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16. Abstract

Several alternatives are used to connect prefabricated bridge deck elements using closure joints but cost and construction considerations limit their use in field applications. In this research, a new detail is proposed to efficiently connect these elements in closure joints. The new detail consists of 90° hooked reinforcement and normal strength concrete in the closure joint. An experimental program was conducted to develop design criteria for the suggested detail. Test results have shown that the longitudinal connection detailed with hooked bars can be a viable alternative for the design and construction of closure joints. This report provides a summary of experimental tests on the new closure joint detail and tentative design recommendations.

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Investigation of Longitudinal Closure Joint Using 90° Hooked Bars in Accelerated Bridge Construction

Final Report

June, 2019

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1.1 Abstract

Several alternatives are used to connect prefabricated bridge deck elements using closure joints but cost and construction considerations limit their use in field applications. In this research, a new detail is proposed to efficiently connect these elements in closure joints. The new detail consists of 90° hooked reinforcement and normal strength concrete in the closure joint. An experimental program was conducted to develop design criteria for the suggested detail. 48 specimens were tested to study the effect of lap splice length, the lateral distance between transverse reinforcement, and bar size. The specimens were tested in flexure and were compared with control specimens. The objective of the experiment was to obtain optimal lap splice length based on modes of failure and ductility ratio. Test results have shown that the longitudinal connection detailed with hooked bars can be a viable alternative for the design and construction of closure joints. This report provides a summary of experimental tests on the new closure joint detail and tentative design recommendations.

1.2 Introduction

A major portion of the bridge infrastructures in the United States is approaching their design life and consequently needs replacement or repair. Many of these bridges are located in crowded roadways, and closure of these bridges for an extended period of time is not feasible. In order to minimize traffic disruptions, the bridge can be temporarily closed while performing construction activities using accelerated bridge construction (ABC) techniques. Minimizing construction time and activities performed in the field not only decrease detour time and traffic jams but also increase safety for workers, vehicles, and the traveling public. One of the most commonly used ABC methodology involves modular bridge systems which consist of precast girder/slab elements. These precast elements are placed on supports and connected to each other using cast-in-place closure joints. Since these modular sections have a pre-topped deck tributary area, extensive forming and scaffolding are eliminated.

In modular bridge superstructure, the shear and moment are transferred through the longitudinal deck joints which run parallel to traffic direction. These joints are functionally required to connect one modular unit to the adjacent units, however, the performance of these joints can be affected by environmental attacks and structural degradations [1]. Durability issues have been encountered in these longitudinal joints with the use of welded steel connectors. In addition, cracking has been observed in bridge overlays of prestressed concrete bridges. These cracks are continuous along the length of the bridges and are prone to deterioration of both superstructure and substructure. A leaking crack can be a potential hazard for vehicular traffic on bridges with highway underpasses [1],[2]. The water leaking through the deck causes serious issues such as corroding deck reinforcement, concrete spalling and expansion due to water freezing and corroding cross-frames if water leaked through the full deck depth. Water leaking is not causing such traffic hazards as it is a serviceability issue.

In order to control deck cracking, several researchers proposed using distributed reinforcement [3]. The use of closely spaced reinforcing bar provides better stress distribution when compared to widely spaced reinforcing bars [4]. In addition, it is desirable to keep the joint width smaller to reduce cost and construction time. However, this reduction in joint width prevents the straight and hooked bars from satisfying AASHTO-LRFD development length requirements [5]. Therefore, special details including spiral wire, U-shaped bar, straight bar, and headed bar have been developed to meet these requirements as shown in Figure 1-1.

One of the disadvantages of using spiral wire is difficulty in its fabrication, which increases the cost of installation [3]. For U-shaped bar, AASHTO-LRFD section 5.10.2 (ACI 7.1 and 7.2) outlines bend diameter which requires the deck to have a thickness greater than 9.5 in [5], [6].

However, this value is typically less than 8.5 in. and thus U-shaped detail cannot be used. On the other hand, headed bars provide satisfactory structural performance but their limited availability, cost, and reduction in concrete cover at the bar head make them a less feasible option.

Figure 1-1 Details of current options for splicing reinforcement in closure joints [7].

On the other hand, hooked bars offer benefits of easy fabrication and reasonable cost. In the past, many studies have been conducted on hooked bar anchorages in beam-column connections [8],[9],[10]. These studies were conducted on typical beam-column connections subjected to varying levels of confinement at the connections including side concrete cover, column axial load, longitudinal column reinforcement, and transverse reinforcement through the connection. The results showed spalling of concrete cover at hooked anchorage level was the main reason of failure [8]. It was also concluded that there is no significant difference between the capacity of 90° hooks and 180° hooks although the slippage at given stress was greater in the case of 180° hooks [8].

The pullout tests in typical beam-column connections were carried out on hooked bars with different straight lead embedment lengths, l_l, and design recommendations of embedment length, l_{dh} of bars were suggested based on these results [9]. Test results concluded that the main factors affecting the capacity of beam-column connections are the level of lateral confinement and embedment length [9]. Limitation values of l_{dh} were provided based on test results in which the failure mode occurred by spalling of the concrete side cover. However, in the closure joint, the adjacent concrete provides large confinement to the hook and the spalling of concrete side cover rarely occurs.

Testing was conducted on hooked bar anchorages with short embedment length ($l_{dh} \leq 7d_b$) and results showed that the failure mode occurred by the loss of concrete cover in front of the hook. This failure mode is different from typical pullout failure and side-splitting [10]. For the range of concrete strength considered, the capacity of hooked bar anchorage was proportional to $\sqrt{f_c'}$ [10].

Many studies have been conducted on tension development length of reinforcing bars in high strength concrete [11],[12],[13],[14]. The results of these experimental studies showed that both tension splice length can be designed to prevent brittle failure if a minimum amount of stirrups are provided over the splice region [11]. This minimum amount of stirrups can be calculated by an expression given in the literature [12]. The recommendations provided in ACI 318-95 for concrete strengths above 10,000 psi were made to ensure sufficient ductility and bond which improve the overall performance of the spliced reinforcement. The lap splice capacity is not increased with any further increase in development length [13].

Other recommendations include a 20% increase in l_{dh} value for the anchorage strength in the case of epoxy coated hooked bars [15]. Also, based on a strut and tie model, recommendations were made on the development length of standard hook anchorage with high strength corrosion resistant reinforcement [16].

A 90^{o} hook in a reinforced concrete joint need to meet AASHTO-LRFD requirements. To develop a bar in concrete, the AASHTO-LRFD code specifies a length of tail extension of $12d_b$

for a 90^{o} hook [5]. Previous studies were used as bases for AASHTO-LRFD and ACI requirement and have shown that the bend portion of the hooks can pop out when either short length of hook or smaller concrete cover are used [8],[9],[10]. However, this scenario is not common in closure joint regions where large confinement is provided by the adjacent concrete.

1.3 Objectives

Although the hooked bars have been used extensively in different applications, their use in closure joints is limited due to lack of experimental data. The objective of this research is focused on studying a hooked detail in the closure joint region through experimentation. To understand the mechanism of development of 90^{o} degree hooks, a detailed experimental program was conducted by incorporating different parameters such as bar size, lap splice length, the lateral distance between reinforcement, and transverse reinforcement. The experimental research incorporating these parameters will fill the gaps in the existing studies and will provide additional details that will help to understand the mechanism of failure in closure joints. This mechanism will be used for revising existing design provisions and possible incorporation of 90^{o} hooked details in construction applications. This report summarizes the results of an experimental study which was conducted to formulate a design procedure for calculating tension development length and tension lap splice length when the hooked bar is used in closure joints in ABC projects.

