DESIGN GUIDELINES OF CIP JOINTS WITH ACCELERATED CONSTRUCTION FEATURES

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ABSTRACT

With the public's demands for reduced construction time and traveling delays, full-depth precast bridge decks or decked concrete girders are being more widely used. For the precast bridge deck system with CIP (cast-in-place) connection, precast elements are brought to the construction site ready to be set in place and quickly joined together, and a concrete closure pour completes the deck connection and ties the individual units together in a manner that is intended to emulate monolithic behavior. This paper focuses on the development of the design guidelines of improved transverse U-bar joints for accelerated bridge construction. Testing results from the University of Tennessee were analyzed to determine the design recommendations. The test variables included concrete strength, U-bar spacing, and overlap length. A strut-and-tie model (STM) is examined according to testing results. A "FBD" (Free Body Diagram) method is proposed to calculate the ultimate moments of the joints, which is capable of identifying the most critical parameters and yields safe and consistent predictions. Design recommendations for U-bars and lacer bars are developed for full-strength joints.

Keywords: U-Bars, Joints, Accelerated Bridge Construction, Design, STM.

INTRODUCTION

The interstate highway system is vitally dependent upon current and future bridges. These bridges must be designed economically and with limited maintenance. For precast bridge construction a portion of the design must consider the bridge connections. Some current connections have had serviceability problems as evident in uncontrolled cracking. In other connections there are uncertainties in the calculations (or lack of calculations) which require design guidelines.

This paper presents design recommendations for precast decking u-bar reinforcement in tension which results from negative moment over a pier. Testing results from the University of Tennessee were analyzed to determine the design recommendations. The calculated capacity of the specimens was determined first by strut and tie modeling by AASHTO, but was shown to be insufficient. Proposed changes to the current calculation of the strut width as specified in AASHTO STM methods were discussed in order to match the test results. However, strut and tie modeling demonstrated that the design for the lacer bar was inadequate.

A triangular shape develops from the flow of forces in the connection joint zone; as a result, a free body diagram (FBD) was developed from the concrete triangular shape. This diagram showed how the forces flow in the in-situ joint as well as how they are resisted. A formula was developed from the FBD to determine the capacity of the joint which accurately reflected the capacities from tests. A FBD was also made of the lacer bar utilizing the forces and geometry calculated from the capacity calculations. A computer analysis program was used to determine the forces in the lacer bar. The lacer bar could then be designed since the required forces to resist (moment and shear) were known. A comparison of the strut and tie model to the triangular method led to the conclusion that both can determine the longitudinal reinforcement spacing, joint overlap length, and concrete strength, but only the triangular method can determine a more sufficient lacer bar size.

SUMMARY OF LAB RESEARCH

Testing of the in-situ joint connection has been done at the University of Tennessee by Sam Lewis (2009) and Beth Chapman (2010). There were two joint directions considered in testing: a longitudinal joint and a transverse joint. The two different joints experienced different forces and had to be tested accordingly. The longitudinal joint was tested in bending since the decking between the girders will experience moment. Tension controlled the transverse joints due to the negative moment in the girder (such as a negative moment over a pier); consequently the joints were tested in pure tension. The type of connection to resist these forces was investigated by Sam Lewis (2009). Lewis tested u-bar and headed bar connections (shown in Figure 1 and Figure 2, respectively) in order to determine which connection performed better. Performance was dependent upon strength, ease of construction, ductility, and cracking (Lewis 2009).

The tensile capacities of the u-bar and headed bar were 414.7 kN (93.24 kips) and 408.2 kN (91.78 kips), respectively. The u-bar specimens also experienced more ductility. During construction it was found that the u-bar joint detail was easier to tie and set in place. The u-bar detail was also found to be less congested than the headed bars which would allow easier deck placement. Lewis (2009) concluded that the u-bar detail should be further considered for the in-situ joint connection.

