Splicing Strength of Welded Steel Mesh in Concrete Bridge Decks

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Abstract: The splicing strength of welded wire fabric (WWF) in concrete bridge decks was investigated by testing three one-way slabs reinforced with spliced WWF, and one slab reinforced with WWF without a splice. The slabs were tested up to their ultimate capacity. The experiments showed that the specimens with transverse wires in the splice zone had an early separation of the spliced steel, thereby resulting in concrete cover failure. A new splice detail without transverse wires in the splice zone resulted in a gradual (more ductile) failure mode and a strain distribution similar to the slab without a splice. It was also determined that the orientation of transverse wires, distribution of longitudinal wires, and weld shear strength are the major factors affecting the splice strength of concrete bridge slabs reinforced with welded wire fabric.

INTRODUCTION

For large slabs or bridge decks, it is necessary to splice mats of welded wire fabric (WWF) in order to transfer the stresses from the longitudinal wires in one mat to the longitudinal wires in the other mat. The objectives of this study are to determine the required overlap or splicing for WWF reinforcement to fully develop or transfer the stresses from one set of longitudinal wires to the overlapping set of longitudinal wires; and to compare the results with the American Concrete Institute (ACI) code ("Building" 1989) and American Association of State Highway and Transportation Officials (AASHTO) (*Standard* 1989) requirements. Also, a new splice detail without transverse wires in the splice zone was investigated. The effects of different factors on the splice strength of WWF reinforced bridge decks was first studied by performing a literature review. The reviewed references included Atlas et al. (1962) and (1964); Donahey and Darwin (1982); "Bond stress—the State of the Art," by ACI Committee 408 (1966); and Ferguson and Thompson (1962). These studies reported that the strength of a splice of welded wire fabric is affected by the overlap length, the transverse wire size, the spacing of the transverse wire, the number of transverse wires in the lap, concrete strength, and the splicing detail (splice type). Two types of splice are commonly used. The first type has the transverse wires in contact such that the transverse wires of one mat would prevent the movement of the second mat under a tensile force. The second type has the transverse wires separated by concrete.

A new splice detail is proposed in this study. The splice detail is without transverse wires in the splice zone as shown in Fig. 1. The advantage of this detail includes constant moment arm for the internal tensile forces in the steel of the bridge deck along the span of a member, and improvement in

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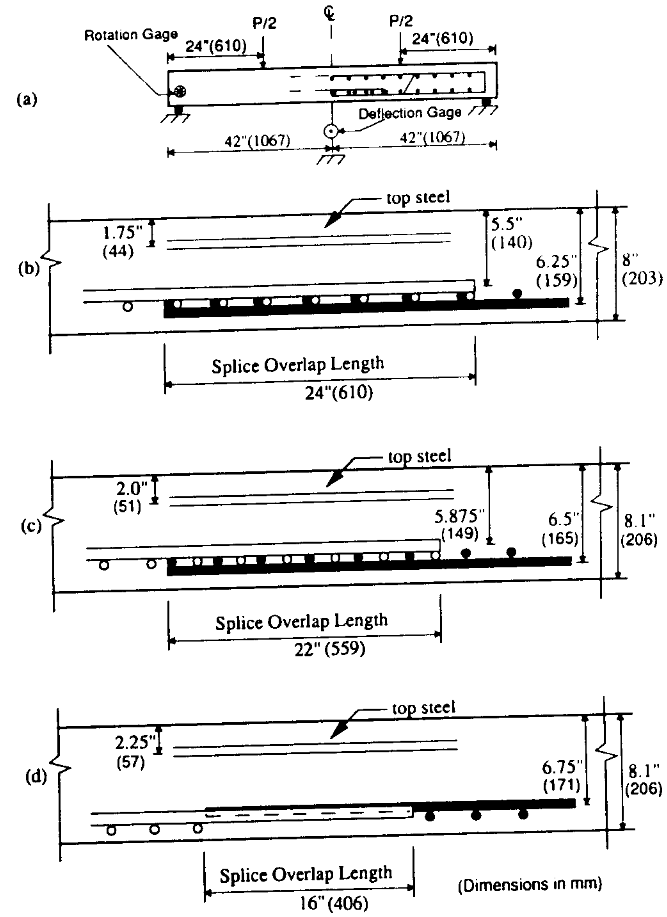


FIG. 1. Splicing of Welded Wire Fabric: (*a*) Test Setup; (*b*) Transverse Wires in Contact; (*c*) Transverse Wires Separated by Concrete; (*d*) No Transverse Wires in Splice Zone

construction efficiency (Bernold and Chang 1992). The constant moment arm was achieved by providing the spliced meshes at one level as opposed to two levels used in the two conventional splice details. The two levels of meshes result in two different moment arms, resulting in different moment capacities on each side of the splice. Also, the presence of transverse wires in the splice zone resulted in an early separation of the spliced steel, resulting in concrete cover failure.