The performance of the proposed 90^{o} hooks meets structural requirements and can be practically implemented in field construction.

1.4 Current Details for Closure Joint

The use of girders with prefabricated deck panels is one of the most commonly used ABC methods for construction of bridge superstructure. These standalone prefabricated units are deployed on site and then connected to the other adjacent units using different closure joints.

One of the possible methods of connecting separate units is the use of straight lap bars in closure joints. However, straight lap bars require wider closure joints to develop sufficient development length based on the AASHTO-LRFD design specifications [5]. The other issue with straight lap bars in wide closure joint is the shrinkage cracking. Alternatively, headed bars are used which are currently in practice for construction of modular bridge systems. NCHRP 12-69 recommends using of headed bars for easier construction practices [17]. Among the many shortcomings of headed bars which may cause service life issues, the primary concern is the increased size of the head at the end, which can cause issues of reducing concrete cover. The headed bars require careful detailing to avoid interfering with adjacent headed bars during placement of two modular units as shown in Figure 1-2 [18]. In addition, the cost of headed bars is relatively higher.

Another common option to reduce the closure joint width is the use of Ultra-High Performance Concrete (UHPC) between the modular deck units. This material offers high compressive strength, high tensile strength compared to normal strength concrete, and low permeable solution to join the prefabricated elements. The higher tensile strength of this material makes it possible to provide adequate strength over short lap splice length [19]. However, the higher cost of this material compared to other available concrete mixtures limits its widespread use.

Lastly, the 180^{o} hooked bars provide detailing of closure joint, which are economical and constructible. The 180^{o} hooked detail provides a solution to the concrete cover problem experienced with headed bars. However, this detail limits the designers to use the same reinforcement size for the top and bottom layers, which is in some cases not the situation for bridge decks [18]. Moreover, meeting the bent bar requirements as outlined in AASHTO-LRFD section

5.10.2 (Section 7.1 and Section 7.2 of the ACI318-08) [5],[6] causes the deck thickness to be greater than 9.5 in., which is a function of girder spacing which is typically less than 8.5 in.

To overcome the shortcomings of the existing details, a new connection detail is proposed in this research by using 90^{o} hooked bars which overcome the concrete cover issue associated with head bars, deck thickness issue associated with 180^{o} hooked bars, and the wide closure joint in case of the use of straight bars. The new detail should be designed for adequate structural performance and service life for ABC applications.

Figure 1-2 Details of headed bar splicing reinforcement in closure joints [7].

1.5 Description of the Proposed ABC Connection

In the proposed closure joint, normal strength concrete along with 90° degree hooked bar is used to connect the pre-topped deck elements in the ABC application. As a part of the research performed by the University of Tennessee, Knoxville a survey was sent to various bridge professionals to determine concerns with current bar details. The primary concerns were the overall width of the closure joint and constructability of each of the details [18],[20]. In order to reduce the closure joint width and to achieve better constructability, hooked bars are commonly used. The objective of the proposed new detail in this study is to investigate the performance of

90^o hooked bars in closure joint. The hooked bars may be obtained from any local steel fabricator which greatly reducing the time and cost of fabrication and shipment to the work site. AASHTO specifies the face of closure joint to be roughened or have shear keys to ensure a proper bond and shear strength between segments [21],[22]. However, the worst-case scenario is when no surface preparation is conducted on adjacent segments which leads to a weaker shear interface due to face smoothness. Based on this assumption, details of the proposed specimens were designed using a smooth interface, as shown in Figure 1-3. The test specimens are designed for unit width of the deck in the transverse direction where moments are transferred through closure joint. The thickness of the slab was chosen to be 8 in. for all specimens, which is a typical thickness for bridge decks. The gap space in Figure 1-3 represents the transverse spacing of noncontact splices, which is limited by the smallest value of one-fifth of lap splice length or 6.0 in. [23]. For this reason, a gap space of 2.0 in. and 4.0 in. are considered as lower and upper limits, respectively. The empirical design of deck typically uses No. 4 bar, but No. 6 bar was also considered as an upper limit. To investigate the optimal lap splice length, a spliced length of 2-in. to 10-in. was considered with a 2-in. increment.

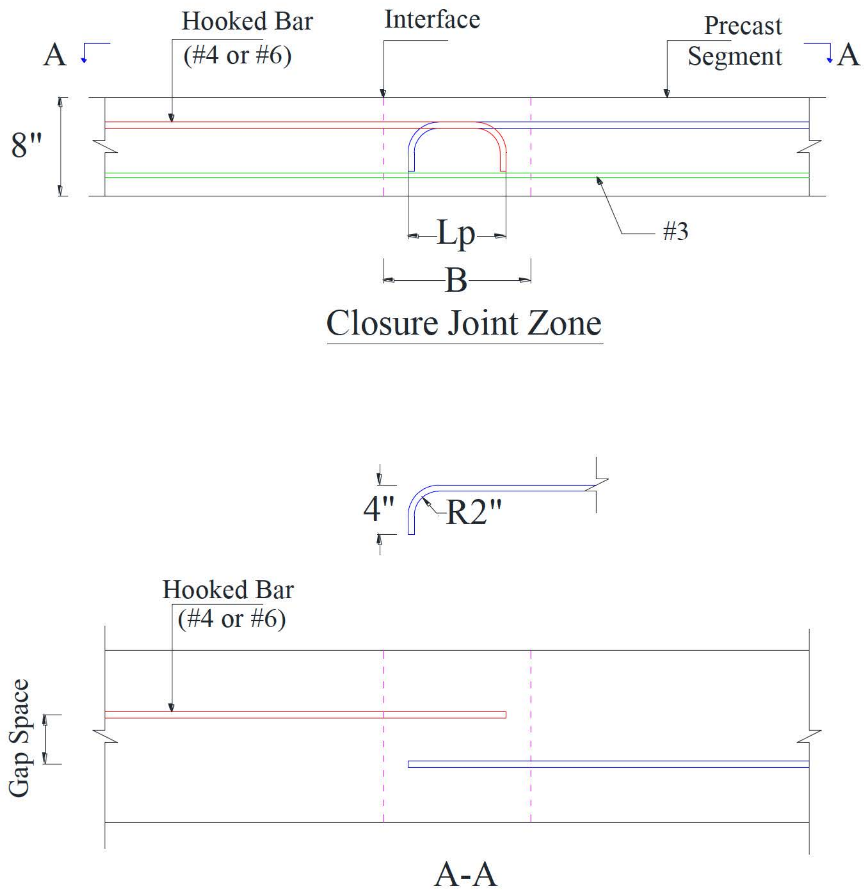

Figure 1-3 Details of hooked bar in the proposed connection (elevation view and section).

The main advantage of the new details includes a reduction in closure joint width. The main advantage of a narrower closure joint is the reduction in shrinkage cracking. The hooked bar can be developed within the deck thickness over a shorter length. Also, the hooked bars have a comparatively lower fabrication cost and their ease of construction saves time which agrees with the ABC goals.

