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EFFECTS OF EPOXY COATING ON ANCHORAGE AND SPLICES OF WELDED WIRE FABRIC

By

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Report on a Research Project
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ABSTRACT

The effects of epoxy-coating on anchorage and splices of WWF were investigated. The program included weld shear tests, pull-out tests of bars with and without cross wires, and slab tests with splices in the welded wire fabric. Comparison slabs containing spliced fabric were cast with no coating on the welded wire fabric for comparison with coated fabric.

The performance of comparison specimens identical in all respects, except for the epoxy coating on one and uncoated wire fabric in the other, indicated that the coating had very little or no influence on strength, stiffness and cracking patterns.

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CHAPTER 1

INTRODUCTION

1.1 General

Welded wire fabric (WWF) has been in use in the United States since the turn of the century. Patent papers for an electric weld apparatus used to produce WWF were filed by John C. Perry from Clinton, Massachusetts in March of 1901. Since that time, the availability and use of WWF has increased greatly.(1)

WWF can be defined as a prefabricated reinforcement consisting of parallel series of cold-drawn or cold-rolled wires welded together in a rectangular grid. The intersecting longitudinal and transverse wires are electrically resistance welded by a continuous automatic welder. The welding process uses a combination of electric current and pressure with no additional metal being added. WWF may consist of plain wires, deformed wires, or combinations of both.(2) Until recently epoxy-coated WWF was not used, but with the continuing concern about corrosion of reinforcing steel in concrete, coated WWF is now being used. The epoxy-coating is typically applied electrostatically to the required thickness and acts as a barrier between the corrosive environment and the steel.

Wires used in WWF are designated with a letter-number combination. The letter "W" or "D" is used in combination with a number designating the cross-sectional area of the wire in hundredths of a square inch. The letter "W" is used to designate a plain wire and the letter "D" is used to designate a deformed wire. WWF is denoted in the same manner, but a pair of numbers used to define the longitudinal and transverse wire spacing is placed before the wire designations. A typical WWF designation would be:

4 x 16 - D20 x D10

where:
- Spacing of longitudinal wires is 4 in.
- Spacing of transverse wires is 16 in.
- Size of longitudinal wires is D20 (0.20 in^2 and deformed)
- Size of transverse wires is D10 (0.10 in^2 and deformed)

WWF is used in many different applications. Some of these applications include continuous one-way slabs, two-way slabs, one-way corridor-type structures, temperature and shrinkage reinforcement, and transverse reinforcement for concrete columns or beams. From these possible applications it is apparent that WWF is a very versatile material that offers certain advantages over conventional reinforcement.(1) Some of these advantages are listed below:

1. Uniform stress distribution - WWF provides a more uniform stress distribution through the use of smaller and closer spaced wires.

2. Crack control - Improved control of crack size and spacing is a result of a more uniform stress distribution and a larger surface bond area.

3. Positive anchorage - The welded cross wires in WWF provide a means of positive mechanical anchorage at each cross wire location.

4. Quick and economical placement - The prefabricated sheets of WWF allow for a quick and easy method of placing reinforcement.

1.2 ASTM and ACI Requirements For Material Properties

ACI 318-89(3) stipulates that certain ASTM standards be met in the manufacture of wire reinforcement and WWF. The physical properties of the wire reinforcement and WWF as stipulated in these ASTM standards are outlined in Table 1.1. The ASTM designation is also listed in Table 1.1.

Table 1.1: Minimum Physical Properties For Wire Reinforcement and WWF

	Yield Strength (psi)	Tensile Strength (psi)	Weld Shear Strength / Cross Section Area (psi)
Plain wire reinforcement [A 82][4]	70,000	80,000	
Deformed wire reinforcement [A 496][5]	75,000	85,000	
Welded plain wire fabric [A 185][6]	65,000	75,000	35,000
Welded deformed wire fabric [A 497][7]	70,000	80,000	35,000

According to ASTM the yield strength is determined at a strain of 0.005 over a recommended gage length of 8 in. A different criterion is specified by ACI 318-89. ACI requires that if a yield strength higher than 60,000 psi is to be used in design, then the yield strength shall be measured at a strain of 0.0035.

In addition to the material properties for the wire reinforcement and WWF, ASTM A 884[8] also stipulates the allowable thickness of epoxy coating on wire reinforcement and WWF. A minimum of 7 mils and a maximum of 12 mils is allowed for reinforcement to be used in concrete.

1.3 ACI Splice and Development Requirements

ACI 318-89 outlines a method for determining the splice and development lengths of both plain and deformed WWF. These requirements are summarized in the following sections.

1.3.1 Development of Welded Deformed Wire Fabric in Tension

In Section 12.7 of ACI 318-89 the requirements for the development of welded deformed wire fabric in tension are specified. Section 12.7.3 states that the development length (l_d), with no cross wires in the development length, shall be determined as for deformed wire. The development length, with cross wires present in the development length as shown in Figure 1.1 and defined in Section 12.7.2 , shall be the product of the basic development length (l_{db}) and the modification factors from Sections 12.2.3 through 12.2.5. l_{db} can be determined from the following equation:

$$l_{db} = 0.03 d_b \, (f_y - 20{,}000)/\sqrt{f_c'} \geq 0.20 \frac{A_w}{s_w} \frac{f_y}{\sqrt{f_c'}} \qquad (1)$$

where: d_b = *nominal diameter of wire, in.*
f_y = *specified yield strength of wire, psi*
f_c' = *specified compressive strength of concrete, psi*
A_w = *area of wire being developed, in*^{2}
s_w = *spacing of wire being developed, in.*

The modification factor for epoxy-coated reinforcement is given in Section 12.2.4.3. Section 12.2.4.3 states that a modification factor of 1.5 be used for epoxy-coated bars with a cover less than $3d_b$ or clear spacing between bars less than $6d_b$, and 1.2 for all other conditions with epoxy-coated bars. The values are a function of d_b , the nominal diameter of the bar or wire being developed, and are based on the work of Treece and Jirsa[9] and Hamad, Jirsa, and d'Abreu[10]. The modification factors for epoxy-coating in Section 12.2.4.3 are also applied to epoxy-coated deformed WWF even though all of the research used to develop this section was conducted on reinforcing bars. Prior to this study, very limited testing had been conducted to determine the effects of epoxy-coating on the bond and anchorage of WWF.

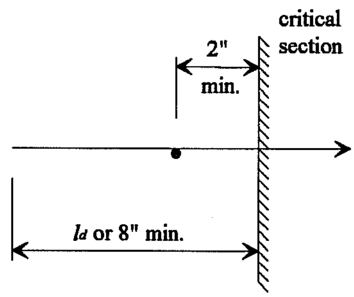


Figure 1.1: Development Requirements for Welded Deformed Wire Fabric

It should be noted that there is a discrepancy in Sections 12.2.3.6 and 12.7.2. Section 12.2.3.6 states that the product of l_{db} and the modification factors shall be greater than or equal to $0.003\, d_b f_y / \sqrt{f_c'}$. If this is the case then no allowance can be made for the mechanical anchorage due to the welded cross wires as indicated in the equation defining l_{db} by subtracting 20,000 from f_y.

