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Behavior of Lap-Spliced Reinforcing Bars Embedded in High-Strength Concrete

by Atorod Azizinamini, Rob Pavel, Erleen Hatfield, and S. K. Ghosh

Safety concerns and a lack of test data on the bond strength of deformed reinforcing bars embedded in high-strength concrete are reasons why the ACI 318 Building Code has imposed an arbitrary limit of 10,000 psi (69 MPa) in calculating the tension development length and the tension lap-splice length. This limitation was first introduced in the 1989 edition of ACI 318.

In an attempt to evaluate the impact of this limitation and develop provisions for its removal, a two-phase investigation was carried out at the University of Nebraska-Lincoln. This paper provides a complete summary of the experimental investigations conducted in both phases of the project. Phase I of the investigation was supported by the Portland Cement Association, while Phase II was supported by the National Science Foundation. During the two phases, 70 beam specimens with tension reinforcement splices were tested. The parameters studied included: bar size, length and deformation type of the reinforcing bars, amount of transverse reinforcement over the splice length, casting position, and concrete compressive strength. Results of the investigation are used to discuss observed differences in how reinforcing bars in tension develop in normal and high-strength concretes (HSCs), to develop hypotheses to explain these differences, and to suggest conditions for removal of the current concrete compressive strength limitation of the ACI 318 Building Code, which applies to the calculation of tension development length and tension splice length. Development of code language provisions for removal of the current limitation, is discussed elsewhere. In this paper, HSC is defined as concrete with compressive strength exceeding 10,000 psi (69 MPa).

Keywords: bond (concrete to reinforcement); building codes; deformed reinforcement; high-strength concretes; reinforcing steels; splicing; structural engineering.

INTRODUCTION

Due to a lack of test data, ACI 318-95 Building Code[1] requirements include an arbitrary limitation of 10,000 psi (69 MPa) on the specified compressive strength of concrete f_c' that may be used in calculating tension development and tension splice lengths. This limitation is contained in Section 12.1.2 of ACI 318-95. To investigate the possible removal of this limitation, a two-phase study was conducted to evaluate the bond performance of reinforcing bars embedded in high-strength concrete (HSC). Phase I of the investigation was initiated in 1990 and completed in 1992. Phase II then followed, with completion in 1997. The results of Phase I of the investigation are reported in Reference 2; however, for the sake of completeness, a brief summary of the test results given in that reference is also included in the tables that are part of this paper. The paper also includes a very brief summary of the main conclusions drawn from Phase I of the investigation. In the two phases of the study, 70 beam specimens with tension reinforcement splices were tested. Nineteen specimens were tested in Phase I and 51 in Phase II. The results of Phase I provided the basis for the development of a behavioral model, in the form of a failure hypothesis, to explain observed differences in the bond characteristics of reinforcing bars embedded in normal strength concrete (NSC) and HSCs. The major conclusion from Phase I of the investigation was that increasing the splice length is not an efficient approach to improving the bond performance of reinforcing bars embedded in HSC and that placing some minimum amount of transverse reinforcement over the splice region is an efficient and safe solution. The purpose of Phase II of the investigation was to develop experimental data to establish this minimum amount of transverse reinforcement over the splice length. This paper provides a summary of all experimental data. Parameters that were varied in the experimental tests included: splice length, bar diameter, deformation type, casting position, concrete compressive strength, and the amount of transverse reinforcement over the splice region.

Development of code provisions for the minimum amount of transverse reinforcement, using the test data discussed in this paper, is presented elsewhere.

RESEARCH SIGNIFICANCE

This paper summarizes the results of an investigation aimed at understanding the behavior of reinforcing bars embedded in HSC. The test results are used to identify a detailing requirement that could result in eliminating the 10,000 psi (69 MPa) limitation placed on the specified compressive strength of concrete that may be used in calculating the tension development or tension lap-splice length (Section 12.1.2 of ACI 318-95).

EXPERIMENTAL WORK

Specimen details

A total of 70 beam specimens with tension reinforcement lap-splices were tested. Table 1 through 4 give details of the specimens. Tests 1, 7, 29, 30, 47, 48, and 56 were conducted at the Construction Technology Laboratories, Skokie, Ill., while the remaining tests were carried out at the University of Nebraska-Lincoln. Fig. 1 shows the general configuration of each test specimen. Each specimen consisted of two or three reinforcing bars spliced at midspan. The longitudinal reinforcement was ASTM A 615 Grade 60 (specified minimum yield strength of 414 MPa), No. 11 (36-mm diameter) or No. 8 (25.4-mm diameter) steel reinforcing. The transverse reinforcement used in some of the specimens over the splice regions consisted of ASTM A 615 Grade 60 (414 MPa yield strength), No. 3 (10-mm diameter) steel bars. The thickness of the side and top concrete cover was equal to one or two times the longitudinal

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bar diameter. The clear spacing between reinforcing bars at midspan of each specimen was approximately two times the side cover. Table 1 through 4 give details of all specimens with: No. 8 (25-mm diameter) bars and side concrete cover thickness of one bar diameter; No. 8 bars and side concrete cover thickness of two bar diameters; No. 11 (36-mm diameter) bars and side concrete cover thickness of one bar diameter; and No. 11 bars and side concrete cover thickness of two bar diameters, respectively. Also shown in each table are the splice lengths, spacing of the stirrups over the splice region, if any, concrete compressive strength at the time of testing, obtained by testing

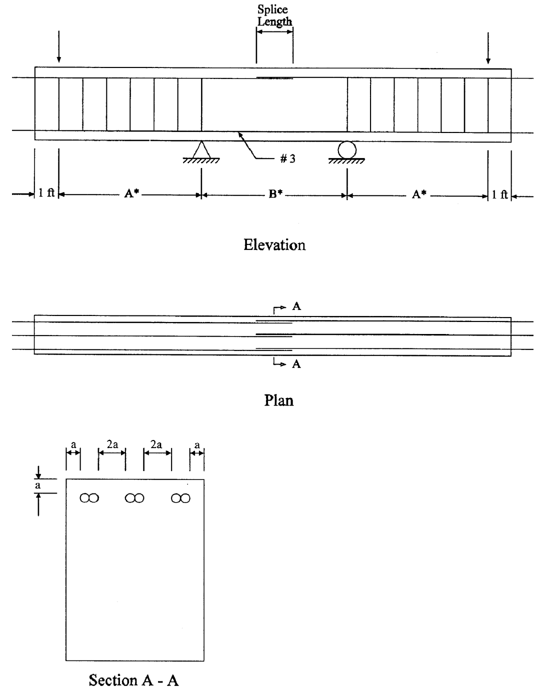


Fig. 1—Test specimen.