1.6 Experimental Program

To evaluate the structural performance of the proposed closure joint detail, an experimental program was conducted at Florida International University (FIU). Based on the test program, the experiments were conducted in three phases. Each phase was investigated for different parameters, including lap splice (L_p) length, the lateral distance between hooked bars (Gap Space (GS)) and the use of transverse bar (TR) (each variable is defined in Figure 1-3). In this detail, lap splice (L_p) was measured from outside to outside of hooked bars as shown in Figure 1-3. To meet the requirement of the AASHTO-LRFD Bridge Design Specifications [22] for the confinement of hooks, the side cover is considered as 2 in. For each bar size, the range of L_P is incrementally varied from 2 in. to 10 in. with 2-in. increment. For bar size (BS), No. 4 and No. 6 are considered which are commonly used in bridge decks. Each group consists of 12 beams with a length of 8 ft. and a depth of 8 in. The design procedure for the design of modular bridge deck is similar to a conventional cast-in-place decks based on the AASHTO-LRFD Bridge Design Specifications [22]. The specimens were tested in a reverse test setup where the tension side was located at the top of the specimens due to some limitations in test setups. Figure 1-4 shows an elevation view of the reversed test setup.

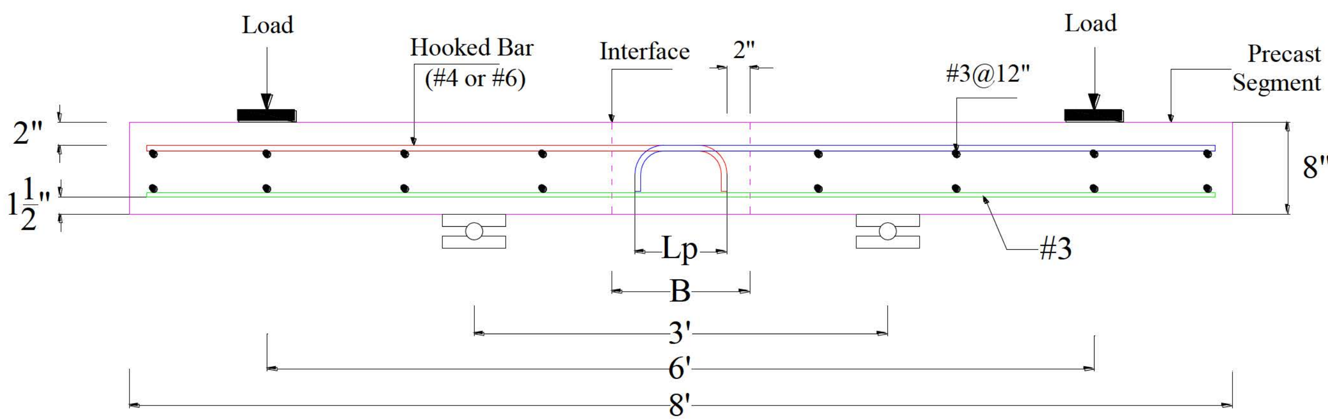

Figure 1-4 Specimen geometry, reinforcement, and reversed test setup.

During the first phase, hereafter, indicated by Group A, 12 test specimens were constructed with different lap splices (2 in. ≤ L_P ≤ 10 in.) with 2-in. increment. The bar sizes for lap splice were No. 4 and No. 6 with a constant gap space of 2 in. In the second phase, hereafter, indicated by Group B, another 12 specimens were constructed with the details identical to Group A, except for a gap of 4-in. between the longitudinal hooked bar in the closure joints region. Finally, Groups C and D test specimens were constructed with details identical to Groups A and B, with additional transverse reinforcement which is not used in Groups A and B. These transverse bars consisting of No. 3 bars at a spacing of 2.5 in. were placed over the hooked bars in the closure joint. This transverse reinforcement represents longitudinal reinforcement that is usually placed in the longitudinal closure joints. All the test specimens designated as S/ST-GS-BS-L_p

where: S is a notation for the specimens without transverse bars;

ST is a notation for the specimens with transverse bars in closure region;

GS is a notation for the lateral distance between the hooked bars (Gap Space) in inches;

BS is a notation for bar size in terms of bar number; and

L_P is a notation for lap splice length in inches.

For instance, S-2-4-2 specifies the specimen with no transverse reinforcement within closure joint, with a 2-in. lateral distance between the hooked bars, No. 4 bar size, and 2-in. lap splice length. Table 1-1 shows the specimen matrix details.

Table 1-1 Specimen Matrix, Unit: in.

Group	**ID**	**G S (in.)**	**Bar Size**	**L_p (in.)**	**Transverse Bar**	**Joint Width (in.)**
A	S-2-4-2	2	#4	2	NO	6
	S-2-4-4	2	#4	4	NO	8
	S-2-4-6	2	#4	6	NO	10
	S-2-4-8	2	#4	8	NO	12
	S-2-4-10	2	#4	10	NO	14
	S-2-6-2	2	#6	2	NO	6
	S-2-6-4	2	#6	4	NO	8
	S-2-6-6	2	#6	6	NO	10
	S-2-6-8	2	#6	8	NO	12
	S-2-6-10	2	#6	10	NO	14
	S-2-4-control	2	#4	Control	NO	N/A
	S-2-6-control	2	#6	Control	NO	N/A

Cont. Table 1-1

B	S-4-4-2	4	#4	2	NO	6
	S-4-4-4	4	#4	4	NO	8
	S-4-4-6	4	#4	6	NO	10
	S-4-4-8	4	#4	8	NO	12
	S-4-4-10	4	#4	10	NO	14
	S-4-6-2	4	#6	2	NO	6
	S-4-6-4	4	#6	4	NO	8
	S-4-6-6	4	#6	6	NO	10
	S-4-6-8	4	#6	8	NO	12
	S-4-6-10	4	#6	10	NO	14
	S-4-4-control	4	#4	Control	NO	N/A
	S-4-6-control	4	#6	Control	NO	N/A
C	ST-2-4-2	2	#4	2	#3	6
	ST-2-4-4	2	#4	4	#3	8
	ST-2-4-6	2	#4	6	#3	10
	ST-2-4-8	2	#4	8	#3	12
	ST-2-4-10	2	#4	10	#3	14
	ST-2-6-2	2	#6	2	#3	6
	ST-2-6-4	2	#6	4	#3	8
	ST-2-6-6	2	#6	6	#3	10
	ST-2-6-8	2	#6	8	#3	12
	ST-2-6-10	2	#6	10	#3	14
	ST-2-4-control	2	#4	Control	#3	N/A
	ST-2-6-control	2	#6	Control	#3	N/A
D	ST-4-4-2	4	#4	2	#3	6
	ST-4-4-4	4	#4	4	#3	8
	ST-4-4-6	4	#4	6	#3	10
	ST-4-4-8	4	#4	8	#3	12
	ST-4-4-10	4	#4	10	#3	14
	ST-4-6-2	4	#6	2	#3	6
	ST-4-6-4	4	#6	4	#3	8
	ST-4-6-6	4	#6	6	#3	10
	ST-4-6-8	4	#6	8	#3	12
	ST-4-6-10	4	#6	10	#3	14
	ST-4-4-control	4	#4	Control	#3	N/A
	ST-4-6-control	4	#6	Control	#3	N/A

1.7 Construction of Test Specimens

The structural performance of a closure joint was replicated under a flexure test for a unit width. The closure joint was cast in two stages. During the first stage, the adjoining decks were cast in a formwork separated by a closure joint. Upon the completion of curing for a minimum of 28 days, the closure joint was cast. The concrete used for both the slab and closure joint had a compressive strength of a minimum of 5 ksi (FDOT CL II deck concrete). Before casting of the closure joints, no surface preparation was done on cast deck portion. This condition represents the most unfavorable condition of a cold joint. In addition to the test group specimens, control specimens of monolithic construction were made. These control specimens replicate the cast-in-place slab of conventional bridges that have no closure joints and contain straight bars. The test specimens were painted in white to document crack initiation and propagation during the test. Construction procedure of test specimens is shown in Figure 1-5.