EXPERIMENTAL PROGRAM

Three one-way slabs reinforced with WWF were tested to investigate the performance of a new splice detail, which has no transverse wires at the splice zone, and to verify the validity of the minimum splice length of the three splicing details according to the ACI Code ("Building" 1989) and

AASHTO Specifications (*Standard* 1989). The slabs used had a rectangular cross section with a dimension of 8 × 36 in. (203 × 914 mm) and overall length of 84 in. (2,134 mm). The splice length for the specimens was designed in accordance with AASHTO Specifications (*Standards* 1989), and ACI Code requirements ("Building" 1989) using the specified yield strength of the WWF reinforcement that accounts for a 10% strength reduction due to welded intersections.

The first specimen had transverse wires in the splice zone which were in contact with each other as shown in Fig. 1. The splice length was designed using 90% of the steel stress at ultimate capacity and was determined to be 24 in. (610 mm). The second specimen also had transverse wires in the splice zone, but the wires were separated by concrete. The splice length for this specimen was determined to be 22 in. (559 mm). The third slab was the model for the new splice detail having no transverse wires in the splice zone, and was designed according to Section 12.15 of the ACI Code ("Building" 1989), and Article 8.33.3.1 of AASHTO Specification (*Standard* 1989). The required splice length according to these specifications was 16 in. (406 mm). It is notable that this splice length is significantly shorter than the other two. The wire arrangement in the splice zones is shown in Fig. 1. Also, a specimen with identical properties but without a splice was tested for the purpose of comparison.

The WWF used in the splice study conformed to ASTM Specifications A 185-85 (*Steel welded wire* 1986) and was fabricated in flat mats from cold-drawn wire conforming to ASTM Specifications A 82-85 (*Steel wire, plain* 1986). The three splice-test slabs were loaded using two concentrated loads at equal distances from the supports, as shown in Fig. 1(*a*), which produced a region of constant moment in the central portion of the slab. Strains were measured at extreme compressive fibers of the concrete, the top steel, the bottom steel, and the concrete at the level of the bottom steel. The slabs were loaded to failure in small load and deformation increments, and strains, deflections, rotations, and load measurements were recorded at each of these increments. The resulting cracks were traced and identified with the load at which they were first visible. Following the test of each slab, control concrete cylinders were tested in compression in accordance with ASTM specifications (*Test method for compressive* 1986), and in tension in accordance with ASTM specifications (*Test method for splitting* 1986). The average concrete strengths were 6,265 psi (43.2 MPa), 5,087 psi (35.1 MPa), and 4,067 psi (28 MPa), while the average concrete splitting strengths were 539 psi (3.7 MPa), 646 psi (4.5 MPa), and 394 psi (2.7 MPa) for splice tests 1, 2, and 3, respectively.

ANALYSIS

The design of spliced WWFs depends on the assumed failure mode of the concrete member. A mathematical development of splice length was presented by Lloyd and Kesler (1969) assuming that the weld shear failure and the fracture of the reinforcement do not occur. Thus, the load carried by a splice is the sum of that carried by the longitudinal overlaps through bond, and by the concrete between the outermost transverse wires in the splice through shear. Considering the weld shear strength to be at least 20,000 psi (138 MPa), the splice length is given by

$$l = 0.045D(f_y - 20{,}000N)/\sqrt{f'_c} \quad (1)$$

where l = splice length (in.); D = nominal diameter of the wire (in.); f_y = yield strength of reinforcement (psi); f'_c = compressive strength of concrete (psi); and N = number of pairs of transverse wires in the splice length. Eq. (5) includes the 20% increase in overlap length to account for a reinforcement spacing closer than 12 diameters, as required by the ACI Building Code ("Building" 1989).

An expression for the splice length that will prevent splitting of the concrete was determined by Lloyd and Kesler (1969), based on experimental results, ultimate strength design concepts, and mathematical development that takes into account the effectiveness of a splice. The expression is

$$l_s = \rho d\left[\frac{f_y}{3.5\sqrt{f'_c}} - \frac{8l_o}{D}\right] \quad (2a)$$

or

$$l_s = \frac{A_w}{S_l}\left[\frac{f_y}{3.5\sqrt{f'_c}} - \frac{8l_o}{D}\right] \quad (2b)$$

where l_s = distance between outermost transverse wire in the splice (in.); ρ = reinforcement ratio; d = effective depth of reinforcement (in.); f_y = yield strength of reinforcement (psi); f'_c = compressive strength of concrete (psi); l_o = total length of longitudinal overhang in lap (in.); D = nominal diameter of reinforcement (in.); A_w = cross-sectional area of the longitudinal wire to be spliced (sq in.); and S_l = spacing of longitudinal wires to be spliced (in.).