Table 1—Details of test specimen No. 8-1db

Test no.	Longitudinal reinforcement	Splice, in.	Stirrups	f_c', psi	Deformation type	Relative rib area	Cross section detail	Dimension A, ft	Dimension B, ft	Yield strength, psi	Tensile strength, psi
1	2	41.00	None	15,120	—	—	9 x 14 in.	6	6	77,850	119,100
2	3	36.00	None	14,450	Bamboo	0.07391	12 x 16 in.	6	6	72,150	107,900
3	2	32.00	None	15,591	Bamboo	0.07391	9 x 16 in.	6	6	72,150	107,900
4	2	32.00	None	15,591	Bamboo	0.07391	9 x 16 in.	6	6	72,150	107,900
5	3	30.00	None	15,034	Bamboo	0.07391	12 x 16 in.	6	6	72,150	107,900
6	2	25.00	None	15,324	Bamboo	0.07391	9 x16 in.	6	6	72,150	107,900
7	2	23.00	None	5,290	—	—	9 x 14 in.	5	5	77,850	119,100
8	2	20.00	None	15,324	Bamboo	0.07391	9 x 16 in.	6	6	72,150	107,900
9	2	25.00	No. 3 @ 4.5 in.	16,003	Bamboo	0.07391	9 x 16 in.	6	6	72,400	110,800
10	2	25.00	No. 3 @ 6.0 in.	16,003	Bamboo	0.07391	9 x 16 in.	6	6	72,400	110,800
11	2	25.00	No. 3 @ 8.0 in.	15,714	Bamboo	0.07391	9 x 16 in.	6	6	72,400	110,800
12	2	32.00	No. 3 @ 7.0 in.	15,884	Bamboo	0.07391	9 x 16 in.	6	6	72,150	107,900
13	2	32.00	No. 3 @ 9.0 in.	15,884	Bamboo	0.07391	9 x 16 in.	6	6	72,150	107,900
14	2	32.00	No. 3 @ 12.0 in.	15,697	Bamboo	0.07391	9 x 16 in.	6	6	72,400	110,800
15	2	32.00	No. 3 @ 15.0 in.	14,578	Bamboo	0.07391	9 x 16 in.	6	6	71,200	108,200

Table 2—Details of test specimen No. 8-2db

Test no.	Longitudinal reinforcement	Splice, in.	Stirrups	f_c', psi	Deformation type	Relative rib area	Cross section detail	Dimension A, ft	Dimension B, ft	Yield strength, psi	Tensile strength, psi
16	2	36.00	None	14,450	Bamboo	0.07391	12 x 16 in.	6	6	72,150	107,900
17	2	20.00	None	15,034	Bamboo	0.07391	12 x 16 in.	6	6	72,150	107,900
18	2	19.00	None	15,591	Bamboo	0.07391	12 x 16 in.	6	6	72,150	107,900
19	2	19.00	None	15,591	Bamboo	0.07391	12 x 16 in.	6	6	72,150	107,900
20	2	15.00	None	15,324	Bamboo	0.07391	12 x 16 in.	6	6	72,150	107,900
21	2	10.00	None	15,324	Bamboo	0.07391	12 x 16 in.	6	6	72,150	107,900
22	2	15.00	No. 3 @ 3.0 in.	15,714	Bamboo	0.07391	12 x 16 in.	6	6	72,400	110,800
23	2	15.00	No. 3 @ 4.5 in.	15,714	Bamboo	0.07391	12 x 16 in.	6	6	72,400	110,800
24	2	15.00	No. 3 @ 6.5 in.	15,714	Bamboo	0.07391	12 x 16 in.	6	6	72,400	110,800
25	2	19.00	No. 3 @ 4.5 in.	15,884	Bamboo	0.07391	12 x 16 in.	6	6	72,150	107,900
26	2	19.00	No. 3 @ 6.0 in.	15,884	Bamboo	0.07391	12 x 16 in.	6	6	72,150	107,900
27	2	19.00	No. 3 @ 8.0 in.	15,697	Bamboo	0.07391	12 x 16 in.	6	6	72,400	110,800
28	2	19.00	No. 3 @ 12.0 in.	14,578	Bamboo	0.07391	12 x 16 in.	6	6	71,200	108,200

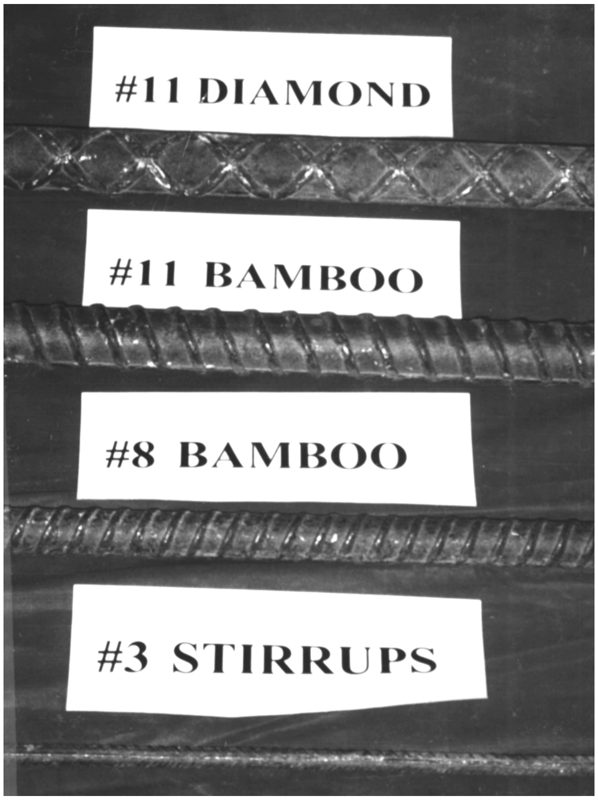


Fig. 2—Different bar sizes used in experimental program.

4 x 8-in. (102 x 204-mm) cylinders cast at the same time as the specimens and cured alongside the specimens, and the yield and tensile strength of the longitudinal reinforcing bars obtained from coupon testing. The cross-sectional dimensions of each beam specimen are also given in Table 1 through 4; the first dimension given is the beam width, while the second dimension is the total beam depth. Fig. 1 designates the distance between the two roller supports as Dimension B, while the distance from each roller to the point of concentrated load application at the near end is identified as Dimension A. Dimensions A and B for each test specimen are given in Table 1 through 4.

Table 1 through 4 also give the deformation types and relative rib areas for longitudinal reinforcing bars used in each test specimen. The relative rib area was calculated based on the procedure specified in Reference 3. Fig. 2 shows a photo of sample reinforcing bars with bamboo and diamond-shaped deformations.

All test specimens shown in Table 1 through 4 contained other than top bars, except Specimens 35, 38, 41, 44, and 47, which contained top bars. Top bars are defined as those having 12 in. (305 mm) or more of concrete cast in one lift underneath the bar.

The HSC mix used in the construction of the test specimens included Type I cement, Class C fly ash, silica fume, and superplasticizer. The maximum aggregate size was 1/2 in. (13 mm). The water-to-cementitious materials ratio *(w/cm)* ranged from 0.21 to 0.27.

