed Bar Detail in a 158.8 mm (6.25”) Deck**

|             |              | $f_y = 420 \text{ MPa (60 ksi)}$ |              |                |              |
|-------------|--------------|----------------------------------|--------------|----------------|--------------|
|             |              | $\gamma_e = .75$                 |              | $\gamma_e = 1$ |              |
|             |              | Bars Size                        | Spacing (mm) | Bars Size      | Spacing (mm) |
| M+ (Bottom) | Transverse   | # 5                              | <b>101.6</b> | # 5            | <b>101.6</b> |
|             | Longitudinal | # 5                              | <b>139.7</b> | # 5            | <b>139.7</b> |
| M- (Top)    | Transverse   | # 5                              | <b>101.6</b> | # 5            | <b>101.6</b> |
|             | Longitudinal | # 4                              | <b>304.8</b> | # 4            | <b>304.8</b> |

Tables 3 and 4 show that the service limit state had little impact on the results of the rebar spacing for the 158.8 mm (6.25”) deck section. The only case that the service limit state governed was the top layer of transverse reinforcement of the u-bar detail design. Also, when comparing the amount of reinforcement required for the 158.8 mm (6.25”) and 203.2 mm (8”) thick deck sections, it can be seen that the 203.2 mm (8”) thick deck requires approximately half the reinforcement that is required for the 158.8 mm (6.25”) thick deck. Although the area of steel required for the 158.8 mm (6.25”) thick deck is larger than that required for the 203.2 mm (8”) thick deck, the decrease in weight of the slimmer deck section and the labor costs saved by the possibility of accelerated construction provide the incentive to investigate this slimmer deck section.

The required steel area for the transverse joint was then determined. The reinforcement in the transverse joint was designed as if the joint was located over an interior pier of a continuous

span bridge system and that the deck was compositely connected to the girder. The negative moment developed in these locations would create large tensile forces in the deck, which would require more longitudinal steel in these regions. The composite cross section and the negative moment used in the flexural calculations were taken from example 9.6 of PCI 2003. The negative moment value that was used in the flexural calculations was 6558.37 kN-m (4837.2 kip-ft). The composite section used for the reinforcement design is shown in Figure 4.

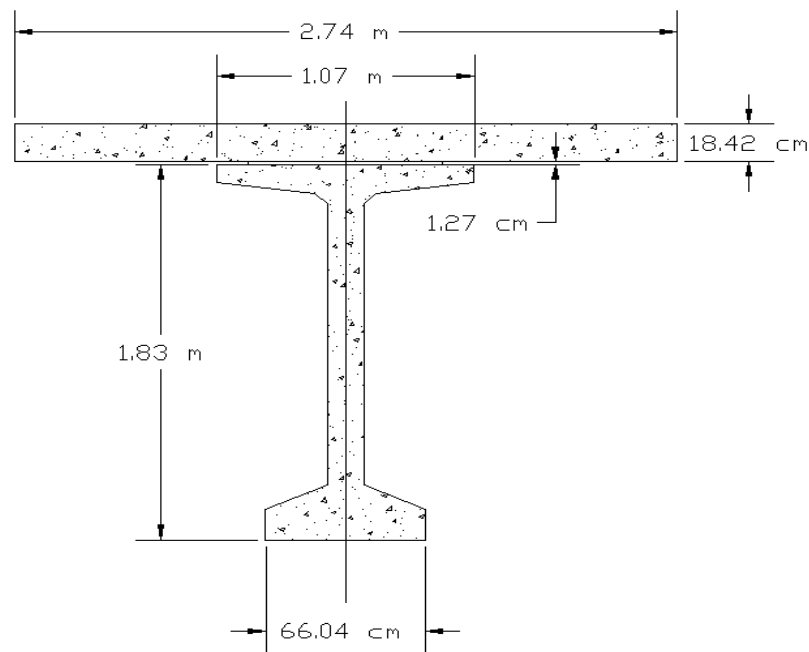


**Figure 4: Composite Section Used for Negative Moment Longitudinal Reinforcement Design**

The amount of longitudinal deck reinforcement was determined by a conventional flexural design using the composite section shown above. The centroid of the reinforcing was assumed to be at mid height of the deck and the required amount of reinforcing was determined for both 420 MPa (60 ksi) and 520 MPa (75 ksi) yield strengths. Table 5 contains the results of the designs.

For easy comparison of the results of the different joint details and joint directions, the deck designs were used to develop a rebar spacing configuration that was used for the construction of all specimens. Due to the use of the u-bar detail, the top and bottom layers of primary joint reinforcement had to have the same spacing in both joint details and directions. The longitudinal specimen reinforcement was determined to consist of a top and bottom layer of #5 rebar spaced at 114.3 mm (4.5"). The top layer of transverse specimen reinforcement was determined to consist of #4 rebar spaced at 304.8 mm (12"), and the bottom layer of transverse reinforcement was determined to be #5 rebar spaced at 152.4 mm (6").

**Table 5: Negative Moment Longitudinal Reinforcement**

| Longitudinal Reinforcement<br>(Mu <sup>-</sup> region) |               |                 |
|--------------------------------------------------------|---------------|-----------------|
| f <sub>y</sub><br>(MPa)                                | rebar<br>size | spacing<br>(mm) |
| 420                                                    | #5            | 114.3           |
| 520                                                    | #5            | 139.7           |

Due to the direction of the longitudinal joint, the longitudinal specimen rebar corresponds to the transverse rebar in a bridge deck, and the transverse specimen rebar corresponds to the longitudinal rebar in a bridge deck. In the transverse joint, the specimen rebar orientation is consistent with the rebar orientation in a bridge deck. The correct spacings of the transverse reinforcement in the longitudinal specimens were not used because that reinforcement was not of primary concern and the spacings used allowed for consistent specimen construction.

The joint overlap length, which is the distance between the reinforcement's bearing surfaces, was determined based on the expected development length of a u-bar and the expected splice length of a headed bar. The approximate development length for a u-bar was determined by the use of the ACI development length equation for standard hooks in tension (ACI 318-08). The expected splice length of a headed bar was determined based on research performed at the University of Texas at Austin (Thompson 2006). The calculated values of the development length for the u-bar and the splice length of the headed bars were used as a reference when the specimen overlap length was determined. The same overlap length was used in the u-bar and headed bar detail for easy comparison of test results.

The ACI equation for determining the development length of a standard hook in tension was used to calculate the approximate development length of a u-bar. This equation does not directly apply to the u-bars that were used, because the u-bars do not meet the dimensional requirement for a standard hook; that is, the  $3d_b$  bend radius used in the u-bar fabrication violates the minimum  $6d_b$  bend radius specified in ACI 318-08. However, the development length equation was used to determine an approximate development length that was used in conjunction with the splice length determined for the headed bar detail to determine an overlap length that

was used in both details. The following equation shows the ACI development length equation for a standard hook in tension.

$$l_{dh} = \left\lceil \frac{.02 \Psi_e \lambda f_y}{\sqrt{f'_c}} \right\rceil d_b \text{ (ACI 12.5.2) Eq. 1}$$

In Equation 1,  $\Psi_e$  and  $\lambda$  were both set equal to one, because the rebar that was used was not epoxy coated and the concrete was not lightweight. The development length was calculated for a #5 bar. A concrete compressive strength of 48.3 MPa (7 ksi) was used as well as a steel yield strength of 520 MPa (75 ksi), because welded wire reinforcement and stainless steel were materials used for this joint detail. Also, the development length modification factor of 0.7 was used, because the specimens met the bar cover parameters of having not less than 63.5 mm (2.5”) of side cover and not less than 50.8 mm (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 199.14 mm (7.84”).

Research from the University of Texas at Austin (UTA) by M. Keith Thompson (Thompson 2006) was used to determine the required overlap length for the headed bar detail. The required anchorage length or development length was found considering the stress provided by the mechanical anchorage of the head and the steel-concrete bond stress from the head to the point of maximum rebar stress. The anchorage length was then used with the suggested strut and tie model, which utilizes a 55 degree strut angle, to calculate the splice length of the headed bars. The required splice length, or in this case the overlap length, was determined to be 127.51 mm (5.02”).

Since the behavior of the specimens with both joint details were to be compared, the overlap lengths for the u-bar and headed bar joint details were made the same. After comparing

the development length of a hooked bar in tension and the splice length calculated for the headed bars, an overlap length of 152.4 mm (6") was used for the both joint details. An overlap length of 152.4 mm (6") was greater than the calculated splice length for the headed bars, but smaller than the 199.14 mm (7.84") required to develop an ACI standard hook in tension. Again the 152.4 mm (6") overlap length is measured between the bearing surfaces of each joint detail.

Two transverse lacer bars were added to each joint detail to provide continuity and confinement of the joints. The inclusion of lacer bars in both joint types was due to previous research conducted by S.R. Gordon and I.M. May (Gordon 2005) on loop bar joints in tension. In their experimental program a loop bar specimen was tested in tension without transverse lacer bars, which resulted in a sudden brittle failure. The transverse lacer bars were installed in the center of the bend in the u-bar detail and in the middle of the headed bar detail.

The specimens and connection details designed for the longitudinal joint direction can be seen in Figures 5 and 6. The specimens and connection details designed for the transverse joint direction can be seen in Figures 7 and 8.

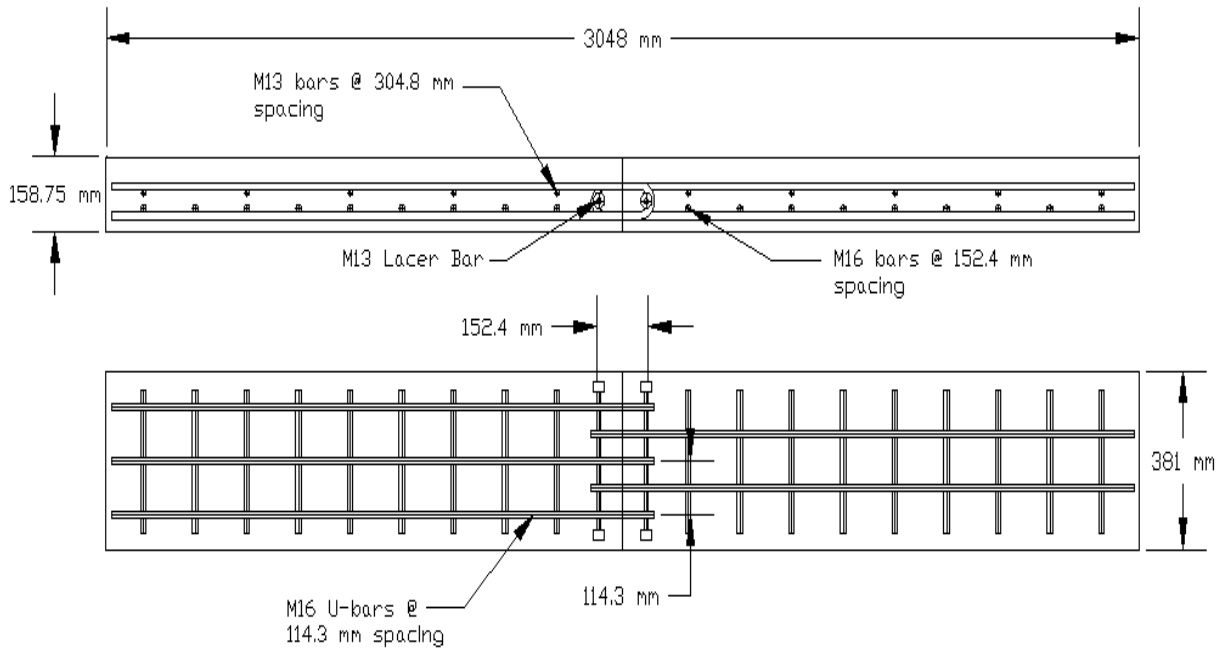


**Figure 5: U-bar Longitudinal Joint Specimen**

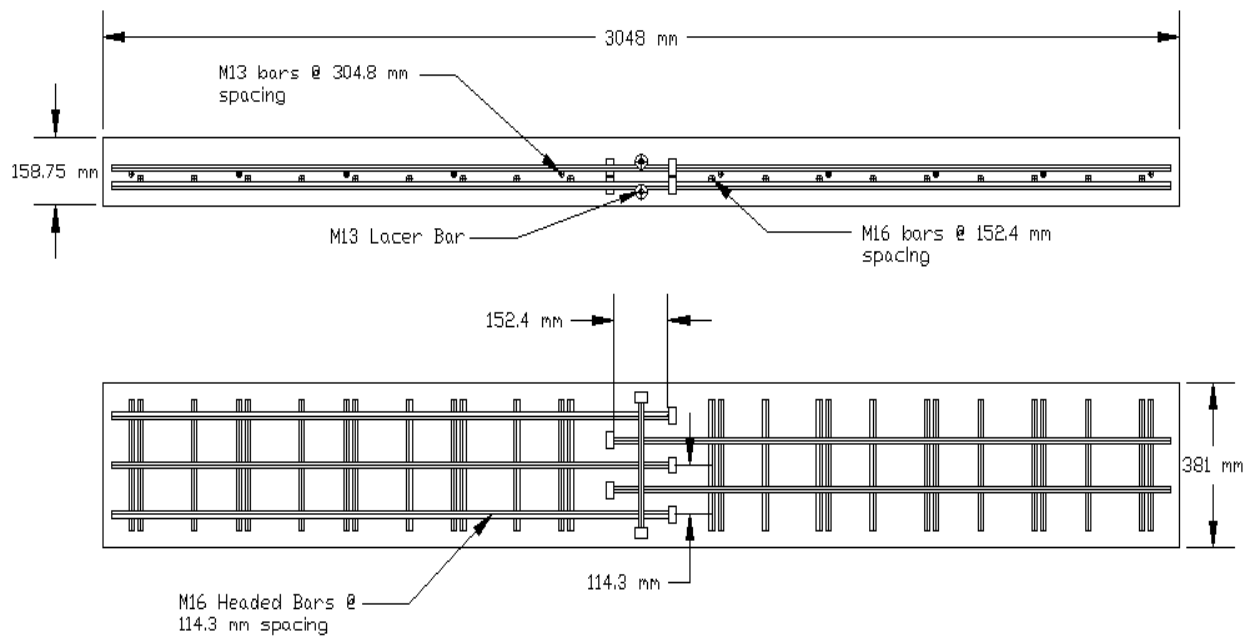


**Figure 6: Headed Bar Longitudinal Joint Specimen**

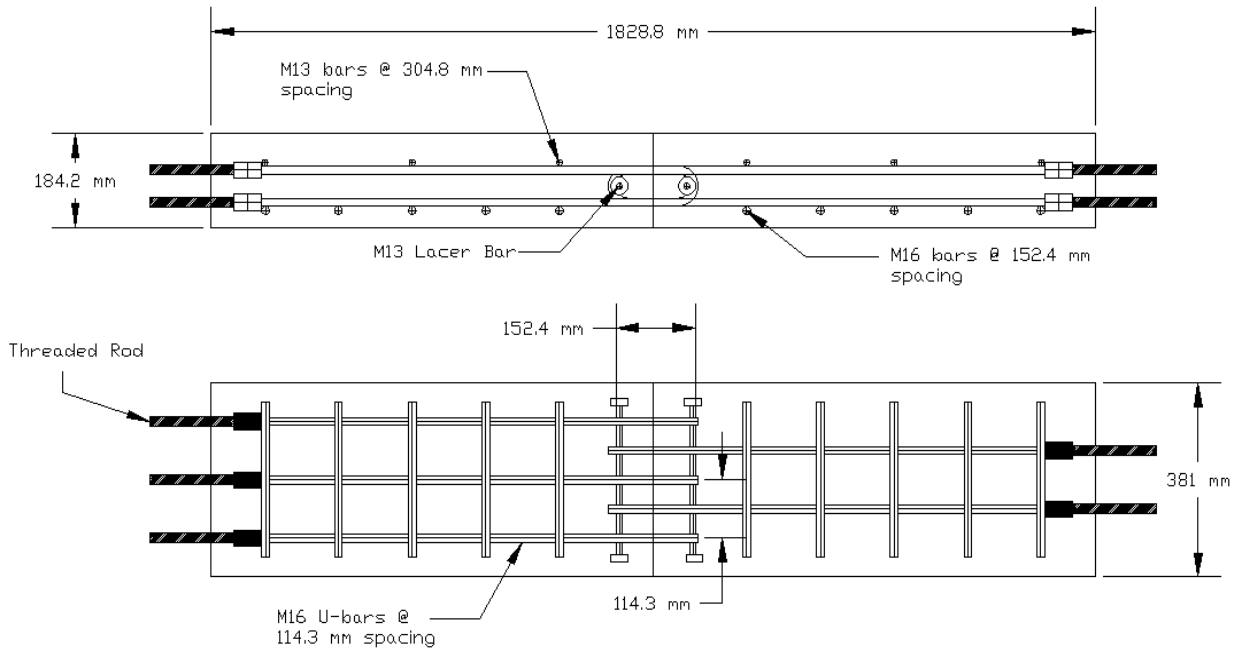


**Figure 7: U-bar Transverse Joint Specimen**

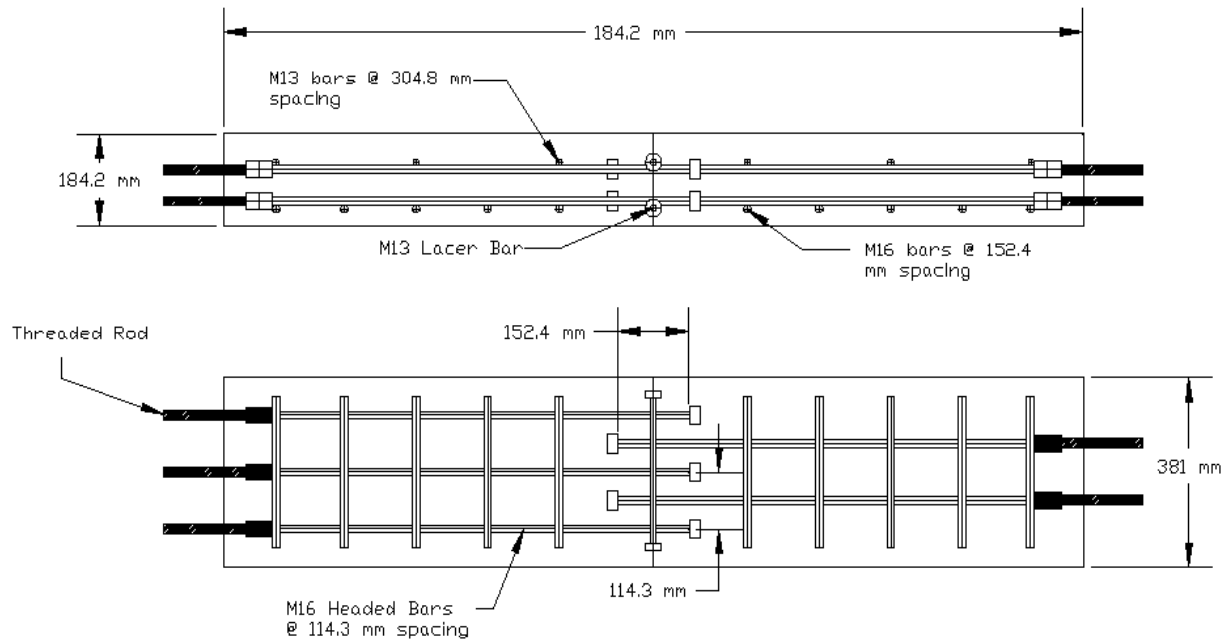


**Figure 8: Headed Bar Transverse Joint Specimen**

### ***Experimental Set-up***

Simple static tests were performed for both the longitudinal connections and transverse connections. The specimens representing the longitudinal joint direction were tested in bending. The specimens representing the transverse joint direction were tested in tension, because of the tensile forces created by negative moment regions. Again, both joint details were tested in both joint directions.

As stated previously, the specimens representing the longitudinal joint direction were tested in bending. A modified version of the four point bending tested was used for the flexural test set-up. The actuators used to apply force were located on the outside of the supports; this set-up produced upward deflection in the specimen producing tension on the top surface of the specimen. Tension on the top surface of the specimen produced safer conditions for observing cracks and crack propagation. This set-up, like the four point bending test, produced a constant maximum moment between the supports where the joint was located. Figure 9 shows the experimental set up used to test the longitudinal joint specimens (Flexural Test Set-Up).