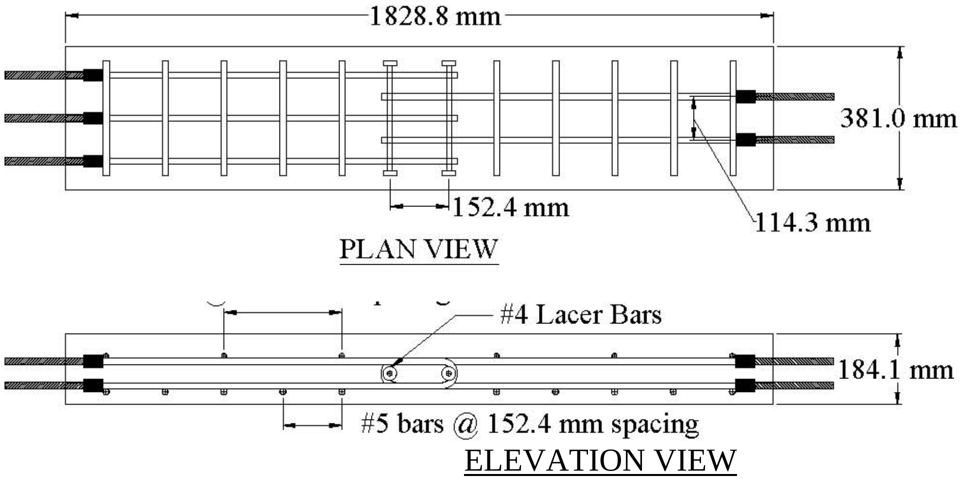

Figure 1: U-bar transverse joint

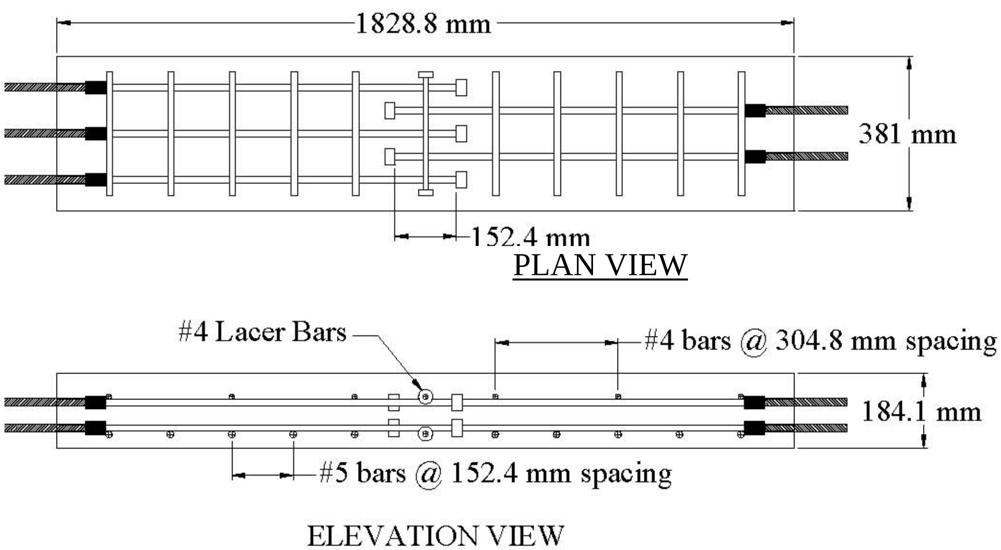

Figure 2: Headed bar transverse joint

Once the u-bar was selected for further testing, Beth Chapman (2010) produced more specimens to test in bending and tension. Figure 3 shows the tensile specimens' dimensions and reinforcement layout. In order to further understand the function of the u-bar joint connection in tension, three different parameters were considered: joint overlap length, u-bar spacing, and concrete compressive strength (values are shown in Table 1). Three different specimens were tested in tension. WT-4 had a different width of 508 mm (20 inches) instead of 381 mm (15 inches) for the other specimens.

The testing results, shown in Table 1, demonstrate that by increasing the u-bar spacing to 152.4 mm (6 inches) from 114.3 mm (4.5 inches) the capacity of the specimen increased approximately fourteen percent. If the joint overlap length is decreased to 101.6 mm (4 inches) from 152.4 mm (6 inches) then there is a decrease of approximately twenty three percent. Finally, if there is a decrease in the concrete compressive strength, then there will be a decrease in the specimen's capacity dependent upon the concrete compressive strength (Chapman 2010).

Peng Zhu (2010) completed testing of four different specimens for static test (represented by ST in Table 1) and fatigue tests (represented by FT in Table 1). The specimens consisted of the same dimensions and reinforcement layout (shown in Figure 3), as well as the same u-bar spacing and joint overlap length (shown in Table 1). Two panels were poured first and after the panels' concrete cured, the in-situ joint zone was poured to connect the panels. The in-situ concrete compressive strength and tested capacities are shown in Table 1 which provides similar results to Chapman's specimens with respect to the concrete compressive strengths (Zhu 2010).

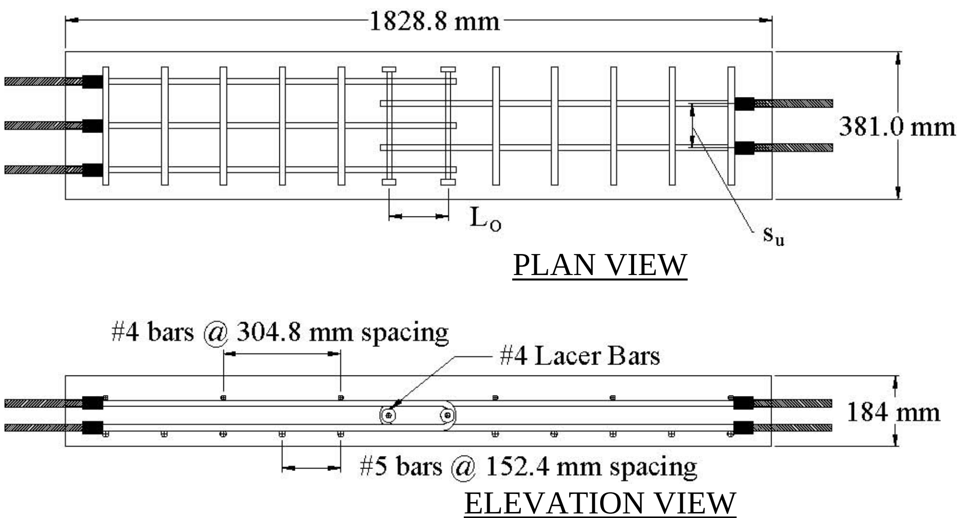

Figure 3: Tensile specimens WT-1, WT-2, WT-3, ST-0, ST-7, FT-0, and FT-7 with varying parameters shown in Table 1