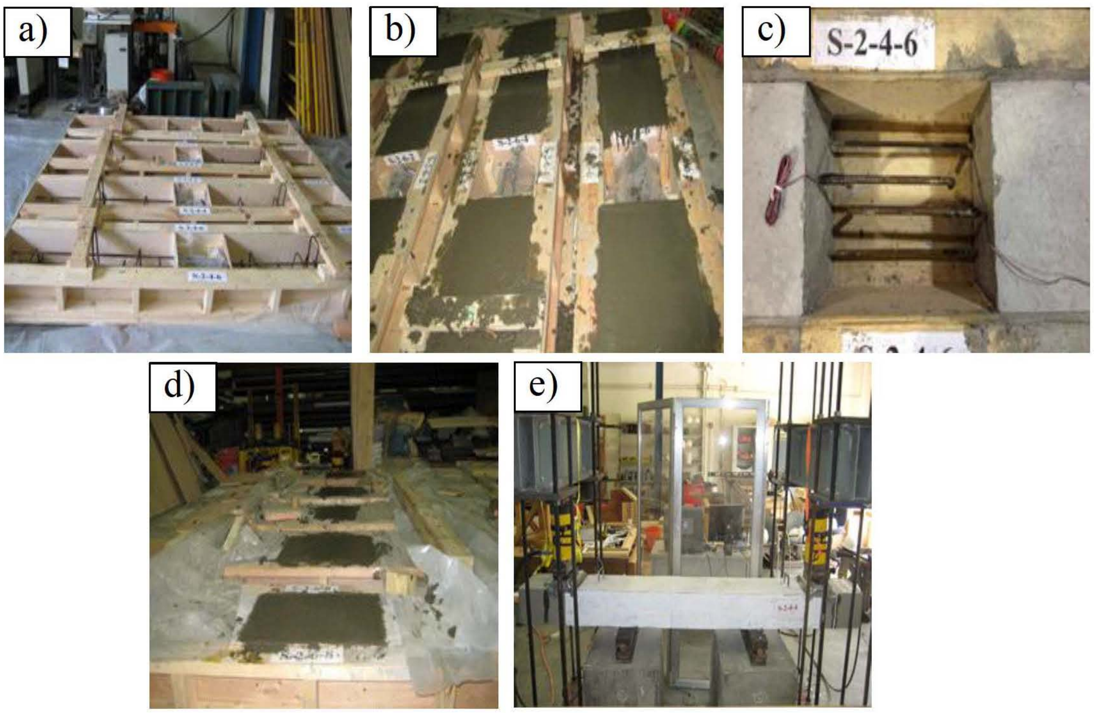

Figure 1-5 Specimen construction procedure a) formwork, b) casting deck portions, c) spliced hooked bars in closure joint, d) casting closure joint, e) final test setup.

1.8 Test Setup and Loading Procedure

The test specimens were tested under a four-point loading setup as the main scope of this study is to investigate the flexural behavior and lap splice length requirement. A displacement controlled monotonic loading was applied to determine the moment capacity of the closure joints. The distance between the hydraulic jacks was 6 ft. and the roller supports were spaced at 3 ft. apart as shown in Figure 1-6 (actual test setup) and Figure 1-7 (schematic test setup). The hydraulic jacks were reacted against a spreader beam, which was anchored to the lab strong floor. To evaluate the behavior of the specimens, the test setup was instrumented with string potentiometers, load cells, and pressure transducers. Each specimen was loaded to failure. The deflection was measured at the mid of the slab and loading points.

Figure 1-6 Actual experimental test setup.

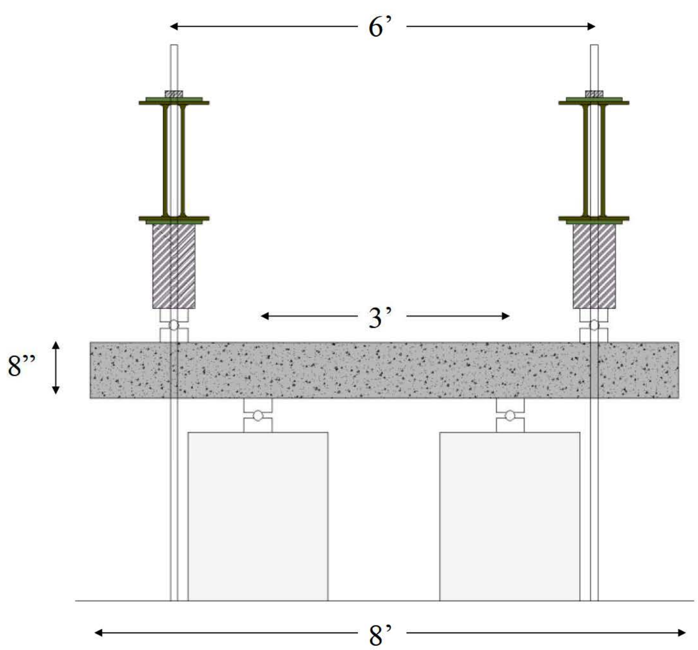

Figure 1-7 Schematic experimental test setup.

1.9 Material Tests

Cylinder test of 4x8 in. based on ASTM C39/C39M [24] specification was used to determine the compressive strength of normal concrete at test days as shown in Figure 1-8. The compressive strength of conventional concrete for the slab and closure joint regions are summarized in Table 1-2. All compressive strength values passed 5,000 psi as required.

ASTM A615 Grade 60, No. 3, No. 4, and No. 6 steel reinforcing bars were used for longitudinal reinforcement in all specimens and transverse bars for specimens in groups C and D. Four segments of each bar were used for tensile testing as shown in Figure 1-9. The resulting values of yield and ultimate strength are mentioned in **Error! Reference source not found.**1-3.

Table 1-2 Compressive Strength of Outer Sections of Slab Specimens

Region	Sample #	Compressive Strength (psi)
Slab	1	7744
	2	6230
	3	7794
	4	7189
	5	6080
	6	6974
	7	6852
	8	6021
	Average	6860
Closure joint	1	6692
	2	6273
	3	6130
	4	6127
	5	6772
	6	6525
	Average	6420

Table 1-3 Bar Test Strengths

Test Specimen	Yield Strength (psi)	Ultimate Strength (psi)
Straight/Hooked No. 4	64,500	103,800
Straight/Hooked No. 6	68,000	113,000
Straight No. 3	61,700	100,100

Figure 1-8 Cylinder test.

Figure 1-9 Reinforcing bar test.

