BEHAVIOR OF ONE-WAY CONCRETE SLABS REINFORCED WITH WELDED WIRE FABRIC WITH AND WITHOUT END LOOPS

by

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THESIS

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To my parents

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Chad Corsten
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ABSTRACT

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Welded Wire Fabric has several advantages for use as reinforcement in concrete members, including uniform steel placement, ease of placement, and increased bond strength from the welded intersections.

The testing of a type of WWF having end loops was the subject of this research program and thesis. The end loops were developed to enhance the development of the tensile stresses in lap splices of WWF. Seven slabs were tested to validate the behavior of standard 12 in lap splices utilizing WWF having end loops. For comparison, one specimen using standard WWF and a 15 in lap splice was tested. The variables of the testing program were: slab thickness (8 in, 9 in, 10 in), bar diameter (10mm, 11mm, 12 mm), concrete strength (f 'c = 3500 psi and 5000 psi), and reinforcement type. Six of the specimens did not satisfy currently recommended ACI Code values for lap splice lengths of deformed WWF, but the guaranteed yield strength (80 ksi) was developed with the fabric having end loops.

Measured values for each test included: load, deflection at load points and midspan, steel stresses, and crack size and spacing.

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CHAPTER 1

INTRODUCTION

1.1 General

Modern construction of reinforced concrete structures involves the use of Welded Wire Fabric (WWF) in numerous applications such as slabs, columns, beams, walls, bridge decks, and piers. Welded Wire Fabric was originally invented around the turn of the century by John C. Perry, of Massachusetts. In 1901, patent papers were filed for an electronic device used to weld steel wires together [1]. Welded Wire Fabric is a manufactured rectangular grid of steel plain or deformed wires welded at each intersection. The welding process applies an electric current and pressure to fuse the wires together, avoiding the use of additional materials [1].

The most common applications for the use of WWF in construction are concrete slabs. Short span and long span one-way slabs, one-way corridor type structures, flat plates and two-way slabs are all common uses for WWF. Several advantages for using WWF include:

1. Less steel required due to higher yield strengths and allowable stresses,
2. Uniform distribution of steel in both the longitudinal and transverse directions for better load distribution and crack control,
3. Quick and economical placement the prefabricated sheets,
4. Better bond by using smaller, more closely spaced bars,
5. Positive mechanical anchorage from the welded transverse wires [1].

Other uses for WWF include shrinkage and temperature steel in numerous applications, and as shear reinforcement for prestressed concrete members [2]. More recently, WWF as confining reinforcement in columns has been utilized. The closely spaced longitudinal and transverse wires can provide confinement as effectively as closely spaced ties in columns [3]. Continued research is being done to study other applications of WWF as confining and regular reinforcement in concrete.

1.2 Background

A particular type of Welded Wire Fabric having features that enhance the development of lapped splices was the subject of this testing program. Welded Wire Fabric used in the United States does not include the looped bar feature shown in Figure 1.1. Another feature of the WWF used in the testing program was the use of deformed bars.

 EMBED Word.Picture.6

Figure 1.1 Looped mesh

Tests of the lap splice of Welded Wire Fabric were carried out by EMPA, a Swiss testing organization, in December 1982. These tests consisted of five reinforced concrete slab strips having lap splices at midspan. The slabs were simply supported and loaded by two equal loads at nearly quarter points of the simple span of approximately 9 ft (280 cm). The variables of the testing program were slab thickness and concrete strength. The reinforcement size for all tests was constant. All of the specimens failed due to wire fracture outside the lap splice, except for one, which failed due to a splice failure [4].

Similar tests were also performed at the University of Texas at Austin in the fall of 1991. These tests differed from the EMPA tests by the following: different splice lengths were tested and three bar sizes were used, WWF mesh without loops was included in the test program, only one concrete strength was specified for all tests [5].

Test specimens in this program differed slightly from the previous testing. These differences include:

1. Different slab thicknesses and bar sizes were used,
2. Rebar mesh without loops was included in the test program,
3. Multiple concrete strengths were specified,
4. A different loading arrangement was utilized.

1.3 ACI 318R-89 Requirements For Splices of Welded Wire Fabric in Tension

The splice requirements for WWF design are outlined in Chapter 12 of the ACI 318R-89 building code. Only deformed WWF is the subject of this research program, and therefore will be discussed. Requirements for plain WWF can also be found in the code [6].

Section 12.18 states that the minimum splice length for deformed WWF shall not be less than 1.3 times the development length, ld, for the bar size and yield strength, nor 8 in The code also states that the overlap of the outermost transverse wires in the splice region shall not be less than 2 in If the splice design should exclude transverse bars, then the code specifies the splice length to be governed as for deformed wire. Figure 1.2 demonstrates the specifications [6].

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a) ACI 318R-89, section 12.18.1

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□□

b) ACI 318R-89, section 12.18.2

Figure 1.2 Requirements for lap splice lengths of deformed WWF [6]

1.4 Objective and Scope

This testing program set out to investigate and validate the behavior of the lap splice region of the different test slabs, compare the behavior of slabs having different concrete strengths and its correlation with splice behavior, and compare the performance of Looped Welded Wire Fabric (WWFL) and standard Welded Wire Fabric (WWF) without end loops.

The advantages of WWFL are the use of a single lap splice length for ease of construction and efficient use of materials, better splice performance through additional anchorage and transfer of tensile stresses, and a standard manufacturing program for quality and cost effectiveness. A previous study at The University of Texas at Austin with 5 mm, 8 mm, and 10 mm diameter WWFL showed excellent lap splice behavior (see Section 2.2.5) [5].

The scope of this research program consisted of the planning, construction and testing of 8 concrete slabs utilizing larger diameter WWFL than the previous study. The slabs were 118 in (300 cm) long and 36 in (91 cm) wide. The first three pairs of slabs were constructed using concrete with a specified strength, f'c = 3,500 psi (24.1 kN/mm2) while the last pair of slabs were constructed using concrete with a specified strength, f'c = 5,000 psi (34.5 kN/mm2). The first pair of slabs were 8 in (20 cm) deep and reinforced with 10 mm diameter WWFL. The second pair of slabs were 9 in (23 cm) deep and reinforced with 11mm diameter WWFL. The third pair were 10 in (25 cm) deep and reinforced with 12 mm diameter bars, but one used WWFL while the other used WWF. The fourth pair utilized higher strength concrete and consisted of companions for the second and third pairs of slabs with WWFL; the first of the pair was 9 in deep and reinforced with 11 mm WWFL, and the second in the pair was 10 in deep and reinforced with 12 mm WWFL. Details of the experimental program are given in Chapter 3.

The testing was carried out at the Phil M. Ferguson Structural Engineering Laboratory at the University of Texas at Austin's J.J. Pickle Research Campus, Austin, Texas, USA.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Welded Wire Fabric has been used since the turn of the century, but little research was done until 30 or 40 years ago. Initial research focused on splices, anchorage and crack widths for WWF. The behavior of WWF is different from other types of concrete reinforcement due to the close spacing and welded intersections of longitudinal and transverse bars. Because of this, splice and anchorage requirements had to be determined from numerous laboratory tests. The cracking behavior of concrete members reinforced with WWF also had to be considered in order to control crack spacing and crack width. Results of the aforementioned topics were compared with similar tests for conventional concrete reinforcement to assure reasonable design recommendations as code provisions were developed.

The following chapter presents research done on splice, anchorage and crack width requirements for WWF. Included is the research done on WWF having end loops for two separate testing programs.

2.2 Splices In One-way Slabs Reinforced With Welded Wire Fabric

The requirements in ACI 318R-89 for splice and anchorage of WWF are based on the research done by Kesler and Lloyd [7]. Presented here are these studies, and testing done on WWF having end loops.