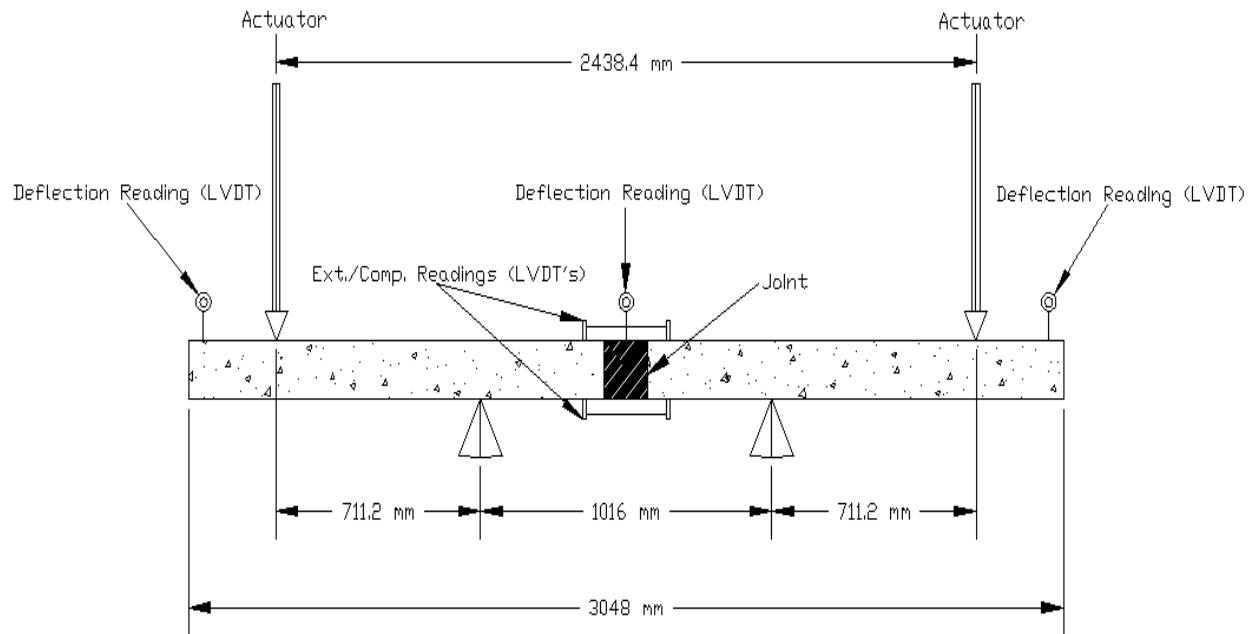


**Figure 9: Flexural Test Set-Up (Longitudinal Joint Test)**

As stated previously the specimens representing the transverse joint direction were tested in tension. The tension test set-up was slightly more complicated than the flexural test set-up. The longitudinal reinforcement in the transverse joint specimens was welded to 19.05 mm ( $\frac{3}{4}$ " ) threaded rods. These threaded rods were used to bolt the tension specimen to support and loading beams. The support beam was connected to the specimens and then placed on top of the load frame. The support beam was then braced and clamped into position, so it would remain stationary. The loading beam was then connected to the specimen and the actuators. The actuators pushed the loading beam down, which applied a tension force to the specimens. Figure 10 shows the tension test set-up.

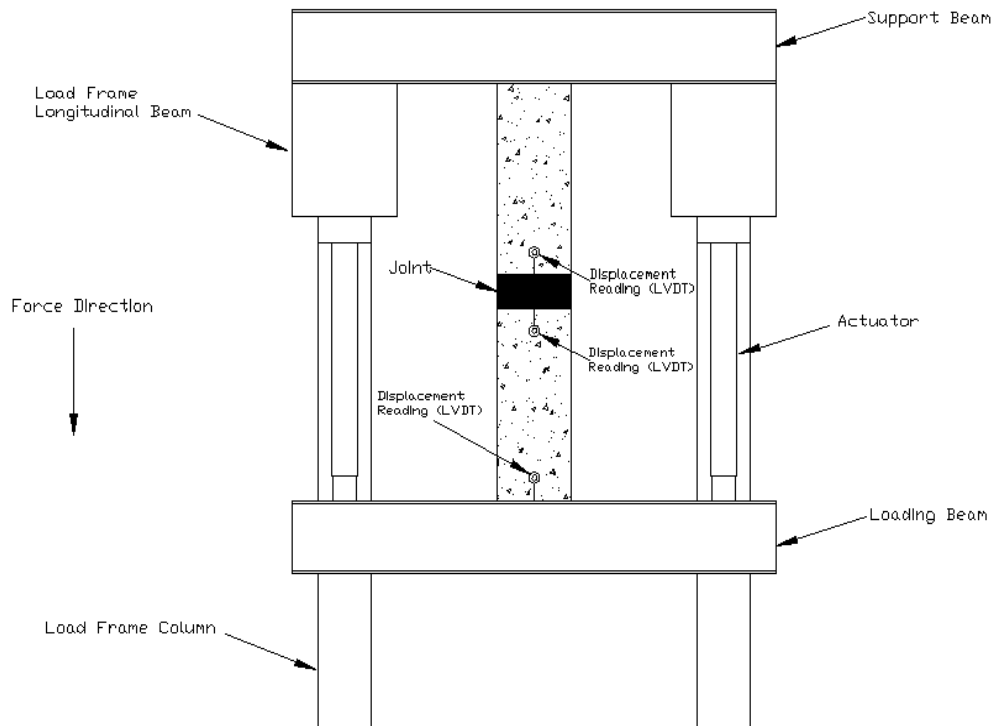


**Figure 10: Tension Test Set-Up (Transverse Joint)**

### ***Instrumentation***

The specimens were fully instrumented to achieve a better understanding of the u-bar and headed bar details in bending and tension. Load cells built into the MTS actuators were used to measure the loads that were applied to the specimens. LVDT's and the locations of the actuator pistons were used to measure the deflection of the specimens. Also, horizontally placed LVDT's were used to calculate the curvature of the longitudinal joint specimens tested in flexure. The strain in the joint reinforcement was measured using strain gages.

As stated previously, strain gages were attached to the joint reinforcement. The strain gages allow for direct strain readings of the rebar in the joint. The main purpose of the strain gages was to determine if the two joint details would yield the rebar in the joint, which would show that the joints would produce a precast deck system that would emulate monolithic behavior. The strain gage configuration was based on the previously determined development length and lap splice length of the u-bar and the headed bars. The development length that corresponded to the u-bars was 199.14 mm (7.84") and the lap splice length that corresponded to the headed bars was 127.51 mm (5.02"). These lengths were used as a reference point for the installation of the gages. Strain gages were placed on either side of these reference points so that the strain along the length of the reinforcement could be measured and the location of rebar yielding could be determined. For the u-bar detail, strain gages were installed on the top and bottom of the three interior u-bars. These gages were installed 50.8 mm (2"), 152.4 mm (6"), 203.2 mm (8") and 254 mm (10") away from the bend of the bar. Also, a strain gage was installed on the outside apex of the bend on each strain gauged u-bar. For the headed bar joint detail, strain gages were installed on the top and bottom headed bars on the three interior sets of

headed bars. The term headed bar set refers to the top and bottom headed bars at a specific location. These gages were installed 25.4 mm (1"), 101.6 mm (4"), 152.6 mm (6"), and 203.2 mm (8") away from the bearing surface of the head on headed bar sets two and three. The strain gages were installed 101.6 mm (4"), 152.4 mm (6"), 203.2 mm (8") and 254 mm (10") away from the bearing surface of the head on headed bar set 4. Figure 11 shows the strain gage configuration used for the u-bar detail. Figure 12 shows the strain gage configuration used in the headed bar detail.

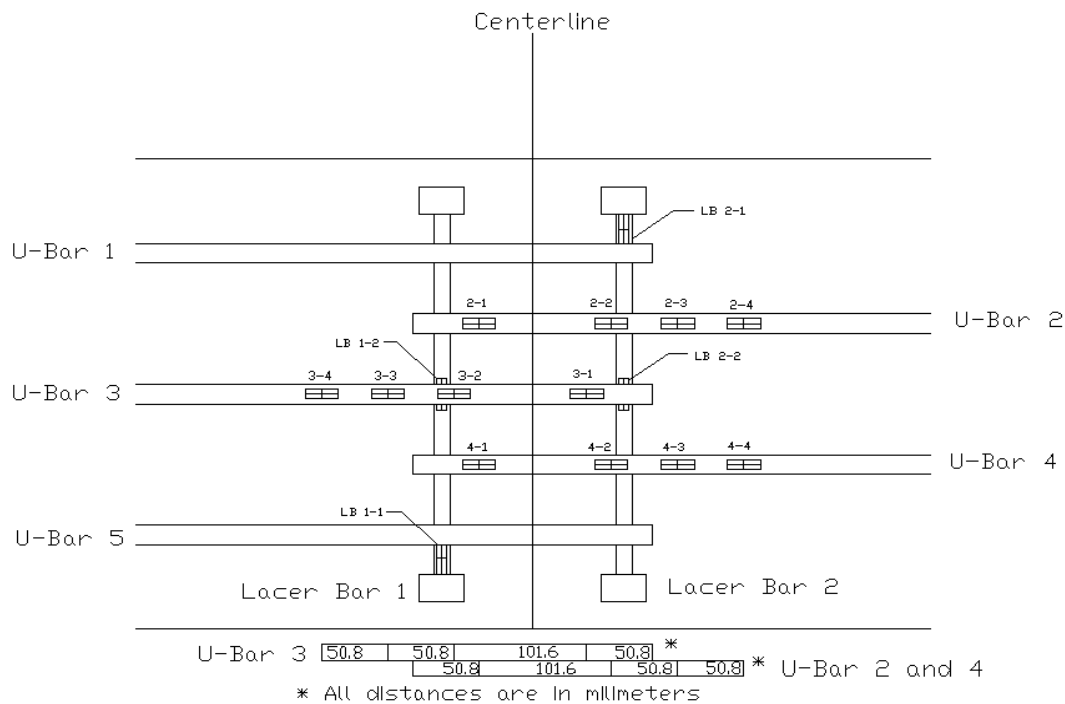


**Figure 11: U-bar Joint Detail Strain Gage Configuration**

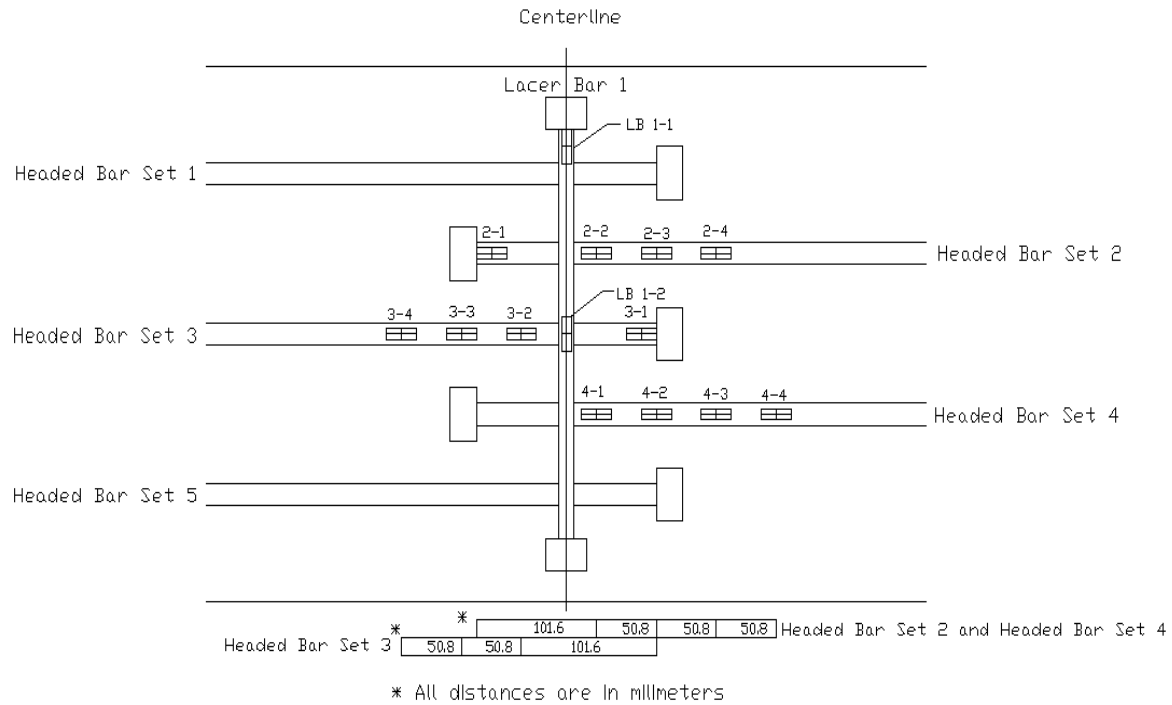


**Figure 12: Headed Bar Joint Detail Strain Gage Configuration**

The strain gage notation used in Figures 11 and 12 indicate the u-bar or the headed bar set where the gage is located and the relative position of that gage. For example, strain gage 2-3 indicates that the gage is located on u-bar 2 or headed bar set 2 and that it is the third gage away from the bearing surface of that bar. In the strain gage results section of this paper, the strain gages are labeled additionally with a “T” or “B” indicating that they are located on the top or bottom of a u-bar or located on the top or bottom bar of a headed bar set. The distances to the centerlines of the strain gages are given at the bottom of Figures 11 and 12 in millimeters. The first length given for each bar is the distance from the centerline of the first gage to the bearing surface of the reinforcement. The other distances shown in the figures represent the center to center spacing between consecutive strain gages in millimeters.

Strain gages were also installed on the transverse lacer bars. A strain gage was installed 25.4 mm (1”) away from the bearing surface of the head, and another strain gage was installed in

the center. All lacer bars used for the construction of the specimens were gauged. Figure 13 shows the strain gage configuration of the lacer bars.

LVDT's were used to determine specimen deflections at various locations. For the longitudinal joint bending specimens, LVDT's were installed in the center of the specimen and at both ends. For the transverse joint tension specimens, LVDT's were installed on the top and the bottom of the joint to measure joint elongation and at the bottom of the specimen to measure the total deflection of the specimens.

The curvature of the longitudinal joint bending specimens was also measured using LVDT's. The LVDT's were installed parallel to the specimen on the top and the bottom of the joint zone. Top and bottom surface strains could then be calculated based on the initial gage length of the LVDT wire and the change in the readings. These surface strains were then used to determine the curvature of the specimens throughout the duration of the test. Figures 9 and 10 show the positions of all previously discussed LVDT's.

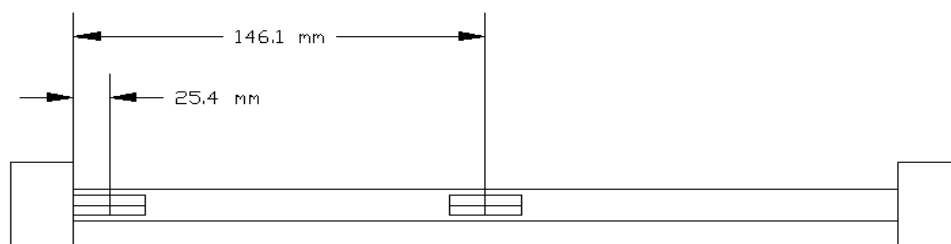


**Figure 13: Lacer Bar Strain Gage Configuration**

## **Chapter 3 Specimen Construction, Reinforcement Cost and Fabrication**

### ***Specimen Construction***

The first step of the specimen construction process was building wood forms so the specimens could be cast. The inside dimensions of the forms were built to the specimen dimensions. The sides and ends of the forms were constructed using 50.8 mm X 254 mm (2" X 10") lumber for the transverse tension specimens and 50.8 mm X 203.2 mm (2" X 8") lumber for the longitudinal flexure specimens. Two different lumber sizes were used, due to the different specimen thicknesses. The form construction for the transverse joint specimens also included drilling holes in the ends of the forms, so that the threaded rods attached to the reinforcement could be extended out of the specimen. The bottoms of the forms were constructed from 9.53 mm (3/8") plywood. A form was built for each specimen, so that several specimens could be cast at the same time.

When strain gage installation was completed, the reinforcement was tied and placed into the forms. The centerline of the joint and the positions for the bearing surfaces of the reinforcement were determined. The longitudinal reinforcement was then put in the correct position in the forms. Rebar chairs were used to give the reinforcement the proper concrete cover. At least 50.8 mm (2") of top concrete cover and 25.4 mm (1") of bottom concrete cover were provided for the reinforcement. Special attention was given to the spacing between the top and bottom layers of longitudinal specimen reinforcement, so that the proper rebar layer separation was provided. The u-bars had the proper separation at the bend, but the separation increased along the length of the bar away from the bend. Thin steel plates of the correct height were tack welded to the free ends of the u-bars to ensure proper layer separation. Proper rebar separation was provided for the headed bar detail in the same way. All plates welded to the

reinforcement were placed away from the joint zone to ensure no interference would take place. The transverse specimen reinforcement was then tied to the longitudinal reinforcement at the correct spacings and the lacer bars were installed in the joints.

The forms with the tied reinforcement were then transported to Ross Prestressed Concrete, Inc. where the concrete was cast. Before the concrete was cast the rebar positions, spacings and overlap lengths were checked and adjusted if necessary. The concrete was then cast. The concrete was first cast into the joint zone, so that the weight of the concrete would prevent the reinforcement in the joint zone from moving throughout the rest of the casting. Concrete vibrators were used to consolidate the concrete and prevent “honey-combing” from occurring. The joint zone was only vibrated on the surface to prevent strain gage damage. After the concrete was finished, the specimens were covered with a tarp and allowed to cure for two days before they were transported. The specimens containing the u-bar detail were cast on July 24, 2008 and the specimens containing the headed bar detail were cast on August 29, 2008. Figure 14 shows several specimens during the construction process.

As can be seen in Figure 14, the specimens were monolithically cast. In practice the joint would consist of precast deck panels with staggered, protruding reinforcement that would then be anchored into a cast-in-place joint. In this experimental program the behavior of the reinforcement of the different joint details was of main concern. The monolithic specimens allowed the behavior of the joint details to be observed without the additional variable of a grouted joint. Additional testing with the best performing detail will contain the grouted joint between the precast panel sections.

When comparing the constructability of the u-bar and the headed bar joint details, the u-bar detail seemed to be the easiest to construct for two reasons. The u-bar detail produces a joint

that is less congested than the headed bar detail and therefore allows for easier placement of precast deck components. The headed bar detail is more congested due to the bearing heads. Instead of the diameter of the bar being placed in the rebar spacing, which is the case for the u-bar detail, the outside diameter of the bearing head must be placed in the same rebar spacing. The head reduces the construction tolerances, which may lead to placement problems in the field. The reinforcement of the u-bar detail was also easier to tie and set in place when compared to the headed bar detail. After the top and bottom of the u-bars were set to the correct height by tack welding a thin plate between the free ends, the u-bars acted as a single reinforcement cage, which made the installation of the rebar easier than the two separate layers of reinforcement produced by the headed bar detail.

During shipment, storage, and placement of precast deck components, bending of the protruding joint reinforcement is a concern. The u-bar detail would also provide benefits when considering this aspect. In the headed bar detail, one headed bar could be accidentally bent at a time, but for the u-bar detail two bars would have to be bent at one time, because the top and bottom layer of reinforcement consists of one bent bar. The bending of two bars at once would require a greater force and would therefore be more unlikely to happen. Also, the possibility of bending a mesh of welded wire reinforcement or stainless steel to form the u-bar bend and at the same time a single rebar cage ready for placement. This would reduce construction time in the precast plant, which would reduce overall project cost.



**Figure 14: Specimen Construction**  
**(a) Reinforcement in the Forms**



**(b) Plate Used to Ensure Proper Layer Separation**



**(c) Threaded Rods Extending through the Form**



**(d) Pouring Concrete**



**(e) Vibrator Use in the Joint Zone**



**(f) Concrete Finishing**

## ***Reinforcement Cost and Fabrication***

In February 2009, the reinforcement suppliers were contacted to determine an approximate cost of reinforcing materials and their fabrication. The cost information received from the suppliers was a representative snap shot of the reinforcement prices at that particular time for construction scale orders. The ease of fabrication of the u-bars was also discussed with the suppliers.

The lowest price quote was given for conventional A615 rebar with attached Lenton Terminator<sup>®</sup> bearing heads. The price per ton of this reinforcement was 800 dollars with an additional cost of 25 dollars for the application of each Lenton Terminator<sup>®</sup> bearing head. Welded wire reinforcement was very competitive with the headed bar price. Two price ranges were given for fabricated welded wire reinforcement, the first price of 850-950 dollars a ton was given for single cut and fabricated wires. The second price range that was given for fabricated wire mesh, which was 900-1000 dollars a ton. The highest price was the price of fabricated stainless steel reinforcement. The price quote given was for Enduramet 32 stainless steel, which was 5000 dollars a ton. The stainless steel price also includes fabrication in a stainless steel only production line which eliminates contamination from black carbon dust. Even though stainless steel has the largest initial material cost, the life cycle cost of a deck using this material could offset the high initial cost.

The ease of the u-bar fabrication process was also discussed with the reinforcement distributors. The u-bar fabrication process was of concern, because of the tight bends that were required in the u-bar detail so the deck thickness could be minimized. The stainless steel supplier stated that the tight bends were not a problem for the stainless steel material, because of its high ductility. The stainless steel supplier also stated that it may be possible to reduce the bend

diameter to less than three times the diameter of the bar without breaking the material. The welded wire reinforcement supplier stated that the tight bend was also not a problem for the material. After determining the best fabrication method for the bends, no welded wire reinforcement was broken during fabrication.

## **Chapter 4 Constituent Testing**

### ***Concrete Testing***

The six specimens that were tested were cast at two different times. All the specimens containing the u-bar detail, a total of four specimens, were cast on July 24, 2008. The two specimens containing the headed bar detail were cast on August 28, 2008. In order to obtain accurate concrete strengths, six concrete cylinders were made at each concrete pour. Three cylinders were used to obtain the 7 day concrete strength, and three cylinders were used to obtain the 28 day concrete strength of each concrete pour.

15.24 cm (6”) by 30.48 cm (12”) concrete cylinders were tested using ASTM C 39 specifications to determine the compressive strength of the concrete used for the construction of the specimens (ASTM C39, 2005). The concrete cylinders were loaded continuously without shock at a rate of 4893 N/sec (1100 lbs/sec), which is within the required loading rate of  $255.1 \pm 48.3$  kPa/sec ( $35 \pm 7$  psi/sec). The concrete cylinders were loaded until failure and the compressive force and stress were recorded. Tables 6 and 7 show the results of the concrete compressive strength tests for the two concrete pours.