The ACI Code ("Building" 1989) and AASHTO (*Standards* 1989) provisions for welded deformed steel mesh are based on the experimental tests conducted by Lloyd and Kesler (1969). According to ACI Building Code (1989) section 12.18, and AASHTO specifications (1989) Article 8.33.5, the minimum WWF overlap in a splice zone measured between the ends of each mat shall not be less than $1.7l_d$ nor 8 in., where l_d is the development length. Also, the overlap measured between the outermost transverse wire of each mat shall not be less than 2 in. (51 mm). However, if no transverse wires exist within a splice length, the overlap should be designed as for deformed wires according to ACI Code ("Building" 1989) section 12.15 or according to the AASHTO Specification (*Standards* 1989) article 8.33.3.1.

The development length, l_d, for welded deformed steel mesh, with at least one transverse wire within the development length not less than 2 in. (51 mm) from the point of critical section, can be calculated using the following equation:

$$l_d = l_{db} \times \text{(Modification Factors)} \quad (3)$$

and

$$l_{db} = 0.03d_b\,\frac{(f_y - 20{,}000)}{\sqrt{f'_c}} > 0.20\,\frac{A_w}{S_w}\,\frac{f_y}{\sqrt{f'_c}} \quad (4)$$

where l_d = development length (in.); l_{db} = basic development length (in.); d_b = nominal diameter of spliced wire (in.); f_y = specified yield strength of reinforcement (psi); f'_c = specified compressive strength of concrete (psi); A_w = area of spliced wire (sq in.); S_w = spacing of spliced wires (in.); and modification factors in (3) should be obtained in accordance with ACI

Building code sections 12.2.3 and 12.2.4 ("Building" 1989), or AASHTO Specifications articles 8.25.2 and 8.25.3 (*Standards* 1989).

The ACI and AASHTO requirements for at least one transverse wire within a splice can be presented in the following form:

$$l_{sp} \geq 1.3 l_d \geq 8 \text{ in.} \tag{5a}$$

and

$$l_{om} \geq 2 \text{ in.} \tag{5b}$$

where l_{sp} = minimum welded wire fabric splice length measured between the ends of spliced mats; l_{om} = overlap length measured between the outermost transverse wires of the mats; and l_d = development length from (3).

The splice length for the three test specimens using 3–5 is 18 in. (457 mm) for specimens 1 and 2, and 16 in. (406 mm) for specimen 3. However, the splice length used in the experimental program is 24 in. (610 mm), 22 in. (559 mm) and 16 in. (406 mm) for specimens 1, 2, and 3, respectively (Al-Mutairi et al. 1987).

TEST RESULTS AND COMMENT

Concrete Strains

Concrete strains were measured in the constant moment region, across the depth of the slab. The strain in the extreme compressive fibers of concrete is considered to be an important indicator of the attainment of useful strain.

Four load-strain curves were recorded for each of the splice specimens, and the average concrete strains at the extreme compressive fibers are plotted in Fig. 2(*a*) for specimens 1, 2, and 3, respectively. For comparison purposes the results for a specimen with identical properties but without a splice are shown in the figure. It was observed that the concrete extreme compressive strain for the spliced specimens compared well with the experimental and analytical results of the specimen without a splice up to 70% of the ultimate load of the latter, after which a slight deviation was noticed. For specimen 1, the failure mode was that of concrete crushing and the ultimate strain was less than that of the specimen without a splice [Fig. 2(*a*)]. However, splice tests 2 and 3 failed by slippage of the reinforcement at the splice zone at a load that was less than that of specimen 1, but had approximately the same concrete compressive strain. All the splice specimens failed at a compressive strain of about −0.0017 which was 45% less than that for the specimen without a splice.

The concrete strain gages, which were used to measure the strain inside the concrete, were embedded in the concrete at the level of the bottom steel. The load-strain curves, thus obtained, are shown in Fig. 2(*b*) for specimens 1, 2, and 3. It can be observed from this figure that two trends of behavior exist. First, the results of specimens 1 and 2 [as shown in Fig. 2(*b*)] indicate that the concrete strain at the level of the tensile reinforcement was in compression up to almost the ultimate load where the strain became in tension and started to increase while the load was nearly constant. Second, the results of specimen 3, which does not have transverse wires at the splice zone, indicate that the concrete strain at the level of the tensile reinforcement was in tension. The behavior observed in specimen 3 is similar to specimens reinforced with conventional reinforcement. However, the existence of transverse wires in the splice zone alters this behavior, and com-