2.2.1 John P. Lloyd and Clyde E. Kesler, 1970 [7]

Lloyd and Kesler tested 48 one-way slabs, of which 23 were reinforced with WWF, to determine the splice and anchorage requirements of deformed WWF and deformed wire. The dimensions of the slabs were 76 in long and 24 in wide, having a depth of either 5 in or 7 in The slabs were simply supported on a span of 6 ft, and loaded symmetrically either 12 in or 18 in from the supports. Only the results from the WWF are pertinent for discussion here.

Of the 23 slabs reinforced with WWF, 18 of them exhibited splice failures, 3 of them exhibited failures in the shear span, and 2 of them exhibited flexural failures in the constant moment region. Specimens having reinforcement ratios less than 0.4% failed from pull out of the wires in the lap splice. Splitting of the concrete in the splice region was also observed, with more severe splitting occurring with higher reinforcement ratios. Design parameters for splice and anchorage strength include: concrete strength, weld strength, and bond strength.

Due to the splitting of the specimens, the splice and anchorage lengths were increased 20% when the longitudinal wires are spaced closer than 12 diameters. The following formula for the required splice length, l, for deformed WWF was recommended:

$$l = (0.045)(D)(fy - 20{,}000\ N) \qquad [7]$$

$$(f'c$$

where N = Number of pairs of transverse wires in the lap

D = Diameter of wires

It was concluded that the splice length in one-way slabs reinforced with deformed WWF depended on bond, weld, and concrete strength.

2.2.2 John P. Lloyd, 1971 [8]

Lloyd studied the factors affecting the strength of lab splices by testing 36 one-way slabs reinforced with plain WWF. All of the slabs were 192 in long, 36 in wide (excluding one 25 in wide slab), and had varied thicknesses of 6 in to 16 in [8].

The tests considered lap splice length, concrete cover, concrete strength, and reinforcement ratio. All of the slabs failed in the splice region from pull-out, except one which failed from the crushing of concrete. One conclusion was that the strength of lap splices increases with increased splice length, and by using more closely spaced smaller wires. The remaining factors, such as concrete cover, concrete strength, and reinforcement ration had a less significant effect on the splice strength of plain WWF. Finally, the bond stress due to an excess length of longitudinal wire beyond the final transverse wire in the splice region added strength to the lap splice [8].

Lloyd suggested the following revisions to the splice requirements [8]:

1. Lap splices of wires carrying more than one-half of the permissible stress should be avoided,
2. The overlap measured between the outermost transverse wires should not be less than 2 in for splices with wire stresses no more than one- half the permissible stress,
3. To prevent splitting failure, the overlap measured between the transverse wires should not be taken less than:

   [40(As required/ft) - (0.8)(Lo)(As provided/ft)],

   where Lo is the total length of wire extending beyond transverse wires for each pair of spliced wire.

2.2.3 Bilal M. Ayyub, Naji Al-Mutairi, and Peter Chang, 1994 [9]

The research program studied the effects that several parameters have on the splice strength in one-way slabs reinforced with WWF. A total of three slabs were tested, each having a different splice arrangement. The first had transverse bars in the splice bearing against each other, and a splice length of 24 in. The second had transverse bars in the splice separated by concrete, and a splice length of 22 in. The third did not have transverse bars in the splice region, and a splice length of 16 in [9].

The following parameters were found to affect the strength of a splice of WWF: 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 type of splice [9].

The results showed that the first specimen, with a splice length of 24 in, had an ultimate resistance close to the required capacity, even though the splice length was 35% larger than the recommended value by the ACI code. The second specimen, with a splice length of 22 in, had an ultimate resistance less than the required capacity, even though the splice length was 26% longer than the recommended value by the ACI code. The tests resulted in a recommendation to adjust the basic development length, ld, to:

$$ldb = 0.03db(fsu - 20{,}000)/((f\,'c)$$

where fsu = specified ultimate strength (psi), replacing fy. Placing the splice in a region of low moment would decrease the ultimate strength required, and thus reduce the required splice length. The third specimen, without transverse bars in the splice zone, exhibited a behavior that was more ductile, similar to that of slabs without a splice. The performance of the splice had an ultimate resistance less than the required capacity, but the performance can be improved with a longer splice length. For the cases with transverse wires in the splice, 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 in the splice region containing transverse bars. Finally, 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 [9].

2.2.4 A. Maissen, and M. Ladnen, 1982 [4]

This test program was the first to study the behavior of one-way slabs reinforced with WWF having end loops (WWFL). The test program consisted of five simply supported one-way slabs with dimensions of 118 in long and 35.5 in wide. The variables of the tests were slab thickness and concrete strength. Each specimen was reinforced with 8mm deformed WWF having end loops (WWFL), and had a splice length of 8 in [4].

The specimens were loaded at about quarter points of the 110 in span, creating a constant moment region of 60 in. Four of the slabs failed due to fracture of reinforcement next to the lap splice, indicating that the 8 in lap splice was adequate to yield the deformed 8 mm bars. Only one of the slabs failed in the lap splice, due in combination to a low concrete strength and high reinforcement ratio (0.46%). From these tests it can be concluded that for slabs reinforced with 8 mm WWF having looped ends (WWFL), and a reinforcement ratio of 0.0030 to 0.0046, the 8 in splice length is adequate to yield the bars.

2.2.5 Oguz Egilmez, 1991 [5]

The purpose of this research program was to provide additional testing of one-way slabs reinforced with WWF having looped ends (WWFL) in order to better understand the behavior. A total of eight reinforced concrete one-way slabs were tested with dimensions of either 54 in, or 118 in long, 35.5 in wide, and depths of 3 in, 6 in, or 8 in. The variables considered were splice lengths, wire diameter, and regular or looped ends. The design concrete strength of 3,500 psi was constant for all tests. Table 2.1 summarizes the specimens tested.

Table 2.1 Test specimens and failure modes [5]

Specimen	Slab Depth (in)	Bar Size[1] (mm)	Mesh Type	Splice Length (in)	Failure Mode[2]
1A	6	8 D	looped	8	F
1B	6	8 D	looped	8	F
2A	8	10 D	looped	12	F
2B	8	10 D	looped	12	F
3A	3	5 P	w/o loops	6	S
3B	3	5 P	looped	6	F
4A	3	5 P	w/o loops	6	F
4B	3	5 P	w/o loops	8	F

Notes: 1. Bar Size, D = Deformed, P = Plain

2. Failure modes, F = Flexural, S = Splice

All of the specimens failed due to the fracture of the reinforcement outside the splice region except 3A, which failed due to concrete splitting in the splice. Specimen 3A did not meet the minimum criteria for splice length according to the ACI building code, and the single pair of transverse wires in the splice region did not provide adequate mechanical anchorage to yield the reinforcing wires. These tests proved that the WWFL aided in the development length of the reinforcement in the splices, and enabled all but one specimen to yield the bars [5].

2.3 Cracking of One-Way Slabs Reinforced with Welded Wire Fabric

Cracking in concrete can result in several problems such as exposing reinforcement to corrosive materials, and aesthetic degradation of a structure. Since cracking in ordinary reinforced concrete construction cannot be eliminated, it is necessary to control the cracking.