**Table 6: Concrete Compressive Strengths, U-bar Specimens**

| <b>U-bar Specimens (Cast on July 24, 2008)</b> |                  |                |              |             |                  |                 |              |
|------------------------------------------------|------------------|----------------|--------------|-------------|------------------|-----------------|--------------|
| 7 Day Test                                     |                  |                |              | 28 Day Test |                  |                 |              |
| Cylinder #                                     | Force (lb)       | Stress (psi)   | Stress (MPa) | Cylinder #  | Force (lb)       | Stress (psi)    | Stress (MPa) |
| 1                                              | 246270           | 8710           | 60.05        | 1           | 292630           | 10350           | 71.36        |
| 2                                              | 275750           | 9753           | 67.24        | 2           | 329430           | 11651           | 80.33        |
| 3                                              | 277660           | 9820           | 67.71        | 3           | 327270           | 11575           | 79.81        |
| <b>Average</b>                                 | <b>266560.00</b> | <b>9427.67</b> | <b>65.00</b> |             | <b>316443.33</b> | <b>11192.00</b> | <b>77.17</b> |

**Table 7: Concrete Compressive Strengths, Headed bar Specimens**

| <b>Headed bar Specimens (Cast on August 28, 2008)</b> |                  |                |              |             |                  |                |              |
|-------------------------------------------------------|------------------|----------------|--------------|-------------|------------------|----------------|--------------|
| 7 Day Test                                            |                  |                |              | 28 Day Test |                  |                |              |
| Cylinder #                                            | Force (lb)       | Stress (psi)   | Stress (MPa) | Cylinder #  | Force (lb)       | Stress (psi)   | Stress (MPa) |
| 1                                                     | 234710           | 8301           | 57.23        | 1           | 281270           | 9948           | 68.59        |
| 2                                                     | 228110           | 8068           | 55.63        | 2           | 265310           | 9383           | 64.69        |
| 3                                                     | 229420           | 8114           | 55.94        | 3           | 266220           | 9416           | 64.92        |
| <b>Average</b>                                        | <b>230746.67</b> | <b>8161.00</b> | <b>56.27</b> |             | <b>270933.33</b> | <b>9582.33</b> | <b>66.07</b> |

### ***Reinforcing Materials Testing***

Tension tests were performed on the welded wire reinforcement and stainless steel reinforcement to obtain accurate material properties. The conventional grade 60 reinforcement used for the headed bars was not tested. Four samples of each reinforcing material were tested. An Instron Universal Testing Machine (UTM) was used to test the reinforcement and to obtain the data necessary to construct stress verses strain curves for both reinforcement types.

An extensometer was used to accurately obtain the modulus of elasticity of each reinforcement. The extensometer was clamped to the specimens at the required 50.8 mm (2") gage length. The extensometer then used the change in the 50.8 mm (2") gage length to determine the strain in the reinforcing materials. When the yield strength of the reinforcing materials was approached, the extensometer was removed as a precautionary measure. The remainder of the strain readings was determined by using an initial gage length of the entire

specimen and the position of the moving platen of the UTM. The stress in the specimens was calculated by using the force readings taken from the load cell in the UTM and simply dividing it by the area of the #5 bars tested. This allowed for the construction of a stress versus strain curve for both the welded wire reinforcement and the stainless steel reinforcement.

As stated previously, to obtain an accurate value for the modulus of elasticity for the welded wire reinforcement and the stainless steel reinforcement, an extensometer was used to obtain strain data. The modulus of elasticity of both reinforcement types was found by plotting the extensometer strain readings versus the calculated reinforcement stress in Microsoft Excel and plotting the trend lines of curve. The slope of the trend line represented the modulus of elasticity of the reinforcing material. Figure 15 shows the stress versus strain curves of both reinforcing materials and the corresponding trend lines.

The modulus of elasticity of welded wire reinforcement (WWR) was determined to be 187461 MPa (27189 ksi). The modulus of elasticity of the stainless steel reinforcement (SS) was determined to be 212382 MPa (30799 ksi). A modulus of elasticity of 199948 MPa (29000 ksi) was used in all theoretical calculations, even though the actual values were known.

The total average stress versus strain curve for the WWR and the SS was also plotted and can be seen in Figure 16. Also, Figure 16 shows that the stainless steel reinforcement is extremely ductile when compared to the welded wire reinforcement.

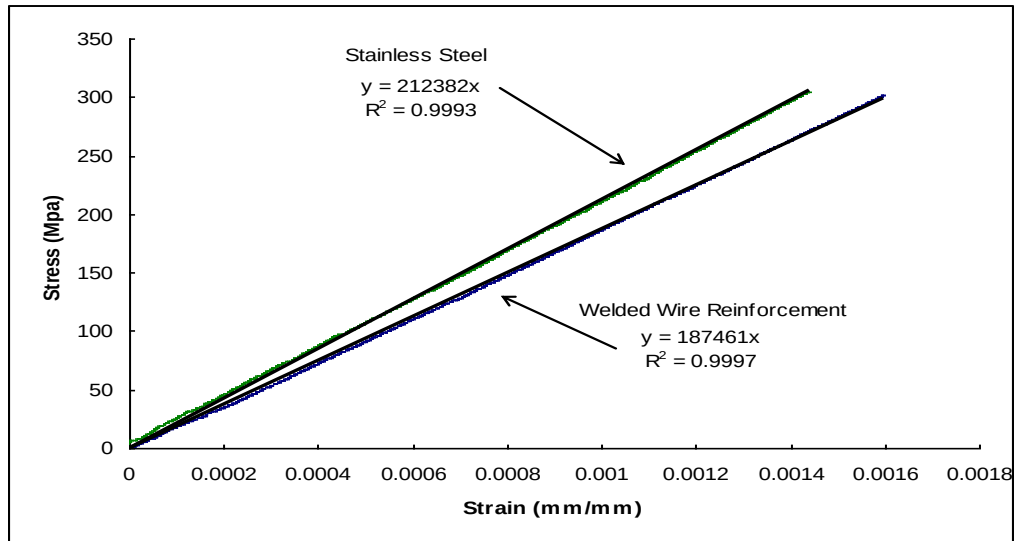


**Figure 15: Determination of Modulus of Elasticity**

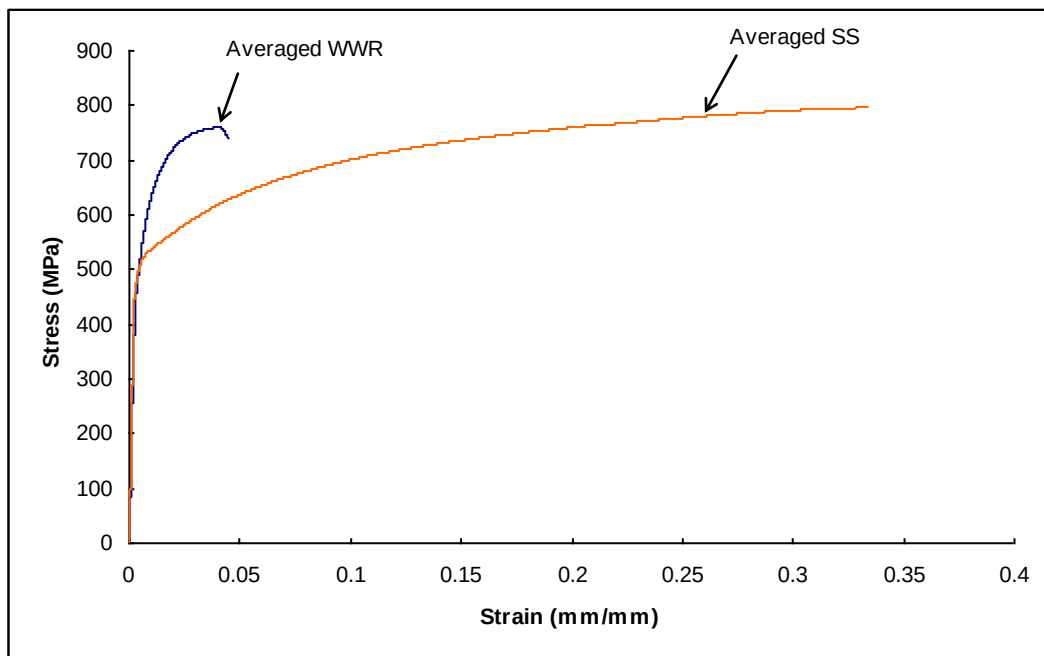


**Figure 16: Average Stress Verses Strain Curves**

### ***Weld Testing***

A preliminary concern of the tension test set-up was the strength of the welds between the longitudinal rebar in the specimens and the threaded rods that were used to connect the specimens to the loading and support beams. If the welds were to fail before the specimen, then the test would be invalid. Tension tests were performed on the rebar to threaded rod welds to ensure that the weld strengths were greater than the rebar yield strengths. The top connection detail used in the tension test set-up is shown in Figures 17 and 18. Figure 19 shows the weld test set-up.

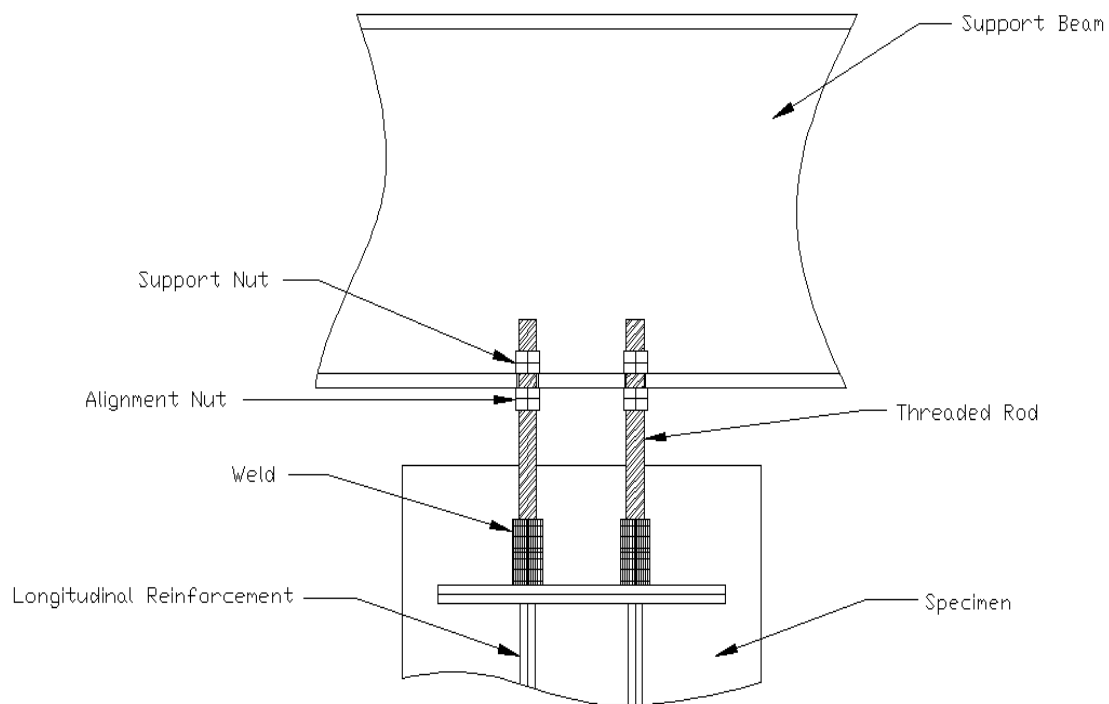


**Figure 17: Connection Detail, Conceptual Drawing**



**Figure 18: Photo of the Top Connection Detail**

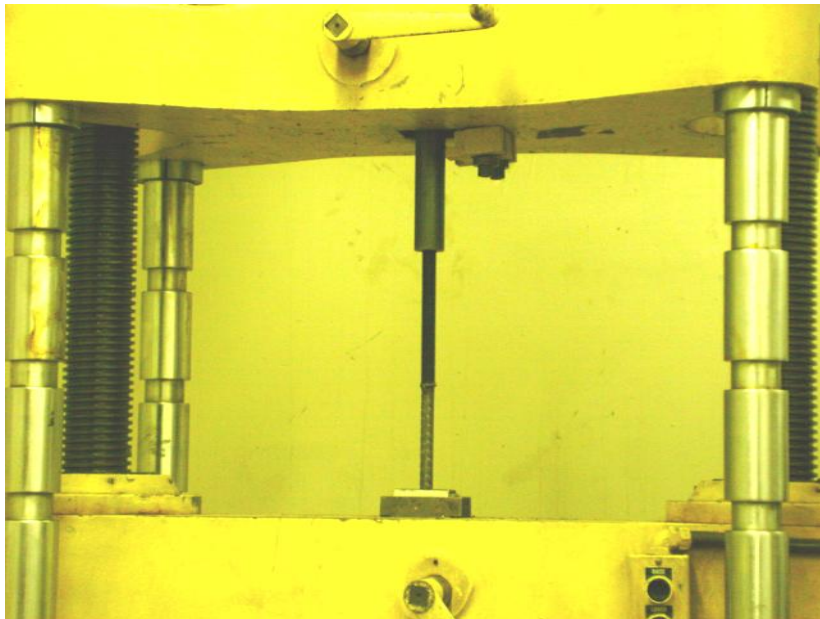


**Figure 19: Weld Test Set-Up**

The welds tested were made by using the MIG, TIG and SMAW welding methods on all rebar materials. Again, conventional reinforcement was used with a yield strength of 420 MPa (60 ksi), and welded wire reinforcement and stainless steel reinforcement were used with a yield strength of 520 MPa (75 ksi). The threaded rods that were used in the experimental program had a diameter of 19.05 mm ( $\frac{3}{4}$ " ) and a minimum yield strength of 758 MPa (110 ksi).

The first set of weld tests that were conducted were one pass welds made by both MIG and TIG welding methods with a 482.6 MPa (70 ksi) carbon welding wire. This welding method produced a small ring of weld material around the outside of the threaded rod and the rebar, and did not fuse the threaded rod and reinforcement together in the center. This welding method produced small cross sectional areas in the weld region, which led to small capacities. Table 8 shows the testing results of the one pass weld tests and Figure 20 shows the typical failure of the one pass weld specimens.

**Table 8: Weld Test Results, One Pass Welds**

| Material                                            | Welding Method | Capacity (kips) | Capacity (kN) | Failure Mode             |
|-----------------------------------------------------|----------------|-----------------|---------------|--------------------------|
| Stainless Steel, $f_y = 520$ MPa (75 ksi)           | MIG            | 15.5            | 68.9          | Weld Broke               |
| Welded Wire Reinforcement, $f_y = 520$ MPa (75 ksi) | MIG            | 13              | 57.8          | Rebar Pulled Out of Weld |
| Conventional Rebar, $f_y = 420$ MPa (60 ksi)        | MIG            | 13.7            | 60.9          | Rebar Pulled Out of Weld |
| Stainless Steel $f_y = 520$ MPa (75 ksi)            | TIG            | 11.8            | 52.5          | Weld Broke               |
| Welded Wire Reinforcement $f_y = 520$ MPa (75 ksi)  | TIG            | 15              | 66.7          | Weld Broke               |
| Conventional Rebar $f_y = 420$ MPa (60 ksi)         | TIG            | 15.8            | 70.3          | Weld Broke               |



**Figure 20: One Pass Weld Failure**

The second set of weld tests consisted of the same welding methods and materials, but the threaded rods and rebar were beveled, so larger weld areas and capacities would be produced. This set of weld tests all produced capacities that were larger than the 103.4 kN (23.25 kips) force required to yield a #5 bar with a yield strength of 520 MPa (75 ksi). These results indicated that the welds could yield the reinforcing material, but to account for any unforeseen forces in the experiment, larger weld capacities were preferred. Table 9 shows the results of the second set of weld tests using the beveled weld geometry.

The third set of weld tests used a 758.4 MPa (110 ksi) welding stick and the beveled weld geometry. The welding method was also changed to shielded metal arc welding (SMAW). These welds were tested on all reinforcing materials. The results of the third set of welding tests can be seen in table 10.

**Table 9: Weld Test Results, Beveled Welds**

| <b>Material</b>                                     | <b>Welding Method</b> | <b>Capacity (kips)</b> | <b>Capacity (kN)</b> | <b>Failure Mode</b> |
|-----------------------------------------------------|-----------------------|------------------------|----------------------|---------------------|
| Stainless Steel, $f_y = 520$ MPa (75 ksi)           | MIG                   | 23.8                   | 105.9                | Weld Broke          |
| Welded Wire Reinforcement, $f_y = 520$ MPa (75 ksi) | MIG                   | 26.7                   | 118.8                | Weld Broke          |
| Conventional Rebar, $f_y = 420$ MPa (60 ksi)        | MIG                   | 26.5                   | 117.9                | Weld Broke          |
| Stainless Steel $f_y = 520$ MPa (75 ksi)            | TIG                   | 25.9                   | 115.2                | Weld Broke          |
| Welded Wire Reinforcement $f_y = 520$ MPa (75 ksi)  | TIG                   | 24.5                   | 109.0                | Weld Broke          |
| Conventional Rebar $f_y = 420$ Mpa (60 ksi)         | TIG                   | 23.8                   | 105.9                | Weld Broke          |

**Table 10: Weld Test Results, Beveled Welds and 110 ksi Welding Stick**

| <b>Material</b>                                     | <b>Welding Method</b> | <b>Capacity (kips)</b> | <b>Capacity (kN)</b> | <b>Failure Mode</b> |
|-----------------------------------------------------|-----------------------|------------------------|----------------------|---------------------|
| Stainless Steel, $f_y = 520$ Mpa (75 ksi)           | SMAW                  | 21.4                   | 95.2                 | Weld Broke          |
| Welded Wire Reinforcement, $f_y = 520$ MPa (75 ksi) | SMAW                  | 33.5                   | 149.0                | Weld Broke          |
| Conventional Rebar, $f_y = 420$ Mpa (60 ksi)        | SMAW                  | 29.5                   | 131.2                | Weld Broke          |

Table 10 shows that the 758.4 MPa (110 ksi) welding sticks and the SMAW welding method produced very strong welds with the welded wire reinforcement and the conventional rebar, but the strength of the weld decreased for the stainless steel reinforcement. The decrease in weld strength for the stainless steel reinforcement may have been because of a weld incompatibility between the 758.4 MPa (110 ksi) welding stick and the stainless steel reinforcement. So the threaded rod to stainless steel weld was tested again using a stainless steel welding stick.

The SMAW welding method using a 308 stainless steel welding rod was used in conjunction with the beveled weld geometry to increase the weld strength between the threaded rod and the stainless steel reinforcement. Only one weld test was conducted using these parameters, due to lack of time and an adequate result. The capacity of the weld using the previously described perimeters produced a strength of 126.3 kN (28.4 kips), which would provide adequate strength of the experiment.

The results of the weld tests indicated that the previously described tension test set-up was possible and that the weld strength between the threaded rod and the longitudinal reinforcement was not a limiting factor.

## Chapter 5 Results and Discussion

### ***Moment Capacity***

The specimens HB-1, SB-1 and WB-1 were tested in flexure. HB-1 corresponds to the first specimen containing the headed bar joint detail that was tested in bending. SB-1 and WB-1 correspond to the first specimens utilizing the u-bar joint detail that were tested in bending. Specimen SB-1 used the stainless steel reinforcing material and specimen WB-1 used the welded wire reinforcement material.

The AASHTO strength I and service limit states were calculated for both the positive and negative moments. A 50.8 mm (2") future wearing surface, the 158.7 mm (6.25") deck thickness and the live load, which was determined from AASTHO Table A4-1, were used to calculate the following moments. The strength I and service limit state unit moments are shown in Table 11.

**Table 11: Required Service and Strength 1 Limit State Unit Moments**

|            | $M^+$ (kN-m/m) | $M^+$ (kip-ft/ft) | $M^-$ (kN-m/m) | $M^-$ (kip-ft/ft) |
|------------|----------------|-------------------|----------------|-------------------|
| Service    | 42.3           | 9.5               | 34.9           | 7.8               |
| Strength I | 71.0           | 16.0              | 58.1           | 13.1              |

The moments shown in Table 11 are unit moments or the moment applied over a 304.8 mm (1') wide strip of deck. Since the specimens were not 304.8mm (1') wide and the reinforcement areas were slightly altered from the deck designs, the moments in Table 11 do not exactly correspond to the service and strength limit state moments of the specimens. An area of steel ratio was used to determine the service and strength limit state moments that would correspond to the specimens. The area of steel ratio was calculated by dividing the actual area of steel in the specimens by the area of steel required by design. The area of steel ratio was then used to adjust the unit moments, so that they would correspond to the specimens. The area of steel ratios were determined to be 1.06 for the specimens containing the u-bar detail and 1.0 for the specimens containing the headed bar joint detail. This shows that the reinforcement in the specimens closely represents the reinforcement required for a 304.8mm (1') wide strip of deck. The required moments only changed slightly for the u-bar specimens and remained the same for the headed bar specimens. Table 12 and 13 shows the service and strength I moments for the specimens.

**Table 12: Modified Specimen Moments, Specimens Containing the U-bar Detail**

| U-Bar Detail Specimens |                       |                         |                       |                         |
|------------------------|-----------------------|-------------------------|-----------------------|-------------------------|
|                        | M <sup>+</sup> (kN-m) | M <sup>+</sup> (kip-ft) | M <sup>-</sup> (kN-m) | M <sup>-</sup> (kip-ft) |
| Service                | 13.7                  | 10.1                    | 11.3                  | 8.3                     |
| Strength I             | 23.0                  | 16.9                    | 18.5                  | 13.6                    |

**Table 13: Modified Specimen Moments, Specimens Containing the Headed Bar Detail**

| Headed Bar Detail Specimen |                       |                         |                       |                         |
|----------------------------|-----------------------|-------------------------|-----------------------|-------------------------|
|                            | M <sup>+</sup> (kN-m) | M <sup>+</sup> (kip-ft) | M <sup>-</sup> (kN-m) | M <sup>-</sup> (kip-ft) |
| Service                    | 12.9                  | 9.5                     | 10.6                  | 7.8                     |
| Strength I                 | 21.7                  | 16.0                    | 17.8                  | 13.1                    |

All specimens produced similar flexural capacities. Specimen HB-1, which contained the headed bar joint detail, produced the lowest flexural capacity, which was 39.48 kN-m (29.12 kip-ft). Specimen WB-1, which contained the u-bar detail made of welded wire reinforcement, produced a flexural capacity of 42.1 kN-m (30.98 kip-ft). The largest flexural capacity was produced by specimen SB-1, which contained the u-bar joint detail made of the stainless steel material; the flexural capacity of this specimen was 43.22 kN-m (31.88 kip-ft).