1.3.2 Development of Welded Plain Wire Fabric in Tension

The development requirements for welded plain wire fabric in tension are stated in Section 12.8 of ACI 318-89, and are shown in Figure 1.2. Section 12.8 says that yield strength shall be considered developed by embedment of two or more cross wires. In addition, l_{db} measured from the critical section to the outermost cross wire shall be greater than or equal to

$$0.27 \frac{A_w}{s_w} \frac{f_y}{\sqrt{f_c'}} .$$

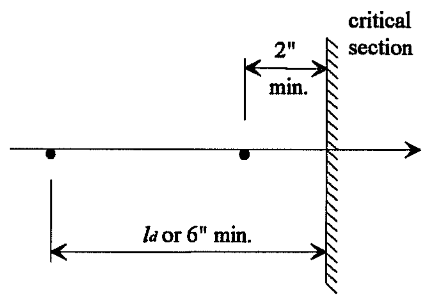


Figure 1.2: Development Requirements for Welded Plain Wire Fabric

1.3.3 Splices of Welded Deformed Wire Fabric in Tension

ACI 318-89 Section 12.18 states the requirements for splices of welded deformed wire fabric in tension. Splices for deformed wire fabric are measured between the ends of the lapped sheets as shown in Figure 1.3. The lap splice, with cross wires present in the lap splice, must be greater than or equal to $1.3l_d$ or 8 in. and there must be 2 in. or more overlap between outermost cross wires. The value of l_d is determined in accordance with Section 12.7. When no cross wires are present in the lap splice, the splice length shall be determined as for deformed wire.

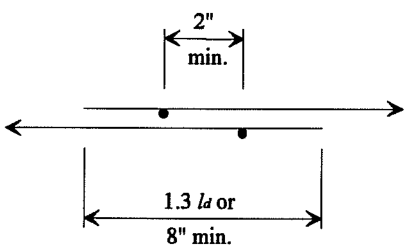
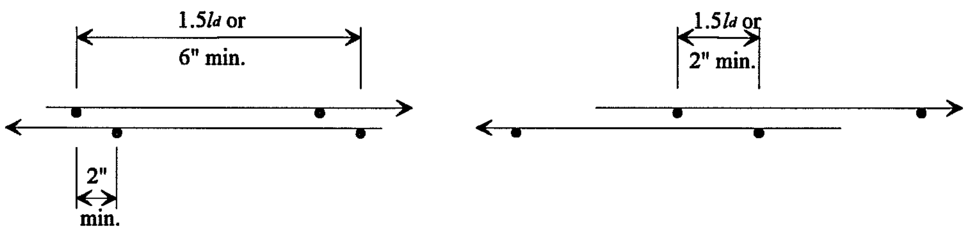


Figure 1.3: Splice Requirements For Welded Deformed Wire Fabric

1.3.4 Splices of Welded Plain Wire Fabric in Tension

Splice requirements for welded plain wire fabric in tension are covered in Section 12.19 of ACI 318-89. Section 12.19 defines two different situations for this splice. First, if the area of reinforcement provided is less than two times the area required then the length of overlap measured between outermost cross wires must be greater than or equal to one spacing of cross wires plus 2 in. or $1.5l_d$ or 6 in. The value of l_d is determined in accordance with Section 12.8. The second situation exists if the area of reinforcement provided is greater than two times the area required, then the length of overlap must be greater than $1.5l_d$ or 2 in. Both of these situations are detailed in Figure 1.4.



Area of Reinforcement Provided Less Than 2 x Area of Reinforcement Required

Area of Reinforcement Provided Greater Than or Equal to 2 x Area of Reinforcement Required

Figure 1.4: Splice Requirements for Welded Deformed Wire Fabric

1.4 Objective

The objective of this thesis was to determine the effects of epoxy-coating on bond and anchorage of WWF. The behavior of lap splices with epoxy-coated and uncoated WWF were studied.

1.5 Scope

The test program included weld shear tests, pull-out tests, and slab tests with splices of WWF. Weld shear tests and pull-out tests were conducted to evaluate the anchorage provided by bond along the wire and by the weld of the transverse wire. Slab tests were conducted using two different types of WWF in three different sizes. Size 4 longitudinal with size 4 transverse wires and size 20 longitudinal with size 10 or 11 transverse wires were used with plain wires and indented deformed wires. All slabs were built in pairs, one with epoxy-coated WWF and the other with uncoated WWF. The slabs were loaded until the splice failed or the wires yielded. Load, deflection, and crack widths were measured.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Considerable research has been done on the use of wire and WWF as a reinforcement since its development in the early 1900's. Some of the major areas of concern have been with splices, development and anchorage, and flexural cracking of members reinforced with wire and WWF. The following sections address and summarize some of the pertinent research that has been conducted in each of these areas.

2.2 Splices of Welded Wire Fabric

2.2.1 Elstner, 1962[11]

A study was conducted by Wiss, Janney, and Associates in 1962 to determine the minimum splice lengths required for welded plain wire fabric and deformed bars to develop full yield strength in continuously reinforced concrete pavement. Twenty-one specimens were tested. All of the specimens constructed with WWF were 8 in. deep, 20 ft. long, and 12⅛ in. wide. The specimens were tested in axial tension and were continuously supported on 1 in. of compacted moist sand.

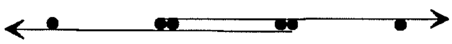
All of the 12 specimens constructed with WWF had 5/0 gauge (0.14556 in^2) longitudinal wires spaced at 3 in. and No. 1 gauge (0.062902 in^2) transverse wires spaced at 12 in. The longitudinal wires extended 6 in. beyond the end transverse wires in each sheet of WWF. Three different splice lengths, measured from the ends of each sheet of WWF, were tested. These splice lengths were 14 in., 18 in., and 25 in.

From this series of tests, Elstner found that the full yield strength of the longitudinal wires could not be developed with a 14 in. or 18 in. lap splice which contained a lap of only one pair of transverse wires. The only specimens using WWF that were able to develop full yield strength had a 25 in. lap splice which contained two pairs of transverse wires. The transverse wires were separated by 1 in. ACI 318-89 requires a minimum splice that includes two pairs of transverse wires each separated by 2 in. The minimum allowable splice according to ACI 318-89 for this type of WWF would have been 26 in.