1.10 Experimental Result

1.10.1 Mode of Failure and Crack Pattern

Resistance against pull-out of deformed reinforcing bars embedded in concrete is mainly provided by bearing of ribs against concrete. Although adhesion and friction are present when a deformed bar is loaded, these bond-transfer mechanisms are quickly lost, leaving the bond to be transferred by bearing on the deformations of the bar [12]. For most structural members, bond failure is governed by concrete splitting.

The mode of failure for a closure joint with an inadequate splice length occurs at the closure joint region. by increasing the applied load, the force is transferred to the closure joint and concrete splits before the yielding occurs due to insufficient lap splice length, which is referred to as a bond failure as shown in Figure 1-10. However, for the flexure failure, the spliced bars reach their yielding, and the subsequent cracks are distributed along the tension zone. The flexure failure is a preferred mode of failure for the closure joints and implies that the splice length is sufficient for these specimens (Figure 1-11). The modes of failure for all test specimens are summarized in Table 1-4.

Figure 1-10 Bond failure.

Figure 1-11 Flexure failure.

The specimens with No. 4 bar in Group A failed in flexure with the exception of the specimen with 2-in. lap splice length which is not surprising due to short lap splice length of only four times bars diameter for bar No. 4.

For specimens with No. 6 in Group A, all failure modes were bond failure except for specimen with 10-in. lap splice length. This concluded that even eleven times the bar diameter of No. 6 is not sufficient to yield the spliced No. 6 bars. The specimens in Group B with No. 4 bars failed in flexure except 2-in. and 4-in. splice length. Since the only difference between Group A and Group B that specimens in Group B have bigger lateral distance between reinforcement which led to the need of at least twelve times bar diameter instead of eight times bar diameter for the case of smaller lateral distance between reinforcement for bar No. 4 for specimens in Group A.

For specimens with No. 6 bars in Group B failed in bond which concluded that increasing the lateral distance between the spliced bars caused that lap splice length to be insufficient even with over 13 times the bar size.

The specimens with No. 4 bars in Group C failed in bond failure mode except 8-in. and 10-in. splice length. For specimens with No. 6 bars in the same group, all specimens failed in bond failure mode except for specimens with 8 in and 10 in. splice length. The specimens with No. 4 in

Group D bars failed in bond except 10-in. splice length. For specimens with No. 6 bars in the same group, all specimens failed in bond. The reason Groups C and D failed in bond was due to a weak plane caused by the presence of transverse bars. The modes of failure for all test specimens are summarized in Table 1-4.

Table 1-4 Test Specimens Modes of Failure

Group	ID	Bar Size	G S (in.)	L_p (in.)	Transverse Bar	Mode of Failure
A	S-2-4-2	#4	2	2	NO	Bond Failure
	S-2-4-4	#4	2	4	NO	Flexural Failure
	S-2-4-6	#4	2	6	NO	Flexural Failure
	S-2-4-8	#4	2	8	NO	Flexural Failure
	S-2-4-10	#4	2	10	NO	Flexural Failure
	S-2-4-control	#4	N/A	N/A	NO	Flexural Failure
	S-2-6-2	#6	2	2	NO	Bond Failure
	S-2-6-4	#6	2	4	NO	Bond Failure
	S-2-6-6	#6	2	6	NO	Bond Failure
	S-2-6-8	#6	2	8	NO	Bond Failure
	S-2-6-10	#6	2	10	NO	Flexural Failure
	S-2-6-control	#6	N/A	N/A	NO	Flexural Failure
B	S-4-4-2	#4	4	2	NO	Bond Failure
	S-4-4-4	#4	4	4	NO	Bond Failure
	S-4-4-6	#4	4	6	NO	Flexural Failure
	S-4-4-8	#4	4	8	NO	Flexural Failure
	S-4-4-10	#4	4	10	NO	Flexural Failure
	S-4-4-control	#4	N/A	N/A	NO	Flexural Failure
	S-4-6-2	#6	4	2	NO	Bond Failure
	S-4-6-4	#6	4	4	NO	Bond Failure
	S-4-6-6	#6	4	6	NO	Bond Failure
	S-4-6-8	#6	4	8	NO	Bond Failure
	S-4-6-10	#6	4	10	NO	Bond Failure
	S-4-6-control	#6	N/A	N/A	NO	Flexural Failure
C	ST-2-4-2	#4	2	2	#3	Bond Failure
	ST-2-4-4	#4	2	4	#3	Bond Failure
	ST-2-4-6	#4	2	6	#3	Bond Failure
	ST-2-4-8	#4	2	8	#3	Flexural Failure
	ST-2-4-10	#4	2	10	#3	Flexural Failure
	ST-2-4-control	#4	N/A	N/A	#3	Flexural Failure
	ST-2-6-2	#6	2	2	#3	Bond Failure
	ST-2-6-4	#6	2	4	#3	Bond Failure
	ST-2-6-6	#6	2	6	#3	Bond Failure
	ST-2-6-8	#6	2	8	#3	Flexural Failure
	ST-2-6-10	#6	2	10	#3	Flexural Failure
	ST-2-6-control	#6	N/A	N/A	#3	Flexural Failure

Cont. Table 4-1

D	ST-4-4-2	#4	4	2	#3	Bond Failure
	ST-4-4-4	#4	4	4	#3	Bond Failure
	ST-4-4-6	#4	4	6	#3	Bond Failure
	ST-4-4-8	#4	4	8	#3	Bond Failure
	ST-4-4-10	#4	4	10	#3	Flexural Failure
	ST-4-4-control	#4	N/A	N/A	#3	Flexural Failure
	ST-4-6-2	#6	4	2	#3	Bond Failure
	ST-4-6-4	#6	4	4	#3	Bond Failure
	ST-4-6-6	#6	4	6	#3	Bond Failure
	ST-4-6-8	#6	4	8	#3	Bond Failure
	ST-4-6-10	#6	4	10	#3	Bond Failure
	ST-4-6-control	#6	N/A	N/A	#3	Flexural Failure

Cracks were mapped during the tests on each specimen to observe the order of cracks formation. At the onset of longitudinal crack formation, the tests were stopped for safety concerns. All remaining cracks were mapped upon the completion of testing, as shown in Figure 1-12. Initiation of cracks with bond failure mode occurred at the cold joints and subsequent crack formation occurred over supports.

Figure 1-12 Cracking pattern in closure joints region.

1.10.2 Load-Displacement Relationship

The comparisons of load-deflection curves for all groups are plotted from Figure 1-13 to Figure 1-16 with fixed horizontal and vertical axes. The test results show that the variables such as the bar diameter, the lateral distance between reinforcement, and lap splice length have a significant influence on the load carrying capacity of specimens.

In Group A, the deflection specimen with No. 4 bar increased with an increase of lap spliced length, while the maximum load carrying capacity of specimens remained the same which is due to better development length which led to better load transfer between the two adjacent deck segments. The test specimen S-2-4-2 exhibited the least displacement before the occurrence of bond failure. For lap splice increments from 2 to 8 in., No. 6 bar was insufficient for tension development length and bond failure occurred in these specimens. However, for lap splice length of 10-in., the failure mode was in flexural. Figure 1-13-b shows that the specimens with No. 6 bars with insufficient lap splice length failed before the bar yielding occurred.