When comparing the flexural capacities of the specimens and the modified required capacities shown on page 45 in table 11, it can be seen that all specimens produced capacities that were much greater than the modified required strength I limit state moments. The flexural capacities of the specimens show that the joint details, both u-bar and headed bar details, are capable of transmitting moment. The moment capacities show that the joint details can produce a precast deck system that can emulate the behavior of a cast-in-place deck system.

Moment verses deflection and moment verses curvature curves were constructed for each specimen. The total applied moment was determined from the actuator readings. The deflection data were taken from the LVDT located in the center of the joint. The curvature data were taken from the horizontally placed LVDT's located above and below the joint. Figure 21 shows the moment verses deflection curves and Figure 22 shows the moment verses curvature curves for all of the flexural specimens.

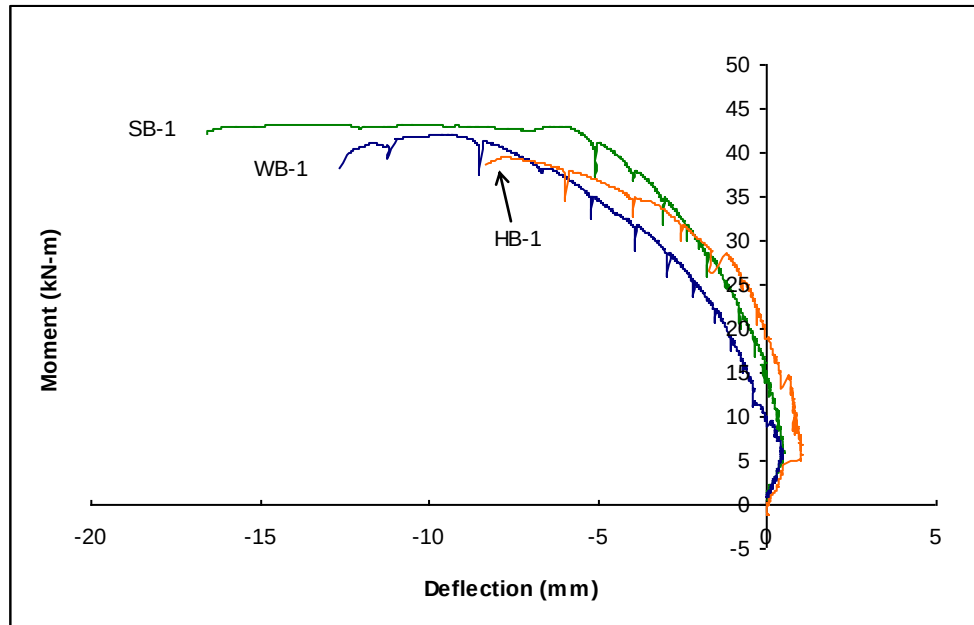


**Figure 21: Moment Verses Deflection Curves**

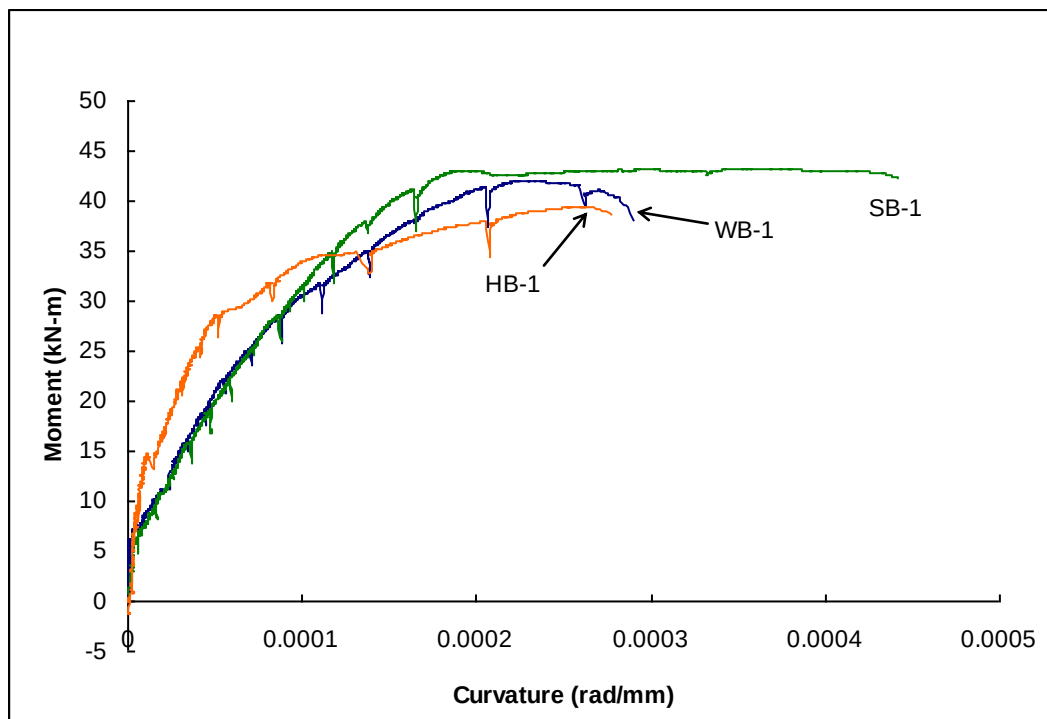


**Figure 22: Moment Verses Curvature Curves**

From Figures 21 and 22, the ductility of the specimens can be seen. Figure 21 shows that the u-bar specimens, SB-1 and WB-1, produce larger deflections than the headed bar detail contained in specimen HB-1. The same trend is also shown in figure 22. Specimens SB-1 and WB-1 produced larger curvatures than HB-1. Even though specimens SB-1 and WB-1 were more ductile than HB-1, they did not sacrifice capacity.

The higher capacities of SB-1 and WB-1 can be attributed to the joint detail, the higher reinforcement yield strength, and the higher compressive strength of the concrete. The higher ductility produced by the u-bar specimens may have been due to the higher concrete strength used, but it may also be due to the reinforcement used and the u-bar joint detail. When comparing the ductilities of SB-1 and WB-1, it can be seen that specimen SB-1 has a significantly larger deflection and curvature. Specimens SB-1 and WB-1 have the same concrete strength, rebar yield strength, and joint detail; the only difference is the reinforcing materials used in them. SB-1 consisted of the stainless steel reinforcing material and WB-1 consisted of welded wire reinforcement. Stainless steel is extremely ductile when compared to welded wire reinforcement; this can be seen in Figure 16. The ductility of the stainless steel reinforcement produced the high ductility of SB-1.

Theoretical moment versus curvature curves were constructed, so that the behavior of the bending specimens could be compared to theoretical values. The theoretical moment versus curvature curves were constructed for both the u-bar detail and the headed bar detail. Two separate moment versus curvature curves had to be determined because of the different concrete strengths, rebar yield strengths and reinforcement configurations used in the specimens. The cracking moment was calculated considering the modulus of concrete rupture stated in ACI 318-05, the additional stiffness added to the cross sections by the rebar and the uncracked moment of

inertia. The moment at yield was calculated considering the cracked section moment of inertia and a steel strain equal to the steel yield strain. The steel yield strain was determined by dividing the yield stress by its corresponding modulus of elasticity. The nominal moment capacity was calculated based on ACI 318-08 procedures, which considered a rectangular concrete stress block and no strain hardening of the reinforcement. Both layers of reinforcement were considering during the calculation of all moments. The specimen cross sections used for the theoretical calculations consisted of the six bar side and can be seen in Figures 23 and 24. The calculated theoretical moments and corresponding curvatures can be seen in Table 12.

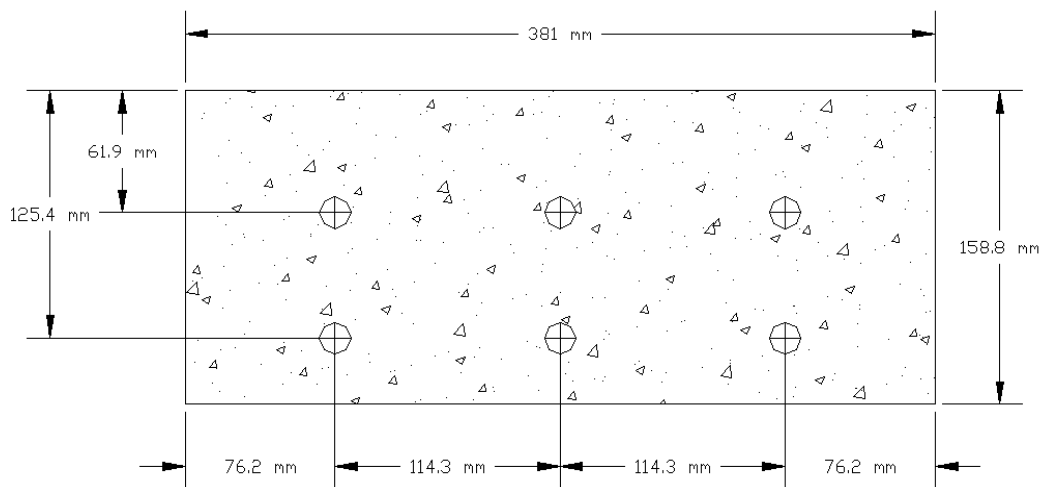


**Figure 23: Cross Section used for Theoretical Calculations of the U-bar Specimens**

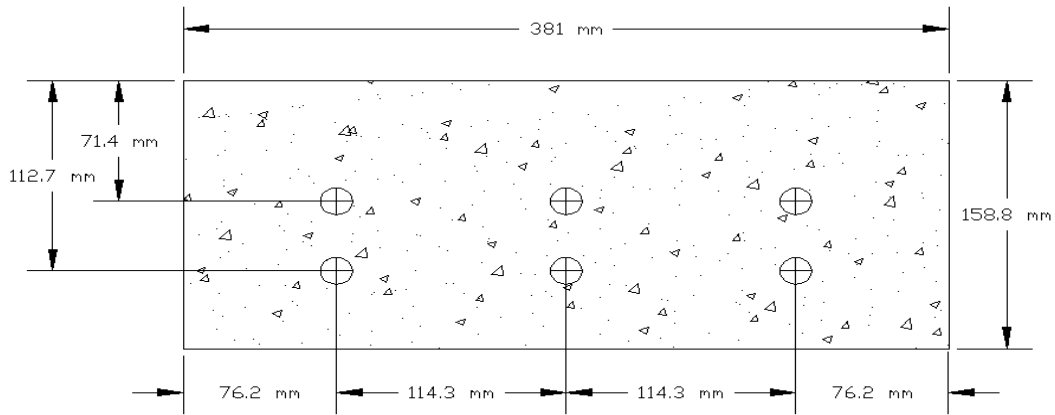


**Figure 24: Cross Section Used for the Theoretical Calculations of the Headed Bar Specimen**

**Table 12: Theoretical Moments and Curvature**

| Headed Bar Detail |       |                  |             | U-bar Detail  |       |                  |             |
|-------------------|-------|------------------|-------------|---------------|-------|------------------|-------------|
| Moment (kN-m)     |       | Curvature (1/mm) |             | Moment (kN-m) |       | Curvature (1/mm) |             |
| $M_{cr}$          | 8.39  | $\Phi_{cr}$      | 0.000001682 | $M_{cr}$      | 9.25  | $\Phi_{cr}$      | 0.000001684 |
| $M_y$             | 28.02 | $\Phi_y$         | 0.000028808 | $M_y$         | 34.98 | $\Phi_y$         | 0.000030326 |
| $M_n$             | 28.02 | $\Phi_n$         | 0.000084184 | $M_n$         | 34.98 | $\Phi_n$         | 0.000084425 |

When comparing the actual specimen moment capacities and the theoretical moment capacities, it can be seen on page 52, that the specimens behaved better than expected. For HB-1, the calculated moment was 28.02 kN-m (20.7 kip-ft), but the actual capacity was 39.48 kN-m (29.12 kip-ft). The actual capacity of HB-1 was 41 percent greater than the calculated value. SB-1 and WB-1 both had the same theoretical nominal moment capacity because they consisted of the same material properties and reinforcement configurations; the calculated moment capacity was 34.98 kN-m (25.8 kip-ft). The actual moment capacities of SB-1 and WB-1 were 43.22 kN-m (31.88 kip-ft) and 42.1 kN-m (30.98 kip-ft) respectively. The actual moment capacities of SB-1 and WB-1 were 23.5 and 20.4 percent greater than the calculated value.

The theoretical moment versus curvature curves were plotted with the corresponding moment versus curvature curves obtained from experimental data. Figure 25 shows the actual moment versus curvature curves of SB-1 and WB-1 plotted against the theoretical moment versus curvature curves calculated for those specimens. Figure 26 shows the actual moment versus curvature curve plotted against the theoretical moment versus curvature curve calculated for that specimen.

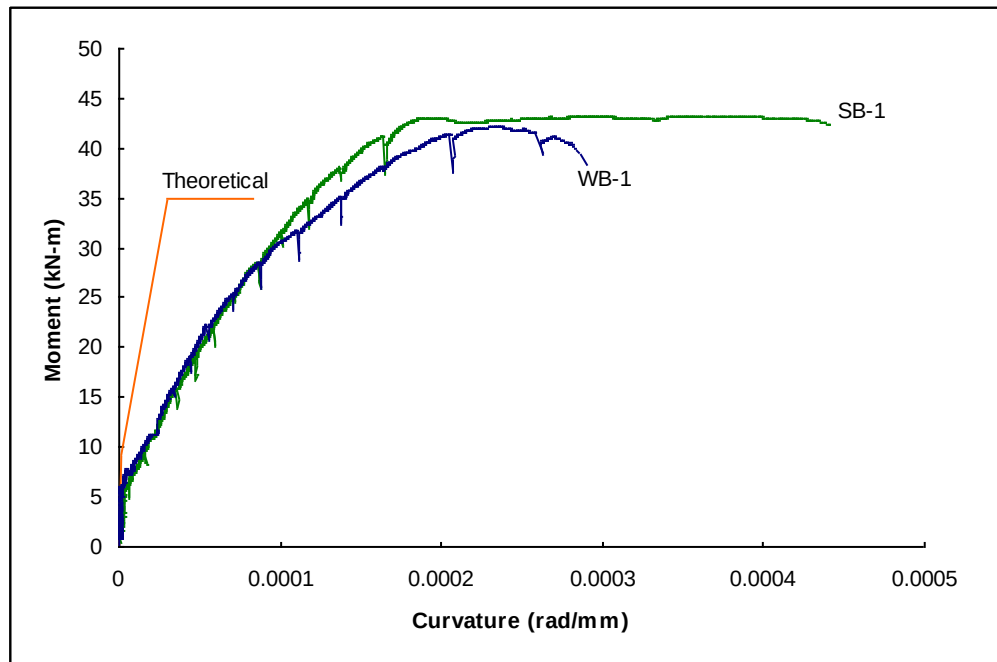


**Figure 25: Actual and Theoretical Moment Verses Curvature Curves for the U-bar Details**

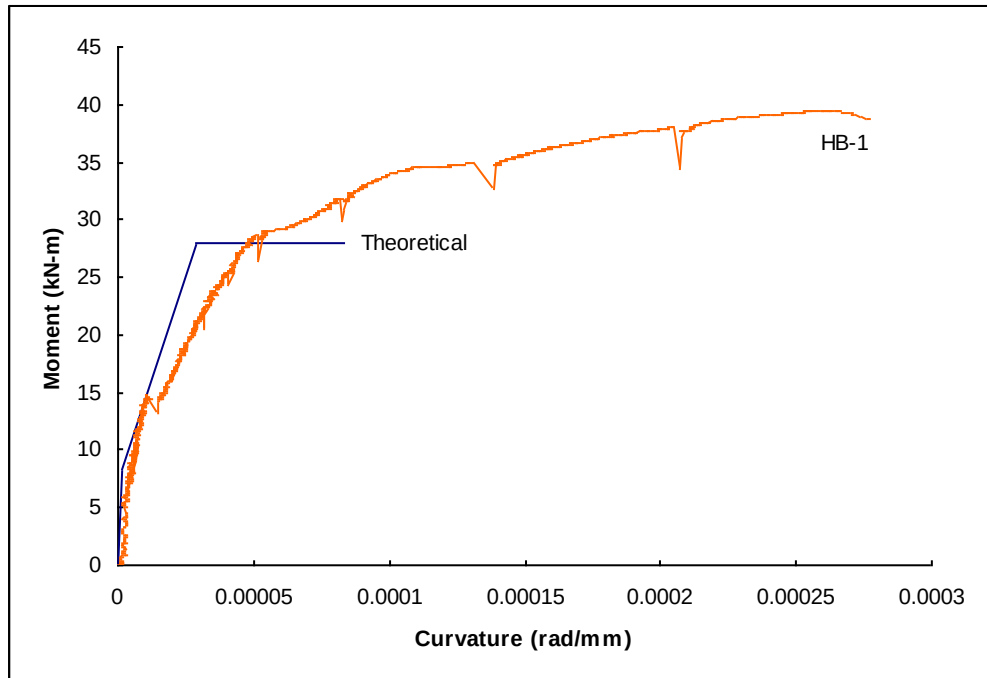


**Figure 26: Actual and Theoretical Moment Verses Curvature Curves for Headed Bar Details**

Figures 25 and 26 show that all specimens produced a larger capacity than the theoretical capacity. Also, the specimens were more ductile than the expected based on the theoretical curvature calculations. The behavior of the specimens shows that the u-bar and headed bar joint details produce a joint that can transmit moment and therefore emulate the behavior of a cast-in-place bridge deck.

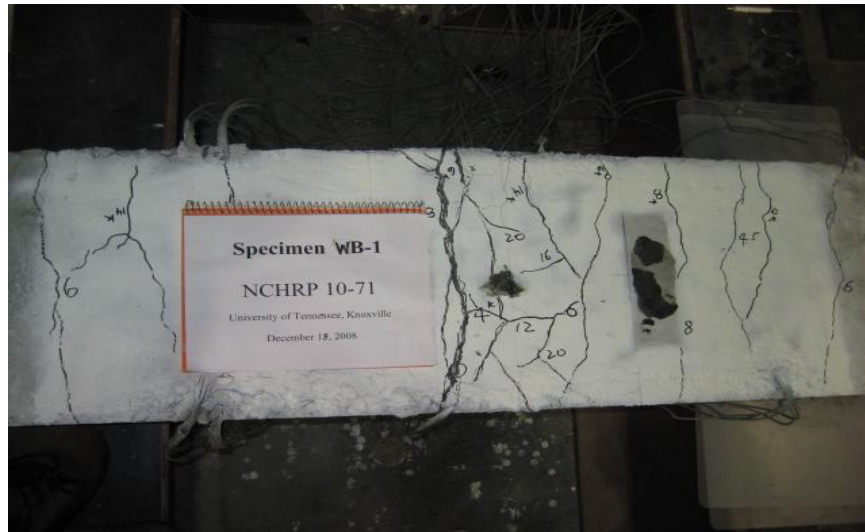
### ***Flexural Specimen Behavior***

The first cracks to appear in all the flexural specimens were transverse cracks uniformly spaced along the length of the specimens. Even though the joint zone had a higher area of steel when compared to the rest of the specimen's body, it also experienced transverse cracks at low moments. In the joint zone the transverse cracks formed at the ends of the longitudinal

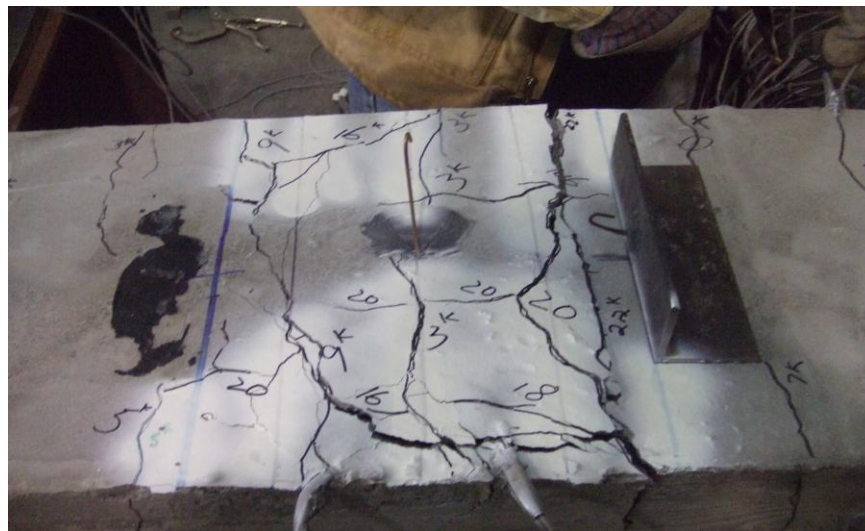
reinforcement. The transverse cracks formed in the top tension face of the specimens first as surface cracks and then propagated deeper into the specimens as the loading progressed. At higher moments, lateral cracks formed in the direction of the longitudinal reinforcement, which corresponds to the transverse reinforcement in an actual bridge deck. These lateral cracks formed over the longitudinal reinforcement that comprised the lightly reinforced half of the specimens. Diagonal cracks then formed from the longitudinal rebar comprising the lightly reinforced half of the specimens and extended to the outside edge of the specimen. Figure 27 shows the crack patterns of specimens SB-1, WB-1 and HB-1 at failure.



**Figure 27: Flexural Crack Patterns at Failure**  
**(a) Specimen SB-1**



**(b) Specimen WB-1**



**(c) Specimen HB-1**

The numbers written next to the cracks shown in Figure 27 represent the total force, in kips, applied to the specimen when the cracks formed.

All flexural specimen failures were ductile, producing yielding in the reinforcement and crushing of the concrete on the compression face of the specimens, under the joint zone.