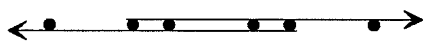
2.2.2 Atlas, Siess, Bianchini, and Kesler, 1964[(12)]

A series of slab tests to study the strength of splices with WWF were conducted at the University of Illinois from 1959 to 1962. All of these slabs were 24 in. wide, 5 in. thick, and 6 ft. long. Welded plain wire fabric was used and the splices consisted of nested laps. As shown in Figure 2.1, two different types of nested laps were used; one with the transverse wires from the two sheets of WWF in contact, and one with the transverse wires separated by concrete. Transverse wire size, transverse wire spacing, and the amount of overlap were also varied. The WWF used in this investigation had longitudinal wires in the splice region with no overhang beyond the end transverse wire.

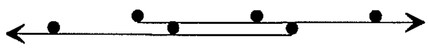
It was found by Atlas, et. al., that when the number and size of overlapped transverse wires were the same, the specimens constructed with splices in which the transverse wires were separated by concrete were more effective. Within the boundaries of the sizes tested, splice effectiveness tended to increase with an increase in the transverse wire size. Spacing of the transverse wires alone did not appear to affect the splice effectiveness unless splice lengths were also increased as a result of increased transverse wire spacing, in which case, splice effectiveness increased. The controlling factor for the transverse wires did not seem to be the size or spacing, but the number of transverse wires included in the splice region.



Nested Lap With Transverse Wires
in Contact



Nested Lap With Trasverse Wires
Separated by Concrete



Standard Lap (Lap Not Nested)

Figure 2.1: Nested Laps

Atlas, et. al., concluded that ultimate flexural capacity of the slabs could be obtained in splices containing at least two overlapped transverse wires with the transverse wires separated by concrete. It was also observed that splices with one set of transverse wires overlapped and separated by concrete could develop at least 40% of the calculated ultimate moment. These conclusions are reflected in the minimum requirements for the splices of welded plain wire fabric allowed by ACI 318-89 Section 12.19.

2.2.3 Lloyd, 1971[13]

Lloyd conducted a study of 36 one-way flexural slabs reinforced with welded plain wire fabric to evaluate the influence of lap length, overhang length beyond last transverse wire, cover, concrete strength, reinforcement ratio, reinforcement style, and nesting on the strength of lap splices of WWF. The only sizes of longitudinal wires considered were W40 and W20. All of the slabs failed in the splice region except one which failed by crushing of the concrete.

As expected, it was found that as the splice length increased the slab strength also increased. A result that was not as obvious was that lap strengths were also very dependent on the length of overhang beyond the last transverse wire. It appears that even though the bars are smooth, a significant amount of bond stress can be developed.

An increase in the cover did not appear to have any significant effect on the strength of the splice, but an increase in the concrete strength did seem to produce a slightly stronger splice strength. Decreasing the reinforcement ratio tended to produce a slightly weaker splice strength.

The aspect of reinforcement style that was studied was the spacing of the transverse wires. One set of slabs showed an increase in splice strength with a decrease in transverse wire spacing, while another set showed a decrease in strength. The effects of nesting were found to be detrimental, possibly because of poorer consolidation of concrete around the steel.

Lloyd concluded that the strength of a splice could be represented by the combination of the strength provided by the bond along the overhang and the strength provided by the shearing of the concrete between the outermost transverse wires in the splice. This was formulated as shown in the following expression:

$$\frac{M_t}{M_u} = \frac{u_u \pi D L_o + C_1 \sqrt{f_c'}\, s_w l_s}{A_w f_y} \tag{2}$$

where: M_t = *maximum test moment.*
M_u = *calculated ultimate moment.*
u_u = *ultimate bond stress, psi.*
D = *nominal diameter of longitudinal wires, in.*
L_o = *total length of longitudinal overhang in lap, in.*
C_1 = *constant to relate* $\sqrt{f_c'}$ *to shear strength of concrete.*
f_c' = *specified compressive strength of concrete, psi.*
s_w = *spacing of longitudinal wires, in.*
l_s = *distance between outermost transverse wires in lap, in.*
A_w = *area of a longitudinal wire,* in^2.
f_y = *specified yield strength of wire, psi.*

The results of all of the slabs were analyzed using this equation, and the values of u_u and C_1 were found to be 340 psi and 2.72 respectively. For design, Lloyd suggested that values of 250 psi and 2.50 be used. If the length of overhang is not known during design, a conservative approach can be taken by neglecting the bond strength in the splice. If this is done and M_t/M_u is taken to be 1.0, then the splice length can be determined as follows:

$$l_s = 0.40 \frac{A_w}{s_w} \frac{f_y}{\sqrt{f_c'}} \tag{3}$$

This equation gives the minimum allowable splice length as required by Section 12.19 of ACI 318-89.

2.2.4 Lloyd and Kesler, 1969[14]

Lloyd and Kesler conducted a study to develop design criteria for splices of deformed wire and welded deformed wire fabric. Of the 23 slabs reinforced with welded deformed wire fabric, 18 failed in the splice region, 3 in the shear span, and 2 in the constant moment region where failure was caused by fracture of the wire. Of the specimens failing in the splice region, none failed in the weld by a pull-out type failure even though some weld strengths were below ASTM standards. All of the slabs failing in the splice region failed by splitting of the concrete between the sheets of WWF.

Lloyd and Kesler hypothesized that the strength of a splice in welded deformed wire fabric could be represented as the sum of the load carried by bond along the overhanging ends and the load carried by the concrete in shear between the sheets of WWF and between the outermost transverse wires in the splice. The effectiveness, *Y*, of a splice based on this type of assumption was calculated as shown by the following equation:

$$Y = \frac{M_{test}}{M_y} = \alpha X_1 + \beta X_2 \tag{4}$$

where: αX_1 = *bond stress contribution.*
βX_2 = *shear strength contribution from concrete.*

The bond stress contribution, αX_1, was defined as $\dfrac{\alpha \, \pi \, l \sqrt{f_c'}}{A_w f_y}$, which was based on results of the slab tests using deformed wire. The shear contribution, βX_2, was assumed to be proportional to $\sqrt{f_c'}$, and the moment in a cracked section was assumed to be proportional to the steel stress. Using these assumptions, Equation 4 was rewritten as shown below:

$$Y = \frac{\alpha\sqrt{f'_c}\ \pi\, l_o + \beta\sqrt{f'_c}\ l_s s_l}{A_w f_y} \tag{5}$$

where: α = *bond stress coeffiiceint.*
f'_c = *specified compressive strength of concrete, psi.*
l_o = *total length of longitudinal overhang in lap, in.*
β = *shear strength coefficient.*
l_s = *distance between the outermost transverse wires in lap, in.*
s_l = *spacing of longitudinal wires, in.*
A_w = *area of a longitudinal wire, in*^{2}.
f_y = *specified yield strength of wire, psi.*

By analyzing the experimental results for the 18 slabs that failed in the splice region, the values of α and β were determined to be 4.95 and 3.56 respectively. It should be noted that Equation 5 is only valid if bond is fully developed and weld strengths are sufficient to cause a shear failure in the layer of concrete between the sheets of fabric and between the outermost transverse wires in the lap.