Table 1 Testing Program

Specimen	f'_c		U-Bar Spacing (S_u)		Joint Overlap Length (L_o)		F_{TESTED}	
	(Mpa)	*(psi)*	*(mm)*	*(in)*	*(mm)*	*(in)*	(kN)	(k)
WT-1	66.1	9582	114.3	4.5	152.4	6	414.6	93.2
WT-2	53.2	7719	114.3	4.5	152.4	6	394.5	88.7
WT-3	65.5	9496	114.3	4.5	101.6	4	336.3	75.6
WT-4	66.0	9576	152.4	6	152.4	6	474.2	106.6
ST-0	32.1	4656	114.3	4.5	152.4	6	301.6	67.8
ST-7	68.8	9979	114.3	4.5	152.4	6	416.0	93.5
FT-0	34.3	4975	114.3	4.5	152.4	6	290.0	65.2
FT-7	65.5	9500	114.3	4.5	152.4	6	450.0	101.2

STRUT AND TIE MODELING

Through testing, researchers have observed a triangular formation of the concrete core in the in-situ joint section and have proposed using the strut and tie modeling method to design the joint zone (Chapman 2010, Lewis 2009). The strut and tie modeling method incorporates the compressive strength of concrete, called a strut, and the tensile strength of the rebar, called a tie. Due to the concrete and rebar interaction the forces will flow in such a way that a model can be developed. Upon simple observation the in-situ joint zone forms a truss shape which naturally is the idealized use for strut and tie modeling.

ACI strut and tie modeling

ACI 318-08 gives strut and tie modeling design criteria in Appendix A (ACI 318-08). Figure 4 demonstrates how a truss model can be formulated utilizing the lacer bar and u-bar spacing. The applied loads at the u-bar are given from the ultimate loads found in testing divided by the number of u-bars applying the load. There is equilibrium of forces in the model since the sum of the forces on both sides equals the ultimate capacity. The outer triangles, represented by the dashed lines, are considered zero bars in the model given that if the method of joints is used at point G the force in strut AG and tie GB are zero. In order to provide an example, the u-bar spacing, joint overlap length, and concrete compressive strength of Specimen WT-1 are applied to Figure 4. The maximum forces flowing through their respective joints are calculated by the method of joints provided in Table 2.

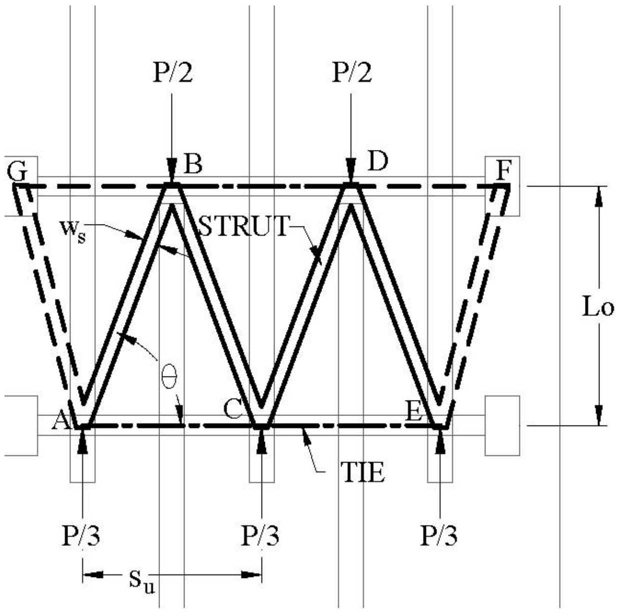

Figure 4: STM of joint section

Table 2: Maximum forces in their respective joints

Joints	Strut		Tie	
	(kN)	(k)	(kN)	(k)
A	147.58	33.18	51.83	11.652
B	147.58	33.18	0	0
C	73.79	16.59	51.83	11.625
D	147.58	33.18	0	0
E	147.58	33.18	51.83	11.625

AASHTO strut and tie modeling

AASHTO also provides STM design specifications in section 5 of concrete structures. AASHTO differentiates itself from ACI in equation 5.6.3.3.3-1 (AASHTO 2010). This equation takes into account the tensile strain of the concrete from the tension tie in equation 5.6.3.3.3-2 (AASHTO 2010). However, the lacer bar does not undergo uniform tension but instead experiences excessive bending deformation (discussed later). Therefore, the tensile strain in the concrete at the tension tie is assumed to be zero. If the tensile strain is not zero a value then a concrete compressive strength less than 0.85f'c would be used, therefore, using 0.85f'c is the maximum that could be used for the calculation. From this assumption the compressive concrete stress is then limited to 0.85f'c, resulting in the same strut capacity as the ACI STM design specifications.