Table 3—Details of test specimen No. 11-1db

Test no.	Longitudinal reinforcement	Splice, in.	Stirrups	f_c', psi	Deformation type	Relative rib area	Cross section detail	Dimension A, ft	Dimension B, ft	Yield strength, psi	Tensile strength, psi
29	2	80.00	None	15,120	—	—	12 x 16 in.	7.5	9.0	73,720	117,900
30	2	57.50	None	13,870	—	—	12 x 16 in.	6.5	8.0	73,720	117,900
31	3	45.00	None	15,750	Bamboo	0.08640	18 x 18 in.	7.0	7.0	71,450	110,750
32	2	45.00	None	15,513	Bamboo	0.08640	12 x 16 in.	6.0	6.0	71,450	110,750
33	2	45.00	None	15,513	Diamond	0.05900	12 x 16 in.	6.0	6.0	70,800	113,000
34	3	45.00	None	10,900	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,000
35	3	45.00	None	10,900	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,00
36	2	40.00	None	13,000	Bamboo	—	12 x 16 in.	6.0	6.0	—	—
37	3	40.00	None	13,600	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,000
38	3	40.00	None	13,600	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,000
39	2	40.00	None	5,080	Bamboo	—	12 x 16 in.	6.0	6.0	—	—
40	3	36.00	None	14,550	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,000
41	3	36.00	None	14,550	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,000
42	3	36.00	None	14,450	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,000
43	3	36.00	None	6,170	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,00
44	3	36.00	None	6,170	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,00
45	2	24.00	None	12,730	Bamboo	—	12 x 16 in.	6.0	6.0	—	—
46	2	24.00	None	5,080	Bamboo	—	12 x 16 in.	6.0	6.0	—	—
47	2	17.00	None	14,330	—	—	12 x 16 in.	6.5	5.0	73,720	117,900
48	2	13.00	None	14,330	—	—	12 x 16 in.	6.5	5.0	73,720	117,900
49	3	40.00	No. 3 @ 5 in.	15,760	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,000
50	3	45.00	No. 3 @ 5 in.	14,900	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,000
51	3	45.00	No. 3 @ 7 in.	15,750	Bamboo	0.08640	18 x 18 in.	7.0	7.0	71,450	100,750
52	3	45.00	No. 3 @ 8 in.	14,850	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,000
53	3	45.00	No. 3 @ 12 in.	14,890	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,000
54	3	57.50	No. 3 @ 5 in.	15,100	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,000
55	3	57.50	No. 3 @ 7 in.	15,100	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,00
56	3	57.50	No. 2 @ 10 in.	15,210	—	—	12 x 16 in.	8.0	8.0	73,720	117,900
57	3	57.50	No. 3 @ 12 in.	16,500	Diamond	0.05900	18 x 18 in.	7.0	7.0	70,800	113,000

Concrete was provided by local ready-mix suppliers. Construction of each specimen consisted of pouring the concrete, starting at one end of the wood form and proceeding to the other end. The concrete slump for the HSC test specimens was approximately 9 in. (229 mm). Little effort was required to finish the top surface of the beams. Immediately following casting, the specimens were covered with a plastic sheet. After approximately five days, the forms were removed and the specimens were covered with wet burlap and plastic sheets generally until two days before testing. The casting of each specimen took approximately 30 min.

Test setup and procedures

The test setup and loading arrangement for each test are shown schematically in Fig. 3. The test setup consisted of a beam specimen placed on two roller-type supports and loaded equally at two ends using two hydraulic rams and spreader beams. The applied load, the resulting deflections at each beam end, and midspan and strains developed in longitudinal bars and stirrups were monitored, and the data stored in a computer.

Each test was begun by applying equal loads at the two ends of a beam. The load at each end was applied in increments ranging from 0.5 to 2 kips (2.22 to 8.89 kN), depending on the estimated strength of the beam specimen. Displacement control was used for the specimens with stirrups, following yielding of the longitudinal bars. The load was held constant for approximately five min after each load or displacement increment, during which time cracks were mapped and test observations recorded. Load or displacement increments continued until the specimen failed.

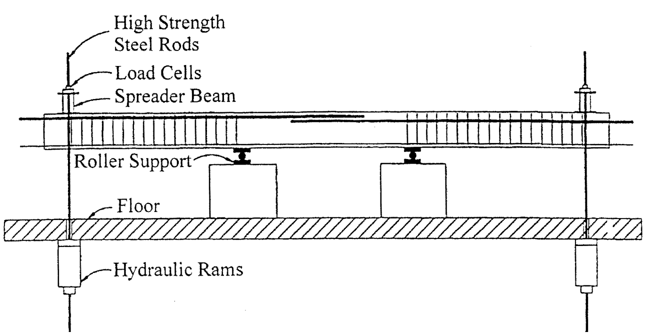


Fig. 3—Test setup.

SUMMARY OF PHASE I INVESTIGATION

A brief summary of all test results is given in Table 5 through 8, which will be discussed in detail later. In Phase I of the investigation, a behavioral model in the form of a failure hypothesis was developed using the results of test data from Specimens 29, 30, 34 through 41, 43 through 48, and 56. A detailed discussion and analysis of these tests are provided in Reference 2. Using this model, an attempt was made to describe the observed differences in the bond performance of deformed reinforcing bars embedded in NSC and HSCs. A brief description of the above noted behavioral model is given as follows.

For the sake of simplicity and to briefly outline the general concept, Fig. 4 shows a segment of a deformed bar embedded in concrete and subjected to different levels of axial tensile

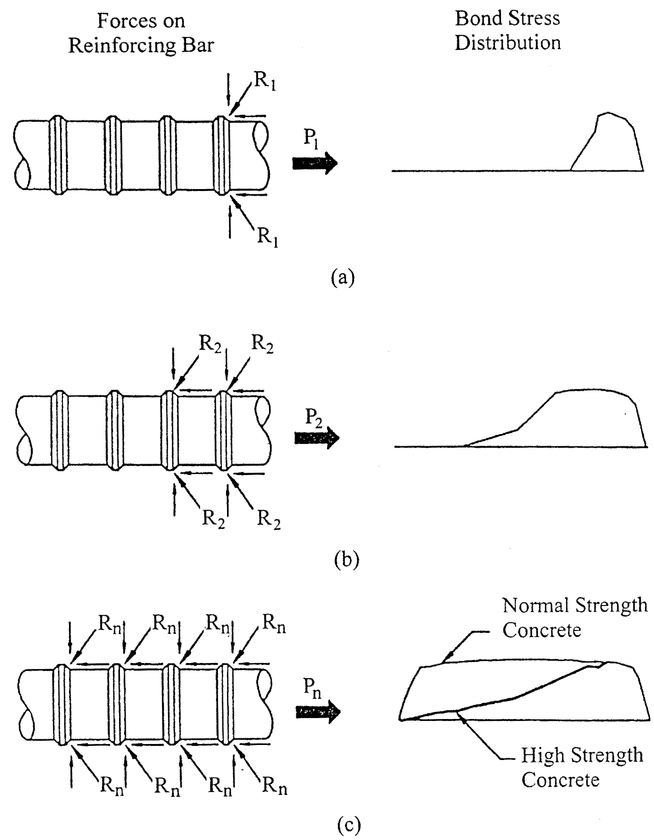


Fig. 4—Freebody diagram of reinforcing bar embedded in concrete and subjected to tension.