2.3.1 Amos Atlas, Chester P. Siess, and Clyde E. Kesler, 1965 [10]

Tests were performed on one-way concrete slabs reinforced with plain WWF to study the overall flexural behavior of the slabs. It was determined that crack spacing was related to the transverse wire spacing, and the ratio of the wire diameter, D, to the effective reinforcement ratio, (e. The formula that was developed for determining the average crack spacing, aavg is [10]:

aavg = Cs(3 + 0.4S) (in) [10]

where Cs is a coefficient dependent on S and D/(e

Cs = 1.0 + 0.024(D/(e - 43), for S = 12 in

Cs = 1.0 + 0.008(D/(e - 33), for S = 6 in

Cs = 1.0 , for S = 3 in

2.3.2 John P. Lloyd, Hassen M. Rejali, and Clyde E. Kesler, 1969 [11]

The research program involved the testing of 23 one-way slabs reinforced with deformed WWF to study the crack controlling capability. As a comparison, several of the specimens were reinforced with deformed bars and wires. The slabs were 6 ft long, 2 ft wide, and 5 in deep.

Crack widths were measured at the reinforcement level and extreme tensile face for steel stresses of 30, 40, 50, and 60 ksi. Initial cracking was observed when the steel strain was 0.00009 and 0.00013 in/in. Cracks generally formed at the transverse wires. It was concluded that deformed WWF, smooth WWF, deformed wires and deformed bars control the maximum crack width equally well.

2.3.3 M.A. Mansur, K. H. Tan, S. L. Lee, and K. Kasiraju, 1987 [12]

The research program consisted of the testing of 14 one-way slabs reinforced with deformed WWF to develop equations for predicting the spacing and width of cracks. The major test parameters included the spacing and diameter of transverse wires, and the concrete cover [12].

The test results showed that crack spacing in slabs depends mainly on the transverse wire spacing. The following formulas were developed for crack prediction, estimating minimum and maximum crack widths, amin and amax [12]:

$$amin = St \qquad , \text{for } St < hc$$
$$amax = 2St$$

$$amin = (St - hc) \qquad , \text{for } hc < St < 2hc$$
$$amax = St$$

$$amin = (St - 2hc) \qquad , \text{for } 2hc < St < 3hc$$
$$amax = 2hc$$

$$amin = (St - nhc) \qquad , \text{for } nhc < St < (n+1)hc$$
$$amax = 2hc$$

where St = Spacing of transverse wires

hc = D[1 + m(a(1 - ((1+2/m())] = height of primary cracks

D = Total slab thickness

a = Ratio of effective depth to total depth of slab

m = Modulus ratio

(= Reinforcement ratio

It was concluded that for better crack control, the spacing of transverse wires should be substantially less than 2hc [12]. It was also concluded that deformed WWF provides better crack control than plain WWF when equally proportioned.

2.4 Strength and Ductility of One-Way Slabs Reinforced with Welded Wire Fabric, 1981 [13]

The researchers tested ten one-way slabs reinforced with WWF to investigate the strength and ductility. Earlier tests of deformed and plain wire used in WWF showed that the average yield strength corresponding to 0.5% strain is between 75 and 85 ksi, but that the ultimate deformations are considerably less than conventional reinforcing bars. While this strength may be adequate, the ability for the steel to deform determines the ductility, which is a highly desirable trait in reinforcing steel.

The results of testing indicated that slabs with a steel ratio of 0.2% failed due to fracture of steel, a non-ductile failure, while slabs with a steel ratio of 0.9% demonstrated a ductile type failure. The testing resulted in a proposed minimum tensile steel ratio to assure ductile failure of slabs reinforced with plain or deformed WWF as follows:

(min = 0.0012 [13]

(An

where, An is the nominal area of one wire (in2).

The preceding equation for a minimum reinforcement ratio results in values greater than specified by ACI 318R-89, (min = 200/fy. Conclusions from the testing stated that in order to assure ductile failures in slabs reinforced with WWF, the reinforcement ratio should be between the proposed minimum value and 0.75 times the balanced reinforcement ratio, but not less than the ACI value [13].

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 Specimen Numbering System

A numbering system for the specimens was developed in order to simplify the presentation and discussion of test results. This notation was devised to describe the characteristics of each specimen, and is in the general form of:

N-A K L

where N = Test Number

1A, 1B, 2A, 2B, 3A, 3B, 4A or 4B

A = Thickness of slab in inches

8, 9 or 10

K = Reinforcement type and size

D10 Deformed 10 mm bars

D11 Deformed 11 mm bars

D12 Deformed 12 mm bars

L = Mesh type and lap splice length

L12 Looped mesh with a splice length of 12 in (30 cm)

N15 Mesh without Loops with a splice length of 15 in (38 cm)

Examples of this numbering system are illustrated in Table 2.1.

Table 3.1 Examples of the specimen numbering system

Test Specimen Number	Slab Depth (in)	Bar type	Bar Size (mm)	Mesh Type	Splice (in)
1A-8D10L12	8	Deformed	10	Looped	12
1B-8D10L12	8	Deformed	10	Looped	12
2A-9D11L12	9	Deformed	11	Looped	12
2B-9D11L12	9	Deformed	11	Looped	12
3A-10D12L12	10	Deformed	12	Looped	12
3B-10D12N15	10	Deformed	12	Non-Looped	15
4A-9D11L12	9	Deformed	11	Looped	12
4B-10D12L12	10	Deformed	12	Looped	12

3.2 Test Specimens

A total of eight reinforced concrete slabs were tested. Seven of these slabs were reinforced with looped fabric (WWFL). Only one of the slabs had the standard WWF without end loops. Figure 3.1 and Figure 3.2 are photographs of 10 mm bar size WWFL and 12 mm bar size WWF respectively, inside the forms.

Figure 3.1 10 mm bar size WWFL (looped)

Figure 3.2	12 mm bar size WWF (non-looped)

The variables of the test program were:

1. Thickness of slab
2. Reinforcing bar size
3. Mesh type (WWFL or WWF)
4. Lap splice length (Differing between WWFL and WWF only)
5. Concrete strength (f 'c).

Reinforcement ratios of the slabs are illustrated in Table 2.2 .

Table 3.2	Reinforcement ratios of the slabs

SpecimenReinforcement Ratio * (1A-8D10L12	1.81 %2A-8D11L12, 2B-8D11L12, 4A-9D11L12	1.79 %3A-10D12L12, 3B-10D12N15, 4B-10D12L12	1.74 %

* (= (As/bd) x 100

According to ACI 318-89R the minimum reinforcement ratio of structural members is 200/fy [6]. For these slabs (fy = 80,000 psi) the minimum reinforcement ratio is 0.25%, and all of the slabs meet this requirement.

Figure 3.3 gives the details of slabs 1A-8D10L12 and 1B-8D10L12. Figure 3.4 gives the details of slabs 2A-9D11L12, 2B-9D11L12 and 4A-10D12L12. Figure 3.5 gives the details of slabs 3A-10D12N12 and 4B-10D12L12. Figure 3.6 gives the details for slab 3B-10D12N15.

Figure 3.3 Details of slabs 1A-8D10L12 and 1B-8D10L12

Figure 3.4 Details of slabs 2A-9D11L12, 2B-9D11L12 and 4A-9D11L12

Figure 3.5 Details of slabs 3A-10D12L12, 4B-10D12L12

Figure 3.6 Details of slab 3B-10D12N15

3.3 Measurements Taken

The following measurements were taken during each test:

1. Six linear potentiometer displacement measurements.

Two at midspan, and two at each load point

2. Three dial gauge measurements. One at midspan and each load point

3. Strain in steel was measured with electric resistance strain gauges

4. Crack widths for all cracks were measured at two different steel stresses

5. First cracking load

6. Peak load, midspan deflection at peak load

7. Failure load, midspan deflection at failure load

8. Failure mode

3.4 Material Properties

3.4.1 Concrete Strengths

The slabs were cast two at a time in order to minimize cost. Concrete for all the specimens was normal weight concrete, purchased from the same local ready-mix plant. The concrete strengths for each pair of slabs is shown in Table 3.3. The plots of strength versus age of concrete are presented in Figure 3.7. The design strength for the first three pairs of slabs was f 'c=3500 psi. (24 N/mm2) and the mix proportions were as follows:

Concrete Mix Design (Proportions for 1 yd3)

Water	260 lb (118 kg)
Cement	370 lb. (168 kg)
Fine Aggregate (sand)	1560 lb (708 kg)

Coarse Agg. 1625 lb (738 kg) Max. size 3/8" (1 cm)

(crushed stone)

Slump for the concrete was about 4 in for slabs 1A/1B and 3A/3B, and about 6 in for slabs 2A/2B. The higher slump for slabs 2A/2B resulted in considerably lower concrete strength as shown in the Figure 3.7.