Adjusted joint strut and tie model

Using STM design methods developed by ACI 318-08 and AASHTO specifications does not produce reasonable results, since these designs are not specific for this type of connection.

The core of the concrete in the in-situ joint is not fully utilized in STM calculations. If STM is to be used, then something must be corrected to fully utilize the concrete core and provide more accurate capacity calculations. Hawkins et al. (2005) mention using the depth of a beam with the angle of the strut to the tie to find the width of the strut. While the concrete core is not a deep beam nor the angle between the strut and tie 45 degree or less (as is the criteria from Hawkins et al. (2005)), this idea may be utilized in the strut capacity calculations in order to give a larger strut area and therefore a larger capacity to be more comparable to testing values. Figure 5 shows the width of the strut which better utilizes the concrete in the in-situ joint zone. This calculated capacity is close to the maximum force in the STM and may be considered as a possible calculation for the capacity of the in-situ joint zone.

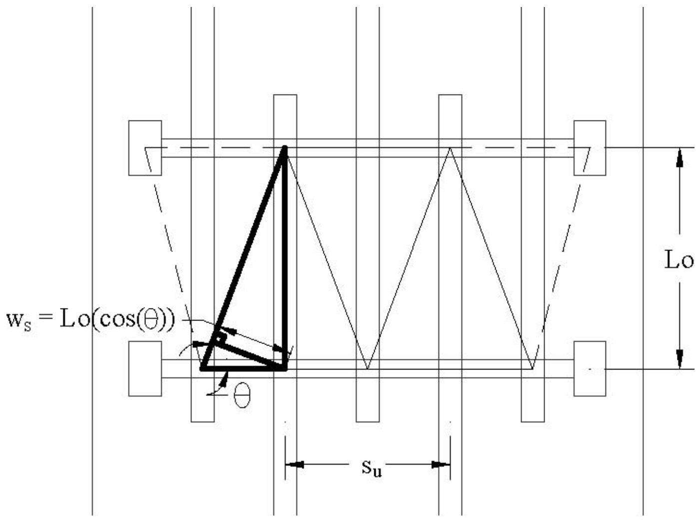

Figure 5: Width of the strut

Figure 6 shows a graph of the tested capacities verses the calculated capacities based on the concrete compressive strength. Any point above one on the vertical axis is considered conservative; therefore when the compressive strength decreases the capacities may become very conservative depending on the reduction in concrete compressive strength. Once the concrete compressive strength reaches approximately 68947.57 kPa (10,000 psi) the trend line reaches one on the vertical axis signifying that the calculation is equal to the tested value.

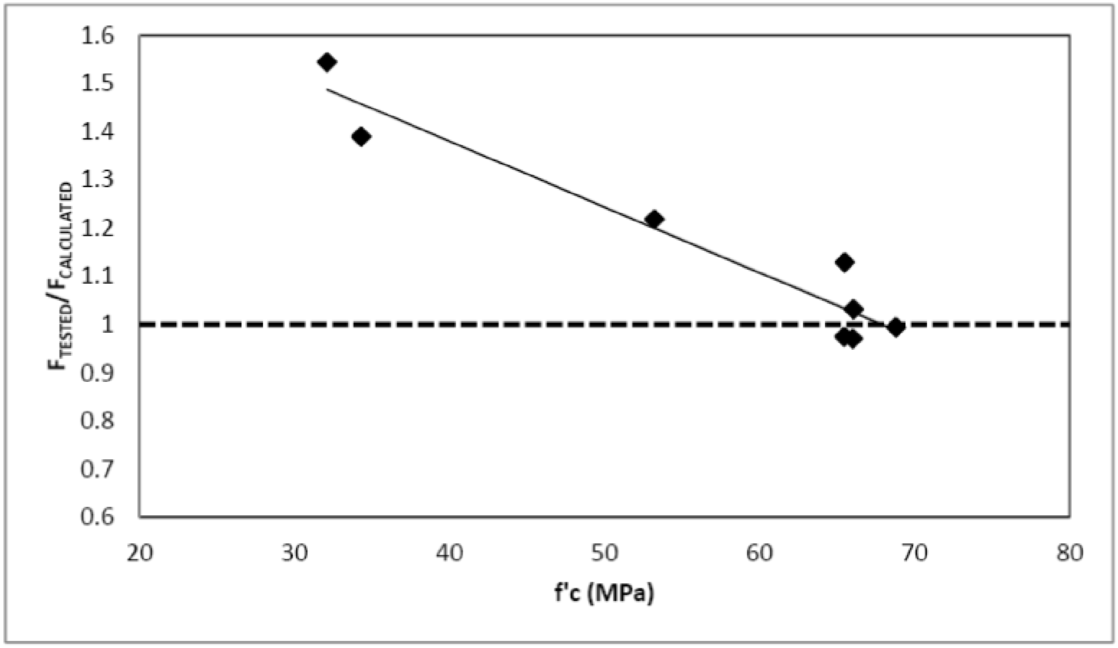

Figure 6 STM tested capacities verses calculated capacities

While the strut and tie model produces reasonable capacities with the widening of the strut, incorrect assumptions are made as the following state:

- Strut and tie modeling assumes the forces flow through the model in such a way as to only produce tension and compression in the members; however, from testing this assumption for the lacer bar is in incorrect. If the lacer bar is designed according to the strut and tie model then a number four rebar should be sufficient for the capacity. However, the lacer bar of this size undergoes excessive bending deformation as is evident from observation after the testing and the lack of uniformity of the strain gauge readings from the lacer bars. If STM does not provide accurate modeling nor design for the lacer bar then another model must be used to provide accurate modeling and design.
- Strut and tie modeling does not accurately model how the in-situ joint zone functions. As stated before, the outer struts in the model, struts AG and EF, have no forces going through them, however, this is incorrect. This section of the concrete core is obviously important as there are forces flowing from A to G and concrete must be in this zone otherwise the in-situ joint will not reach capacity. The concrete within the dashed triangles of Figure 4 must be accounted for in design. According to the model shown in Figure 4 there are no forces acting on the top lacer bar. From testing, however, there is deformation in both of the lacer bars therefore a conclusion can be made that this model is not accurate for the top lacer bar. Even if the design of the bottom lacer bar is used the designer would not know if the top lacer bar controls due to the fact that there are different loading scenarios on both of the lacer bars.

PROPOSED FBD MODEL

The new design method for determining the capacity of the in-situ joint zone must be able to model the joint zone correctly, determine an accurate capacity, and design the lacer bar appropriately. Figure 7 shows the specimen examined after failure.

Figure 7 Failure of Specimen

The triangular shape shown in Figure 7 signifies the failure in the concrete of the transverse joint specimen. The concrete center triangle remains intact which helps to illustrate how the forces are transferred through the concrete. The geometry of the triangle should compose of the u-bar spacing and lacer bar spacing as is shown in joint zone in Figure 8.

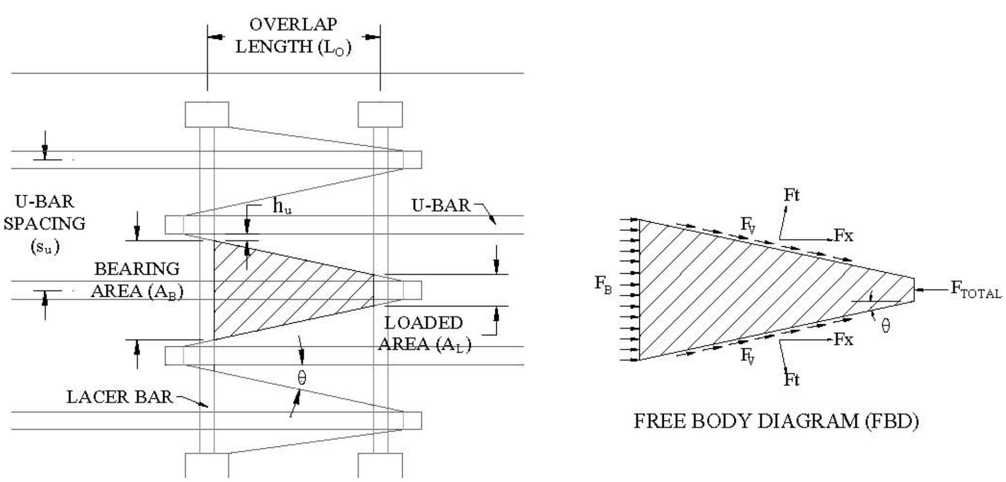

Figure 8: Proposed “FBD” Model

The loaded area (A_L) at the radius of the u-bar for the concrete center triangle is obviously taken from a combination of the u-bar and the lacer bar as is evident in Figure 7 and Figure 8.

The force flows from the u-bar to the loaded area and is then distributed directly to the bearing area (A_B). The bearing area shall b e defined as the area of concrete which bears on the lacer bar and u-bars on either side of the u-bar applying the load, as a reaction to the applied load of the u-bar; therefore, one of the strength parameters of the concrete shall be the bearing of the concrete (F_B). In using the joint zone of Figure 8 as a reference, the length shall be the inner edge of the lacer bars of the overlap length. A line going from the inside radius of the opposing u-bars intersects the lacer bars which gives the width of the triangular specimen.