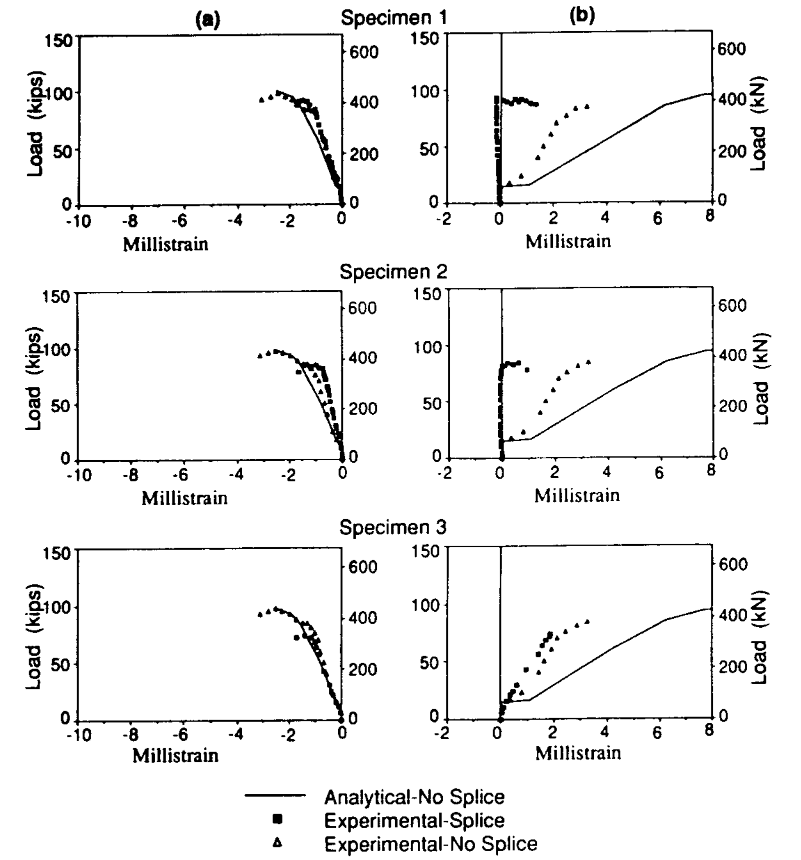


FIG. 2. Load-Strain Curve for Splice Test Specimens: (*a*) Top of Concrete; (*b*) Concrete at Level of Bottom Steel

TABLE 1. Strains at Failure Load for Splice Test Slabs

Splice test number (1)	Failure load kips (kN) (2)	FAILURE STRAINS: Top concrete (3)	FAILURE STRAINS: Top steel (4)	FAILURE STRAINS: Bottom Steel: Bottom mesh (5)	FAILURE STRAINS: Bottom Steel: Top mesh (6)
1	87 (387)	−0.0017	0.0022	0.0020	0.0021
2	78 (347)	−0.0016	0.0037	0.0019	0.0014
3	72 (320)	−0.0016	0.0008	0.0057	0.0020

pressive strains appear as the net effect of two factors. The first factor is the compression forces due to the transfer of load from the WWF to the concrete, which causes the two spliced mats (top and bottom WWF) to be under tension, which puts the concrete between the transverse wires in compression. The second factor is in the tensile strains due to the flexural bending, which causes the tensile strain to increase as the curvature increases and the fibers at the level of the strain gages to elongate. The simultaneous effect of these two factors may put the concrete either in compression or in tension depending on which effect is larger. The test results of splice tests 1 and 2 indicated that the concrete strain was in compression up to the load that initiated splice failure. The slippage of the reinforcement at the spliced zone increased the members curvature resulting in increased tensile strains and a reduction in the compression strain. At a load close to ultimate, the concrete strain shifted from compression to tension and tensile strains continued to increase until failure. The ultimate strain measured in the concrete at the level of the bottom steel was about 0.0014, 0.0010, and 0.0018 for specimens 1, 2, and 3, respectively. The ultimate strains for the splice test specimens are summarized in Table 1.

Steel Strains

The WWF reinforcement strains were measured for the top and bottom steel of the three splice-test specimens. The average of the strain measurements at each level was obtained and the resulting load strain curves were