### ***Flexural Crack Widths at Service Level Loading***

As previously stated, the service level moments of the specimens were determined from the service level moments calculated in the deck design and the steel area ratio. The positive service level moments were used when comparing the crack widths of the specimens. The positive service moments were used, because the bending specimens represent a longitudinal joint that would primarily resist positive moment. The positive service moments for the u-bar detail and the headed bar detail are 13.7 kN-m (10.1 kip-ft) and 12.9 kN-m (9.5 kip-ft), respectively. For specimens SB-1 and WB-1 crack widths were measured at 12.6 kN-m (9.29 kip-ft) and 15.8 kN-m (11.65 kip-ft). For specimen HB-1 the crack widths were measured at 11 kN-m (8.11 kip-ft) and 14.2 kN-m (10.47 kip-ft). Since the crack widths were not measured at the corresponding service moments, the crack widths at the service level loading were found by interpolating between the measured values. SB-1 was found to have an average crack width of 0.19 mm (0.00748 in) at service level loading. The average crack width at service level loading for specimen HB-1 was found to be 0.21 mm (0.0083 in). WB-1 was found to have an average crack width of 0.26 mm (0.01 in) at service level loading. So the u-bar detail produced both the largest and the smallest crack widths at service level loading, which were 0.26mm (0.01 in) and 0.19 mm (0.0075 in). The headed bar detail produced the mid-range crack widths.

Crack widths were measured visually by using a crack width gage. The crack width gage contained numerous lines of labeled widths. The cracks in the specimens were compared to the lines on the crack width gage; the width of the line that most accurately represented the crack was recorded as the crack width. Crack widths were measured systematically throughout the testing of all bending specimens.

### ***Tensile Capacity***

Specimens ST-1, WT-1 and HT-1 were tested in tension. Specimen HT-1 utilized a headed bar joint detail made of conventional reinforcement. Specimens ST-1 and WT-1 both utilized the u-bar joint detail. The reinforcement used in ST-1 was stainless steel, and the reinforcement used in specimen WT-1 was welded wire reinforcement.

The largest tension capacity that was expected from the tension specimens was the force determined by multiplying the area of steel of the lightly reinforced side of the specimens by the appropriate rebar yield strength. HT-1 was reinforced with conventional rebar with a yield strength of 420 MPa (60 ksi) and was expected to have a maximum tensile capacity of 330.9 kN (74.4 kips). The maximum capacity of HT-1 was calculated considering the four #5 bars comprising the lightly reinforced side of the specimen and the rebar yield strength. The u-bar specimens both used reinforcement that had a yield strength of 520 MPa (75 ksi), considering the yield strength of the reinforcement and the area of the four #5 bars in the lightly reinforced side of the specimen the maximum expected tensile capacity of the u-bar specimens was 413.7 kN (93 kips).

All specimens produced similar tensile capacities. HT-1 produced the lowest tensile capacity, which was 399.51 kN (89.81 kip). This was to be expected because HT-1 contained

conventional rebar with the lowest rebar yield strength. The tensile capacity of HT-1 indicated that the yield strength of the rebar was approximately 496.4 MPa (72 ksi), which was between the minimum yield strength of 420 MPa (60 ksi) and the maximum yield strength of 551.6 MPa (80 ksi) for A615 rebar. The second highest tensile capacity was produced by the u-bar detail using stainless steel reinforcement (ST-1). ST-1 produced a tensile capacity of 408.24 kN (91.78 kip). The largest tensile capacity was produced by WT-1, which was 414.75 kN (93.24 kips).

Both WT-1 and HT-1 exceeded the expected tensile capacity. The additional capacity of specimens WT-1 and HT-1 may be explained by strain hardening of the reinforcement. However, specimen ST-1 did not meet the expected capacity of 93 kips, it only had a capacity of 91.78 kips.

ST-1 had a capacity that was 1.3 percent less than the expected tensile capacity. The low capacity could have been due to uneven loading or bending stresses caused by the reinforcement not being centered in the specimen. Another cause specific to ST-1 was that the welds broke during testing. The specimen was rewelded and tested to failure, but the specimen may have been damaged during the first unsuccessful test. ST-1 may have experienced damage that could have affected its behavior and tensile capacity during the second successful test.

As stated previously, the welds of ST-1 failed during testing. The welds broke at a load of approximately 289 kN (65 kips). The specimen was rewelded and tested to failure. During the second test of ST-1, no strain gage or LVDT data was collected. The only data collected from the second test were the applied force and the actuator location, which was taken from the MTS actuator system. The actuator location allowed for the determination of the total specimen deflection. The data collected from both tests were spliced together to form a complete load versus deflection curve.

Load verses deflection curves were also constructed for specimens HT-1 and WT-1. The applied load was determined from the load data provided by the MTS actuator system. The total specimen deflection was used in the construction of the curves; these data were taken from the LVDT that was attached to the bottom of the specimens. The load verses deflection curves for specimens HT-1, WT-1 and ST-1 were plotted together and can be seen in Figure 28. The end of the bold line for specimen ST-1 signifies where the data from the first test ends and where the data from the second test begins.

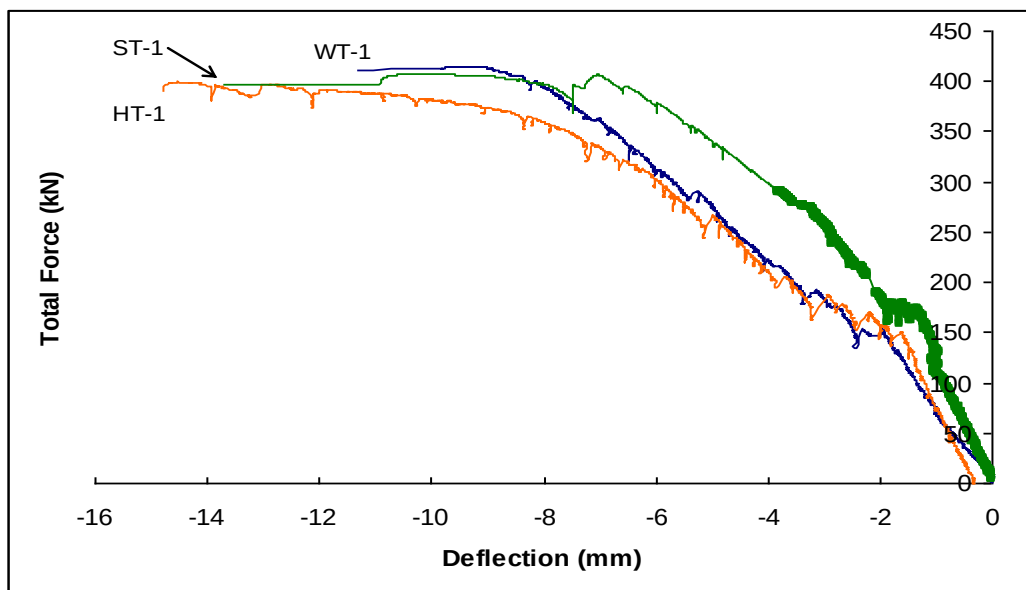


**Figure 28: Total Applied Force Verses Deflection Curves**

Figure 28 shows the load deflection curves of all the specimens leveling off at approximately the force required to yield the reinforcement. The increase in deflection while holding constant load signifies that the reinforcement in the specimens was yielding. Both the shape of the load verses deflection curves and the capacities of the specimens indicate that it can be safely assumed that the reinforcement in all the specimens yielded.

The behavior of the specimens shows that the u-bar and the headed bar joint details can successfully yield the joint reinforcement without brittle failure. This result shows that both joint details could effectively be used as a transverse joint in a negative moment region, which would mainly produce global tension in the deck.

### ***Tension Specimen Behavior***

All specimens produced similar crack patterns up to and beyond the service loading. The first cracks to appear were transverse cracks evenly spaced along the length of the specimens. The joint zone usually experienced transverse cracking after several other transverse cracks already had formed in other locations. Delayed transverse cracking in the joint zone may have been due to the larger area of reinforcement in the joint zones, when compared to the bodies of the specimens. The transverse cracks initially were found only in the surface of the concrete, and as the loading progressed the cracks propagated through the entire thickness of the specimens. Additional loading produced longitudinal cracks that appeared above the main longitudinal reinforcement in the specimens. These longitudinal cracks appeared above the longitudinal reinforcement that comprised the lightly reinforced half of the specimen, or the top half of the specimens in this particular set-up. When approaching the capacities of the specimens, diagonal

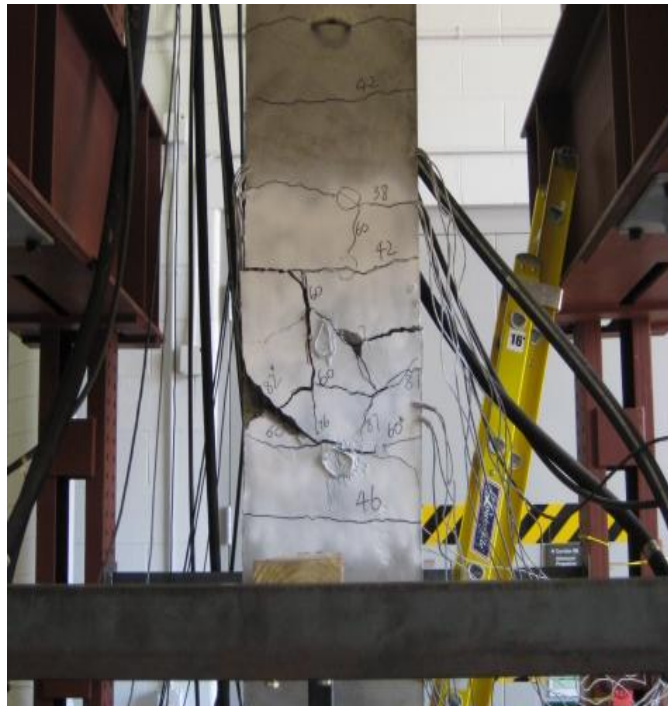
cracks appeared close to the sides of the specimens. These diagonal cracks would usually propagate toward a transverse crack in the joint zone and cause the failure surface for the specimens. Figure 29 shows the cracks patterns at failure for specimens ST-1, WT-1 and HT-1.



**Figure 29: Tension Crack Patterns at Failure**  
**(a) Specimen ST-1**



**(b) Specimen WT-1**



**(c) Specimen HT-1**

The numbers written by the cracks in Figure 29 represent the total force applied in kips when the crack was formed.

### ***Tensile Crack Widths at Service Level Loading***

The tensile service load was calculated by using the service level negative moment found in design example 9.6 of the PCI Bridge Design Manual (PCI 2003). The neutral axis was then found for the cracked composite cross section, which was then used in conjunction with the service level negative moment to determine the total tensile service load. The area of steel in the specimens was compared to the total required area of steel to determine the tensile service loads for the specimens. The tensile service load for HT-1 was determined to be 240 kN (54 kips), and the tensile service load for ST-1 and WT-1 was determined to be 287 kN (64.5 kips). Different tensile service loads were determined for the u-bar and headed bar specimens because of the different yield strengths of their reinforcing materials.

Crack widths in the tension specimens were measured in the same way as the flexural specimens. A crack width gage was used to visually determine the widths of the specimens. Crack widths were measured systematically throughout the testing of HT-1 and ST-1, but only two crack widths were measured at two different loads for specimen WT-1

Crack widths were measured at a total applied load of 289 kN (65 kips) for specimen ST-1, which was very close to the tensile service load of 287 kN (64.5 kips). The average crack width at 289 kN (65 kips) was found to be 0.3 mm (0.012 in) for ST-1. Crack widths were measured for specimen HT-1 at a total applied force of 293.5 kN (66 kips), which was very close to the service level loading. The average crack width at 293.5 kN (66 kips) for specimen HT-1 was determined to be 0.55 mm (0.022 in). The last crack width measurement for specimen WT-1 was taken at 178 kN (40 kip) and the average crack width was found to be 0.2 mm (0.0079 in).

Comparable crack width measurements were taken at 200 kN (45 kips) for specimen ST-1 and at 187 kN (42 kips) for specimen HT-1, and the average crack widths were found to be 0.15 mm (0.006 in) and 0.32 mm (0.012 in), respectively. When comparing the crack widths of the two joint details, one can see that the headed bar detail of specimen HT-1 created the largest crack widths at approximately 177.9 kN (40 kips) and at its service level loading, when compared to the crack widths created by the u-bar details of specimens ST-1 at its service level loading and WT-1 at approximately 177.9 kN (40 kips).

### ***Strain Gage Data***

Strain gages were installed on the specimen reinforcement in the configurations shown in Figures 10 and 11. The purpose of the strain gage data was to verify the required overlap length that would develop the yield strength of the reinforcement. The strain gage data were expected to show increasing strain as the distance away from the bearing surface of the reinforcement increased. The strain gage data did not show this trend. Instead of a useful trend in the strain readings, the strain gage locations that were crossed by a crack produced high strain readings, while the other gage locations had smaller strain readings. The joint cracking patterns produced scattered results that could not be used to verify the required overlap length. The strain gage data do show that both joint details allow the reinforcement to develop large strains at crack locations. Figures 30 and 31 show the force versus reinforcement strain curves for specimens WT-1 and HT-1, respectively. The strain gage data for ST-1 were lost due to weld failure. Figures 32, 33, and 34 show the moment versus reinforcement strain curves for specimens SB-1, WB-1 and HB-1, respectively.

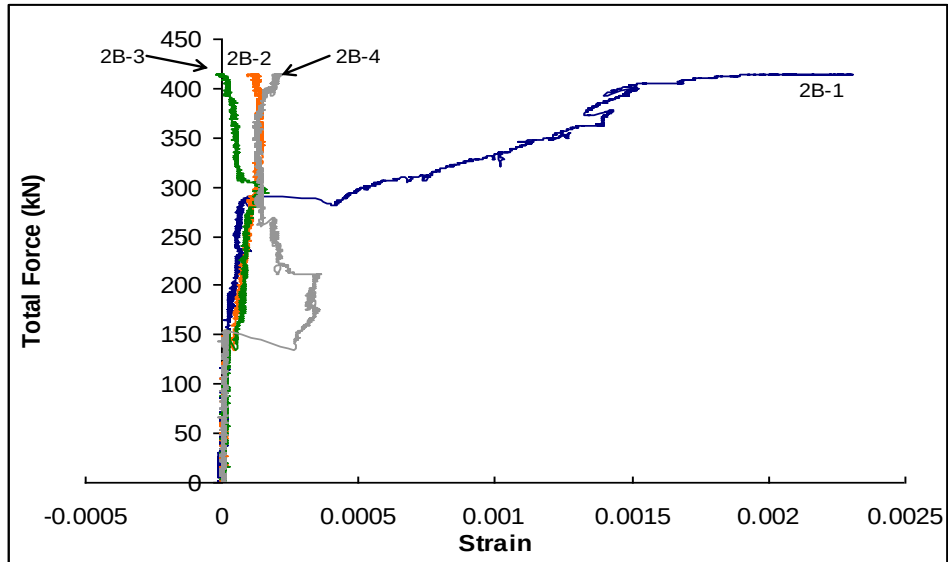
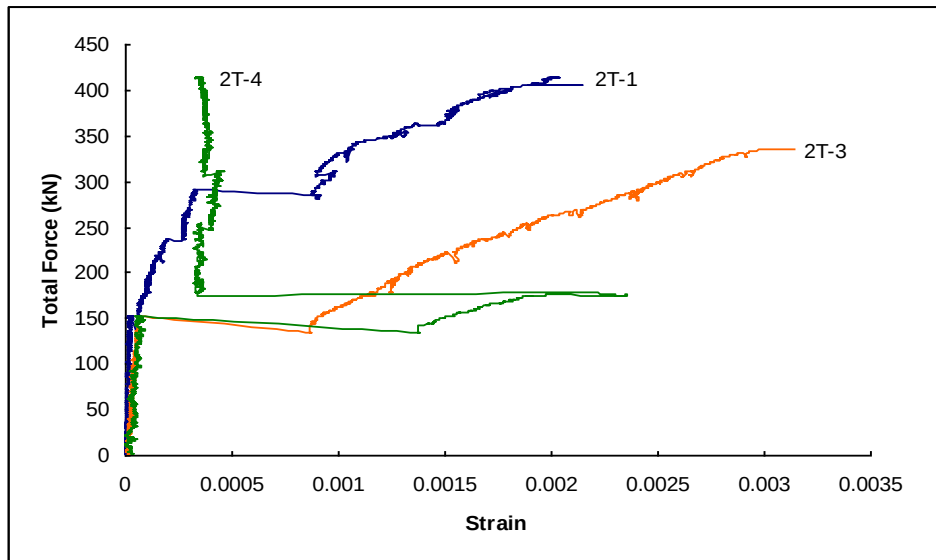
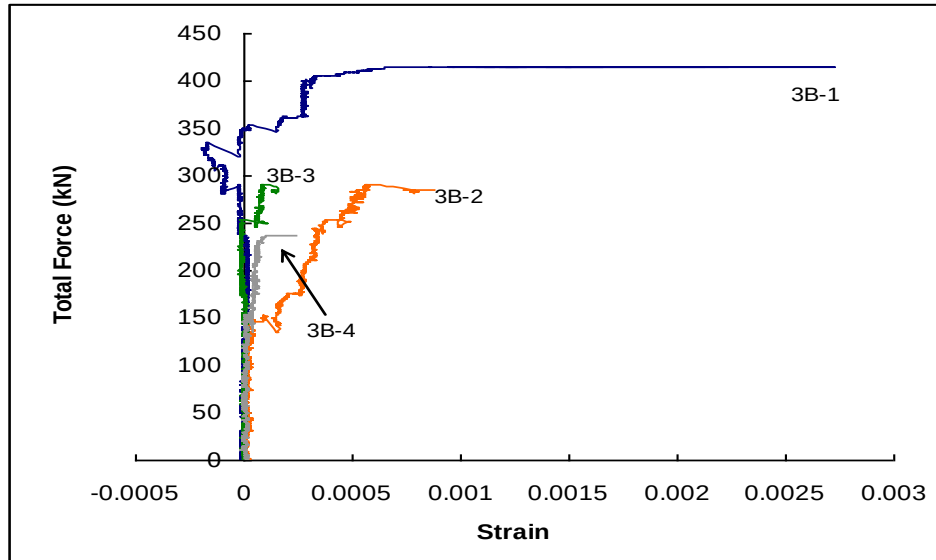


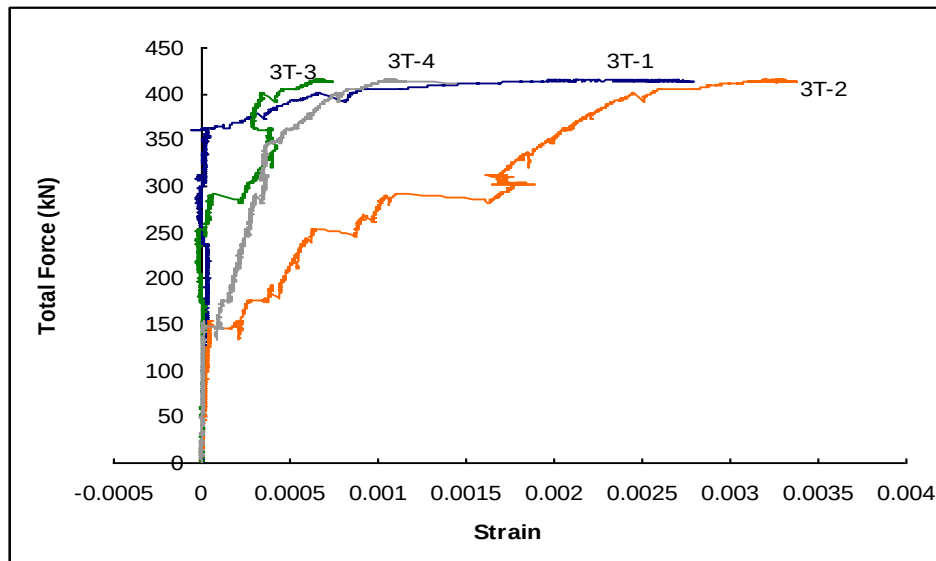
**Figure 30: Total Force Verses Rebar Strain Figures for Specimen WT-1**  
**(a) Bottom of U-bar 2 (Specimen WT-1)**