Table 4—Details of test specimen No. 11-2db

Test no.	Longitudinal reinforcement	Splice, in.	Stirrups	f_c', psi	Deformation type	Relative rib area	Cross section detail	Dimension A, ft	Dimension B, ft	Yield strength, psi	Tensile strength, psi
58	2	42.00	None	15,034	Bamboo	0.08640	18 x 18 in.	7	7	71,450	110,750
59	2	36.00	None	14,450	Diamond	0.05900	18 x 18 in.	7	7	71,450	110,750
60	2	28.00	None	15,034	Bamboo	0.08640	18 x 18 in.	7	7	71,450	110,750
61	2	20.00	No. 3 @ 2.5 in.	15,191	Bamboo	0.08640	18 x 18 in.	7	7	75,000	113,900
62	2	20.00	No. 3 @ 4.5 in.	16,003	Bamboo	0.08640	18 x 18 in.	7	7	71,450	110,750
63	2	20.00	No. 3 @ 6.0 in.	16,003	Bamboo	0.08640	18 x 18 in.	7	7	71,450	110,750
64	2	24.00	No. 3 @ 2.5 in.	15,191	Bamboo	0.08640	18 x 18 in.	7	7	75,000	113,900
65	2	24.00	No. 3 @ 5.0 in.	14,578	Bamboo	0.08640	18 x 18 in.	7	7	71,450	110,750
66	2	24.00	No. 3 @ 7.0 in.	14,578	Bamboo	0.08640	18 x 18 in.	7	7	71,450	110,750
67	2	28.00	No. 3 @ 4.0 in.	15,417	Bamboo	0.08640	18 x 18 in.	7	7	75,000	113,900
68	2	28.00	No. 3 @ 6.0 in.	15,417	Bamboo	0.08640	18 x 18 in.	7	7	75,000	113,900
69	2	28.00	No. 3 @ 8.0 in.	15,697	Bamboo	0.08640	18 x 18 in.	7	7	71,450	110,750
70	2	28.00	No. 3 @ 12.0 in.	15,697	Bamboo	0.08640	18 x 18 in.	7	7	71,450	110,750

forces. This figure shows the free body diagram of the reinforcing bar at several load stages. At low axial load levels [Fig. 4(a)], the outermost lug (i.e., the one closest to the loading point) exerts a bearing force on the concrete. The horizontal component of this force produces bond stress. (The horizontal component of the friction force is not shown in this figure but also adds to the bond stress). Fig. 4 also shows the corresponding bond stress distribution. As load increases, this bearing force causes crushing of concrete in the vicinity of the lug. This action allows the next adjacent lug to exert a bearing pressure on the concrete and participate in resisting the applied axial tension [Fig. 4(b)]. The ACI Building Code assumes that at ultimate the bond stress distribution is uniform that implies that all the lugs bear against concrete under that load [Fig. 4(c)] and help in resisting the applied axial force. This is a reasonable assumption to make for NSC and has been shown to be valid by experimental testing.[4] In the investigation reported in Reference 2, experimental evidence did not indicate the same behavior for HSC. This observation could be explained as follows.

Referring to Fig. 4, when the first lug bears on the concrete, a bearing force acting against the lug is created. The horizontal component of this bearing force results in what is referred to as bond stress. The vertical component of the bearing force creates a radial force that is responsible for splitting the surrounding concrete. Note also that the bearing strength of concrete is related to f_c', whereas the tensile strength is related to $\sqrt{f_c'}$. Therefore, as an example, assuming that the bearing strength and tensile strength of concrete are given by $0.85f_c'$ and $5\sqrt{f_c'}$, respectively, the ratio of the bearing strength of 15,000 psi (103 MPa) concrete over that of 5000 psi (35 MPa) concrete would be 3, whereas the ratio of tensile strength of 15,000 psi (103

Table 5—Summary of test results: No. 8-1db

Test no.	Maximum displacement ductility	Yield displacement, in.	Failure type	Maximum end load, kips	Maximum bar stress, ksi
1	—	—	—	—	69.90
2	4.30	0.11	Splitting	31.40	73.05
3	1.90	0.11	Splitting	19.27	69.33
4	2.10	0.11	Splitting	18.75	67.77
5	1.30	0.11	Splitting	28.09	67.84
6	0.87+	—	Splitting	17.41	63.06
7	—	—	Splitting	—	44.50
8	—	—	Splitting	14.50	52.90
9	7.70	0.11	Comp	24.44	>78.28
10	5.50	0.12	Splitting	23.83	>78.28
11	4.90	0.12	Splitting	22.66	76.11
12	8.60	0.11	Comp	24.39	>80.21
13	7.80	0.10	Comp	22.09	75.61
14	6.70	0.11	Splitting	23.87	>78.87
15	6.30	0.10	Splitting	22.34	75.05

Table 6—Summary of test results: No. 8-2db

Test no.	Maximum displacement ductility	Yield displacement, in.	Failure type	Maximum end load, kips	Maximum bar stress, ksi
16	7.70	0.100	Splitting	20.14	74.83
17	2.40	0.115	Splitting	18.42	71.00
18	1.90	0.140	Splitting	17.32	67.50
19	1.30	0.120	Splitting	17.42	67.92
20	0.91+	—	Splitting	16.79	65.54
21	—	—	Splitting	10.33	41.50
22	5.00	0.120	Splitting	21.04	76.36
23	1.40	0.120	Splitting	17.53	68.05
24	0.91+	—	Splitting	16.86	65.56
25	5.70	0.110	Splitting	20.31	75.25
26	6.30	0.120	Splitting	19.46	73.13
27	3.80	0.120	Splitting	19.64	73.25
28	4.20	0.132	Splitting	18.60	71.35

Table 7—Summary of test results: No. 11-1db

Test no.	Maximum displacement ductility	Yield displacement, in.	Failure type	Maximum end load, kips	Maximum bar stress, ksi
29	1.24	0.24	—	—	71.10
30	0.92+	—	—	—	67.80
31	2.60	0.13	Splitting	56.05	71.67
32	—	—	Splitting	35.91	68.84
33	—	—	Splitting	36.50	69.67
34	0.66+	—	Splitting	37.00	46.90
35	—	—	Splitting	39.80	50.50
36	—	—	Splitting	30.50	57.20
37	0.79+	—	Splitting	34.80	56.00
38	—	—	Splitting	44.20	56.80
39	—	—	Splitting	21.50	41.10
40	—	—	Splitting	44.00	55.70
41	—	—	Splitting	46.00	58.30
42	—	—	Splitting	44.20	59.14
43	—	—	Splitting	34.80	44.70
44	—	—	Splitting	31.80	40.90
45	—	—	Splitting	23.00	43.10
46	—	—	Splitting	14.60	27.90
47	—	—	Splitting	—	31.80
48	—	—	Splitting	—	28.10
49	2.70	0.18	Splitting	56.60	70.61
50	2.70	0.20	Comp	70.20	>78.98
51	3.40	0.13	Comp	61.88	75.30
52	2.80	0.20	Splitting	67.60	>78.93
53	1.90	0.17	Splitting	59.30	73.19
54	2.90	0.20	Comp	64.80	78.60
55	2.70	0.20	Comp	68.20	>78.82
56	1.90	0.22	—	—	73.70
57	3.10	0.14	Comp	64.80	78.48