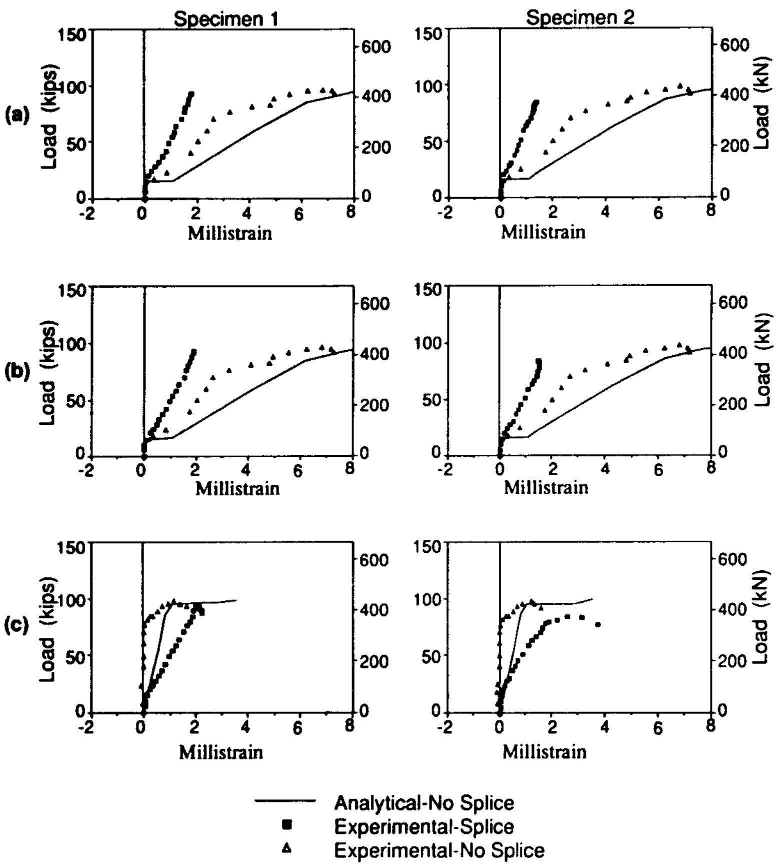


FIG. 3. Load-Strain Curve for Splice Tests 1 and 2: (*a*) Top Mesh of Bottom Steel; (*b*) Bottom of Mesh of Bottom Steel; (c) Top Steel

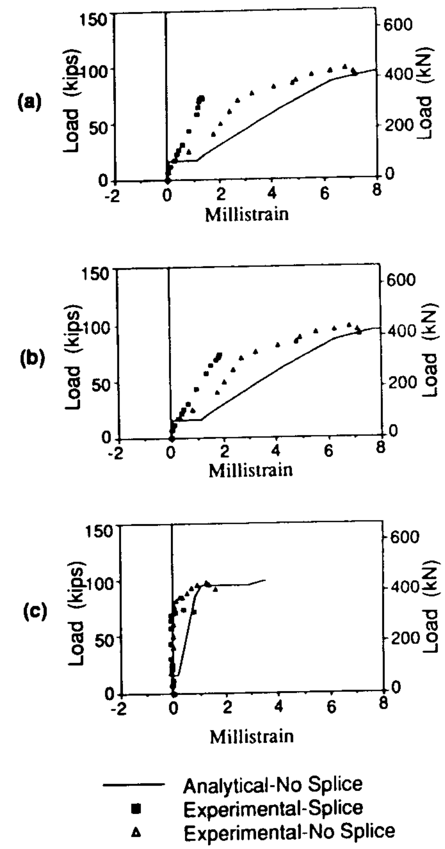


FIG. 4. Load-Strain Curve for Splice Test 3: (*a*) Top Mesh of Bottom Steel; (*b*) Bottom Mesh of Bottom Steel; (c) Top Steel

plotted. For comparison purposes, the plots contain the experimental and analytical results of an identical specimen but without a splice.

The load-strain curves for the spliced tensile reinforcement are shown in Figs. 3(*a* and *b*) and 4(*a* and *b*) for the two steel meshes at the bottom of the three splice-test specimens. The figures show that the splice test specimens with transverse wires in the splice zone had less strain at the same load (i.e., stiffer) than the splice test specimen without transverse wires at the splice zone or the specimen without a splice. However, specimen 3 (without transverse wires in the splice zone) had a larger strain than the other two splice-test specimens. This is a clear indication of the effect of transverse wires on ductility which is discussed in the following sections. The average ultimate strain was approximately 0.0020, 0.0014, and 0.0020 for the top mesh of the bottom steel; and 0.0021, 0.0019, and 0.0057 for the bottom mesh of the bottom steel for specimens 1, 2, and 3, respectively. These average strains, with the exception of the 0.0057 reading, were less than the ACI and the ASTM specified yield strains, which are 0.0035 and 0.0050, respectively. These low strains are due to slippage of meshes at the splice zone. Therefore, the reinforcement did not reach the yield condition.

This performance can be improved, in the case of no transverse wires, by increasing the development length or splice length as commonly done for conventional reinforcement. However, for cases with transverse wires, the observed early cracking and cover separation below the bottom mesh of the bottom steel resulted in reducing the development length, increasing the possibility of reinforcement slippage, and limiting the amount of performance improvement by increasing the splice length. This behavior is attributed to the rigidity of the meshes within the splice zone.

The load-strain curves for the top steel are plotted in Figs. 3(*c*) and 4(*c*) for the three splice-test specimens. These figures show that the top steel for the splice tests with transverse wires in the splice zone started to carry tensile strains at an early stage of loading until failure by crushing of the concrete extreme compressive fibers (specimen 1) or slippage of the spliced reinforcement (specimen 2). However, the top steel of specimen 3 had compressive strains in the early loading stages and shifted to tension at high loads. This behavior is common for specimens without splices as indicated by the experimental results reported by Al-Mutairi et al. (1987).

Strain Distribution

The strain distributions for a section at different loading stages is essential in investigating its behavior. The strain distributions for specimens 1, 2, and 3 are shown in Fig. 5.