The design strength for the fourth pair of specimens was f 'c=5000 psi (34 N/mm2) and the mix proportions were as follows:

Concrete Mix Design (Proportions for 1 yd3)

Water 250 lb. (113 kg)

Cement 470 lb. (213 kg)

Fine Aggregate (sand) 1385 lb. (628 kg)

Coarse Agg. 1870 lb. (848 kg) Max. size 3/4" (1.9 cm)

(crushed stone)

Slump for the concrete was 4 in for slabs 4A/4B with the revised mix shown above for higher strength concrete. The higher strength shown in Figure 3.3 for slabs 4A/4B is due to the revised mix design.

Table 3.3 Concrete strength data

Test SpecimenAge of Concrete - daysStrength - psi1A/1B83279284180Day of test 1A814548Day of test 1B9344282A/2B52393283138Day of test 2A543333Day of test 2B6334883A/3B72864143177Day of test 3B243531Day of test 3A2835784A/4B63065144863255701Day of test 4A and 4B345723

Figure 3.7 Concrete strength vs. Age

3.4.2 Reinforcement

Three different bar sizes were used: deformed 10 mm bars, deformed 11mm bars and deformed 12mm bars. Figures 3.8, 3.9, 3.10 show the stress-strain curves for these bars, respectively. Table 3.4 illustrates the bar areas and yield strengths based on 0.2% extension under load following the appropriate ASTM specification [14]. The nominal yield for all three sizes of bars was 80 ksi (550 N/mm2) and the range of measured yield was significantly above this value.

Table 3.4 Steel bar areas and yield strengths

Bar Size	Area in2 (mm)	Yield Strength ksi (kN/mm2)
10 mm	0.122 (78.5)	89 (613)
11 mm	0.147 (95.0)	100 (689)
12 mm	0.175 (113.1)	97 (668)

Figure 3.8 Stress-strain curve for deformed 10 mm bars

Figure 3.9 Stress-strain curve for deformed 11mm bars

Figure 3.10 Stress-strain curve for deformed 12 mm bars

3.5 Test Setup

3.5.1 Description of Test Setup

The testing apparatus consisted of two concrete blocks at approximately quarter points with a roller and fixed bar for the slab to rest upon. The blocks and bearing rods were securely fastened to their respective surfaces with hydrostone. A steel frame consisting of angles and channels was placed on the slab, providing loading points for the hydraulic rams at a distance of 7 in from the slab ends. The hydraulic rams were bolted to the frame with steel rods anchored to the strong-floor. Loading was provided by four hydraulic rams, one at each corner, and applied to the specimens by means of the channels. A schematic of the test setup is shown in Figure 3.11.

□ EMBED Charisma □□□

Figure 3.11 Test setup

3.5.2 Testing Instrumentation

The behavior of the specimens to the applied load was monitored by several types of instruments and recorded by a data acquisition system. The instruments included a load cell, pressure gauge, pressure transducer, six linear potentiometers, three displacement dial gauges and six electric strain gauges.

The applied load was determined by the pressure transducer readings and the hydraulic rams, which were calibrated prior to testing. The load cell was used to check the calibrated rams and pressure transducer, which proved to be very close to the pressure transducer.

Deflections were measured by the linear potentiometers and dial gauges. The steel stresses were measured by electric resistance strain gauges that were applied to the steel bars before casting the concrete.

3.5.3 Test Procedure

The test specimens were loaded incrementally and data was recorded at each load increase. Loading was load-controlled in the elastic range and deflection controlled in the plastic range. The applied load, deflections and the strain in the steel were recorded at each load level. Cracks were marked on the slab as they appeared, and their widths were measured at two steel stresses, typically 40 ksi (276 kN/mm2) and 50 ksi (345 kN/mm2). Loading continued until failure occurred. The duration of the tests was between two and four hours for each slab.

3.6 Data Recording and Processing

A personal computer was used to obtain and store data by means of a scanner to read the electrical instruments. Crack data and dial gauges were recorded manually after each loading cycle. A computer program was used to read and store the individual voltages from the instruments. The data was then changed from voltages to engineering units by another computer program. This data was then transferred to a spreadsheet program for analysis.

CHAPTER 4

PRESENTATION OF TEST RESULTS

4.1 Introduction

All of the data obtained from the testing of the eight specimens is presented in this chapter. A schematic of the loading for all specimens is shown in Figure 4.1.

All test specimens were cast as shown in Figure 4.1 except that the steel was in the bottom of the form as shown in the reinforcement photographs. The specimen was inverted (steel on top) for testing to simplify the marking of the tension cracks with the loading arrangement shown in Figure 4.1 . Loading was applied to the overhanging part of the slab resulting in a region of constant moment from the applied loading. The 12 in splice was located in the 53 in long constant moment region at 2h from the support as shown in Figure 4.1 . The splice for specimen 3B-10D12N15 was placed 19 in from each support in order to center the 15 in splice.

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Figure 4.1 Schematic of loading

4.2 Test 1A-8D10L12

This 8 in deep test specimen was unloaded twice during testing, and then loaded up to failure, as shown in the load-deflection plot in Figure 4.2. The pauses to mark cracks resulted in slight unloading which was recovered when loading resumed. The load, P, is the total load applied to each end of the specimen. The test data for load, midspan and quarter point deflections are presented in Appendix-A, Table A-1.

A total of six strain gauges were mounted on bars of the reinforcement fabric. The gauges are designated: NL1, NL2, NL3, SU1, SU2. SU3. The locations of these strain gauges are shown in Figure 4.3. Figure 4.4 shows the load-strain curve for strain gauge SU1. The strain data for this strain gauge is presented in Appendix-B, Table B-1.

Crack patterns and average crack widths are shown in Figures 4.5 and 4.6 for steel stresses of 49.8 ksi (337 N/mm2) and 63.5 ksi (438 N/mm2), respectively. These steel stresses correspond to the average of strain gauges SU1, SU2 and SU3. The crack width values are the average values of the three measurements taken from each crack. Figure 4.7 shows the distribution of crack sizes for the specimen. A total of 12 cracks formed throughout the test.

The 12 in splice length enabled the 10 mm bars to fully develop their ultimate strength. Failure of the slab was due to fracture of the bars immediately south of the splice region. The maximum load achieved was 21.2 kips (94.3 kN) and the maximum deflection was 1.15 in (2.92 cm). The lap splice performed well, without any observed slippage. This was verified by the lack of any sudden decreases in strain due to decreases in load, as seen in Figure 4.4 . Test data for this and other specimens are presented in Table 4.1.

Figures 4.8, 4.9 and 4.10 are photographs of this specimen taken before the test, during the test and after failure occurred respectively.