Sam Lewis (2009) noted that the transverse specimen would crack in the transverse direction above the lacer bar first. Longitudinal cracking would then occur between the transverse cracks and failure would occur when the longitudinal cracks would reach the transverse cracks (Lewis 2009). From Lewis' cracking observations and the free body diagram ("FBD"), shown in Figure 8, there is a tensile strength (F_T) and a shear strength (F_V) of the specimen; however, a pre-cracking and post-cracking stage of the in-situ joint zone must be considered. The pre-cracking stage for the triangular concrete specimen is composed of the horizontal strengths of the tensile and shear strengths, but once the in-situ joint zone cracks (post-cracking), no more tensile strength can be developed. The ultimate strength is then dependent upon the shear strength. The post-cracking stage will only be considered since this calculation is for the ultimate capacity; therefore, the tensile strength will be understood to be zero. Once cracking has occurred, the shear strength can be developed from the interlocking of aggregate and the friction of the two interlocking faces of the opposing triangles.

Based on the proposed "FBD" model, Figure 9 represents the calculation verses actual failure (y-axis) depending on the concrete compressive strength (x-axis). Any Tested/Calculated value (y-value) above one is considered conservative. Most cases are slightly conservative with the exception of specimen WT-3 which has a value of 0.99, representing a valid calculation theory for this specimen.

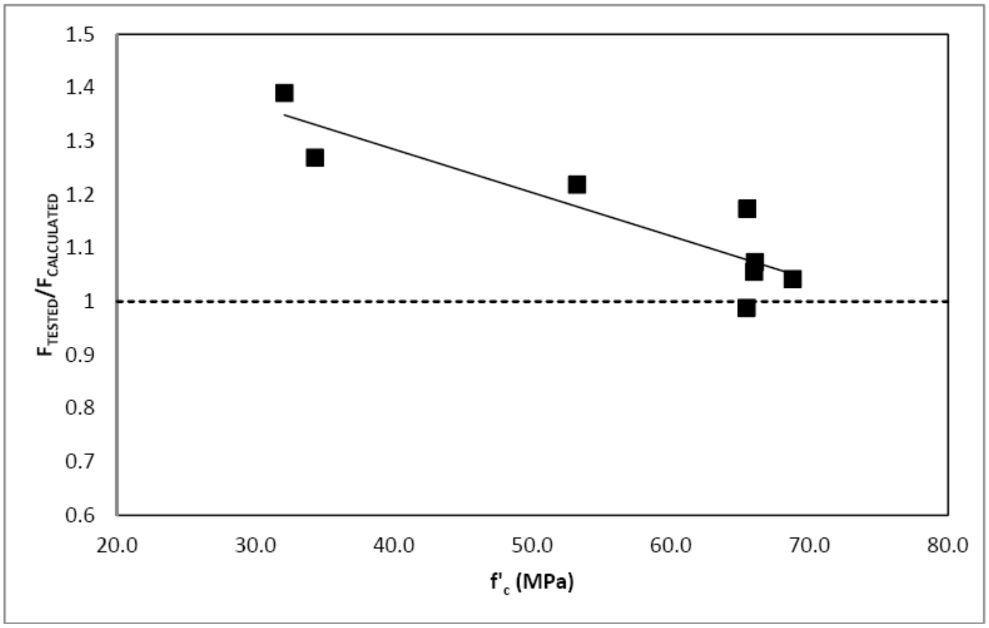

Figure 9 Verification of the proposed "FBD" model

LACER BAR

Previous research projects have not provided sufficient information about the lacer bar. Most researchers understand that it allows the joint zone to be more ductile as there have been many tests which show the joint zone is brittle without the lacer bars (Gordon et al. 2005). The strain gauge configuration on the lacer bar of Sam Lewis' (2009) and Beth Chapman's (2010) research assumes the lacer bar acts in tension. The test results, however, show the lacer bar acts in bending, as shown in Figure 7. Further observation of the deflection shows that the lacer bar deforms where the u-bar applies the force and where the lacer bar bears against the concrete. This observation would verify that force flows in the order of the following: The tensile force pulls the u-bar, the u-bar bears on the lacer bar, the lacer bar transfers the force to the concrete (the loaded area), the concrete distributes the force in a triangular pattern in the direction of the opposite lacer bar where the concrete bears against the lacer bar (bearing area).

The failure of the lacer bar should be carefully considered. The lacer bar allows ductility but also gives excessive cracking along the in-situ joint section which may not satisfy cracking and serviceability requirements. Therefore, modeling the lacer bar would prove beneficial in understanding and assisting with the design of the lacer bar. The proposed "FBD" method has been used to model the lacer bar. In order to model the lacer bar the following parameters and restraints were used: The length of the lacer bar is from center to center of the heads of the lacer bar. The ends are assumed fixed due to a tangent line of the deflected shape approximately perpendicular to the head and also due to the concrete surrounding the head on the inside of the lacer bar which mostly stays intact after failure. There are three loads applied to the lacer bar as follows: firstly, the direct load from the u-bar taken as a uniformly distributed load for a distance equal to the diameter of the u-bar), secondly, a uniformly distributed bearing load from the base of the concrete triangle, and lastly, the bearing strength of the concrete as the lacer bar deflects and bears against the concrete from the u-bar loading.