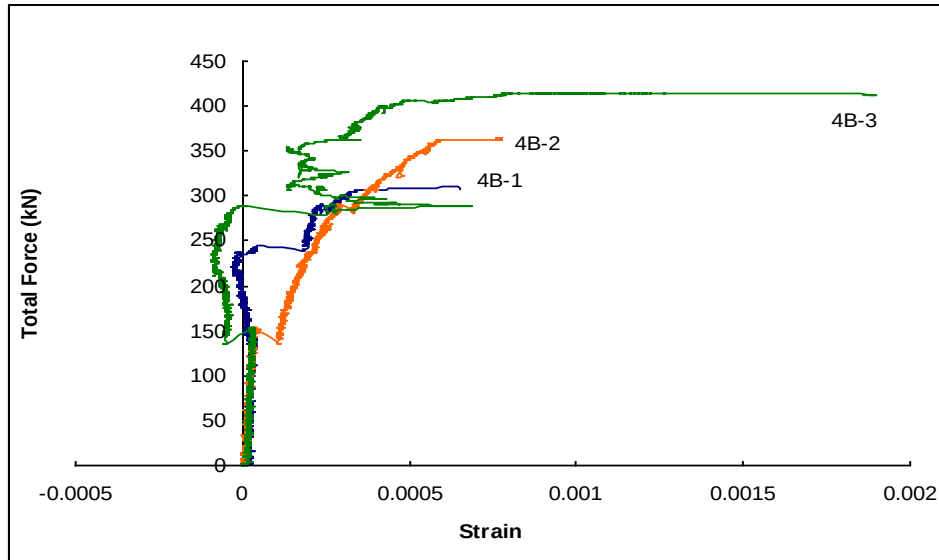
**(b) Top of U-bar 2 (Specimens WT-1)**



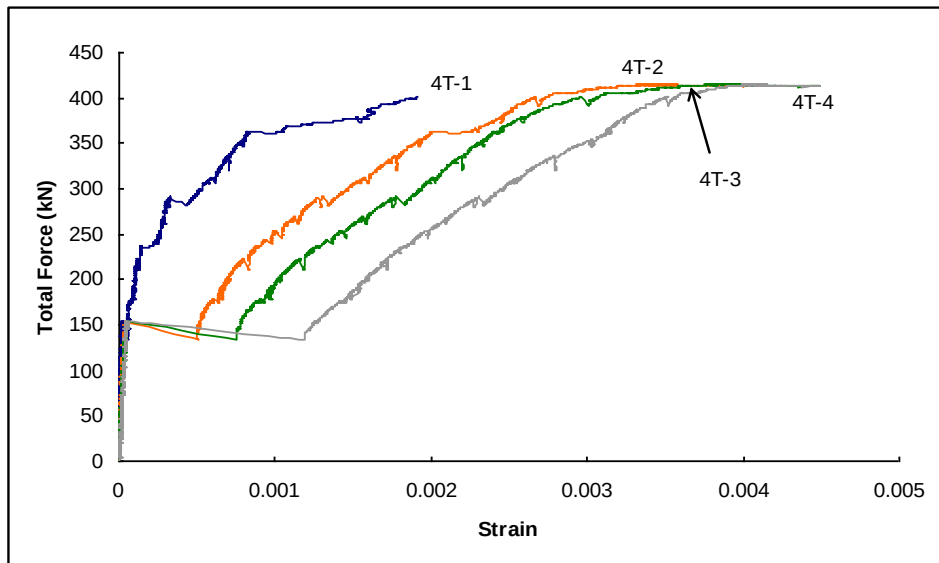
**(c) Bottom of U-bar 3 (Specimen WT-1)**



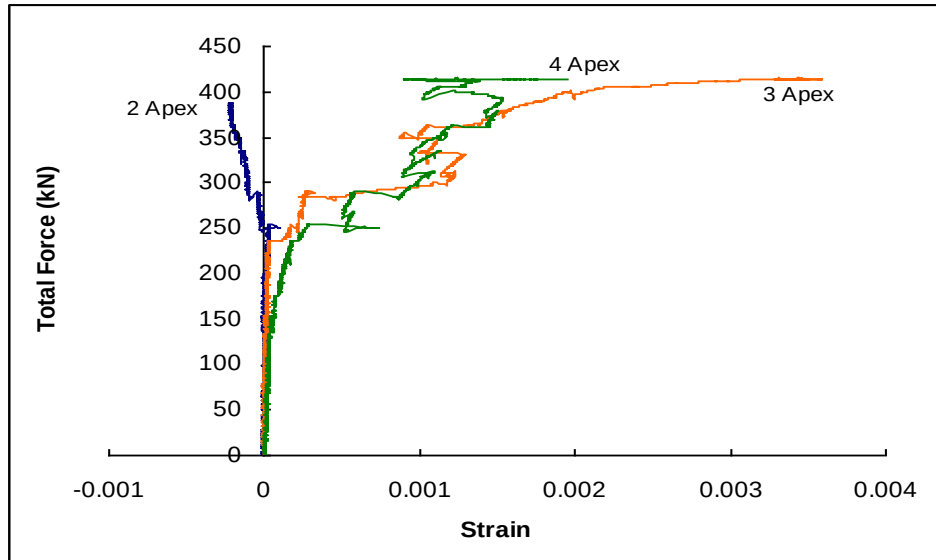
**(d) Top of U-bar 3 (Specimen WT-1)**



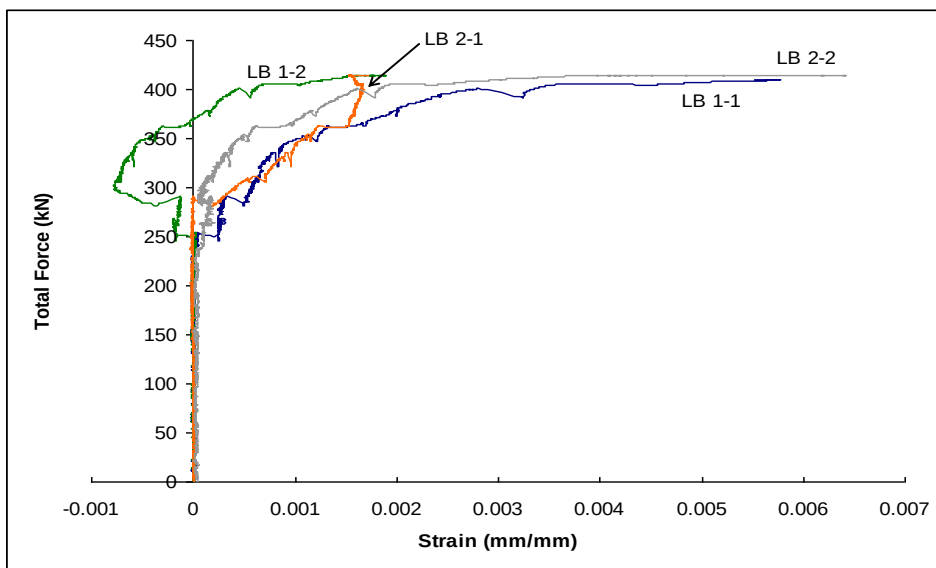
**(e) Bottom of U-bar 4 (Specimen WT-1)**



**(f) Top of U-bar 4 (Specimen WT-1)**



**(g) Apex of U-bars 2, 3, and 4 (Specimen WT-1)**



**(h) Transverse Lacer Bars 1 and 2 (Specimen WT-1)**

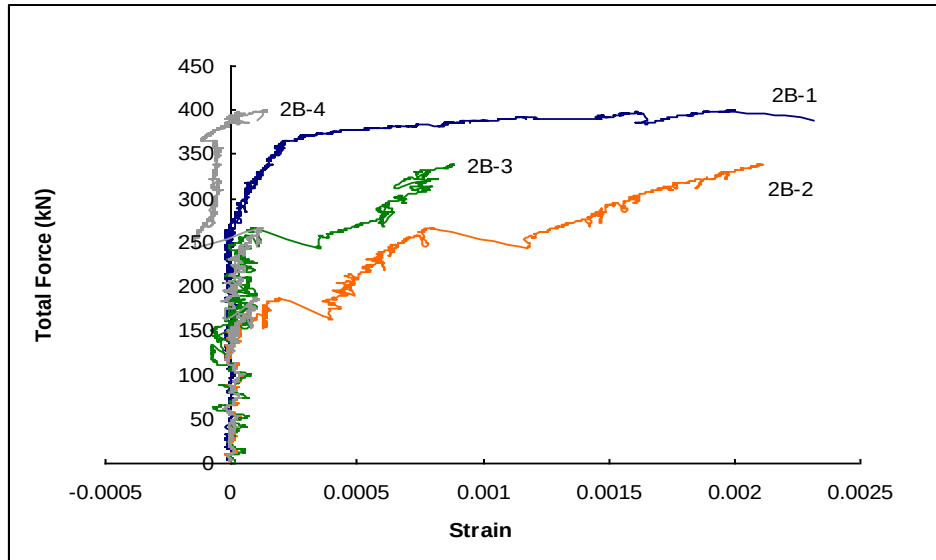
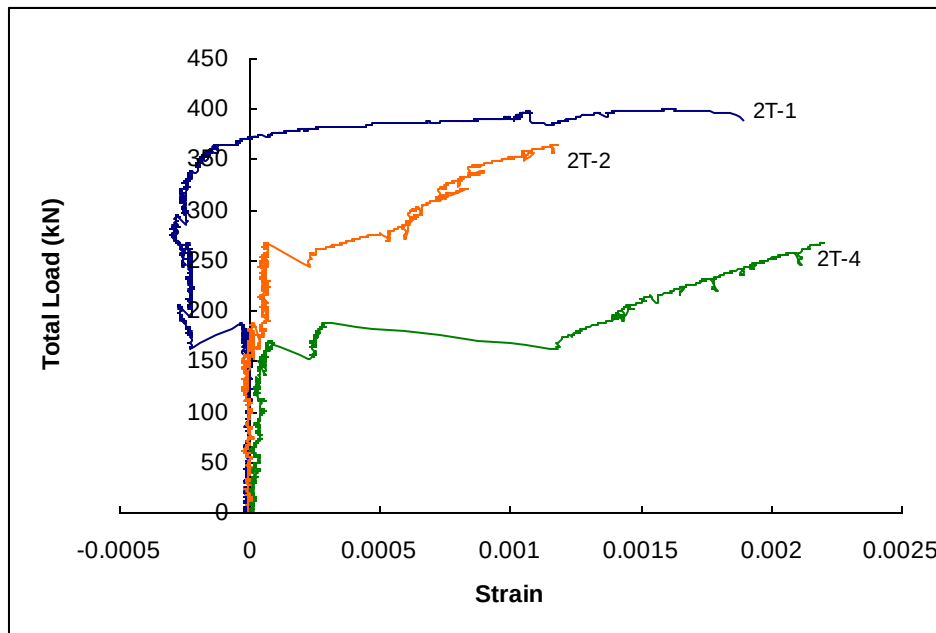
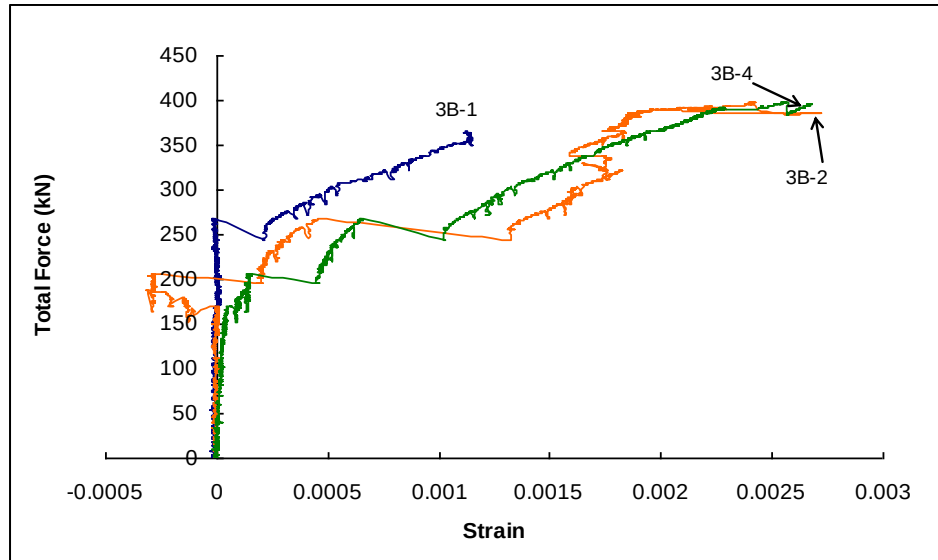


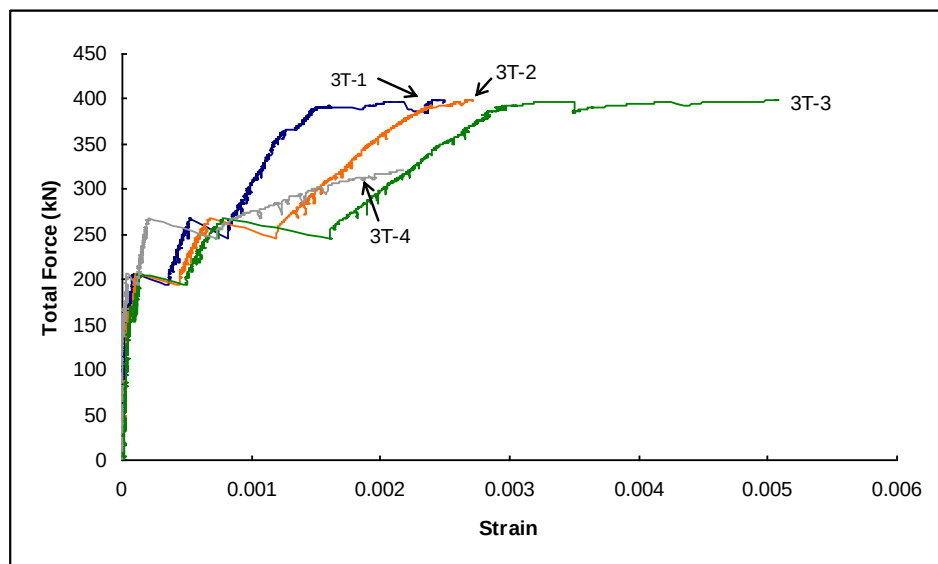
**Figure 31: Total Force Verses Rebar Strain for Specimen HT-1**  
**(a) Bottom Bar of Headed Bar Set 2 (Specimen HT-1)**



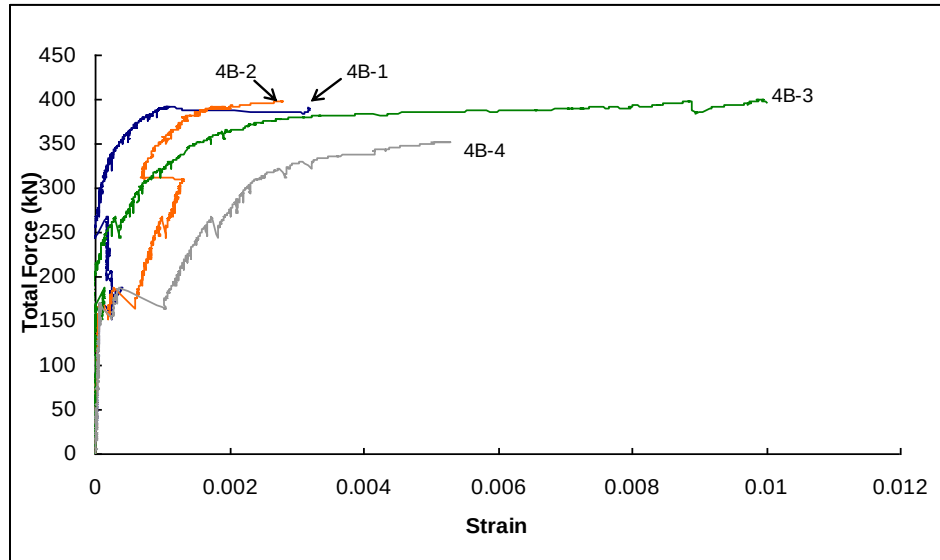
**(b) Top Bar of Headed Bar Set 2 (Specimen HT-1)**



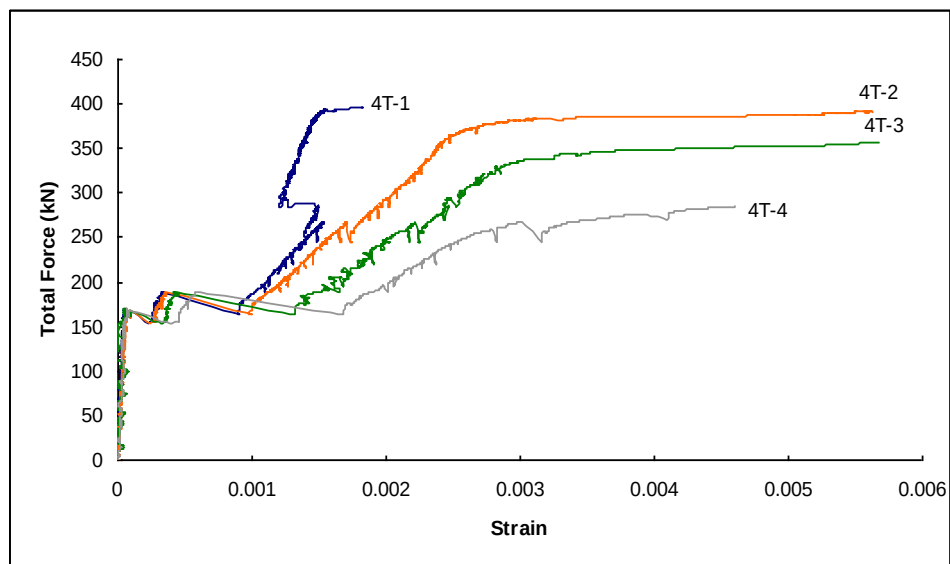
**(c) Bottom Bar of Headed Bar Set 3 (Specimen HT-1)**



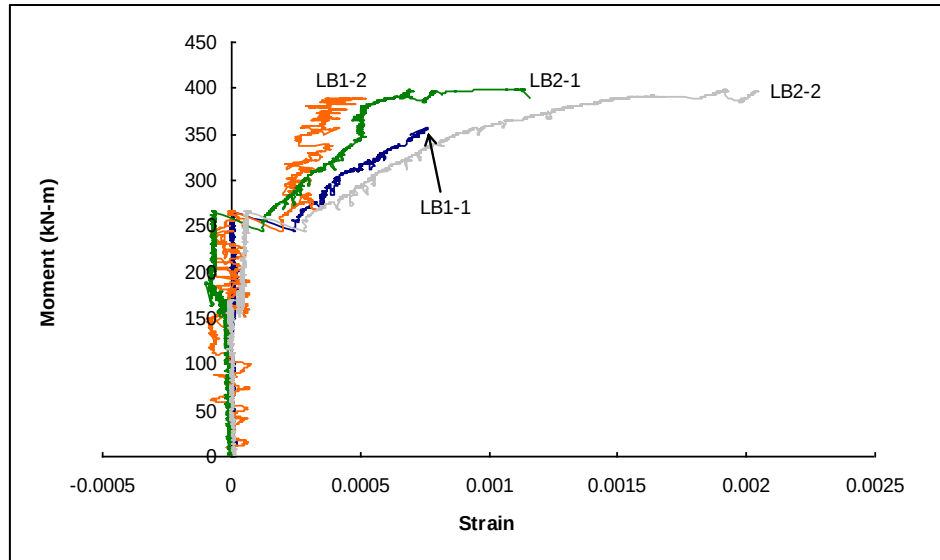
**(d) Top Bar of Headed Bar Set 3 (Specimen HT-1)**



**(e) Bottom Bar of Headed Bar Set 4 (Specimen HT-1)**



**(f) Top Bar of Headed Bar Set 4 (Specimen HT-1)**



**(g) Transverse Lacer Bars 1 and 2 (Specimen HT-1)**

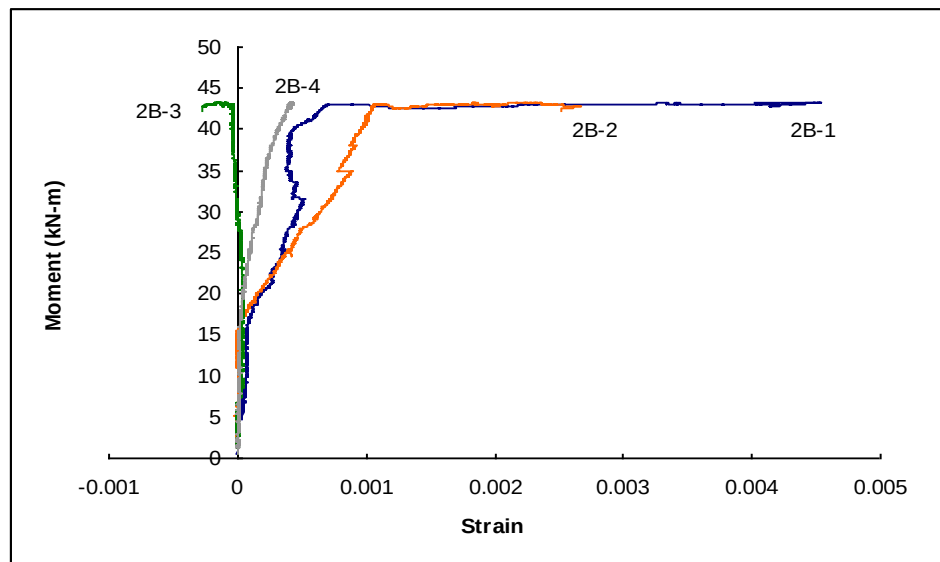
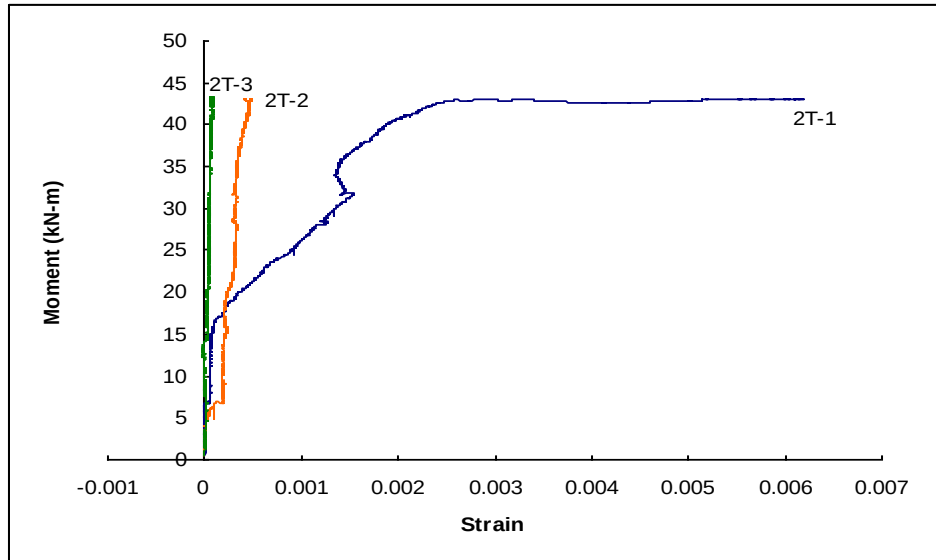
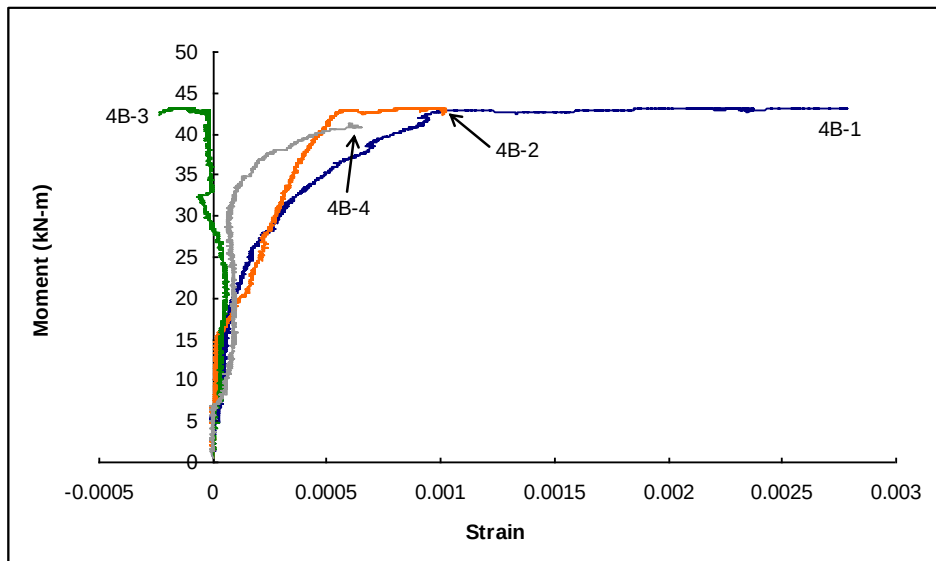