Table 8—Summary of test results: No. 11-2db

Test no.	Maximum displacement ductility	Yield displacement, in.	Failure type	Maximum end load, kips	Maximum bar stress, ksi
58	3.30	0.155	Splitting	35.25	73.46
59	1.30	0.160	Splitting	32.60	70.52
60	0.98+	—	Splitting	32.27	70.14
61	2.70	0.180	Splitting	33.99	74.40
62	0.94+	—	Splitting	30.61	67.40
63	0.83+	—	Splitting	26.91	59.41
64	6.50	0.140	Splitting	36.66	76.30
65	1.60	0.196	Splitting	30.06	66.14
66	1.40	0.193	Splitting	30.74	67.56
67	5.10	0.160	Splitting	39.72	80.67
68	4.70	0.150	Splitting	37.46	77.15
69	3.60	0.175	Splitting	36.25	74.79
70	3.10	0.167	Splitting	35.88	74.29

MPa) concrete over that of 5000 psi (35 MPa) concrete would be 1.73. In other words, increasing the compressive strength from 5000 psi (35 MPa) to 15,000 psi (103 MPa) would result in a bearing strength 3 times as large, while the tensile strength increases only 1.73 times.

In the case of HSC, the higher bearing strength of the concrete will prevent crushing of the concrete in the vicinity of each lug to the extent that would otherwise take place in NSC. This implies that, at ultimate, all lugs may not participate in resisting applied axial force and demands that the first few lugs contribute the most. With the first few lugs being more active, and considering the fact that in HSC tensile strength does not increase at the same rate as bearing strength, it may be concluded that in the case of HSC failure would be by splitting of concrete prior to achieving a uniform load distribution.

The experimental evidence and the above behavioral model[2] led to the following major observations and conclusions.

1. In the case of HSC, especially when the concrete cover is small, increasing the lap-splice length is not an efficient approach to increasing bond strength and, in fact, may be totally ineffective. A mechanism that could delay splitting of the concrete over the tension development or the lap-splice length would be more effective in increasing the bond strength of deformed reinforcing bars embedded in HSC. This mechanism could be provided by requiring a minimum amount of stirrups over the tension development or the lap-splice length.

2. In the case of HSC, the assumption of uniform bond stress distribution over the development length may not be valid.

3. In the case of HSC, top cast bars exhibit slightly higher bond strength than bottom cast bars, which is opposite to what is typically observed in NSC. Using the behavioral model described above, Reference 2 provides a possible explanation for this observation.

4. In the past, researchers have used the ratio of bond stress obtained from tests U_{TEST} over bond stress implied by ACI 318 (U_{ACI}) as an index when investigating the bond strength of reinforcing bars embedded in NSC.

U_{TEST} is defined by the following equation

$$U_{TEST} = f_s d_b / 4 l_s$$

where f_s is the maximum bar stress in the reinforcement obtained from tests, d_b is the diameter of the reinforcement, and l_s is the splice length.

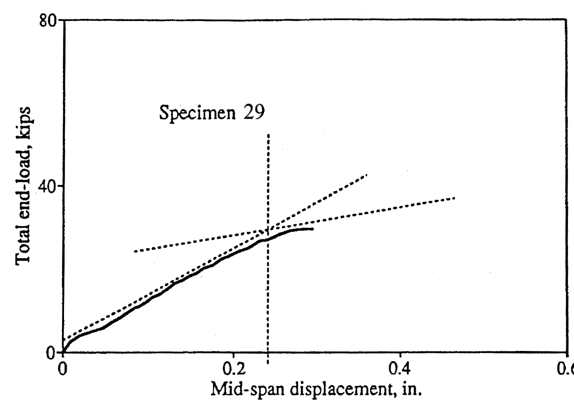
When the value of the U_{TEST}/U_{ACI} index exceeds unity, it is assumed that the bond strength is adequate. However, in the case of HSC, it was concluded that the U_{TEST}/U_{ACI} ratio exceeding one is a criterion that is necessary but not sufficient in assessing the bond strength of reinforcing bars. It was shown that this criterion does not insure that members with tension lap-splices will fail in a ductile manner.

PHASE II INVESTIGATION

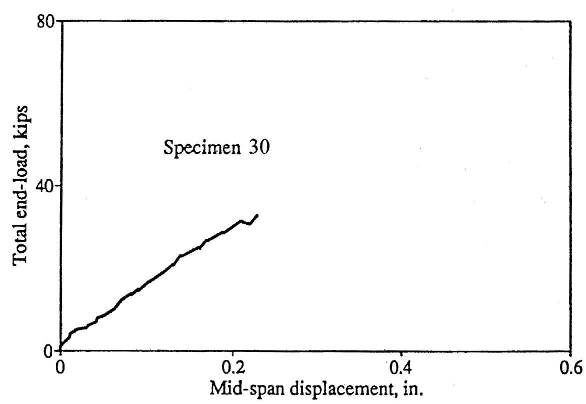
The main purpose of Phase II of the investigation was to develop information that could lead to establishing the required minimum amount of transverse reinforcement over the splice region.

One of the questions raised in Phase I of the investigation was the possibility of not requiring minimum transverse reinforcement over the splice regions for smaller bars or in cases where larger concrete cover thicknesses are used. Consequently, specimens with No. 8 (25-mm diameter) bars and cover thicknesses as large as 2 times the bar diameter were used in several test specimens.

As stated earlier, the use of the U_{TEST}/U_{ACI} ratio as a criterion to study the safety of reinforcing bars embedded in HSC is not adequate. For instance, for Tests 29 and 30, the splice lengths were 80 in. (2030 mm) and 57.5 in. (1460 mm), respectively. The splice lengths for Tests 29 and 30 were calculated based on ACI 318-89 (ignoring the f_c' limitation) and ACI 318-83 Building Code requirements, respectively. The ACI 318-95 Code for the same condition (No. 11 reinforcing bars, concrete cover of one bar diameter, and f_c' of 15,000 psi or 103 MPa) would require approximately 45 in. (1140 mm) splice length (ignoring the f_c' limitation). The U_{TEST}/U_{ACI} ratios for Specimens 29 and 30 were 1.17 and 1.63, respectively, which are greater than 1. However, both specimens failed in a very brittle and violent manner without exhibiting ductility. Fig. 5 gives total end load versus the midspan displacement plots for these two tests. As will be shown later, providing some minimum amount of transverse reinforcement results in a significant increase in ductility. In this study, the displacement ductility ratio was selected as an index to assess the test results. Fig. 6 gives the definition of the displacement-ductility ratio. For most tests, when transverse reinforcement over the splice region was incorporated, the plot of applied end load versus midspan displacement exhibited a relationship of the type shown in Fig. 6. The displacement-ductility ratio is defined as the ratio of the maximum midspan displacement over the first yield displacement. The first yield displacement is defined as corresponding to



(a)



(b)

Fig. 5—Load-displacement response of specimens: (a) Specimen 29; and (b) Specimen 30.

the intersection of the tangents to the load displacement curve at the origin and at maximum displacement (Fig. 6). A displacement ductility ratio of greater than 1 signifies first that longitudinal bars are capable of developing at least their actual yield strength and second that specimens will exhibit some level of ductility. Therefore, the use of the displacement-ductility ratio addresses the dual criteria of strength and ductility, whereas the bond stress ratio is only a strength criterion.