The proposed design method assumes the concrete in the in-situ joint zone will develop a triangle. This assumed triangle's capacity is calculated based on a free body diagram of one triangle taking the observed confinement of the in-situ joint zone into consideration. It should be noted that this method calculates the capacity of the concrete in the in-situ joint zone, but this is not the only failure mode of the decking joint which must be checked. The objective of this calculation is to check for u-bar failure before the concrete joint failure since the u-bar will provide more ductility. The u-bar allowable tension and serviceability must be checked. In the in-situ joint zone the serviceability is related to how the lacer bar deforms, indicating that certain design criteria must be developed for the lacer bar.

DESIGN GUIDELINES

The University of Tennessee has completed u-bar transverse joint tests using three different parameters: concrete compressive strength, u-bar spacing, and joint overlap length. Some limitations must be given to these parameters.

As the concrete compressive strengths decreased the calculated values became more conservative. Figure 9 shows that if the concrete compressive strength reaches approximately 69 MPa (10,000 psi) then the calculated capacities mirror the tested capacities and are no longer considered conservative since the values would approximate one in Figure 9. This calculation would then not be applicable for high strength concrete and should only be used for normal weight concrete up to 69 MPa (10,000 psi).

Specimen WT-4's u-bar spacing was increased to six inches from 4.5 inches resulting in an increase in ultimate capacity. This can be explained in the proposed method due to the increase in loading and bearing areas. However, to increase the tensile capacity of the joint zone a lower u-bar spacing should be used. Figure 10 shows that the total connection's capacity is increased if a smaller spacing is used. The minimum u-bar spacing is limited by the spacing requirements in ACI 318-08 section 7.6.1 which states that the spacing cannot be less than the diameter of the bar or 25.4 mm (one inch). In the case of a number 5 rebar, the smallest u-bar spacing allowed by ACI 318-08 would be 82.55 mm (3.25 inches). In terms of strength it is not logical to increase the u-bar spacing but in order to reduce costs a maximum spacing may be desired. There has been no testing done for the maximum u-bar spacing by the University of Tennessee. Eventually, however, there would be an angle which would not allow the shear capacity to develop strength once the cracks have formed. If u-bar spacing is desired to exceed six inches then it would be recommended to run further tests to determine the capacity of horizontal strengths developed once cracked.

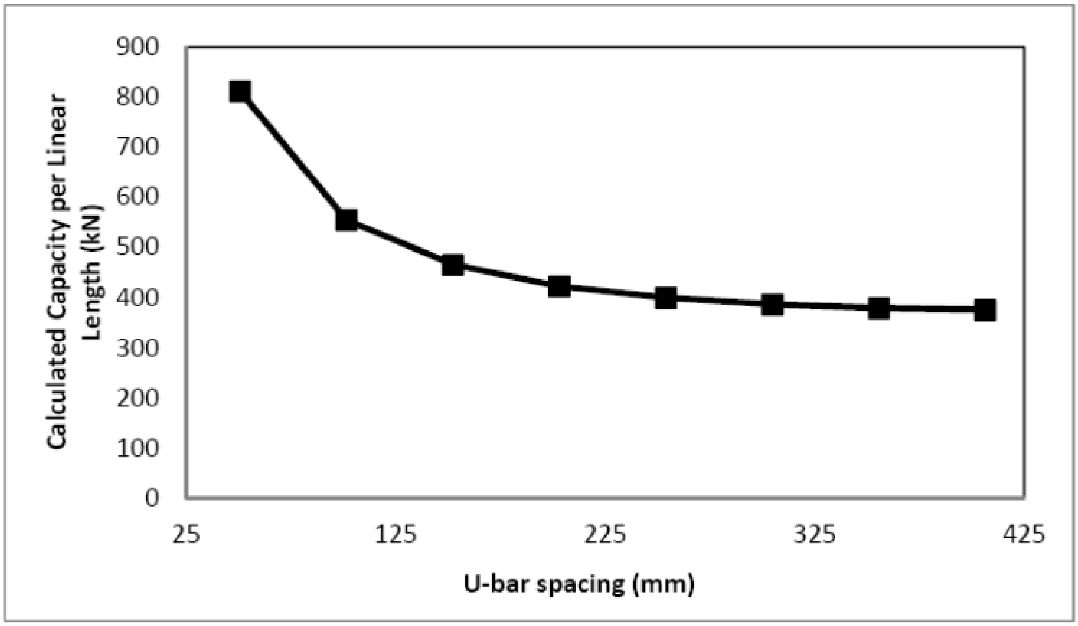

Figure 10 Impact of U-bar spacing

Once the joint overlap length was reduced to 101.6 mm (4 inches) from 152.4 mm (6 inches) the capacity of the specimen was also reduced. Figure 11 shows that with the increase of the joint overlap length there is an increase in the joint's strength. Therefore, with any increase of the joint overlap length there would be an increase in the joints capacity. The limitation would then come from economics since with a larger overlap length would be more concrete to pour on site. The University of Tennessee has not done testing on joint overlap lengths above 152.4 mm (6 inches). Theoretically this would increase the capacity but this would need to be verified by testing. Chapman (2010) recommended not decreasing the joint overlap length below 152.4 mm (6 inches) since the crack widths were enlarged and inadequate ductility was experienced with an overlap length of 101.6 mm (4 inches).