The lateral distance between the reinforcement (Gap Space) for specimens in Group B with No. 4 and No. 6 bars was increased from 2 in. to 4 in. The results show that increasing this gap space requires an increase of lap splice length, which is shown in Figure 1-14-a and Figure 1-14-b. The 4-in. gap space for specimens with No. 6 bars was insufficient for all splice lengths and failed in bond. Figures 1-13 and 1-14 show that reducing the gap spacing and increasing the bar diameter can improve the ultimate load capacity of the specimens due to an increase in reinforcement ratio.

Groups C and D have test parameters similar to Groups A and B but include transverse reinforcement in closure joint region. The test results of Groups C and D are presented in Figures 1-15 and 1-16. General design practice requires the use of these transverse bars for thermal and shrinkage requirements.

In Group C, the deflection specimen with No. 4 bar increased with an increase of lap spliced length, while the maximum load carrying capacity of specimens remained the same which is due to better development length which led to better load transfer between the two adjacent deck segments. The test specimens ST-2-4-2 and ST-2-4-4 exhibited the least displacement before the occurrence of bond failure. For lap splice increments from 2 to 6 in., No. 6 bar was insufficient for tension development length and bond failure occurred in these specimens. However, for lap splice lengths of 8-in. and 10-in., the failure mode was in flexural. Figure 1-15-b shows that the specimens with No. 6 bars with insufficient lap splice length failed before the bar yielding occurred.

The lateral distance between the reinforcement (Gap Space) for specimens in Group D with No. 4 and No. 6 bars was increased from 2 in. to 4 in. The results show that increasing this gap space requires an increase of lap splice length, which is shown in Figure 1-16-a and Figure 1-16-b. The 4-in. gap space for specimens with No. 6 bars was insufficient for all splice lengths and failed in bond. Figures 1-15 and 1-16 show that reducing the gap spacing and increasing the bar diameter can improve the ultimate load capacity of the specimens due to an increase in reinforcement ratio.

Adding transverse reinforcement in the closure joints acts as ties for spliced bars (due to their short length) to reduce cracks initiation and propagation, however, specimens in groups C and D failed in bond since the overall width of specimens were small, but in the case of large scale closure joints, this would not happen.

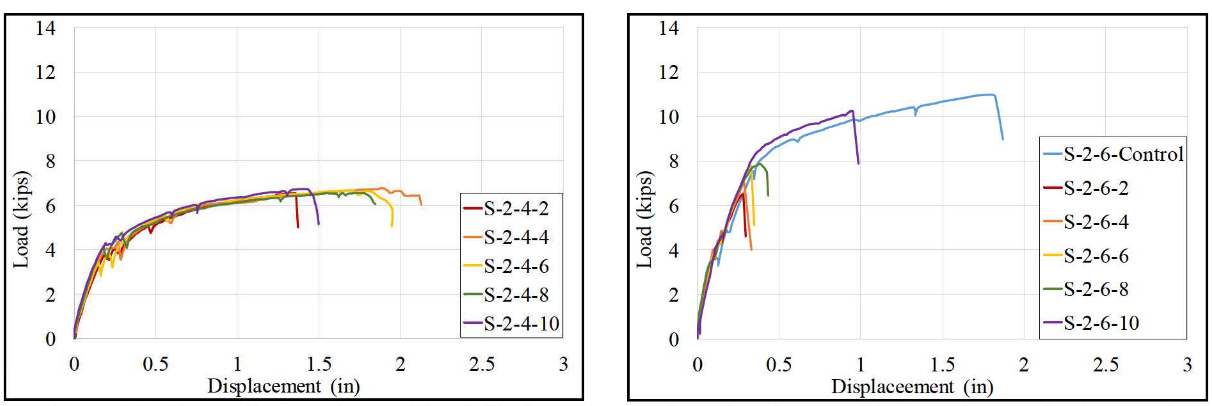

Figure 1-13 Group A, Experimental load-displacement result a) Specimens with No. 4 bars and 2-in. gap, b) Specimens with No. 6 bars and 2-in. gap.

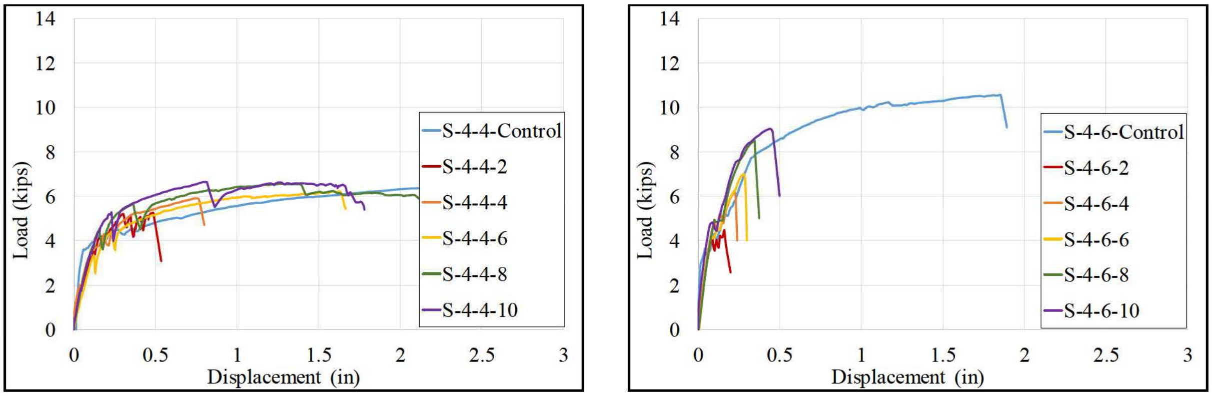

Figure 1-14 Group B, Experimental load-displacement result a) Specimens with No. 4 bars and 4-in. gap, b) Specimens with No. 6 bars and 4-in. gap.

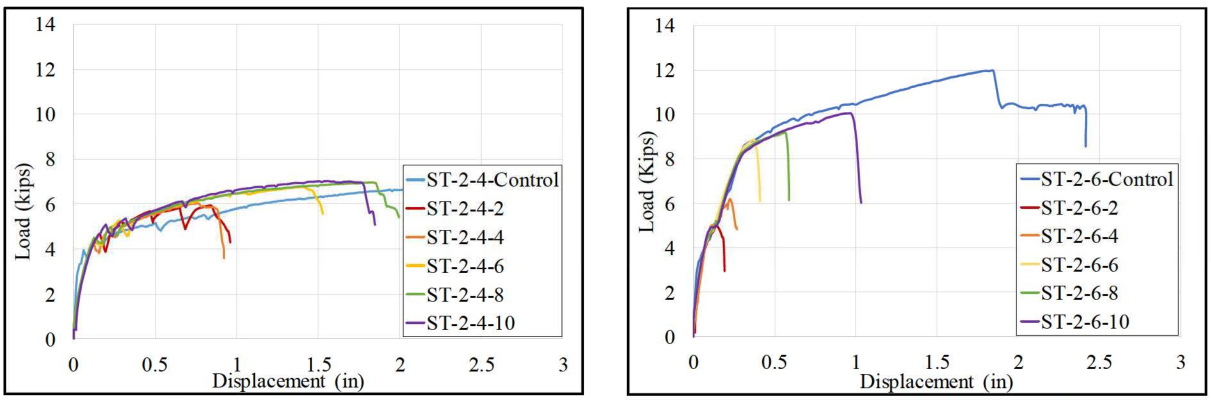

Figure 1-15 Group C, Experimental load-displacement result a) Specimens with No. 4 bars and 2-in. gap, b) Specimens with No. 6 bars and 2-in. gap.