Figure 4.2 Load-midspan deflection for specimen 1A-8D10L12

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Figure 4.3 Strain gauge locations for specimen 1A-8D10L12

Figure 4.4 Load-strain curve for strain gauge SU1, specimen 1A-8D10L12

Figure 4.5 Average crack widths at steel stress of 49.8 ksi (337 N/mm2) for specimen 1A-8D10L12

Figure 4.6 Average crack widths at steel stress of 63.5 ksi (438 N/mm2) for specimen 1A-8D10L12

Figure 4.7 Crack width-% of smaller cracks of specimen 1A-8D10L12

Figure 4.8 Photograph of specimen 1A-8D10L12 before testing

Figure 4.9 Photograph of specimen 1A-8D10L12 during test

Figure 4.10 Photograph of specimen 1A-8D10L12 after failure

4.3 Test 1B-8D10L12 (Companion to specimen 1A with same concrete mix)

This 8 in deep test specimen was unloaded three times during testing, and then loaded up to failure, as shown in the load-deflection plot in Figure 4.11. The test data for load, midspan and quarter point deflections are presented in Appendix-A, Table A-2.

Strain gauge data is not presented for this test specimen. Problems during removal of the forms led to the fracture of all but one lead wire. The remaining gauge did provide some strain data, but a load-strain plot of this data was erratic. Similar results to those in test 1A-8D10L12 are expected.

Crack patterns and average crack widths are shown in Figures 4.12 and 4.13 for steel stresses of 40.5 ksi (279 N/mm2) and 46.5 ksi (321 N/mm2), respectively. These steel stresses correspond to strain gauge SU2 readings. The crack width values are the average values of the three measurements taken from each crack. A total of 12 cracks formed throughout the test. Figure 4.14 shows the distribution of crack sizes for the specimen.

The 12 in splice length enabled the 10 mm bars to fully develop their ultimate strength. Failure of the slab was due to fracture of the bars immediately south of the splice region. The maximum load achieved was 21.4 kips (95.2 kN) and the maximum deflection was 1.19 in (3.02 cm). The lap splice performed well, without any observed slippage. Test data for this and other specimens are presented in Table 4.1.

Figures 4.15, 4.16 and 4.17 are photographs of this specimen taken before the test, during the test and after failure occurred respectively.

Figure 4.11 Load-midspan deflection for specimen 1B-8D10L12

Figure 4.12 Average crack widths at steel stress of 40.5 ksi (279 N/mm2) for specimen 1B-8D10L12

Figure 4.13 Average crack widths at steel stress of 46.5 ksi (321 N/mm2) for specimen 1B-8D10L12

Figure 4.14 Crack width-% of smaller cracks of specimen 1B-8D10L12

Figure 4.15 Photograph of specimen 1B-8D10L12 before testing

Figure 4.16 Photograph of specimen 1B-8D10L12 during test

Figure 4.17 Photograph of specimen 1B-8D10L12 after failure

4.4 Test 2A-9D11L12

This 9 in deep test specimen was loaded continuously up to failure, as shown in the load-deflection plot in Figure 4.18. The test data for load, midspan and quarter point deflections are presented in Appendix-A, Table A-3.

A total of six strain gauges were mounted on bars of the reinforcement fabric. The gauges are designated: NL1, NL2, NL3, SU1, SU2. SU3. The locations of these strain gauges are shown in Figure 4.19. Figure 4.20 shows the load-strain curve for strain gauge SU1. The strain data for this strain gauge is presented in Appendix-B, Table B-2.

Crack patterns and average crack widths are shown in Figures 4.21 and 4.22 for steel stresses of 43.5 ksi (300 N/mm2) and 50.0 ksi (345 N/mm2), respectively. These steel stresses correspond to the average of strain gauges SU1, SU2 and SU3. The crack width values are the average values of the three measurements taken from each crack. Figure 4.23 shows the distribution of crack sizes for the specimen. A total of 11 cracks formed throughout the test.

The 12 in splice length enabled the 11 mm bars to nearly reach their yield strength, but not ultimate. Failure was sudden with horizontal splitting over the 12 in splice length, but flexural cracking up to failure was quite similar to that of tests 1A and 1B. It should be noted that the specimen did not meet ACI 318R-89 lap splice requirements for deformed WWF. The splice length was inadequate for the specified yield strength of the steel bars, and was therefore inadequate for the actual steel strength, which was higher. It was therefore expected that the splice would fail. The transverse bars in the splice region and the looped WWF did not provide enough mechanical anchorage for the bars to yield. The maximum load achieved was 28.5 kips (126.8 kN) and the maximum deflection was 0.61 in (1.55 cm.). Test data for this and other specimens are presented in Table 4.1.

Figures 4.24, 4.25 and 4.26 are photographs of this specimen taken during the test, after failure occurred, and of the failed splice region, respectively.

Figure 4.18 Load-midspan deflection for specimen 2A-9D11L12

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Figure 4.19 Strain gauge locations for specimen 2A-9D11L12

Figure 4.20 Load-strain curve for strain gauge SU1, specimen 2A-9D11L12

Figure 4.21 Average crack widths at steel stress of 43.5 ksi (300 N/mm2)

for specimen 2A-9D11L12

Figure 4.22 Average crack widths at steel stress of 50.0 ksi (345 N/mm2)

for specimen 2A-9D11L12

Figure 4.23 Crack width-% of smaller cracks of specimen 2A-9D11L12

Figure 4.24 Photograph of specimen 2A-9D11L12 during testing

Figure 4.25 Photograph of specimen 2A-9D11L12 after failure

Figure 4.26 Photograph of the failed splice of 2A-9D11L12

4.5 Test 2B-9D11L12 (Companion specimen to 2A with same concrete mix)

This 9 in deep test specimen was loaded continuously up to failure, as shown in the load-deflection plot in Figure 4.27. The test data for load, midspan and quarter point deflections are presented in Appendix-A, Table A-4.

A total of six strain gauges were mounted on bars of the reinforcement fabric. The gauges are designated: NL1, NL2, NL3, SU1, SU2. SU3. The locations of these strain gauges are shown in Figure 4.28. Figure 4.29 shows the load-strain curve for strain gauge SU2. The strain data for this strain gauge is presented in Appendix-B, Table B-3.

Crack patterns and average crack widths are shown in Figures 4.30 and 4.31 for steel stresses of 41.3 ksi (285 N/mm2) and 47.8 ksi (330 N/mm2), respectively. These steel stresses correspond to the average of strain gauges SU1, SU2 and SU3. The crack width values are the average values of the three measurements taken from each crack. Figure 4.32 shows the distribution of crack sizes for the specimen. A total of 10 cracks formed throughout the test.

The 12 in splice length enabled the 11 mm bars to nearly reach their yield strength, but not ultimate. Failure was sudden with horizontal splitting over the 12 in splice length, but flexural cracking up to failure was quite similar to that of tests 1A and 1B. Again, the splice length for this specimen was less than the ACI code requirements, resulting in a splice failure. The maximum load achieved was 29.3 kips (130.3 kN) and the maximum deflection was 0.65 in (1.65 cm.). Test data for this and other specimens are presented in Table 4.1.

Figures 4.33 and 4.34 are photographs of this specimen taken during the test and after failure occurred, respectively.