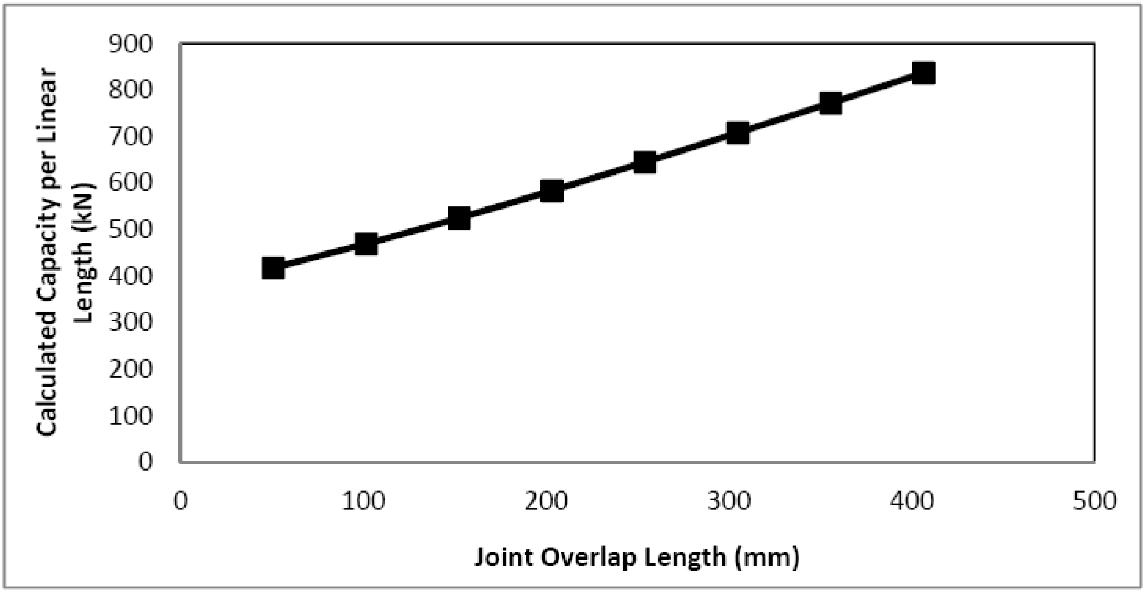

Figure 11 Impact of joint overlap length

The lacer bar is imperative in the design given that it provides ductility, confinement, and bearing for the concrete. In order to accurately design for the lacer bar, a bending analysis can be done for the lacer bar configuration based on the proposed model. As the lacer bar deflects it causes cracking in the in-situ joint region, which is not desired for serviceability. This cracking can be decreased if the lacer bar diameter is large enough to resist the applied moment from the tensile forces. The options to consider are to increase the lacer bar size or to add additional lacer bar(s). However, the lacer bar size is limited by the required spacing between rebar according to ACI 318-08 spacing criteria which limits the size or configuration of the lacer bar(s) providing sufficient concrete flow in the joint zone. Increasing the area of the lacer bar will decrease the deflection, however, this is not a strength criteria but a serviceability requirement.

SUMMARY

The University of Tennessee has proposed a u-bar connection to increase flexural and tensile capacities, thereby decreasing the cracking in the joint zone. To further understand the connection, different parameters were given to the specimens as follows: concrete compressive strength, u-bar spacing, and joint overlap length. It was found that, as the u-bar spacing was increased, the capacity increased. Also, if the joint overlap length was decreased, then the capacity was decreased.

Two different methods were examined to mathematically determine the capacity of the connection. Strut and tie modeling (STM) was first examined, but if ACI's or AASHTO's STM criteria were followed the calculated capacities were significantly lower than that obtained from testing. If an increase of the strut's width was allowed, however, then the increase in capacity compared reasonably to the tested capacity.

The “FBD” method was proposed to determine the capacity which analyzed a triangular concrete shape. A free body diagram (FBD) of the triangular shape of the concrete in-situ joint, bounded by the u-bar and lacer bar spacing, could be analyzed and used to determine the specimen’s capacity. Both the strut and tie modeling and the triangular method produced accurate and reasonable calculated capacities compared to the tested capacities. The observed design difference was the analysis and design of the lacer bar. From testing and observation the lacer bar underwent bending deformation.

The proposed method allowed the lacer bar to be analyzed in bending, similar to the testing results. The lacer bar should be designed for serviceability since the lacer bar assists in controlling the cracking. Also, serviceability design would prevent over-designing the lacer bars, thus allowing the concrete to flow freely around the u-bar bend.

ACKNOWLEDGEMENTS

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