Table 5 through 8 provide summaries of all test results. For each specimen, the maximum displacement-ductility ratio, the first yield displacement, type of failure, maximum applied end load, and maximum tensile stress in the reinforcing bars are given. The maximum stress in the reinforcement was calculated using actual material properties and the moment curvature analysis approach.

The observed effect of transverse reinforcement over the splice region on the bond performance of reinforcing bars for test specimens with No. 8 (25-mm diameter) or No. 11 (35-mm diameter) bars and concrete cover thicknesses of 1 or 2 times the bar diameter were similar, in general. A more detailed discussion of the effect of stirrups is provided below for specimens using No. 11 (35-mm diameter) bars and having one bar diameter as the concrete cover thickness, with specific observations provided for other cases.

No. 11 (35-mm diameter) reinforcing bars with one bar diameter concrete cover

As discussed previously, Specimens 29 and 30 had 77 and 27 percent more splice length than required by ACI 318-95

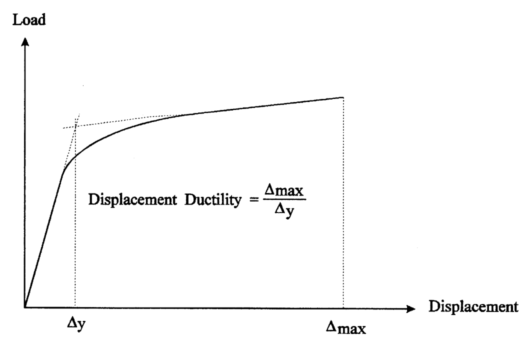


Fig. 6—Definition of displacement-ductility ratio.

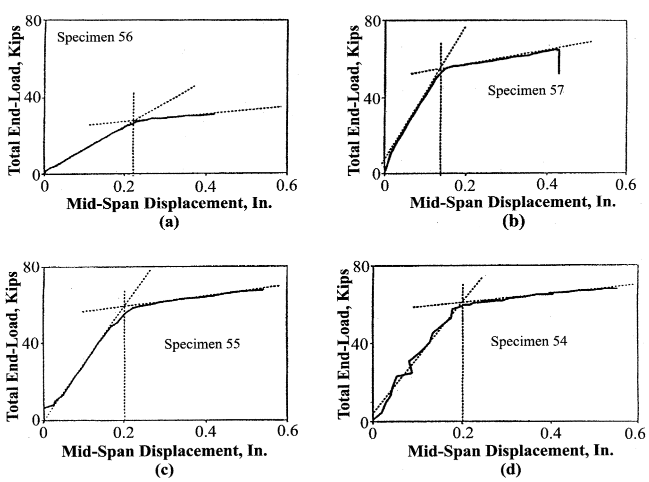


Fig. 7—Load-displacement response of specimens with stirrups: (a) Specimen 56; (b) Specimen 57; (c) Specimen 55; and (d) Specimen 54.

provisions, if the f_c' limitation was ignored. However, both specimens failed in a very brittle and violent manner. Fig. 7 shows the midspan displacement versus total applied end load curves for test specimens with the same splice length as that of 30; however, with different amounts of transverse reinforcement over the splice region. Fig. 7(a), (b), (c), and (d) give load-displacement responses of Specimens 56, 57, 55, and 54, respectively. Fig. 8(a), (b), and (c) give load-displacement responses of specimens (53, 52, and 50, respectively) having the same bar sizes and concrete cover thicknesses [No. 11 (35-mm diameter) bars and one bar diameter concrete cover thickness] as Specimen 30, except that splice lengths for these specimens were 45 in. (1140 mm), rather than 57.5 in. (1460 mm). Fig. 8(d) gives the load displacement response of Specimen 49, which was similar to specimen 30, except that it had a splice length of 40 in. (1020 mm). Fig. 7 and 8 indicate that incorporating some transverse reinforcement over the splice region results in significant increases in ductility.

For most of the test specimens, each stirrup over the splice region was instrumented using electrical strain gages. Fig. 9 through 11 give the strain distributions for the stirrups located over the splice regions for Specimens 57, 50, and 49, respectively. In these figures, the horizontal axis shows the gage position along the splice region. Strain distributions are shown at several different midspan displacements for each specimen. For

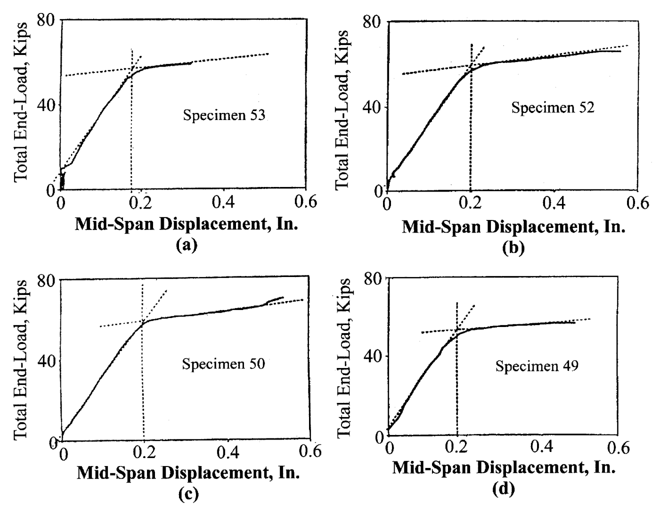


Fig. 8—Load-displacement response of specimens with stirrups: (a) Specimen 53; (b) Specimen 52; (c) Specimen 50; and (d) Specimen 49.

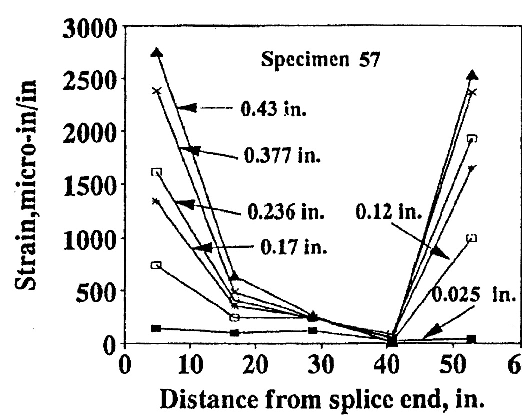


Fig. 9—Strain in stirrups placed over splice region: Specimen 57.

the sake of clarity, data points corresponding to any given midspan displacement are connected by straight lines.

Fig. 9 and 10 indicate that, for specimens with 57.5- and 45-in. (1460- and 1140-mm) splice lengths, the distribution of strains in stirrups over the splice region is quite nonuniform near the maximum midspan displacement, which coincides with specimen failure. This nonuniform strain distribution is more pronounced for the specimen with 57.5 in. (1460 mm) splice length (Fig. 9), compared to the specimen with 45-in. (1140-mm) splice length (Fig. 10). On the other hand, as the splice length decreases to 40 in. in the case of Specimen 49, the strains in stirrups over the splice region assumes a more uniform distribution at maximum midspan displacement, as indicated in Fig. 11.

Fig. 9 through 11 indicate that, in general, the outermost stirrups experience larger strains. This behavior is more pronounced for larger splice lengths.