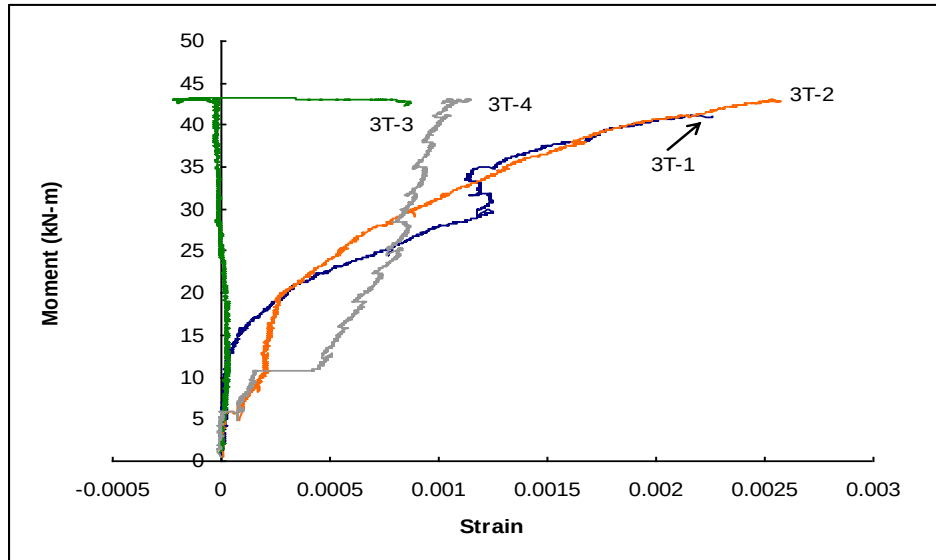
**Figure 32: Moment Verses Rebar Strain Curves of Specimen SB-1**  
**(a) Bottom of U-bar 2 (Specimen SB-1)**



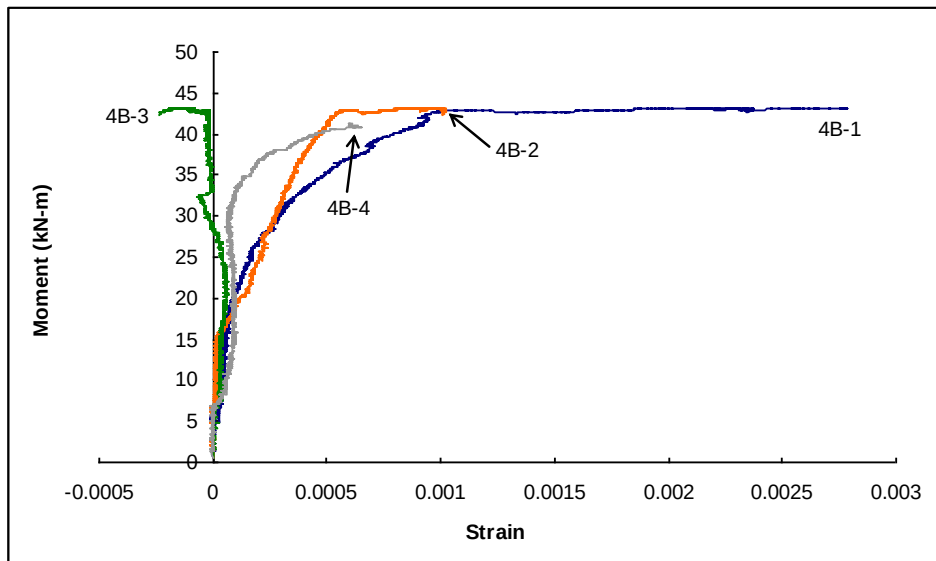
**(b) Top of U-bar 2 (Specimen SB-1)**



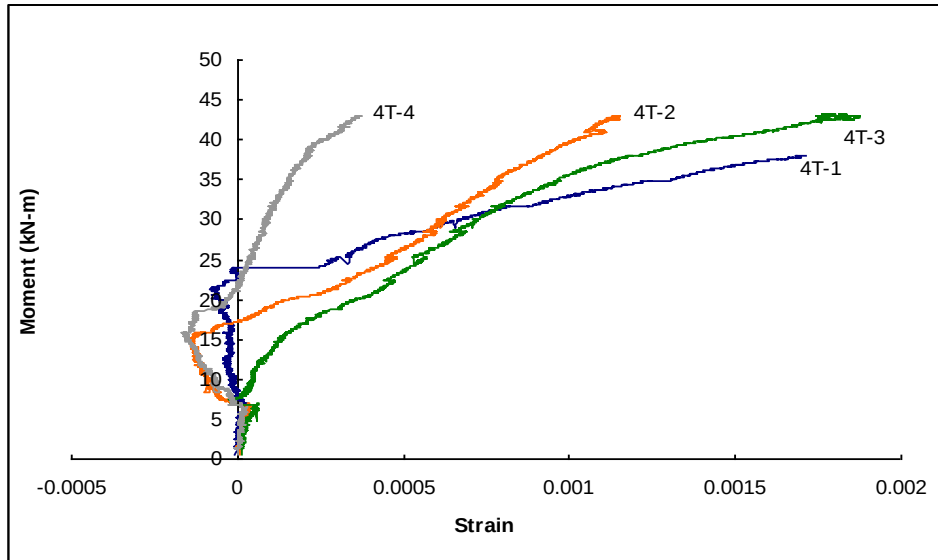
**(c) Bottom of U-bar 3 (Specimen SB-1)**



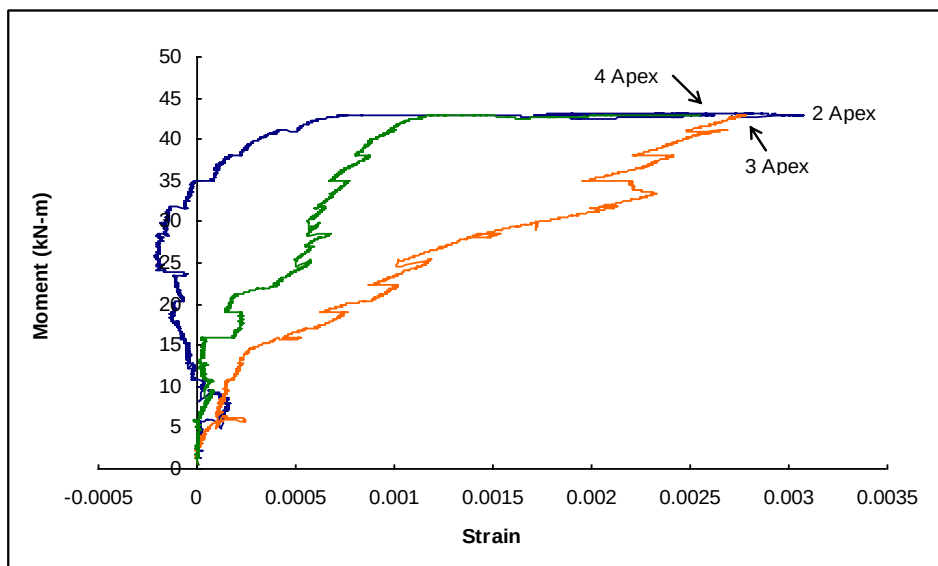
**(d) Top of U-bar 3 (Specimen SB-1)**



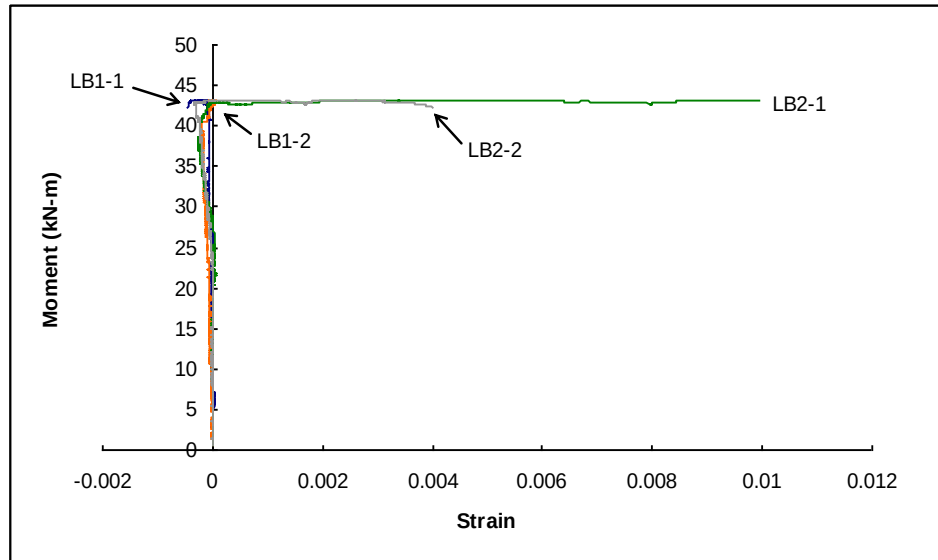
**(e) Bottom of U-bar 4 (Specimen SB-1)**



**(f) Top of U-bar 4 (Specimen SB-1)**



**(g) Apex of U-bars 2, 3, and 4 (Specimen SB-1)**



**(h) Lacer Bars 1 and 2 (Specimen SB-1)**

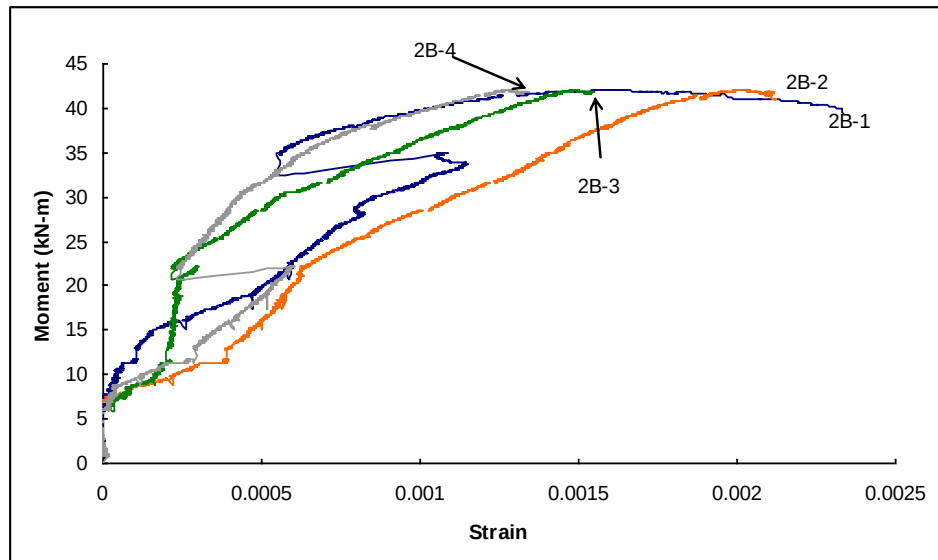
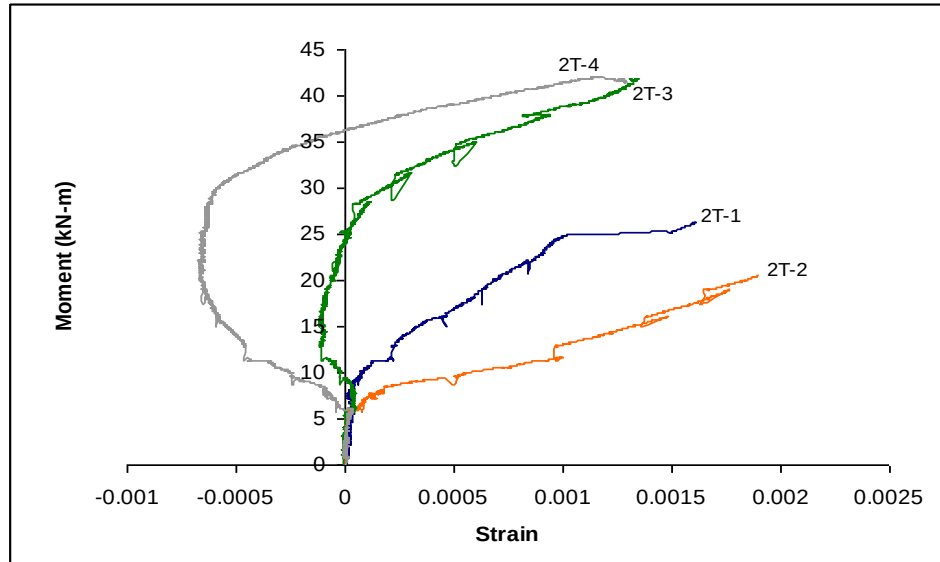
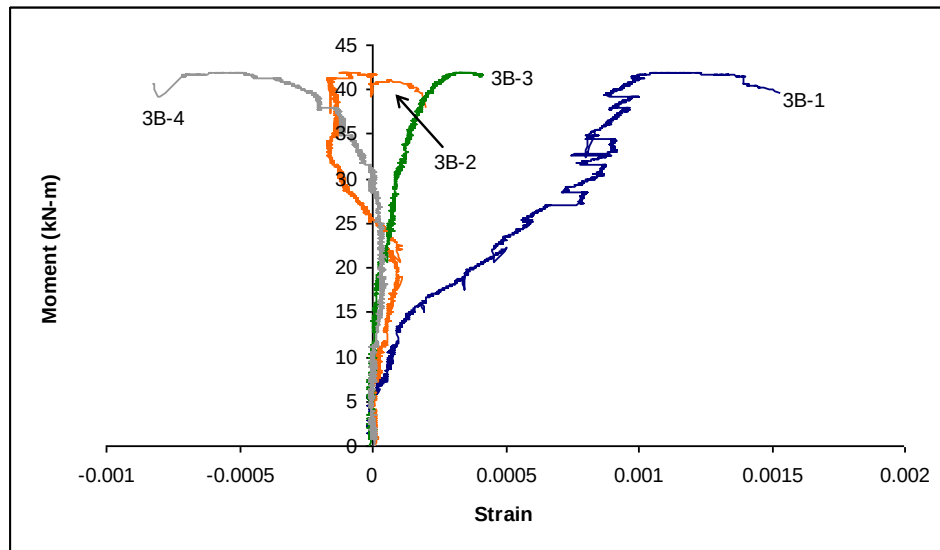


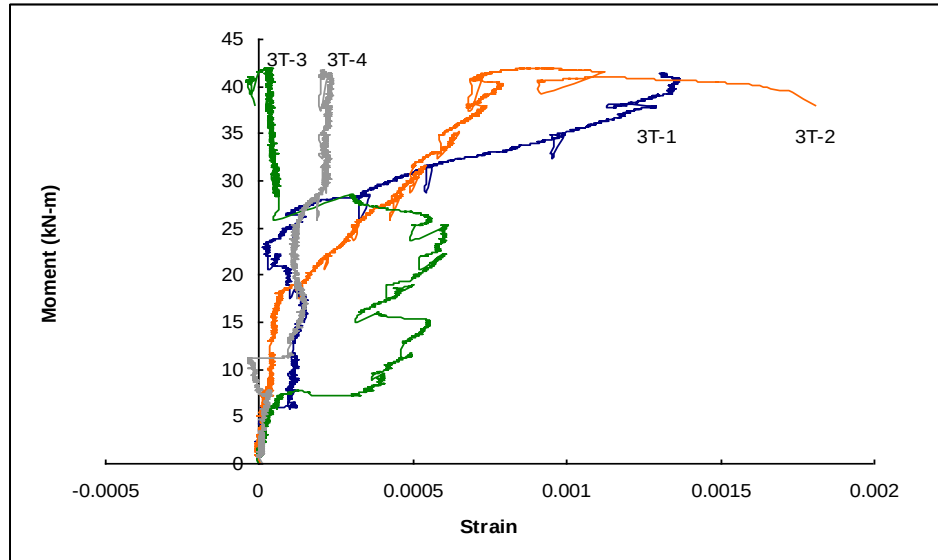
**Figure 33: Moment Verses Rebar Strain Curves of Specimen WB-1**  
**(a) Bottom of U-bar 3 (Specimen WB-1)**



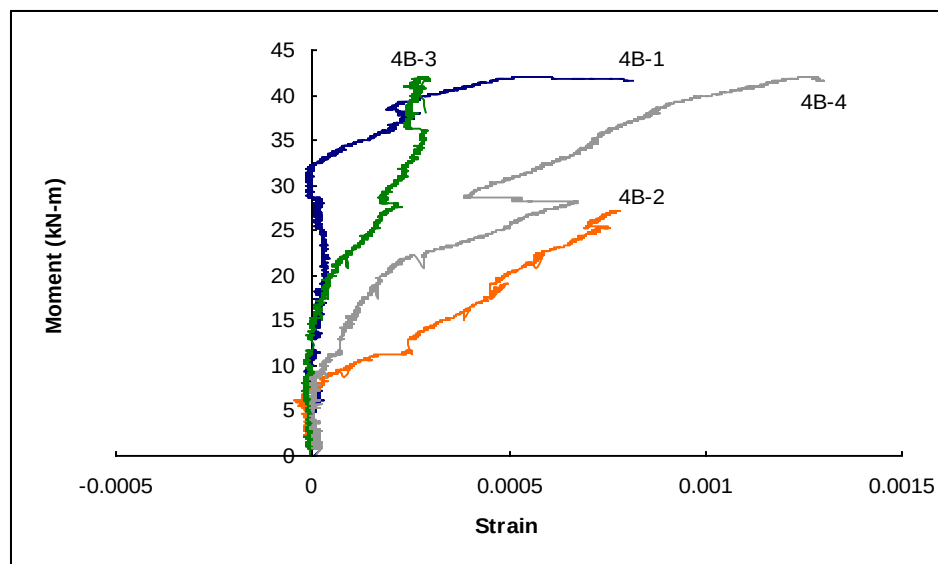
**(b) Top of U-bar 2 (Specimen WB-1)**



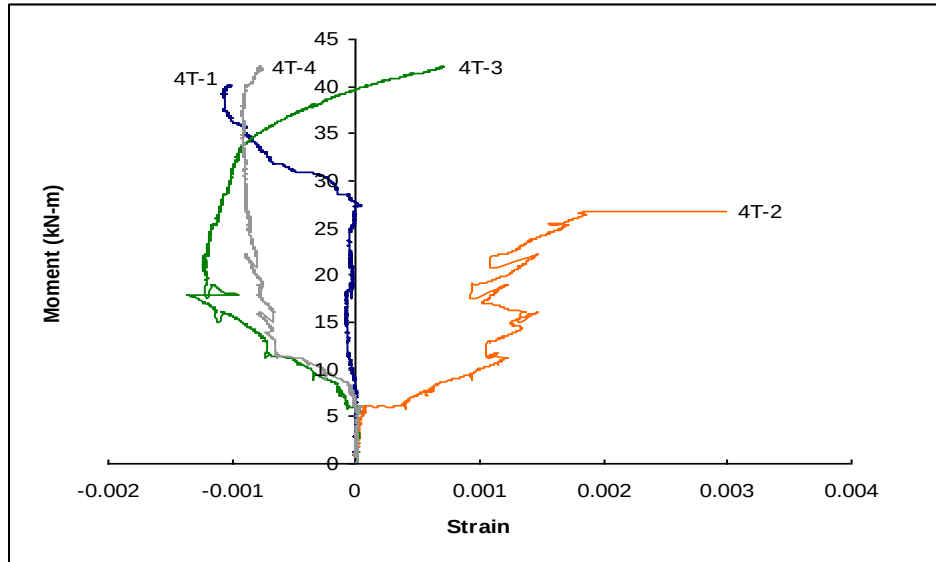
**(c) Bottom of U-bar 3 (Specimen WB-1)**