It was suggested by Lloyd and Kesler that if the longitudinal wires were spaced too closely together that a shear failure of the concrete may occur in the overhanging ends of the lap before the full bond stress could be developed. By equating the bond stress to the shear stress in the overhang of the lap, a value of 4.5 in. was obtained for the minimum longitudinal spacing needed to develop full bond stress. If the longitudinal spacing is less than 4.5 in., then the splice effectiveness could be computed by the following equation, where l is the total splice length:

$$Y = \frac{3.56\sqrt{f'_c}\ l s_w}{A_w f_y} \tag{6}$$

Design equations for the splices of welded deformed wire fabric could be based upon Equations 5 and 6, but Lloyd and Kesler concluded that these would not be conservative since they are based on results obtained from welded deformed wire fabric which exceeded ASTM requirements for wire deformation geometry. Because of this it was desirable to develop design equations based on an earlier investigation which used pull-out tests to determine bond stresses for deformed wires and welded deformed wire fabric.[15] From these tests, and assuming that bond stress is proportional to $\sqrt{f_c'}$, it was found that the ultimate bond stress was equal to $10\sqrt{f_c'}$. Since variations in wire diameter did not produce appreciable variations in ultimate bond stress, this value is not dependent on wire size. ACI 318-63[3] limited the allowable bond stress for deformed bars to ¾ of the ultimate bond stress, therefore this value was reduced to $7.5\sqrt{f_c'}$. One additional reduction in the allowable bond stress was required to account for the smaller effective depth of one sheet of WWF in a splice region. Assuming that design will be based on the largest effective depth, the allowable bond stress was then reduced to $6.6\sqrt{f_c'}$.

It was desired to have the design equations for welded deformed wire fabric compatible with the equations for deformed bars. As a result, Lloyd and Kesler proposed equations that were also based upon the steel yield strength, f_y, and the concrete compressive strength, f_c'. In addition, the splice length was increased by a factor of 1.2 so that it could be reduced by 20% when the spacing of the longitudinal bars was greater than 6", as stipulated in ACI 318-71[3] for deformed bars. By equating the force transmitted by the allowable bond stress to the total force in a wire at a critical section, and subtracting the weld shear strength for each transverse wire in the lap, the following design equation was obtained:

$$l = 0.045\, D\, (f_y - 20{,}000\, N)\, /\, \sqrt{f_c'} \qquad (7)$$

where: l = *length of lap, in.*
D = *nominal diameter of longitudinal wires, in.*
f_y = *specified yield strength of wire, psi.*
N = *number of pairs of transverse wires in lap.*
f_c' = *specified compressive strength of concrete, psi.*

This equation is nearly identical to that specified in Section 12.18 of ACI 318-89 with the exception that ACI does not allow the weld shear strength of more than one pair of transverse wires to be subtracted from f_y. With the current ASTM provisions, the weld shear strength for welded deformed wire fabric is no longer 20,000 psi, but is 35,000 psi times the wire area.

By using Equation 5 with a bond stress of $6.6\sqrt{f_c'}$ and letting Y equal one, an expression which considers the possibility of a splitting failure of the concrete between the outermost transverse wires was obtained as shown below:

$$l_s = \rho d \left[\frac{f_y}{3.5\sqrt{f_c'}} - \frac{8\, l_o}{D} \right] \tag{8}$$

where: l_s = *distance between outermost transverse wires in lap, in.*
ρ = *reinforcement ratio.*
d = *effective depth of reinforcement, in.*
f_y = *specified yield strength of wire,psi.*
f_c' = *specified compressive strength of concrete, psi.*
l_o = *total length of longitudinal overhang in lap, in.*
D = *nominal diameter of longitudinal wires, in.*

If l_o is taken to be zero and ρ is $A_w/s_w d$ then an equation similar to that shown in Section 12.7.2 of ACI 318-89 is obtained.

2.3 Development and Anchorage of Welded Wire Fabric

2.3.1 Atlas, Siess, Bianchini, and Kesler, 1964[12]

A number of tests were conducted on one-way slabs reinforced with welded plain wire fabric at the University of Illinois during the period from 1953 to 1963. Atlas, et. al., used the results from these slab tests to examine the anchorage characteristics of the slabs in the shear span. It was found that anchorage failure occurred when the bond along the WWF, between the end of the WWF and the location of a shear crack, could not transfer the additional tensile stress resulting from the redistribution of stress after the formation of the shear crack.

Atlas, et. al., concluded from the available test data that the embedment of one transverse wire beyond a shear crack location was not sufficient to develop the full yield strength of the longitudinal wire. The results of this investigation are evident in ACI 318-89 section 12.8 which requires that a minimum of two cross wires must be embedded beyond a critical section in order to develop the full yield strength of the longitudinal wire.

2.3.2 Lloyd and Kesler, 1969[14]

Lloyd and Kesler studied a total of 28 one-way slabs to determine development and anchorage characteristics of deformed wire and welded deformed wire fabric. Of the 28 slabs considered, 15 failed in the shear span. The other 13 slabs which did not fail in the shear span contribute to an understanding of anchorage behavior.

Lloyd and Kesler assumed that the anchorage capability of welded deformed wire fabric could be represented as the sum of the bond strength and the weld strengths present in an anchorage or development length, *l*. The maximum stress capable of being developed in the longitudinal wires could then be represented as shown below:

$$\sigma_B = \frac{\alpha \sqrt{f_c'}\ \pi l}{A_w} + N f_w \tag{9}$$

where: σ_B = *ultimate stress developed via bond and transverse wires, psi.*
α = *constant relating bond strength to* $\sqrt{f_c'}\,/D$.
f_c' = *specified compressive strength of concrete, psi.*
l = *anchorage or development length, in.*
A_w = *area of longitudinal wire, in*^{2}.
N = *number of transverse wires in anchorage length,* l.
f_w = *weld shear strength, psi.*

If the ultimate bond stress can not be achieved because of splitting of the concrete, then the maximum allowable stress in the wires could be determined as follows:

$$\sigma_S = \frac{\beta \sqrt{f_c'}\ b\, l}{A_s} \tag{10}$$

where: σ_S = *ultimate stress developed by splitting, psi.*
β = *constant relating shear strength of concrete to* $\sqrt{f_c'}$.
f_c' = *specified compressive strength of concrete, psi.*
b = *width of slab, in.*
l = *anchorage length, in.*
A_s = *area of tensile reinforcement,* in^2.

By comparing the estimated steel stresses at failure with the stresses computed by Equations 9 and 10 the values of α and β were found to be 6.0 and 5.25 respectively.