The effect of stirrups for a given splice length could be studied by comparing the results of Tests 30 and 54 through 57 for the 57.5 in. (1460 mm) splice length; Specimens 33, 50, 52, and 53 for the 45-in. (1140-mm) splice length; and Specimens 37, 38, and 49 for 40-in. (1020-mm) splice length. By comparing the results of these tests, the following observations can be made.

1. The failure mode for Specimens 30, 33, 37, and 38 (that did not include any stirrups over the splice region) was very brittle and violent, even though Specimen 30 had a 57.5-in. (1560-mm) splice length (27 percent more than the ACI 318-95 requirement, ignoring the f_c' limitation).

2. The failure mode for Specimen 56 [that had small diameter stirrups No. 2 (6-mm diameter)] at 10 in. (254 mm) on center was by fracture of one of the stirrups and splitting of concrete cover over the splice region. However, longitudinal bars had already yielded at the time of failure. On the other hand, Specimen 57 [which had No. 3 (10-mm diameter) Grade 60 (414 MPa yield strength) bars at 12 in. on center] failed by the crushing of concrete on the compression side of the specimen in the constant moment region. Splitting of concrete cover over the splice region did not occur in this specimen. The failure mode for Specimens 55 and 54 was similar to the failure mode observed in Specimen 57. The results of Tests 54, 55, and 57 indicate that for the 57.5 in. (1460 mm) splice length, decreasing the spacing of stirrups over the splice region from 12 in. (305 mm) (Test 57) to 7 in. (178 mm) (Test 55) or 5 in. (127 mm) (Test 54) did not significantly influence the specimens' behavior. For Specimens 57, 55, and 54, the displacement ductility ratios obtained were 3.1, 2.71, and 2.9, respectively (Table 7). The displacement-ductility ratio obtained for Specimen 56 was 1.9.

3. For specimens with 45-in. (1140-mm) splice length, the results of Tests 50, 52, and 53 could be studied to investigate the effect of stirrup spacing. These specimens used No. 3 (10-mm diameter) Grade 60 (414 MPa yield strength) stirrups at 12, 8, and 5 in. (305, 203, and 127 mm) on center over the splice region. The mode of failure for Specimens 53 and 52 was by splitting of the concrete cover over the splice region, whereas for Specimen 50 the failure mode was by crushing of the concrete on the compression side in the constant moment region. It should be noted, however, that for Specimen 52, splitting of concrete took place after the specimen achieved a significant level of ductility. The displacement ductility ratios obtained for Specimens 53, 52, and 50 were 1.9, 2.8, and 2.7, respectively.

4. The mode of failure for Tests 37 and 38, which had no stirrups over the splice region, was very brittle. The mode of failure for Specimen 49, which had the same splice length [40 in. (102 mm)], but stirrups over the splice region at 5 in. (127 mm) on center, was similar to that of Specimen 52. In other words, this specimen failed by splitting of concrete over the splice region after achieving a significant level of ductility (2.7).

5. From the discussion presented in Observations 2 to 4 it can be concluded that for specimens with 40, 45, and 57.5 in. (102, 114, and 146 mm) splice lengths, providing No. 3 (100-mm diameter) Grade 60 (414 MPa yield strength) stirrups over the splice region, spaced at approximately 5, 8, and 12 in. (127, 203, and 305 mm), respectively, insures that an adequate level of ductility is achieved before failure.

6. For No. 11 (35-mm diameter) reinforcing bars and one bar diameter cover thickness, tension splices designed on the basis of ACI 318-95 provisions and neglecting the current 10,000 psi (69 MPa) concrete compressive strength limitation will result in an unsafe design situation. Requiring some minimum amount of transverse reinforcement over the length of the tension splices calculated above will result in safe designs.

No. 11 (35-mm diameter) reinforcing bars with two bar diameter concrete cover

It may be speculated that increasing the thickness of cover concrete will achieve the same objective as providing some minimum amount of stirrups over the splice region. Therefore, specimens having concrete cover thicknesses equal to 2 times the bar diameter were included in the testing program. Table 4 gives details of the specimens with and without stir-

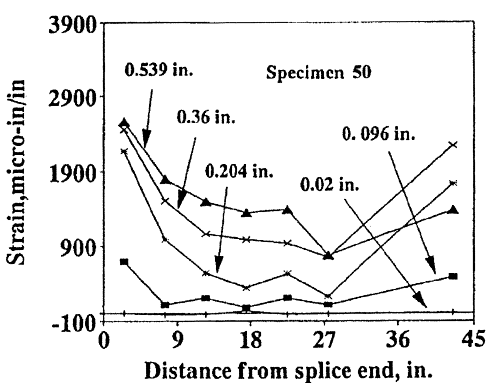


Fig. 10—Strain in stirrups placed over splice region: Specimen 50.

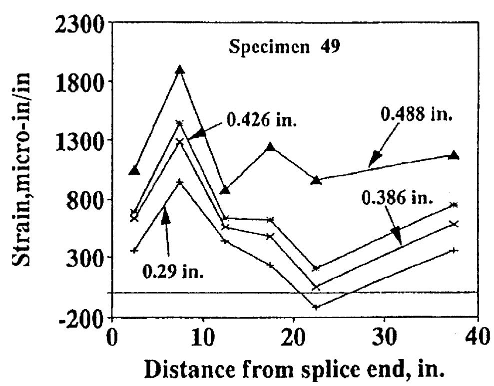


Fig. 11—Strain in stirrups placed over splice region: Specimen 49.

rups, using No. 11 (35-mm diameter) reinforcing bars and a concrete cover thickness of two times the bar diameter. Results of these tests are given in Table 8. For No. 11 (35-mm diameter) bars with f_c' of 15,000 psi (103 MPa) and a concrete cover of 2 times the bar diameter, ACI 318-95 provisions will require a tension splice length of approximately 28 in. (710 mm), ignoring the f_c' limitation. Specimen 60 satisfies this ACI 318-95 requirement. However, this specimen failed in a very brittle manner. Specimen 59, which had approximately 29 percent more splice length than ACI 318-95 requirements, also failed in a brittle manner, exhibiting a small level of ductility (1.3) and little warning before failure. Specimen 58, which had 50 percent more splice length than called for by ACI 318-95 provisions, did exhibit an adequate level of ductility. However, failure of this specimen was very violent by way of splitting of cover concrete over the splice region.

Specimens using stirrups had either 20-in. (508-mm) splice length (Tests 61 through 63), 24-in. (610 mm) (Tests 64 through 66), or 28-in. (710-mm) splice lengths (Tests 67 through 70).

By studying the results of tests with stirrups, as reported in Table 8, the following specific observations could be made.

1. Results of Tests 61 through 63 could be compared to study the behavior of specimens with splice lengths 29 percent shorter than ACI 318-95 requirements, having instead, a relatively large amount of stirrups. Specimen 61, which had stirrups at 2.5-in. (64-mm) spacing, achieved a displacement-ductility ratio of 2.7. The other two specimens failed before achieving any level of ductility. This behavior indicates that the use of stirrups should be in conjunction with some minimum splice length.