Figure 4.27 Load-midspan deflection for specimen 2B-9D11L12

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Figure 4.28 Strain gauge locations for specimen 2B-9D11L12

Figure 4.29 Load-strain curve for strain gauge SU2, specimen 2B-9D11L12

Figure 4.30 Average crack widths at steel stress of 41.3 ksi (285 N/mm2) for specimen 2B-9D11L12

Figure 4.31 Average crack widths at steel stress of 47.8 ksi (330 N/mm2) for specimen 2B-9D11L12

Figure 4.32 Crack width-% of smaller cracks of specimen 2B-9D11L12

Figure 4.33 Photograph of specimen 2B-9D11L12 during test

Figure 4.34 Photograph of specimen 2B-9D11L12 after failure

4.6 Test 3A-10D12L12

This 10 in deep test specimen was loaded continuously up to failure, as shown in the load-deflection plot in Figure 4.35. The test data for load, midspan and quarter point deflections are presented in Appendix-A, Table A-5.

A total of six strain gauges were mounted on bars of the reinforcement fabric. Figure 4.36 shows the load-strain curve for strain gauge SU2. The strain data for this strain gauge is presented in Appendix-B, Table B-4.

Crack patterns and average crack widths are shown in Figures 4.37 and 4.38 for steel stresses of 42.5 ksi (293 N/mm2) and 54.1 ksi (373 N/mm2), respectively. These steel stresses correspond to the average of strain gauges SU1, SU2 and SU3. The crack width values are the average values of the three measurements taken from each crack. Figure 4.39 shows the distribution of crack sizes for the specimen. A total of 13 cracks formed throughout the test.

The 12 in splice length did not enable the 12 mm bars to reach their yield strength. Failure was sudden with horizontal splitting over the 12 in splice length, but flexural cracking up to failure was quite similar to that of tests 1A and 1B. The splice length for this specimen did not satisfy the ACI code requirements for deformed WWF, resulting in an expected splice failure. The maximum load achieved was 31.1 kips (138.3 kN) and the maximum deflection was 0.44 in (1.12 cm.). Test data for this and other specimens are presented in Table 4.1.

Figures 4.40 and 4.41 are photographs of this specimen taken during the test and after failure occurred, respectively.

Figure 4.35 Load-midspan deflection for specimen 3A-10D12L12

Figure 4.36 Load-strain curve for strain gauge SU2, specimen 3A-10D12L12

Figure 4.37 Average crack widths at steel stress of 42.5 ksi (293 N/mm2) for specimen 3A-10D12L12

Figure 4.38 Average crack widths at steel stress of 54.1 ksi (373 N/mm2)

for specimen 3A-10D12L12

Figure 4.39 Crack width-% of smaller cracks of specimen 3A-10D12L12

Figure 4.40 Photograph of specimen 3A-10D12L12 during test

Figure 4.41 Photograph of specimen 3A-10D12L12 after failure

4.7 Test 3B-10D12N15 (Companion specimen to 3A with same concrete mix)

This 10 in deep test specimen was loaded continuously up to failure, as shown in the load-deflection plot in Figure 4.42. The test data for load, midspan and quarter point deflections are presented in Appendix-A, Table A-6.

A total of six strain gauges were mounted on bars of the reinforcement fabric. Figure 4.43 shows the load-strain curve for strain gauge SU2. The strain data for this strain gauge is presented in Appendix-B, Table B-5.

Crack patterns and average crack widths are shown in Figures 4.44 and 4.45 for steel stresses of 39.9 ksi (275 N/mm2) and 51.1 ksi (352 N/mm2), respectively. These steel stresses correspond to the average of strain gauges SU1, SU2 and SU3. The crack width values are the average values of the three measurements taken from each crack. Figure 4.46 shows the distribution of crack sizes for the specimen. A total of 10 cracks formed throughout the test.

The 15 in splice length enabled the 12 mm bars to reach their yield strength, but not ultimate. Failure was sudden with horizontal splitting over the 15 in splice length, but flexural cracking up to failure was similar to that of tests 1A and 1B. The splice length of 15 in did not satisfy the ACI code requirements for deformed WWF, and therefore a splice failure was expected. As seen in the load-deflection plot, though, the specimen exhibited some deflection after the peak load was reached. The maximum load achieved was 35.3 kips (157.0 kN) and the maximum deflection was 0.60 in (1.52 cm.). Test data for this and other specimens are presented in Table 4.1.

Figures 4.47 and 4.48 are photographs of this specimen taken during the test and the failed splice region, respectively.

Figure 4.42 Load-midspan deflection for specimen 3B-10D12N15

Figure 4.43 Load-strain curve for strain gauge SU2, specimen 3B-10D12N15

Figure 4.44 Average crack widths at steel stress of 39.9 ksi (275 N/mm2) for specimen 3B-10D12N15

Figure 4.45 Average crack widths at steel stress of 51.1 ksi (352 N/mm2) for specimen 3B-10D12N15

Figure 4.46 Crack width-% of smaller cracks of specimen 3B-10D12N15

Figure 4.47 Photograph of specimen 3B-10D12N15 during test

Figure 4.48 Photograph of specimen 3B-10D12N15 after failure

4.8 Test 4A-9D11L12 (Companion to specimens 2A/2B with revised concrete mix having higher f'c)

This 9 in deep test specimen was loaded continuously up to failure, as shown in the load-deflection plot in Figure 4.49. The test data for load, midspan and quarter point deflections are presented in Appendix-A, Table A-7.

A total of six strain gauges were mounted on bars of the reinforcement fabric. Figure 4.50 shows the load-strain curve for strain gauge SU1. The strain data for this strain gauge is presented in Appendix-B, Table B-6.

Crack patterns and average crack widths are shown in Figures 4.51 and 4.52 for steel stresses of 42.5 ksi (293 N/mm2) and 52.3 ksi (361 N/mm2), respectively. These steel stresses correspond to the average of strain gauges SU1, SU2 and SU3. The crack width values are the average values of the three measurements taken from each crack. Figure 4.53 shows the distribution of crack sizes for the specimen. A total of 13 cracks formed throughout the test.

The 12 in splice length enabled the 12 mm bars to reach their yield strength, but not ultimate. Failure was sudden with horizontal splitting over the 12 in splice length, but flexural cracking up to failure was quite similar to that of tests 1A and 1B. One of the outside reinforcing bars in the looped mesh did fracture, but it is not certain whether the splice failure contributed to the bar fracture. Fracture of a bar (Figure 4.56) clearly indicated that the steel was well above the initial yield strength. As seen in the plot of load versus deflection, the specimen exhibited some deflection after the peak load was reached. The splice length of the specimen, though, did not meet the ACI code requirements for deformed WWF, and thus a splice failure was expected. The maximum load achieved was 32.9 kips (146.3 kN) and the maximum deflection was 0.84 in (2.13 cm.). Test data for this and other specimens are presented in Table 4.1.

Figures 4.54, 4.55 and 4.56 are photographs of this specimen taken during the test, after failure occurred and of the fractured bar in the splice region, respectively.

Figure 4.49 Load-midspan deflection for specimen 4A-9D11L12

Figure 4.50 Load-strain curve for strain gauge SU1, specimen 4A-9D11L12

Figure 4.51 Average crack widths at steel stress of 42.5 ksi (293 N/mm2) for specimen 4A-9D11L12

Figure 4.52 Average crack widths at steel stress of 52.3 ksi (361 N/mm2)

for specimen 4A-9D11L12

Figure 4.53 Crack width-% of smaller cracks of specimen 4A-9D11L12

Figure 4.54 Photograph of specimen 4A-9D11L12 during test

Figure 4.55 Photograph of specimen 4A-9D11L12 after failure

Figure 4.56 Photograph of fractured bar in splice of specimen 4A-9D11L12

4.9 Test 4B-10D12L12 (Companion to specimen 3A with revised concrete mix having higher f'c)

This 10 in deep test specimen was loaded continuously up to failure, as shown in the load-deflection plot in Figure 4.57. The test data for load, midspan and quarter point deflections are presented in Appendix-A, Table A-8.