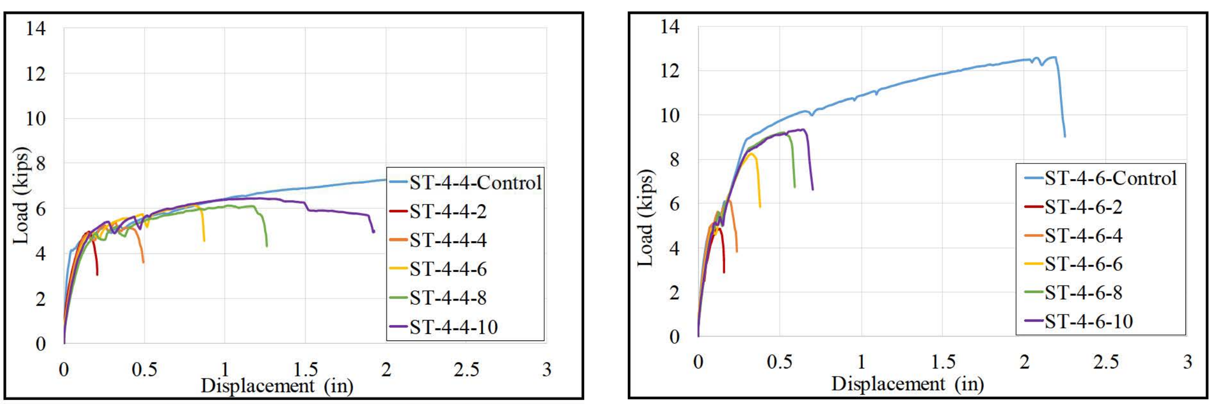

Figure 1-16 Group D, Experimental load-displacement result a) Specimens with No. 4 bars and 4-in. gap, b) Specimens with No. 6 bars and 4-in. gap.

1.10.3 Measured Ductility

Since the behavior of the concrete beams is not perfectly elastic-plastic, the ductility parameters (Δ_y and Δ_{max}) were obtained from the idealization load-displacement curve. The idealized curve consists of two regions: linear elastic and plastic. The yield displacement (Δ_y) was calculated based on initial stiffness and maximum load applied as shown in Figure 1-17.

Displacement ductility capacity (μ) of a member cab be calculated using the following equation:

$$\mu = \frac{\Delta_{max}}{\Delta_y} \tag{1-1}$$

Where;

Δy: Idealized elastic displacement of the tested beam

Δmax: the maximum displacement of the tested beam

Considering the described equation above, the ductility of the specimens related to lap splice length is plotted in Figures 1-18 and 1-19. The presence of transverse reinforcement in closure joint improves displacement ductility and the tension development is possible with shorter spliced length. Based on the test results, it is concluded that a certain level of displacement ductility and load carrying capacity is needed for bar development length. These limits control the design of lap splice length for hooked bar and failure by either flexure or bond is of secondary importance.

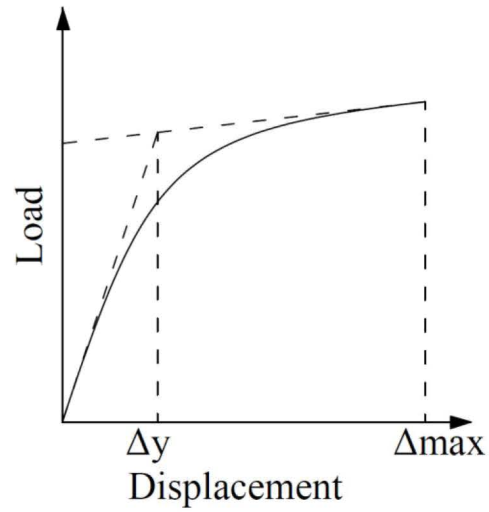

Figure 1-17 Measuring ductility approach.

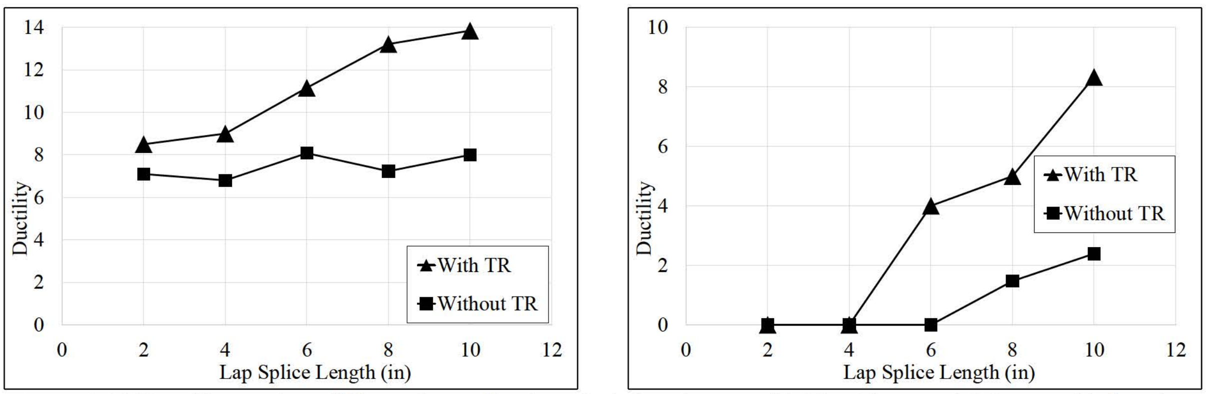

Figure 1-18 Ductility against different lap splice length a) Specimens with No. 4 bars and 2-in. gap, b) Specimens with No. 6 bars and 2-in. gap.

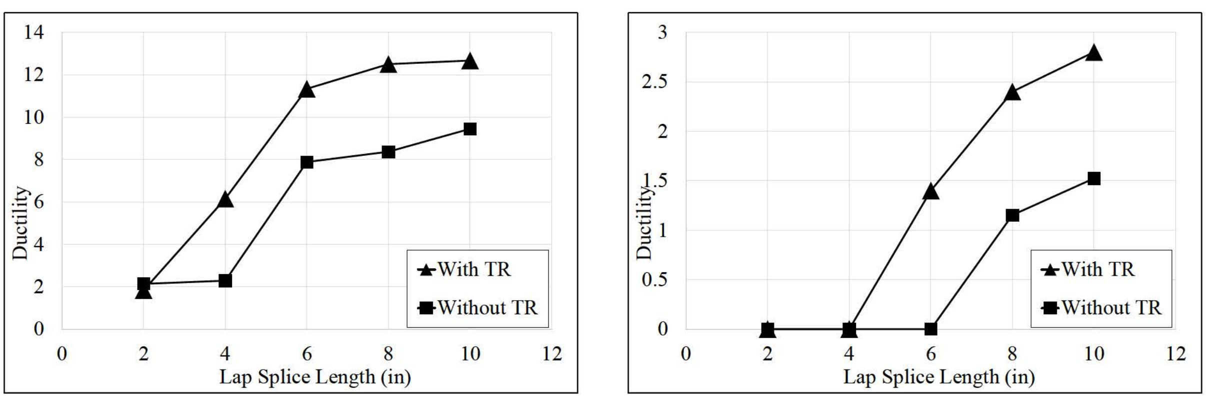

Figure 1-19 Ductility against different lap splice length a) Specimens with No. 4 bars and 4-in. gap, b) Specimens with No. 6 bars and 4-in. gap

1.11 Tentative Design Recommendations

The performance of a tension splice is considered similar to that of an identical component in which the reinforcing bars are continuous. Thus, to comply with general ACI design philosophy due to the lack of the same requirement in AASHTO-LRFD, the members with tension splice should exhibit some level of ductility. The ACI requirement specifying $1.25f_y$ (section 12.14) provides insufficient knowledge of the level of ductility (curvature of displacement) in a member with tension splice. Therefore, in order to develop design criteria, strength and ductility criteria should be incorporated. The displacement ductility ratio, described in the previous section, was used to express the ductility criteria. A displacement ductility ratio greater than one signifies: firstly, those longitudinal bars are capable of developing at least their actual yield stress, and secondly, specimens are capable of reaching deformation levels corresponding to limits beyond the first yield displacement [12].