Since the deformed wires used in this study exceeded the ASTM deformation requirements, Lloyd and Kesler decided to use the results from an earlier investigation in which pull-out tests were conducted to determine the maximum bond stresses for deformed wires and welded deformed wire fabric.(15) Using a value of $10\sqrt{f_c'}$ for the allowable bond stress and a weld shear strength of 20,000 psi times the wire area, an equation for the minimum development length of welded deformed wire fabric was suggested as shown below:

$$L'' = 0.03\, D\, (f_y - 20{,}000\, N) / \sqrt{f_c'} \tag{11}$$

where: L'' = *development length, in.*
D = *nominal diameter of longitudinal wire, in.*
N = *# of welds in development length at least 2 in. from critical section.*
f_y = *specified yield strength of wire, psi.*
f_c' = *specified compressive strength of concrete, psi.*

This equation is identical to that shown in Section 12.7.2 of ACI 318-89 with the exception that ACI does not allow N to be taken greater than one.

To prevent a splitting failure the force in the steel was equated to the force which could be carried by shear in the concrete. By using $5.25\sqrt{f_c'}$ for the shear stress capable of being carried by the concrete, the minimum development length, L'', required to prevent a splitting failure was determined to be:

$$L'' = \frac{\rho\, d f_y}{5.25\sqrt{f_c'}} \qquad (12)$$

where: ρ = *reinforcement ratio.*
d = *effective depth of reinforcement, in.*
f_y = *specified yield strength of wire, psi.*
f_c' = *specified compressive strength of concrete, psi.*

By substituting A_s/bd for ρ in Equation 12, the equation shown in Section 12.7.2 of ACI 318-89 can derived.

2.3.3 Schmitt and Darwin, 1992[16]

Schmitt and Darwin conducted a study at the University of Kansas to determine the effects of epoxy-coating on the bond strength of smooth and deformed wire. The experimental program consisted of three groups of beam-end test specimens using W11 and D11 wires. These specimens were 9 in. wide by 24 in. long by 11 in. deep and were cast with either a 2 in. or ¾ in. cover. The tests used a 3 in. bond length and were essentially pull-out type tests.

A summary of the beam-end test results can be seen in Table 2.1. In order to compare the individual tests on an equal basis, the ultimate bond forces were corrected for variations in concrete strength by normalizing the test results to a nominal concrete strength of 5000 psi. This was done using the assumption that within the concrete strength range used, bond strength is proportional to $\sqrt{f_c'}$. These modified bond forces are also shown in Table 2.1. It should also be noted that the ultimate bond forces reported for Group 2 are only accurate to ± 0.1 kips because of a problem with the load cell.

Table 2.1: Summary of Beam-End Tests[16]

Group	Specimen Label*	Cover (in.)	Concrete Strength (psi)	Ultimate Bond Force (kips)	Modified Bond Force (kips)**
1	U1	2-1/16	4130	0.625	0.688
	U2	2-1/8	4130	0.264	0.291
	AVERAGE				0.489
	C1	2-1/16	4130	1.123	1.236
	C2	2-1/16	4130	0.486	0.535
	AVERAGE				0.885
	UD1	2-1/8	4130	3.139	3.454
	UD2	2-1/16	4130	3.896	4.487
	AVERAGE				3.870
	CD1	2-1/16	4130	4.817	5.300
	CD2	2-1/16	4130	3.987	4.387
	AVERAGE				4.844
2	U1	2-1/8	4910	0.054	0.055
	U2	2-1/8	4910	0.284	0.287
	AVERAGE				0.171
	C1	2-1/16	4910	0.030	0.030
	C2	2-1/8	4910	0.562	0.567
	AVERAGE				0.299
	UD1	2-1/16	4910	3.748	3.782
	UD2	2-1/8	4910	4.959	5.004
	AVERAGE				4.393
	CD1	2-1/16	4910	5.145	5.192
	CD2	2-1/8	4910	4.747	4.790
	AVERAGE				4.991

* Specimen Label

U - Uncoated Plain Wire CD - Coated Deformed Wire

C - Coated Plain Wire UD - Uncoated Deformed Wire

** Modified Bond Force = Ultimate Bond Force x $(5000/f_c')^{1/2}$

Table 2.1 (continued): Summary of Beam-End Tests[16]

Group	Specimen Label	Cover (in.)	Concrete Strength (psi)	Ultimate Bond Force (kips)	Modified Bond Force (kips)
3	U1	13/16	5230	0.436	0.417
	U2	13/16	5230	0.156	0.149
	AVERAGE				0.283
	C1	3/4	5230	1.040	0.994
	C2	13/16	5230	1.290	1.233
	AVERAGE				1.114
	UD1	13/16	5230	3.888	3.717
	UD2	13/16	5230	4.118	3.937
	UD3	13/16	5230	4.669	4.464
	AVERAGE				4.039
	CD1	13/16	5230	4.113	3.932
	CD2	3/4	5230	3.953	3.779
	AVERAGE				3.856

* Specimen Label

U - Uncoated Plain Wire   CD - Coated Deformed Wire

C - Coated Plain Wire   UD - Uncoated Deformed Wire

** Modified Bond Force = Ultimate Bond Force x $(5000/f_c')^{1/2}$

From the tests results, Schmitt and Darwin concluded that coated plain wire provides higher bond strength than uncoated plain wire, and that epoxy-coating appears to have little effect on the bond strength of deformed wire. These conclusions are based on very limited test data and there is very large scatter in the data. This suggests that more tests would need to be conducted in order for the trends to be conclusive.

2.4 Flexural Cracking of Slabs Reinforced With Welded Wire Fabric

2.4.1 Atlas, Siess, Bianchini, and Kesler, 1964[12]

Atlas, et. al., conducted a series of one-way slab tests at the University of Illinois to examine the width and spacing of cracks in flexural members reinforced with welded plain wire fabric. It was proposed that rigidly welded transverse wires, smooth longitudinal wires, and high tensile strength of the WWF were the factors affecting the cracking of flexural members reinforced with WWF.

The following equation was recommended for determining the average crack spacing in slabs reinforced with WWF:

$$a_{ave} = c_s\,(3.0 + 0.4s) \tag{13}$$

where: a_{ave} = *average crack spacing, in.*
$c_s = 1 + 0.024\,(D/P_e - 43)$, *for* $s = 12$ *in.*
$c_s = 1 + 0.008\,(D/P_e - 33)$, *for* $s = 6$ *in.*
$c_s = 1$, *for* $s = 3$*in.*
s = *transverse wire spacing.*
D = *nominal diameter of longitudinal wire.*
P_e = *effective reinforcement ratio.*

In addition, it was found that the maximum crack width could be determined from the following equation:

$$w_{max} = a_{ave}\,\frac{f_s}{K} \tag{14}$$

where: w_{max} = *maximum crack width, in.*
a_{ave} = *average crack spacing, in.*
f_s = *steel stess at the crack.*

In Equation 14, K depends primarily on the modulus of elasticity of the reinforcement and the variations in steel stress between the cracks. Tentative values of K are presented by Atlas, et.

al., but they are based on very limited experimental data and are thus not recommended for use.