A total of six strain gauges were mounted on bars of the reinforcement fabric. Figure 4.58 shows the load-strain curve for strain gauge SU2. The strain data for this strain gauge is presented in Appendix-B, Table B-7.

Crack patterns and average crack widths are shown in Figures 4.59 and 4.60 for steel stresses of 43.7 ksi (301 N/mm2) and 54.4 ksi (375 N/mm2) respectively. These steel stresses correspond to the average of strain gauges SU1, SU2 and SU3. The crack width values are the average values of the three measurements taken from each crack. Figure 4.61 shows the distribution of crack sizes for the specimen. A total of 13 cracks formed throughout the test.

The 12 in splice length enabled the 12 mm bars to reach their yield strength, but not ultimate. The increased concrete strength of f 'c = 5700 psi in this specimen, as compared to f 'c = 3500 psi in the companion specimens, did in improve the load and deformation capacity of the specimen, but did not enable fracture of the bars. Failure was sudden with horizontal splitting over the 12 in splice length, but flexural cracking up to failure was similar to that of tests 1A and 1B. The 12 in lap splice length did not satisfy the minimum ACI code requirements for deformed WWF, resulting in an expected splice failure. The maximum load achieved was 36.6 kips (162.8 kN) and the maximum deflection was 0.69 in (1.75 cm.). Test data for this and other specimens are presented in Table 4.1.

Figures 4.62 and 4.63 are photographs of this specimen taken during the test and after failure occurred, respectively.

Figure 4.57 Load-midspan deflection for specimen 4B-10D12L12

Figure 4.58 Load-strain curve for strain gauge SU2, specimen 4B-10D12L12

Figure 4.59 Average crack widths at steel stress of 43.7 ksi (301 N/mm2) for specimen 4B-10D12L12

Figure 4.60 Average crack widths at steel stress of 54.4 ksi (375 N/mm2)

for specimen 4B-10D12L12

Figure 4.61 Crack width - % of smaller cracks of specimen 4B-10D12L12

Figure 4.62 Photograph of specimen 4B-10D12L12 during test

Figure 4.63 Photograph of specimen 4B-10D12L12 after failure

Table 4.1 Test data summary

Specimen

Cracking Load

kips

Peak Load

Failure Load

Failure TypeLoad

kipsDeflection

inLoad

kipsDeflection

in1A921.20.8419.11.15Bar1B721.40.95201.19Bar2A928.50.6128.50.61Splice2B9.329.30.6529.30.65Splice3A11.531.10.4431.10.44Splice3B13.035.30.5532.30.60Splice4A11.432.30.8432.90.84Splice *4B11.036.60.6936.60.69Splice   * one bar fracture noted

4.10 Measured vs. Predicted Flexural Strength

The measured flexural strength and the predicted strength of the specimens are compared in terms of moments. Predicted ultimate moments were calculated using an equivalent rectangular compressive stress block, the actual yield strength of bars based on 0.2% extension under load, and the actual compressive strength of concrete at the day of testing. The ultimate strength of bars was used for those that fractured, and the actual bar stress was used for specimen 3A, since the bars did not yield.

The predicted values for the ultimate moments of all specimens were conservative, with the total measured moment exceeding the predicted values. The degree by which the total measured moment exceeded the predicted moment ranged from 5% for specimen 3B-10D12N15 up to 25% for specimen 4A-9D11L12. The results are shown in Table 4.2.

Table 4.2 Measured vs. predicted flexural strength of specimens

Specimen	Measured Flexural Strength	Moment due to Self Weight and Test Frame	Total Moment	Predicted Ultimate Moment
	Kip-in / kN-m	Kip-in / kN-m	Kip-in / kN-m	Kip-in / kN-m
1A-8D10L12	540.6/61.1	13.2/1.49	553.8/62.6	475.2/53.7
1B-6D10L12	545.7/61.7	13.2/1.49	558.9/63.1	475.2/53.7
2A-9D11L12	726.8/82.1	14.85/1.67	741.6/83.8	668.6/75.5
2B-9D11L12	747.2/84.4	14.85/1.67	762.0/86.1	670.4/75.7
3A-10D12L12	793.1/89.6	16.5/1.86	809.6/91.5	706.9/79.9
3B-10D12N15	900.2/101.7	16.5/1.86	916.7/103.6	870.3/98.3
4A-9D11L12	839.0/94.8	14.85/1.67	855.5/96.7	684.6/77.3
4B-10D12L12	933.3/105.4	16.5/1.86	949.8/107.3	888.0/100.3

CHAPTER 5

DISCUSSION

5.1 Strength of Lap Splices

The failure mechanism of specimens 1A-8D10L12 and 1B-8D10L12 was fracture of the bars outside the splice region. The performance of the first two specimens were in agreement with similar tests done on 10 mm deformed bars using WWFL, providing further proof of the effectiveness of the looped mesh [5]. The remaining specimens all failed due to concrete splitting in the plane of the splice prior to bar fracture. Table 5.1 compares the ACI 318R-89 required splice lengths for deformed WWF with actual splice lengths used. The table also shows that splice length requirements increase when the actual steel stresses at failure are used. The only splices that satisfy the code requirements are those of specimens 1A and 1B. The 12 in splice lengths that were provided in these specimens actually exceeded the code requirements, meaning that the splices were more than adequate to yield the bars. The additional anchorage from the looped ends and multiple transverse bars in the splice region added to the splice performance, enabling the splice to perform after the reinforcement had yielded. The remaining specimens required splice lengths from between 2.2 in to 11.5 in greater than what was provided. There is strong evidence that the inclusion of the looped mesh greatly increased the performance of the splices, considering the degree of inadequacy of many specimens when compared to code requirements.

Table 5.1 Actual vs. required splice lengths

Specimenf 'c

(psi)fy

(ksi)ld

(in)fS *

(ksi)ld

(in)Splice length tested

(in)1A/B-8D10L1245008010.695.512.6122A-9D11L1233008014.98014.9122B-9D11L1235008014.58014.5123A-10D12L1235008017.27015.1123B-10D12N1535008017.39821.1154A-9D11L1257008011.39012.8124B-10D12L1257008013.59516.012 * fs = actual steel stress at failure

The comparison between companion specimens 2A-9D11L12, 2B-9D11L12 and 4A-9D11L12, all with 12 in splice lengths, show the difference between increasing f 'c = 3500 psi to 5000 psi. Figure 5.1 compares the load-deflection plots for each specimen, indicating a substantial increase in load and deformation capacity. The maximum strain values attained for each specimen are 2700 micro in/in for 2A (Figure 4.21), 3100 micro in/in for 2B (Figure 4.29), and 3750 micro in/in for 4A (Figure 4.50). The steel stresses for these strains (Figure 3.9) are 80 ksi for specimens 2A and 2B, and 90 ksi for specimen 4A. The increased concrete strength enabled the steel to reach an additional stress of 10 ksi before a splice failure occurred. To further demonstrate the additional capacity of the increased f 'c, one of the longitudinal bars in specimen 4A did fracture, proving the higher stresses of the steel (Figure 4.56). These findings agree with earlier studies showing that the concrete strength is directly related to lap splice strength [7 and 9].

The comparison of companion specimens 3A-10D12L12 and 3B-10D12N15, both with f 'c = 3500 psi, show the difference between a 12 in WWFL

Figure 5.1 Load-Deflection for tests 2A/2B-9D11L12 and 4A-9D11L12

splice length and a 15 in WWF splice length. The load-deflection plots for each specimen (Figures 4.35 and 4.42), show that specimen 3B not only demonstrated a higher load capacity, but a higher deformation capacity as well, reaching a load plateau in the inelastic region. The maximum strain values attained for each specimen, 2700 micro in/in for 3A and 3100 micro in/in for 3B (Figures 4.20 and 4.30), indicate steel stresses of 70 ksi at failure for 3A and 98 ksi at failure for 3B (Figure 3.10). The 25% increase in splice length from 12 in with loops to 15 in without loops resulted in a higher strength and ductility of the lap splice.