Each data point in Figures 1-18 and 1-19 represents the displacement ductility ratio achieved by each of 48 test specimens. Based on the structural performance of the specimens compared to the calculated ductility ratio, it might be concluded that the specimens achieving a displacement ductility ratio of greater than 3 shall be considered satisfactory. Graybeal et al [25] conducted several tests for UHPC closure joints and the corresponding ductility was checked by the author herein, and it was estimated that the UHPC closure joint specimens reach a ductility ratio of 3 to 4. Since the UHPC closure joints have been implemented in practice in many bridges, with same ductility range. It can be concluded that performance of closure joints with 90^o degree hooked reinforcement is satisfactory since it reaches the same ductility range as UHPC closure joints with straight bars.

Results of the experimental tests are summarized in Table 1-5. Following is a suggested design recommendation for the closure joint detail recommended in this research. It should be noted that until further tests are performed, this is a conservative recommendation, based on the results shown in Table 1-5.

For specimens with No. 4 and No. 6 reinforcement, using the 90^{o} degree hook detail, as shown in Figure 16, lap splice length should be at least 12 times diameter of the reinforcement. Further, at least three No. 3 reinforcement, acting as confining reinforcement and running parallel to the closure joints, as shown in Figure 1-20 and should be provided over the spliced hooked reinforcement. In case of the absence of the transverse bars, lap splice length should be at least 14 times diameter of the reinforcement. The maximum stagger spacing should be limited to 4 in. until further research is conducted.

Table 1-5 Design Recommendations

Bar Size	Gap Space (in.)	Without Transverse Bar	With Transverse Bar
#4	2	$8d_b$ (4 in.)	$8d_b$ (4 in.)
#6	2	$14d_b$ (10 in.)	$8d_b$ (6 in.)
#4	4	$12d_b$ (6 in.)	$8d_b$ (4 in.)
#6	4	$14d_b$ (10 in.)	$11d_b$ (8 in.)

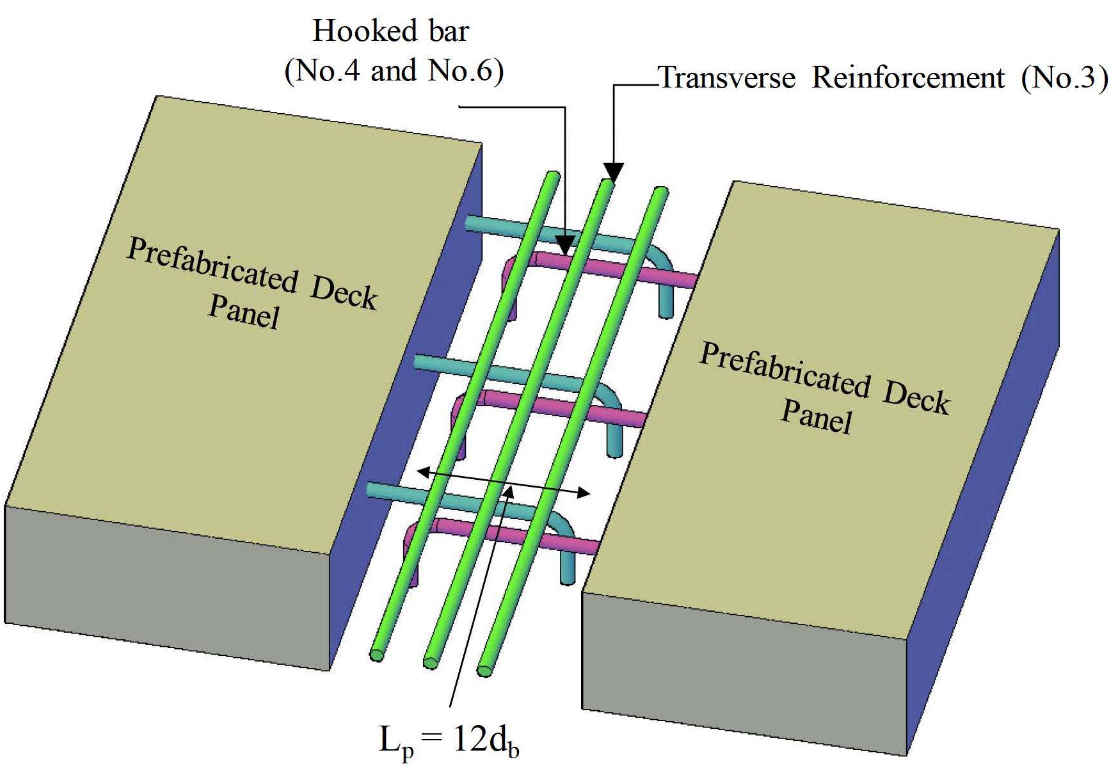

Figure 1-20 Design recommendation details.

1.12 Conclusions

The main objective of this research was to develop design provisions for 90^{o} degree hooked reinforcement detail in closure joint regions. An experimental investigation was designed to develop information that could lead to design recommendations. The parameters investigated experimentally included reinforcement diameter, the lateral distance between reinforcement, transverse reinforcement, and lap splice length. Testing was carried out on a four-point setup and the results are reported in terms of load-deflection curves and ductility ratio.

Based on the conducted research, the following specific conclusions are made:

1. For the reinforcement types and 90^{o} degree hook detail, AASHTO-LRFD (ACI 318-11) Specification requires 18 times the diameter of the reinforcement as lap splice length, as compared to 12 times the diameter of the reinforcement recommended in this study. The main reason for this observed behavior is that AASHTO-LRFD and ACI design recommendations were developed based on test specimens simulating 90^{o} degree hook detail in beam-column connections. In the case of deck slab, significant confinement for hooked detail is provided by concrete to either side of closure joints. This additional confinement helps to reduce the required tension splice length.
2. The design recommendations in this research are based on providing sufficient ductility by including confining reinforcement, increasing in tension lap splice length, or decreasing the lateral distance between spliced reinforcement. The recommended design provisions ensure achieving acceptable levels of ductility before failure, which could be by either flexure or bond failures as the mode of failure is considered of secondary importance.
3. Performance of closure joints with 90^{o} degree hooked reinforcement is considered satisfactory as it reaches a ductility ratio of 3 to 4 range as compared to UHPC closure joints with straight bars.

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