2.4.2 Lloyd and Kesler, 1969[14]

Lloyd and Kesler conducted a study of 63 one-way flexural slabs to determine the crack controlling properties of deformed wire and welded deformed wire fabric. From this investigation and the results of other researchers, recommended equations were developed for estimating average crack spacing and maximum crack widths at the extreme tensile fiber. Since average crack spacing is not of direct importance, Lloyd and Kesler did not compare estimated crack spacing to actual measured results. Instead, concentration was focused on maximum crack widths.

As determined by Lloyd and Kesler, the maximum crack width at the extreme tensile fiber can be estimated by the following equation:

$$w'_{max} = 0.076\,(t_b A)^{1/3} R f_s \; x \; 10^{-6} \qquad (15)$$

where: w'_{max} = *maximum crack width at extreme tensile fiber, in.*
t_b = *thickness of concrete cover measured from extreme tensile fiber to center of wire, in.*
A = *average effective concrete area around a wire,* in^2.
f_s = *tensile steel stress, psi.*
R = h_2 / h_1.
h_1 = *distance from centroid of tensile reinforcement to neutral axis, in.*
h_2 = *distance from extreme tensile fiber to neutral axis, in.*

In addition to considering the deformed wire and welded deformed wire fabric in this crack control study, Lloyd and Kesler looked at a series of tests that were conducted with welded plain wire fabric and compared the results. From this study it was found that there was no significant difference between the crack control properties of the welded plain wire fabric, deformed wire, and welded deformed wire fabric. It was noted in the study involving welded plain wire fabric that a light coating of rust covered the wires which may have increased the bond characteristics. Lloyd and Kesler also concluded that the transverse wire spacing does not significantly influence crack widths in slabs reinforced with WWF.

2.4.3 Lee, Mansur, Tan, and Kasirju, 1989[17]

Lee, et. al. conducted a study to determine the cracking behavior of one-way slabs reinforced with welded plain wire fabric and welded deformed wire fabric. This investigation included 14 slabs reinforced with welded plain wire fabric and 6 with welded deformed wire fabric. It was found that the crack spacing depended mainly on the spacing of transverse wires, and that crack widths could be calculated on the basis of crack spacing, stress in longitudinal wires, and bond properties of the reinforcement.

The following equation was developed to estimate the minimum and maximum crack spacing:

$$\begin{array}{lll} \text{when } s_t < h_c, & a_{\min} = s_t, \text{ and} & a_{\max} = 2s_t \\ \text{when } h_c < s_t < 2h_c, & a_{\min} = (s_t - h_c), \text{ and} & a_{\max} = s_t \\ \text{when } nh_c < s_t < (n+1)h_c, & a_{\min} = (s_t - nh_c), \text{ and} & a_{\max} = 2h_c \end{array} \tag{16}$$

where: s_t = spacing of transverse wires.
h_c = distance from neutral axis to extreme tensile fiber.
$a_{\min}$ = minimum crack spacing.
$a_{\max}$ = maximum crack spacing.
n = 2, 3, . . .

Figure 2.2 shows the variation in minimum and maximum crack spacing as a function of s_t. From this figure it can be seen that the best crack control using WWF is provided when s_t is considerably less than h_c, and when s_t is slightly greater than h_c, but much less than $2h_c$.

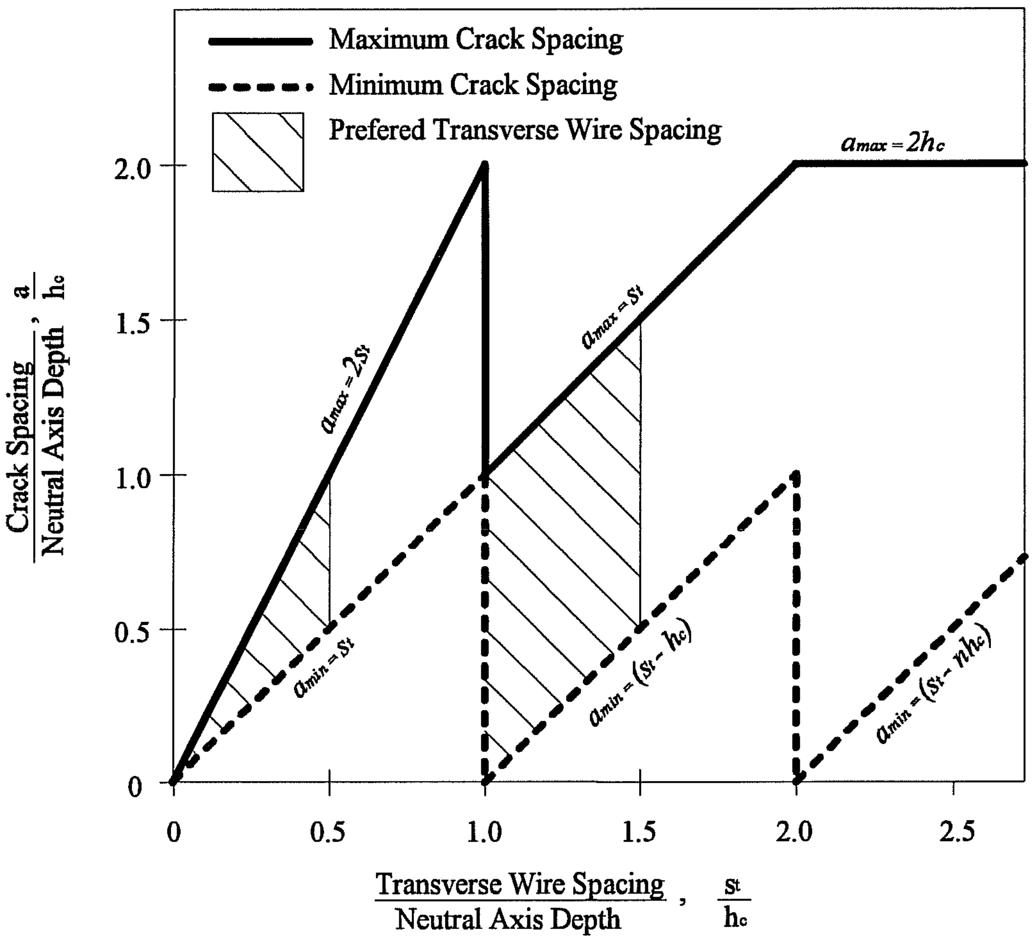


Figure 2.2: Variation in Crack Spacing as a Function of Transverse Wire Spacing[17]

Using the value for the maximum crack spacing obtained from Equation 16, the maximum crack width at the extreme tensile fiber can be calculated using the following equation:

$$w_{max} = \epsilon_s a_{max} R \qquad (17)$$

where: w_{max} = *maximum crack width at extreme tensile fiber.*
ϵ_s = *strain in steel reinforcement.*
a_{max} = *maximum crack spacing, in.*
$R = h_c / h_s$.
h_c = *distance from neutral axis to extreme tensile fiber.*
h_s = *distance from centroid of tensile reinforcement to neutral axis, in.*

In this equation, the bond stress developed along the longitudinal wires is neglected and thus is only applicable to slabs reinforced with welded plain wire fabric where the bond stress developed is considerably less than that developed for welded deformed wire fabric. The estimated maximum crack widths based on Equation 17 correlate very closely with the measured values for the 14 slabs reinforced with welded plain wire fabric.