The strain distributions for the splice-test specimens can be categorized in two groups: (1) Strain distributions for splice specimens with transverse wires in the splice zone; and (2) strain distributions for splice specimens without transverse wires in the splice zone. The first category is distinguished by large tensile forces carried by the top reinforcement, and a small concrete compressive zone or neutral axis depth from the extreme compressive fibers. Fig. 5(*a*) for specimen 1 shows a tensile strain carried by the compressive reinforcement that is slightly greater than the tensile strain carried by the tensile reinforcement. The results of specimen 2 [Fig. 5(*b*)] show a much bigger difference in tensile strain with the top reinforcement carrying a larger stress than the tensile reinforcement. These strain distributions indicate that as the splice length decreases the effectiveness of the splice decreases, thereby forcing the top steel to provide tensile strength. The behavior of the splice zone of the second category is more common and is distinguished by the linearly varying strain along the depth of the section at early load stages. The level of nonlinearity in this case increases as the load is increased. However, as shown in Fig. 5(*c*) for specimen 3, the strain distribution is almost linear for all loading stages until ultimate failure by slippage of the spliced reinforcement. This latter strain distribution is similar to those corresponding to test specimens without splices as presented by Al-Mutairi et al. (1987). The difference in the strain distribution and behavior for the two categories are partly attributed to the splice details. The splice detail for the first category includes transverse wires in the splice zone (specimens 1 and 2), while the second category does not include transverse wires in the splice zone. Although the existence of transverse wires improves the anchorage and bonding strength (Al-Mutairi et al. 1987), early separation of the two bottom steel meshes, resulting in concrete cover separation, is expected due to the increased mesh stiffness. This further results in a shift in tensile strains from the bottom reinforcement to the top reinforcement that depends in magnitude on the splice length. Finally, note that the second category of splice detail had a smaller splice length than the first category.

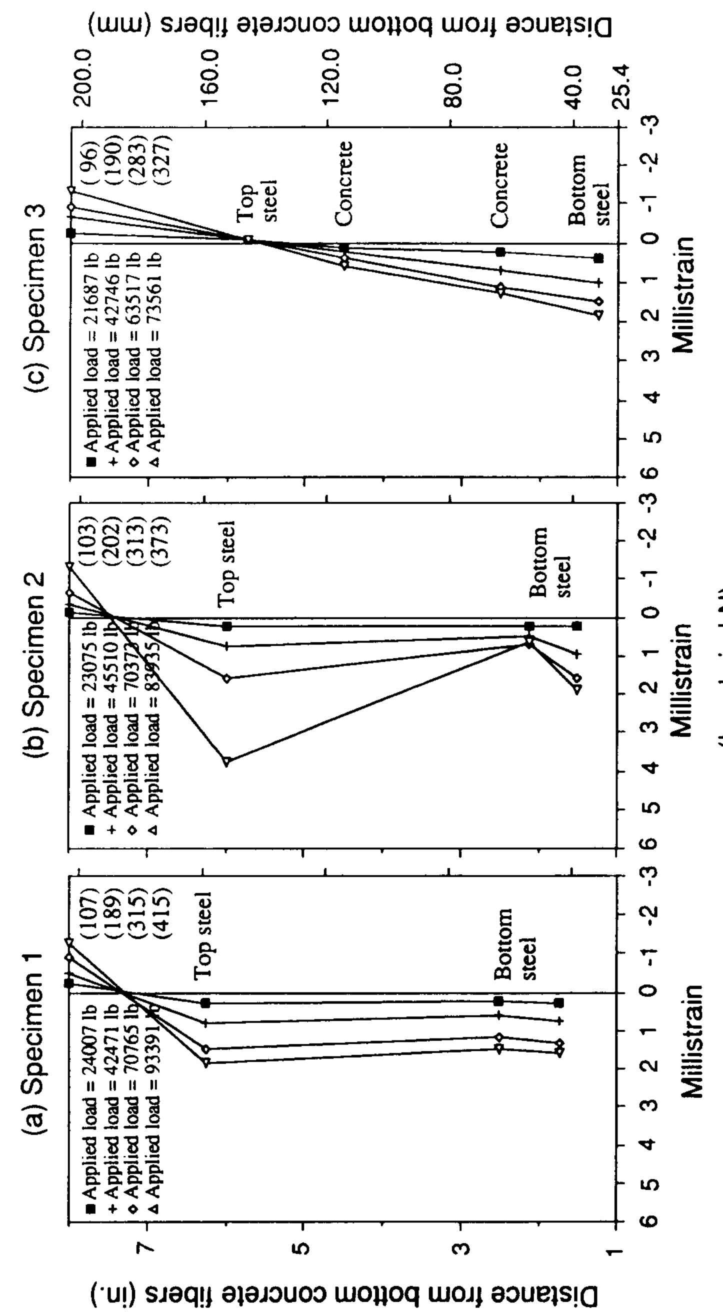


FIG. 5. Strain Distributions: (a) Splice Test 1; (b) Splice Test 2; (c) Splice Test 3

Therefore, the ultimate strength of specimen 3 was smaller than the corresponding loads for specimens 1 and 2. Also, the splice zone is usually placed in the minimum bending moment location, which results in small splice length.