2. Results of Tests 64 through 66 could be studied for behavior of specimens with splice lengths 14 percent shorter than the ACI 318-95 requirements. Specimen 64, which had stirrups at 2.5-in. (64-mm) spacing, achieved a large displacement ductility ratio of 6.5. However, the other two specimens failed while exhibiting a small level of ductility (approximately 1.6).

3. Results of Tests 67 through 70 could be used to study the stirrup requirements for splice length calculated on the basis of ACI 318-95 requirements. Inspection of the test results indicate that all specimens failed after achieving a displacement- ductility ratio greater than 3.0. Specimen 70, which had No. 3 (10-mm diameter) stirrups at 12 in. (305 mm) on center, achieved a displacement ductility ratio of 3.1. This behavior again indicates that an approach consisting of calculating the splice length based on current ACI 318-95 requirements (neglecting the f_c' limitation) and providing some level of stirrups over the splice region will lead to safe designs.

From the discussion presented previously, it could be concluded that for No. 11 (35-mm diameter) reinforcing bars with even larger concrete cover thicknesses, some minimum amount of stirrups over the splice region will still be required.

No. 8 (25-mm diameter) reinforcing bars with one bar diameter concrete cover

To investigate the effect of stirrups on the performance of smaller bar sizes embedded in HSC, specimens using No. 8 (25-mm diameter) bars were tested. Details of specimens using No. 8 (25-mm diameter) bars and having cover concrete thickness equal to one bar diameter are given in Table 1. Table 5 gives the test results from these specimens. The following specific observations could be made.

1. The required tension splice length for No. 8 (25-mm diameter) reinforcing bars, according to ACI 318-95 provisions (neglecting the f_c' limitation), embedded in 15,000 psi (103 MPa) concrete and having a concrete cover of one bar diameter is approximately 32 in. (812 mm). This required length was used in Specimens 3 and 4. These two identical specimens were cast at the same time and tested on the same day. Both specimens failed in a brittle manner while exhibiting a relatively small level of ductility (approximately 2). However, the level of ductility exhibited was higher than that observed for No. 11 (35-mm diameter) bars under similar conditions. Specimen 2, which had approximately 12 percent more splice length than required by ACI 318-95, demonstrated a displacement ductility level of 4.3. However, one general observation must be highlighted. Specimens that did not have any stirrups over the splice length and demonstrated an adequate level of ductility by having longer splice lengths than those required by current ACI 318-95 provisions, neglecting the f_c' limitation, eventually failed violently by splitting of the concrete cover over the splice region. The behavior of these specimens was also much harder to predict and demonstrated much larger variations compared to specimens containing any amount of stirrups. The large variations in the behavior of specimens with small bar sizes (No. 8 [25-mm diameter] and smaller), without stirrups and tension splice lengths larger than required by ACI 318-95 could be attributed to the variation in tensile capacity of concrete, which is usually significant. On the other hand, specimens that used any amount of stirrups over the splice region produced much better and predictable results and safety was less of a concern during testing.

2. Two different splice lengths were used for specimens containing stirrups. Specimens 9 through 11 used a 25-in. (635-mm) splice length, whereas Specimens 12 through 15 used a 32-in. (812-mm) splice length.

3. Specimens 9 through 11, which had a shorter splice length than required by ACI 318-95, exhibited a good level of ductility. Specimens 12 through 15 [32-in. (812-mm) splice length] all exhibited a good level of ductility, even Specimen 15, which only had three stirrups over the splice region. In summary, the observed behavior indicates that for No. 8 (25-mm diameter) and smaller bar sizes, the amount of required minimum stirrups should be smaller.

No. 8 (25-mm diameter) reinforcing bars with two bar diameter concrete cover

Details of the specimens using No. 8 (25-mm diameter) bars and having a thickness of concrete cover equal to 2 times the bar diameter are given in Table 2. Table 6 gives the test results from these specimens. The following specific observations could be made.

1. The required tension splice length for No. 8 (25-mm diameter) reinforcing bars, according to ACI 318-95 provisions (neglecting the f_c' limitation), embedded in 15,000 psi (103 MPa) concrete and having a concrete cover of two times the bar diameter is approximately 19 in. (482 mm). This splice length was used in Specimens 18 and 19. These two identical specimens were cast at the same time and tested on the same day. Both specimens failed in a brittle manner while exhibiting displacement ductility ratios of 1.3 and 1.9. Specimen 16, which had a much longer splice length (89 percent longer)

than required by ACI 318-95, demonstrated a very good level of displacement ductility (7.7).

2. Two different splice lengths were used for specimens containing stirrups. Specimens 22 through 24 used a 15-in. (381-mm) splice length, whereas Specimens 25 through 28 used a 19-in. (483-mm) splice length.

Specimens 23 and 24, which had only 15-in. (381-mm) splice lengths and stirrups at 4.5 and 6.5 in. (114 and 165 mm) on center, failed in a brittle manner while demonstrating a very small level of ductility. For the same splice length, when the spacing of the stirrups was reduced to 3 in. (76 mm), the behavior was satisfactory.

Specimens 25 through 28, which had splice lengths as required by ACI 318-95 provisions (ignoring the f_c' limitation), all exhibited a good level of ductility before failure.

In summary, the observed behavior indicates that for No. 8 (25-mm diameter) reinforcing bars and a concrete cover thickness of 2 times the bar diameter, some minimum amount of stirrups over the splice region will still be required.

CONCLUSIONS

To develop an alternative to the current f_c' limitation in Section 12.1.2 of ACI 318-95 for calculating the tension development length and the tension lap splices, an investigation was carried out. Using the information from the experimental phase of the project, a behavioral model was developed that attempts to describe the observed differences in the behavior of reinforcing bars embedded in NSC and HSC. Further, the paper identifies that providing some minimum amount of stirrups over the tension development length or the tension lap splice length will produce design conditions that will be in compliance with the general ACI design philosophy of ductile reinforced concrete behavior. By reviewing the information generated during the investigation, the following main conclusions were drawn.

1. In the case of HSC, especially in the presence of a small cover, increasing the splice length is not an efficient approach to increasing bond strength. A mechanism that could delay splitting of the concrete cover over the tension development length or tension splice length would be more effective in increasing the bond strength of deformed reinforcing bars embedded in HSC. This mechanism could be provided by requiring some minimum amount of stirrups over the development or the splice length.

2. In the past, researchers have used the ratio of bond stress obtained from tests U_{TEST} over bond stress implied by ACI Codes U_{ACI} as an index when investigating the bond strength of reinforcing bars embedded in NSC. When the value of this index exceeds unity, it is assumed that the bond strength is adequate. However, in the case of HSC, it was concluded that the U_{TEST}/U_{ACI} ratio exceeding 1 is a criterion that is necessary, but not sufficient, in assessing the bond strength of reinforcing bars. It was shown that this criterion does not insure that members with tension splices will fail in a ductile manner.

3. By studying the behavior of specimens having No. 8 (25-mm diameter) and No. 11 (35-mm diameter) reinforcing bars and varying amounts of concrete cover, it was concluded that even in the case of larger concrete cover thicknesses, one needs to provide some minimum amount of stirrups over the tension development length or the tension splice length.

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