Lee, et. al. also proposed a much more detailed method for determining maximum crack widths which includes the effect of bond. This method is not detailed here because it is considered to be much too complicated for any type of a design calculation.

2.5 Development and Anchorage of Epoxy-Coated Reinforcing Bars

2.5.1 Treece and Jirsa, 1987[9]

Treece and Jirsa conducted a study of 21 beam specimens to determine the bond strength of epoxy-coated and uncoated reinforcing bars in tension. The specimens were constructed with #6 or #11 bars spliced at the center of the beam. The bars were either uncoated or epoxy-coated with a nominal coating thickness of 5 mils or 12 mils. These values represent the minimum and maximum allowable coating thickness as stipulated by ASTM. In addition to coating thickness, concrete strengths were also varied.

The performance of these tests were evaluated on the basis of measured bond strength, crack width and spacing, and stiffness of the beams. The results showed that epoxy-coated bars developed approximately 65% of the bond strength of uncoated bars, however, no loss in stiffness was observed for epoxy-coated specimens. This result was independent of bar size, coating thickness, and concrete strength. In addition, it was found that average crack widths were approximately 50% greater in coated specimens than uncoated specimens.

Figure 2.3 shows a typical plot of load versus deflection and Figure 2.4 shows a typical plot of steel stress versus crack width for a pair of coated and uncoated specimens. In these figures specimen 0-11-12b is uncoated and specimen 12-11-12b is coated. As can be seen in Figure 2.3, the coated specimen reached only a fraction of the load carried by the uncoated specimen, but there does not appear to be any difference in stiffness. Figure 2.4 shows that at the same steel stress, uncoated specimens had much smaller average crack widths than coated specimens.

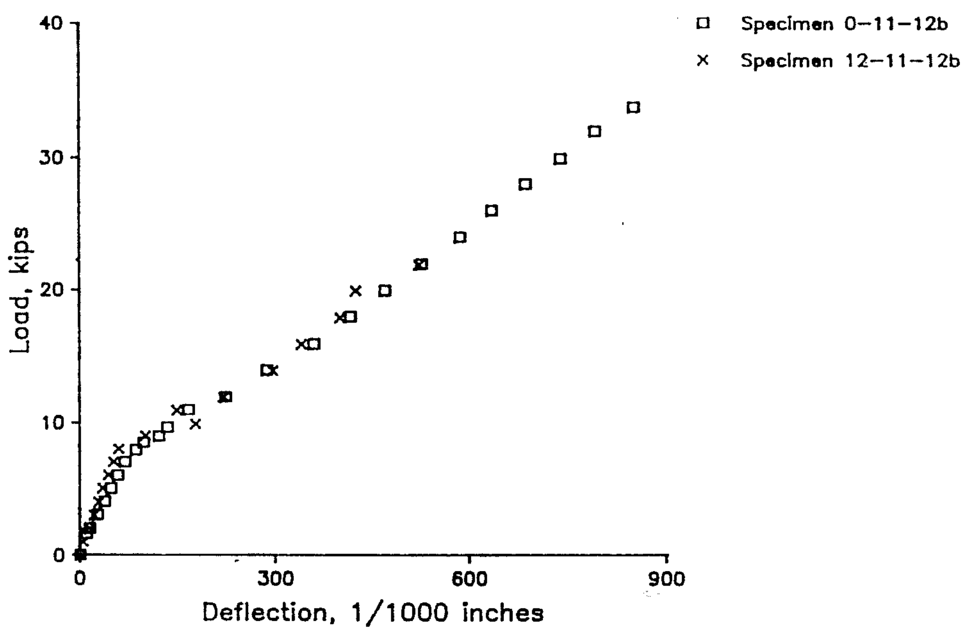


Figure 2.3: Load Versus Deflection for Typical Pair of Specimens[9]

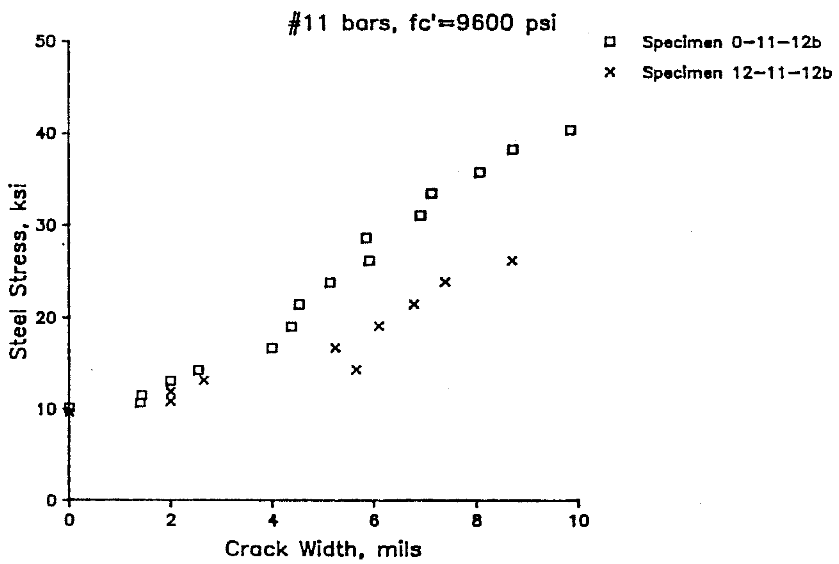


Figure 2.4: Steel Stress Versus Crack Width for Typical Pair of Specimens(9)

2.5.2 Hamad, Jirsa, and d'Abreu, 1990(10)

Hamad, Jirsa, and d'Abreu conducted a study to provide a better and more complete understanding of the effects of epoxy-coating on the bond of reinforcing bars. This study included pull-out tests, beam splice tests, and hooked bar tests.

The pull-out test results showed that epoxy-coated bars consistently developed a lower bond strength than uncoated bars. This reduction in bond strength ranged from about 10% to 25% and was not affected by the concrete strength or level of confinement. The beam tests conducted showed approximately the same results as those found by Treece and Jirsa(9). Hamad, Jirsa, and d'Abreu included an additional variable that was not considered by Treece and Jirsa by also studying the effects of transverse reinforcement in the splice region on the bond of epoxy-coated reinforcing bars. It was found that the bond strength of epoxy-coated #11 bar splices were 74% of the uncoated splices in the absence of transverse reinforcement and 80% when transverse reinforcement was provided. For #6 bar splices these values were 67% and 74% in the absence and presence of transverse reinforcement, respectively.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 Introduction

An experimental program was developed to determine the effects of epoxy coating on the bond of welded plain wire fabric and welded deformed wire fabric. This program included a series of weld shear tests, pull-out tests, and one-way slab tests. The details of each of these tests will be outlined in the following sections.