Deflection and Rotation

The load-deflection curves for the three specimens are shown in Fig. 6(*a*) along with their theoretical curves, which were obtained by using the incremental deformation method (Al-Mutairi et al. 1987). The figure shows that the experimental results for specimen 1 indicate a larger slab stiffness than the theoretical values. However, it is close to the experimental curve of a slab with identical reinforcement but without a splice. On the other hand, the theoretical load-deflection curve for specimens 2 and 3 are below the experimental curve but above the theoretical curve of the specimen without a splice, thereby indicating a lower stiffness that specimen 1. Note that specimen 1 failed by crushing the extreme compressive fiber of the

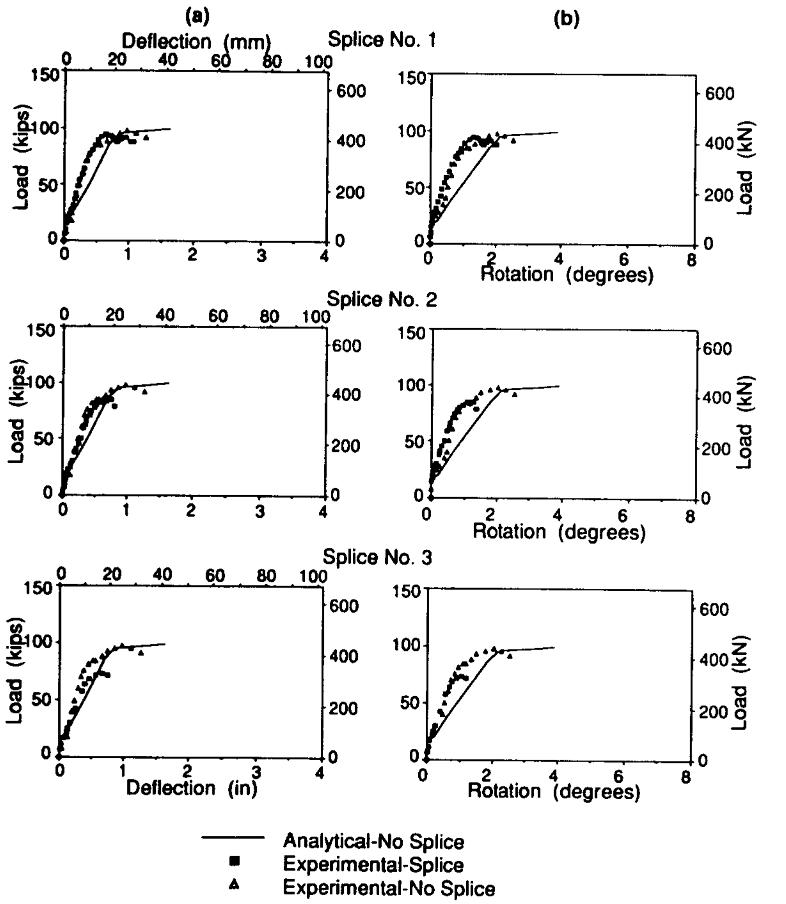


FIG. 6. Load-Deflection and Load-Rotation Curves: (*a*) Load Deflection; (*b*) Load Rotation

concrete, while specimens 2 and 3 failed by slippage of the spliced reinforcement. The difference in the splice length between specimens 1 and 2 of only 2.0 in. (51 mm) indicates that specimen 1 has just the right length to fully develop its ultimate strength. However, specimen 3 failed by steel slippage due to the relatively short splice length. In this case, the splice length was designed as deformed wires, not welded deformed steel mesh and had a splice detail similar to conventional steel.

The load-rotation curves are plotted in Fig. 6(*b*) for specimens 1, 2, and 3. The shape of these curves is similar to the shape of the experimental load-deflection curves shown in Fig. 6(*a*). Thus, these curves illustrate behaviors similar to those concluded from the load-deflection curves. Fig. 7 shows a comparison of load-deflection characteristics of the three specimens with splices and the specimen without a splice. It can be observed that the splices result in a slight reduction in ductility. Among the spliced specimens, specimen 3 (without transverse wires in the splice) has the largest ductility. Therefore, the proposed splice detail improves ductility. Also note that specimens 1 and 2 showed a unique cracking behavior that included a major crack initiated at the end of the bottom mesh of the bottom steel as shown in Fig. 8. This cracking behavior can be attributed to the relatively stiff steel meshes in the splice zone. Similar cracking characteristic was not observed in specimen 3.