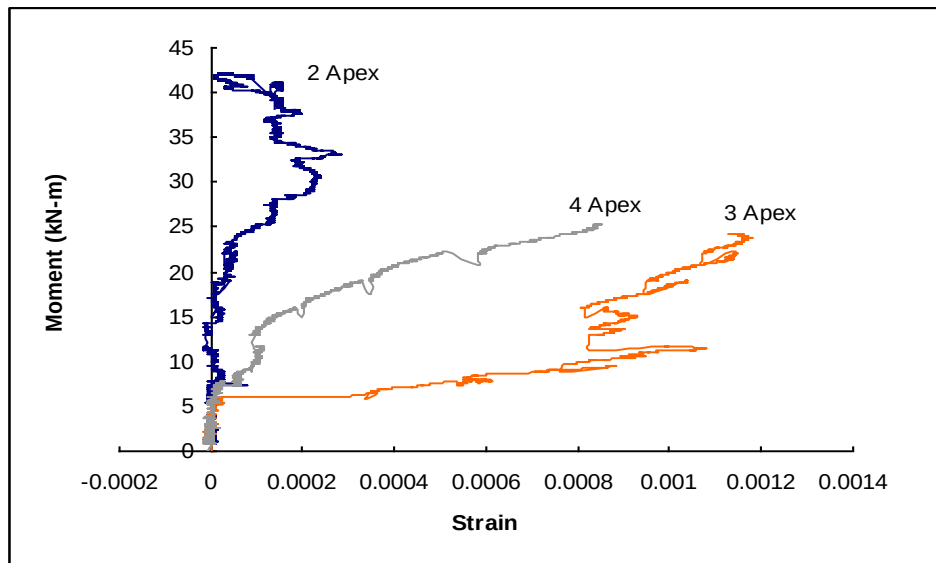
**(d) Top of U-bar 3 (Specimen WB-1)**



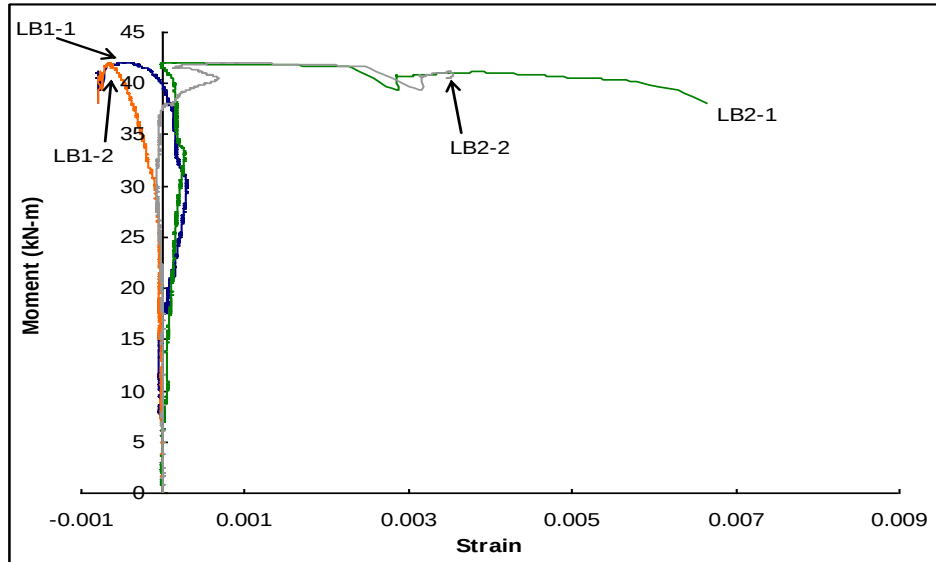
**(e) Bottom of U-bar 4 (Specimen WB-1)**



**(f) Top of U-bar 4 (Specimen WB-1)**



**(g) Apex of U-bars 2, 3, and 4 (Specimen WB-1)**



**(h) Transverse Lacers Bars 1 and 2 (Specimen WB-1)**

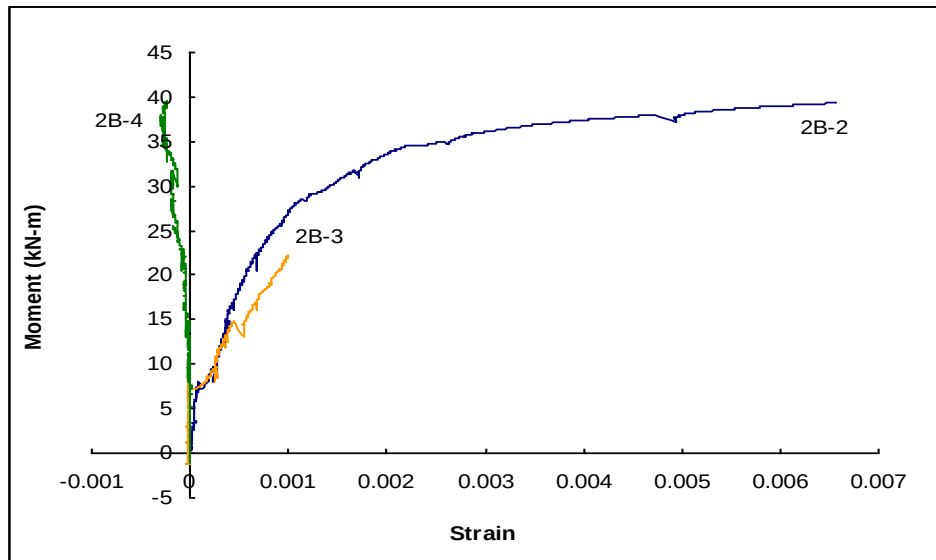
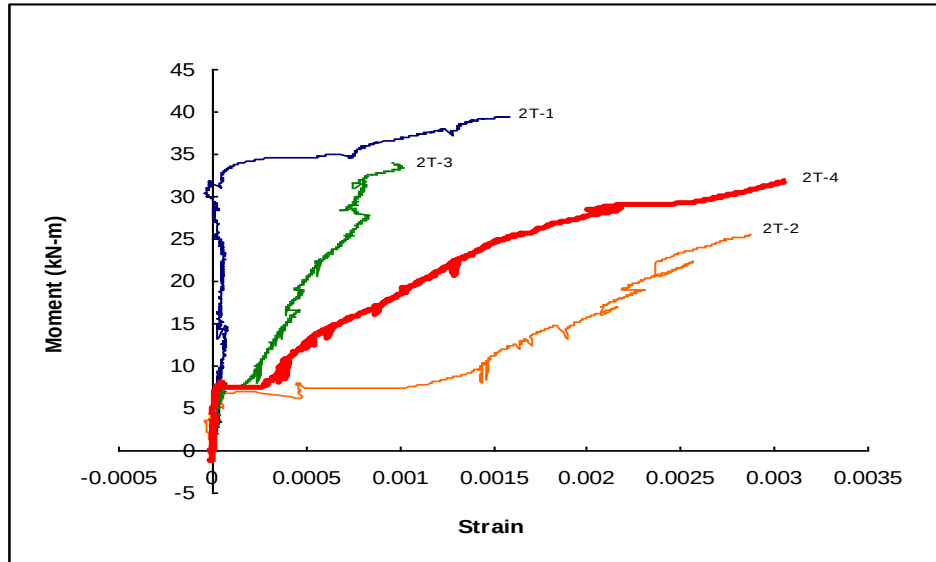
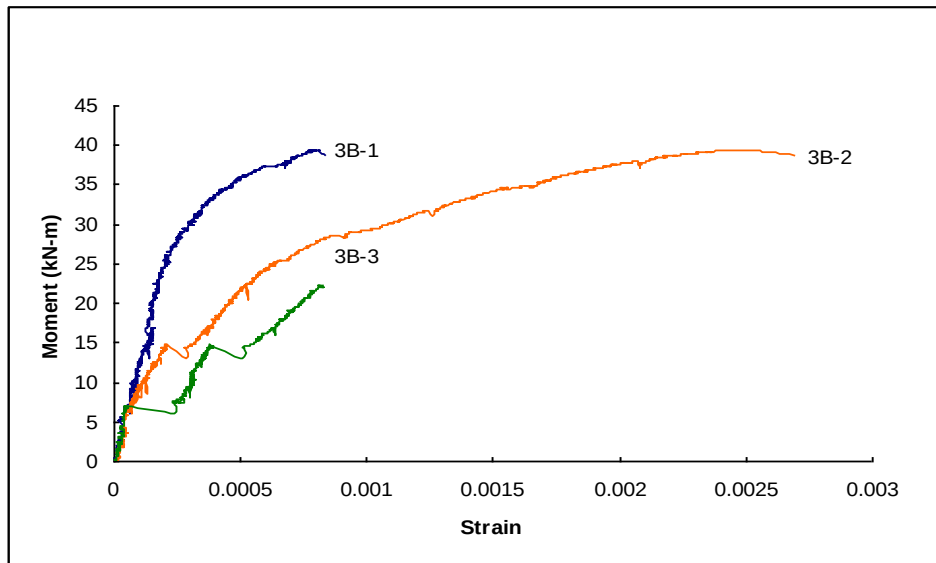


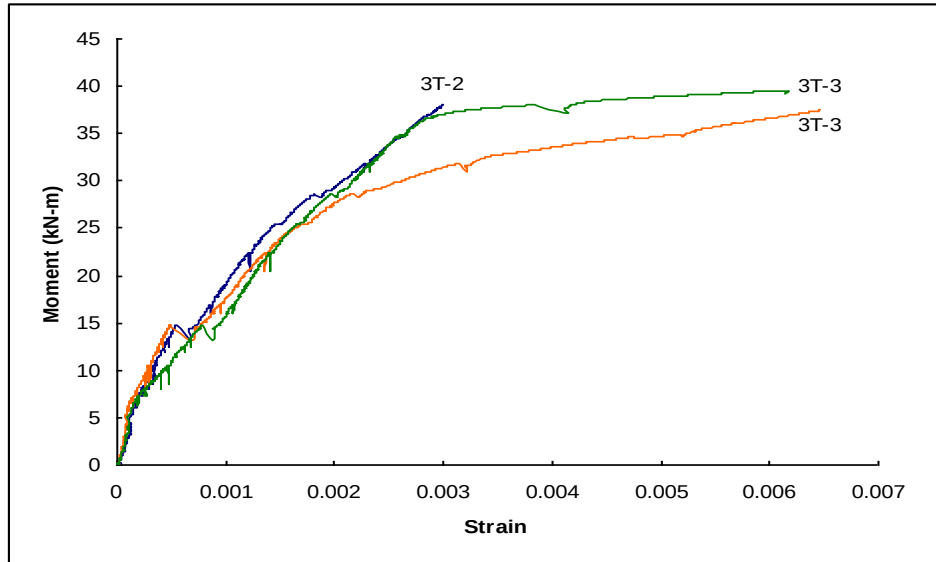
**Figure 34: Moment Verses Rebar Strain Curves of Specimen HB-1**  
**(a) Bottom of U-bar 2 (Specimen HB-1)**



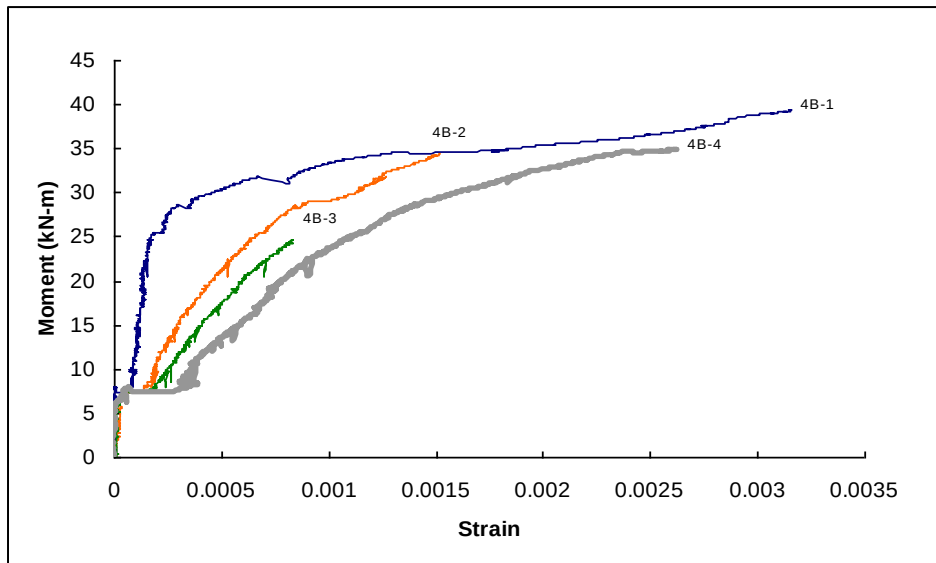
**(b) Top of U-bar 2 (Specimen HB-1)**



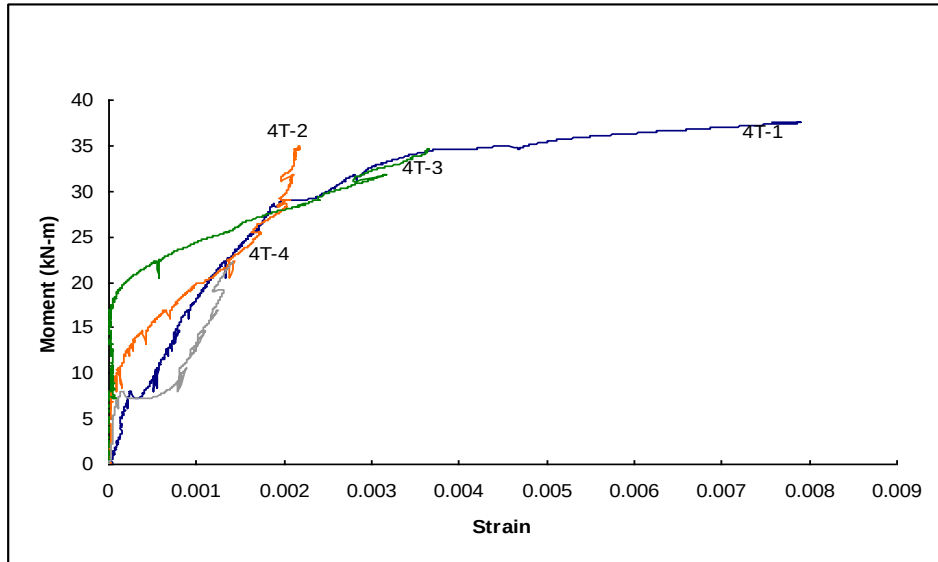
**(c) Bottom of U-bar 3 (Specimen HB-1)**



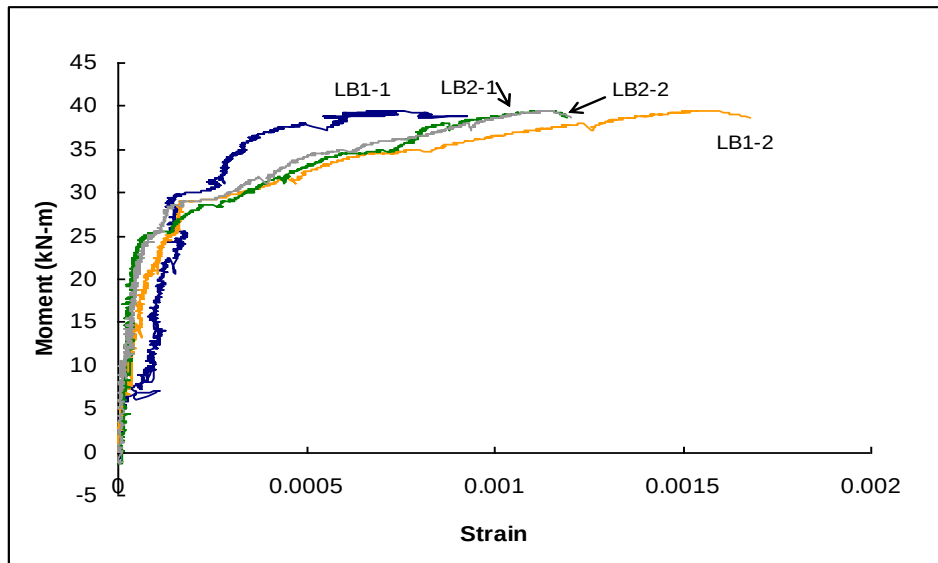
**(d) Top of U-bar 3 (Specimen HB-1)**



**(e) Bottom of U-bar 4 (Specimen HB-1)**



**(f) Top of U-bar 4 (Specimen HB-1)**



**(g) Transverse Lacer Bars 1 and 2 (Specimen HB-1)**

## **Chapter 6 Conclusions and Recommended Future Research**

As stated previously, the main objective of this research was to test two joint details and select the best performing joint detail for further testing. The additional testing of the best performing detail will then be the first step toward the development of design guidelines and standard details for longitudinal and transverse precast deck joints.

Specimens containing both headed bar and u-bar joint details were tested in tension and in bending to ensure that the proposed joints could produce a precast deck system that would act monolithically. The capacities of the joint details were used for comparison and the selection of the best performing joint detail. All joint details produced adequate capacities and ductility in both the tension and bending tests. Specimens containing the u-bar joint detail produced the largest capacities in both the bending and tension tests. Specimen WT-1 produced the largest tensile capacity which was 414.75 kN (93.24 kips), and specimen SB-1 produced the largest flexural capacity which was 43.22 kN-m (31.88 kip-ft). The u-bar detail produced the largest capacities without compromising ductility. Smaller crack widths at service level loading were also produced by the u-bar detail when compared to the headed bar detail. The development of small crack widths increase durability by decreasing decrease reinforcement corrosion, thus leading to longer deck life.

The constructability and reinforcement costs of the joint details were also compared. The u-bar detail created a less congested joint, which made it the easiest to construct. The bearing heads of the headed bar detail require more space in a given rebar spacing due to the larger diameter of the rebar heads. This extra space would reduce construction tolerances and could therefore cause problems in placement of precast deck components. The u-bars can also be easily tied together to form a rebar cage, which would allow for easy construction in the precast yard

when compared to the two single layers of reinforcement in the headed bar detail. The lowest material cost was the conventional rebar used for the headed bars. The material costs were competitive between the conventional rebar used in the headed bars and the welded wire reinforcement. The cost for conventional reinforcement was approximately 800 dollars a ton with an additional cost of 25 dollars for the installation of each Lenton Terminator bearing head. The welded wire reinforcement cost was 850-900 dollars a ton for single fabricated cut wires or 900-1000 dollars a ton for fabricated wire mesh. The stainless steel reinforcement had the highest cost of 5000 dollars a ton including fabrication. Even though the initial cost of stainless steel is high, it should still be considered due to its potential to increase the life span of the structure.

After consideration of capacity, service level crack widths, constructability, and cost, the u-bar detail constructed of welded wire reinforcement was chosen for further testing. This detail produced adequate capacities in both the tension and flexural testing, while still producing adequate ductility. The constructability of the u-bar detail as well as the cost of the welded wire reinforcement make this detail an economical choice.

The previously conducted experiments have shown that the u-bar detail can develop adequate capacity with an overlap length of 152.4 mm (6"), a rebar spacing of 114.3 mm (4.5") and two transverse lacer bars, but they do not give a sense of how each variable will affect the behavior of the joint. Additional tests should be conducted with varying overlap lengths, rebar spacings, amount of transverse reinforcement, and concrete strengths. Understanding the effects that each variable has on the overall behavior of the joint will provide engineers the ability to use the u-bar joint detail in various situations. Also, A706 reinforcing material should be tested as the primary reinforcement in a u-bar bar joint detail. The A706 material is suggested for testing

due to its ductility, its ASTM standard (ASTM A706 2008) states that it must be able to be bent to an inside diameter equal to  $3d_b$ .

A strut and tie model should be developed as a design tool for the transverse u-bar joints. The crack patterns in the u-bar tension specimens indicate that the strut and tie model would be an accurate model for the prediction of the ultimate capacity of this joint type.

## **List of References**

ACI Committee 318, (2005) “Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05),” American Concrete Institute, Farmington Hills, MI.

American Association of State Highway and Transportation Officials, (2007) “AASHTO LRFD Bridge Design Specifications,” Washington, D.C., 4<sup>th</sup> Edition.

ASTM A 496, “Standard Specification for Steel wire, Deformed for Concrete Reinforcement,” American Society of Testing and Materials, West Conshohocken, PA, 2005.

ASTM A 706, “Standard Specification for Low Alloy Steel Deformed and Plain Bars for Concrete Reinforcement,” American Society of Testing and Materials, West Conshohocken, PA, 2008.

ASTM A 955, “Standard Specification for Deformed and Plain Stainless-Steel Bars for Concrete reinforcement,” American Society of Testing and Materials, West Conshohocken, PA, 2007.

ASTM C 39, “Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens,” American Society of Testing and Materials, West Conshohocken, PA, 2005.

Dragosavic, M.; van den Beukel, A.; Gijssbers, F.B.J., (1975) “Loop Connections between Precast Concrete Components Loaded in Bending,” Heron, Vol. 20, No. 3, pp. 3-35.

French, C. E.; Ma, J.; Eriksson, R.; Marsh, L.; Seguirant, S.; Prussack, C., “Cast-in-Place Concrete Connections for Precast Deck Systems” NCHRP 10-71 Interim Report, pp. 1-4, 7-20, 23-26.

Gordon, S. R.; May, I. M., (2005) “Development of In Situ Joints for Pre-Cast Bridge Deck Units,” Bridge Engineering 000, Issue BEO, pp. 1-14.

Precast Prestressed Concrete Institute, (2003) “Precast Prestressed Concrete Bridge Design Manual,” Chicago, IL., Sections 9.6 and 9.8.

Thompson, M. K.; Jirsa, J.O.; and Breen, J.E., (2006) “Behavior and Capacity of Headed Reinforcement,” *ACI Structural Journal*, V.103, No. 4, pp. 522-530.

## **Vita**

Samuel Lewis was born in Ironton, Ohio, on October 28, 1984. He was raised by his parents Joseph and Clara Lewis along with two younger brothers Matthew and Joshua in Ironton. In 2003, Samuel graduated from Rock Hill High School in Pedro, Ohio. Following high school, Samuel attended Ohio University to study Civil Engineering, receiving his Bachelor of Science degree in Civil Engineering in 2007. Samuel is currently pursuing his Master of Science degree in Civil Engineering with an emphasis in structures at the University of Tennessee, Knoxville.