Finally, the comparison of companion specimens 3A-10D12L12 and 4B-10D12L12, both with 12 in splice lengths, also show the difference in splice performance due to f 'c = 3500 psi and 5000 psi. Figure 5.2 compares the load-deflection curves for the two specimens, and indicates the additional load and deformation capacity of the higher strength concrete in specimen 4B. The maximum strain values for each specimen are 2700 micro in/in for 3A (Figure 4.20) and 4600 micro in/in for 4B (Figure 4.58) indicating steel stresses of 70 ksi and 95 ksi, respectively (Figure 3.10). This increased steel stress in specimen 4B is similar to that of specimen 3B-10D12N15 with a 15 in splice length. The higher concrete strength produced better load and deformation capacity (Table 4.1) with a 12 in splice (WWFL), than lower strength concrete with a 15 in splice (WWF). This comparison again supports earlier studies suggesting that concrete strength is related to lap splice strength [7 and 9].

5.2 Crack Control

The formation of cracks in every specimen was generally at the location of a transverse bar, which were spaced at 6 in. The average crack spacing in the

Figure 5.2 Load-Deflection for tests 3A-10D12L12 and 4B-10D12L12

slabs was close to the transverse bar spacing of 6 in. Each slab distributed cracks similarly, with differences only arising due to slab thickness and bar size. Since the concrete cover was held constant at 0.75 in, specimens with larger bar diameters exhibited larger cracks. The maximum crack width recorded was 0.021 in for specimen 3A-10D12L12 at a steel stress of 54 ksi.

One observation that should be noted from the average crack width drawings for tests 1A, 1B, 4A and 4B were the closely spaced pairs of cracks at the ends of the splice region directly above the transverse wires in the splice of the looped mesh. The inclusion of the extra transverse bars may result in additional splitting cracks at the ends of the splices, where stress concentrations occur. Since the test specimens all had similar arrangements of deformed WWFL, the data obtained is limited for further crack control studies.

5.3 Ductility

Ductile behavior in reinforced concrete slabs can occur when the reinforcement ratio, (, is between the code minimum, and that of balanced conditions. This corresponds with deformation of the reinforcement before the concrete crushes. The two initial slabs, 1A/B-8D10L12, showed little ductility by fracture of the steel bars outside of the splice region, although some deformation did occur in the steel. The other slabs also failed suddenly, in a non-ductile manner by splitting of the concrete in the splice region. The plots of load versus midspan deflection (Figures 4.2, 4.11, 4.18, 4.27, 4.35, 4.42, 4.49, 4.57) better reflect the level of ductility in individual specimens. Most of the tests resulted in very low displacement ductility ratios, the largest being just over two. While this number may be low, WWF is generally used as slab reinforcement or shrinkage and temperature steel, and not as primary reinforcement in highly stressed members requiring large deformation capacity.

The behavior of the steel reinforcement that is used in WWF is less ductile than regular reinforcement. The manufacturing process that creates the individual bars, in combination with the welding process, produces high yield strengths while sacrificing ductility. The stress-strain curves of the steel reinforcement (Figures 3.8, 3.9 and 3.10) indicate the lack of a defined yielding point and limited inelastic deformation as compared to regular reinforcement with lower yield strengths. Upon yielding of the steel, less deformation than regular reinforcement will occur before fracture.

The standard design practice for assuring ductility is proportioning of the steel to allow the reinforcement to yield and deform substantially before failure. Improving the ductility for slabs that failed due to fracture of the reinforcement could be accomplished by relocating splices away from high moment regions, or by using more ductile reinforcement. The ductility of the specimens that failed from concrete splitting in the splice region can improve with longer lap splice lengths, locating the splices in regions of lower moment, or by using more ductile reinforcement.

CHAPTER 6

SUMMARY AND CONCLUSIONS

The focus of this research program included seven specimens reinforced with Welded Wire Fabric having looped ends (WWFL), 1A-8D10L12, 1B-8D10L12, 2A-9D11L12, 2B-9D11L12, 3A-10D12L12, 4A-9D11L12 and 4B-10D12L12, and one specimen reinforced with Welded Wire Fabric without loops, (WWF), 3B-10D12L15. Specimens 1A-8D10L12 and 1B-8D10L12 failed by fracture of the bars just south of the splice region where the distance from the extreme compression fiber and reinforcing steel was smallest. The remainder of the slabs failed from the splitting of concrete in the splice region. The splice length was sufficient to yield the bars for all of the specimens except 3A-10D12L12, where the splice length was inadequate. The increased concrete strength for specimens 4A-9D11L12 and 4B-10D12L12 improved the load-deflection responses of the corresponding companion specimens, (2A/2B-9D11L12 for specimen 4A, and 3A-10D12L12 for specimen 4B). The bars in specimen 4B-10D12L12 did yield, unlike those of the companion specimen 3A-10D12L12, demonstrating the increased capacity of the splice with a concrete mix having a higher f 'c.

The lap splices in the specimens performed well considering all but the first two specimens failed to meet the ACI 318R-89 code requirements for deformed WWF. The specimens also had reinforcement ratios near the minimum allowable by the ACI Code.

The results of this test program are similar to those performed in Switzerland by EMPA [4], and previous tests done at the University of Texas at Austin, USA [5]. The overall behavior and types of failure for all test programs produced similar results for comparable specimens.

The predicted ultimate moment capacities for all specimens were conservative to the measured test values. In all cases, the test values were greater than those predicted. The total moment calculated from the tests exceeded the predicted ultimate moment by 5% for specimen 3B-10D12N15 and up to 25% for specimen 4A-9D11L12. The predicted values utilized actual steel stresses as accurately as possible, taking into account yielding and fracture conditions.

Based on these tests of WWFL, it can be concluded that:

1. With concrete strength f 'c = 3500 psi, Welded Wire Fabric having looped ends (WWFL), and concrete cover of 0.75 in, it was possible to fully develop the ultimate strength of the 10 mm deformed steel bars with a 12 in lapped splice. Specimens 1A and 1B with 10 mm diameter deformed steel bars both failed in this manner even considering the actual steel stress was approximately 90 ksi at failure in flexure, without distress in the 12 in lapped splice.
2. With a concrete strength f 'c = 3500 psi, WWFL with a 12 in lapped splice, and 0.75 in cover for 11 mm and 12 mm bars, a splice failure occurred before fully developing the actual yield strength of the steel. The assumed steel yield stress of 80 ksi maximum was developed for the 11 mm bars, but since the steel had an actual yield strength of 90-100 ksi, failure occurred in the lap splice.
3. With concrete strength f 'c = 5000 psi, WWFL with a 12 in lap splice for 11 mm and 12 mm bars, and 0.75 in cover, failure occurred in the splice after developing the

actual yield strength of the steel. Failure only occurred in the splice due to the inability of the lap splice to develop the ultimate strength of the steel.

4. With concrete strength f 'c = 3500 psi, WWF with a 15 in lap splice for 12 mm bars, and 0.75 in cover, failure occurred in the splice after nearly developing the actual yield strength of the steel. Failure occurred in the splice due to the inability of the lap splice to develop the ultimate strength of the steel.

APPENDIX - A

LOAD DEFLECTION DATA

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APPENDIX - B

STRAIN GAUGE TEST DATA

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