Recommendations for Splice Length

The experimental program results indicated that specimen 1, with an overlap splice length of 24 in. (610 mm), is more effective than specimen 2 with a 22 in. (59 mm) overlap length, or specimen 3 with a 16 in. (406 mm) overlap length and no transverse wires in the spliced zone. The 22 in. (559 mm) overlap length for specimen 2 is approximately 26% larger than the value recommended by ACI Code ("Building" 1989) and AASHTO Specifications (*Standards* 1989). However, this splice did not perform satisfactorily. On the other hand, the splice length in specimen 1 had an ultimate

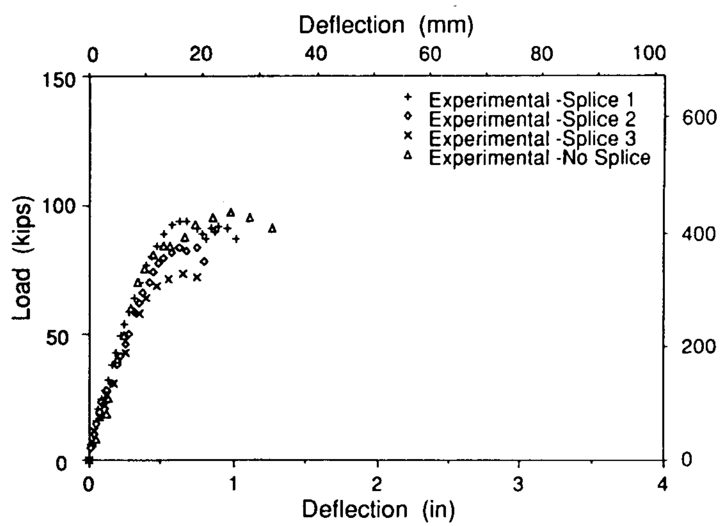
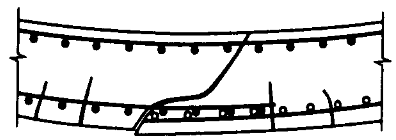


FIG. 7. Comparison for Splice Tests

(a) Example Failure Mode for Specimen 1



(b) Effect of Wire Stiffness

FIG. 8. Concrete Cracking in Tension: (*a*) Example Failure Mode for Specimen 1; (*b*) Effect of Wire Stiffness

resistance that was close to the required capacity. Therefore, the splice length of specimen 1, which is approximately 35% larger than the recommended value by ACI code and AASHTO specifications, is closer to the minimum overlap length required to fully develop the ultimate capacity. As a result, it is recommended that the ultimate tensile stress of the tensile reinforcement be used instead of the yield stress in (3)–(5). The basic development length is, therefore, revised to

$$l_{db} = 0.03 d_b \frac{(f_{su} - 20{,}000)}{\sqrt{f'_c}} \tag{6}$$

where f_{su} = specified ultimate strength (psi). Alternatively, the constant 0.03 in (3)–(5) can be increased instead of changing f_y to f_{su} in these equations. Also, the splice zone should be located in the minimum moment region if possible. Having the splice zone at this location reduces the required ultimate resistance needed from the splice which in turn increases its effectiveness. To reduce the chances of concrete-cover cracking at the end of the bottom layer mat of the bottom tensile steel, it is recommended to offset the ends of the lapped splices. This procedure will increase the length of the crack path, which increases the concrete resistance. In addition, ACI Committee 408 (1966) recommends using vertical tie stirrups to confine the spliced zone and to increase the ductility of the slabs.

SUMMARY AND CONCLUSIONS

The splicing strength of WWF in concrete bridge decks was investigated by testing three one-way slabs with spliced WWF, and one slab without a splice up to their ultimate capacity. Based on the experimental and analytical study of slabs reinforced with WWF, the following can be concluded:

The ultimate tensile strength should be used to improve the estimate of the basic development length for WWF instead of the yield tensile stress.

Slabs without transverse wires in the splice zone exhibit a gradual failure mode (more ductile) and behave similar to slabs without a splice. On the other hand, slabs with transverse wires in the splice zone had a strain distribution, which was different than that of the slabs without a splice. Also, their compression reinforcement carried a large portion of tension force, depending on the splice length.

Offsetting ends of lapped splice helps to prevent concrete cracking at the edges of the spliced bottom tension reinforcement.

Only three spliced slabs were tested in this study. Therefore, the preceding conclusions should be viewed as tentative. Additional testing is needed in order to develop design equations or guidelines.

ACKNOWLEDGMENT

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APPENDIX II. NOTATION

The following symbols are used in this paper:

A_w = area of individual wire to be developed or spliced;
D = nominal diameter of wire;
d = effective depth of reinforcement;
d_b = nominal diameter of wire;
f'_c = compressive strength of concrete;
f_{su} = specified ultimate strength of reinforcement;
f_y = yield strength of reinforcement;
l = splice length;
l_d = basic development length;
l_{db} = basic development length;
l_o = total length of longitudinal overhang in lap;
l_{om} = overlap length measured between outermost transverse wire of each mat;
l_s = distance between outermost transverse wire in lap;
l_{sp} = minimum welded wire fabric splice length measured between ends of each mat;
N = number of pairs of transverse wires in lap;
P = applied load;
S_l = spacing of wires to be spliced;
S_w = spacing of wire to be developed or spliced; and
ρ = reinforcement ratio.