3.2 Weld Shear Tests

Weld shear tests were conducted on the coated and uncoated 4x16-W20xW10 and 4x16-D20xD10 WWF to determine the weld shear strength of the weld between the longitudinal and transverse wires. A number of weld shear tests were also conducted on the WWF with size 4 longitudinal wires. In each of these cases it was found that fracture occurred in the wire before shearing of the weld. For this reason, only a few weld shear tests on the WWF with size 4 longitudinal wires were conducted.

3.2.1 Numbering System

The specimens used for the weld shear tests were numbered according to the location of the weld being tested in the particular sheet of WWF. Each of the longitudinal and transverse wires in a sheet of WWF were numbered from 1 to the number of longitudinal or transverse bars. The weld shear tests were then given a designation in the form of B#X#-#. B# represents the longitudinal wire number, X# represents the transverse or cross wire number, and -# designates the type of WWF used for the weld shear test specimen. This numbering system was developed so that both weld shear tests and pull-out tests could be numbered in the same manner. Table 3.1 shows the WWF designations for the pull-out specimens, which are the same designations as those used for the weld shear tests. Figure 3.1 shows how a typical sheet of WWF was divided into weld shear specimens and pull-out specimens.

Table 3.1: Pull-Out Test Specimen Designations

B#X#-#	Type of WWF	Length of Bond (in.)
B#X#-1	uncoated W20xW10	0
B#X#-2	coated W20xW10	0
B#X#-3	uncoated D20xD10	0
B#X#-4	coated D20xD10	0
B#X#-5	uncoated W20xW10	4
B#X#-6	coated W20xW10	4
B#X#-7	uncoated D20xD10	4
B#X#-8	coated D20xD10	4
B#-9	uncoated D20	4
B#-10	coated D20	4

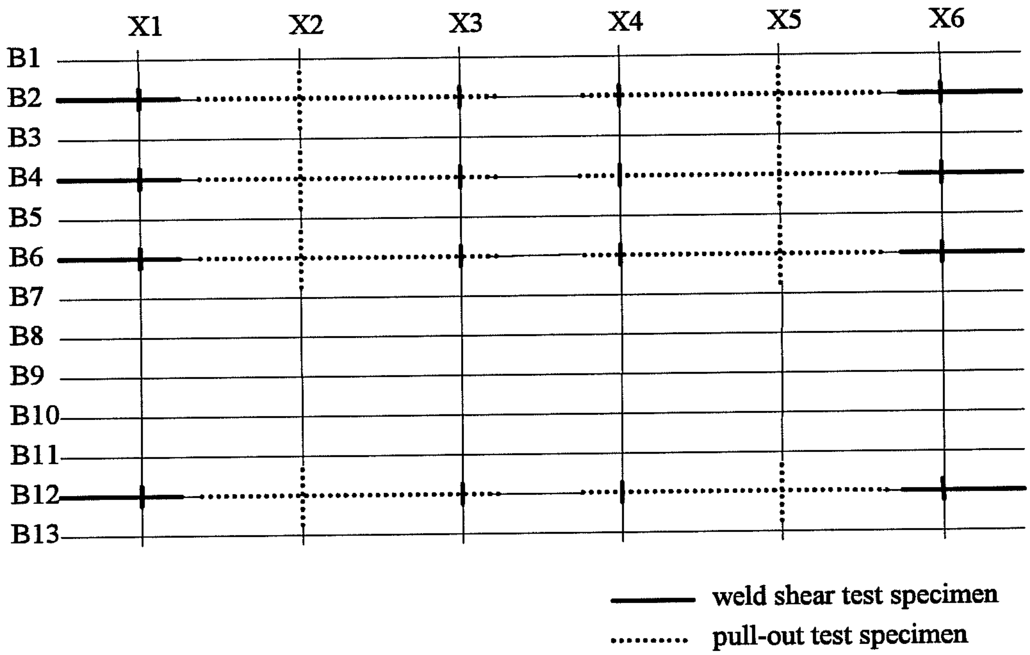


Figure 3.1: WWF Divided into Weld Shear and Pull-Out Specimens

3.2.2 Test Specimens

The test specimens were cut from 4 ft. 2 in. wide by 8 ft. long sheets of WWF. The test wires were sized according to the requirements of ASTM A497; the transverse wire on each specimen extended approximately 1 in. on each side of the longitudinal wire, and the longitudinal wire extended 8 in. below the transverse wire and 4 in. above the transverse wire. The 8 in. length was sufficient to engage the grips of the testing machine and the 4 in. length was sufficient to clear the upper bearing of the testing device. A detail of a typical test specimen can be seen in Figure 3.2.

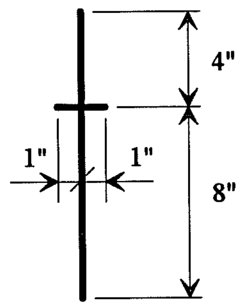


Figure 3.2: Weld Shear Test Specimen

3.2.3 Test Setup

The details for the testing apparatus were obtained from ASTM and a photograph of the completed apparatus can be seen in Figure 3.3. The testing apparatus was used in conjunction with a Southwark Emery 60 kip hydraulic load machine to determine the weld shear strength of the specimens.

3.2.4 Test Procedure

Each of the test specimens was tested to failure to determine the shear strength of the weld. The tests were run at a constant rate of stress and the ultimate shear strength was recorded. Only the load was monitored.

3.3 Pull-Out Tests

A total of 31 pull-out tests were conducted. 27 pull-out tests were conducted on the coated and uncoated 4x16-W20xW10 and 4x16-D20xD10 WWF, and 4 tests were conducted on coated and uncoated D20 wires.

3.5 Material Properties

All slabs and pull-out specimens were cast using the same concrete mix design. A maximum coarse aggregate size of ¾ in. and a water to cement ratio of 0.72 was used. The cylinder strengths at 7, 14, and 28 days for each of the slab and pull-out specimens can be seen in Table 3.4. Also shown in this table is the slump at the time of casting and the cylinder strength at the time of testing.

Table 3.5 shows the yield strength for the various sizes of WWF used in this investigation. Yield strengths were measured at both a strain of 0.0035 as stipulated in ACI 318-89 and at a strain of 0.0050 as stipulated in ASTM. Strains were measured over an 8 in. gage length with a cross wire located at the center of the gage length. Table 3.5 does not make any distinction between coated and uncoated WWF since both were manufactured at the same time using the same lot of wire, and therefore have the same mechanical properties.

Table 3.4: Concrete Strength for Slab Specimens and Pull-Out Specimens

Specimen Number	7 Day Strength	14 Day Strength	28 Day Strength	Test Day Strength	Slump
UW4-1, CW4-1	3560	3900	4060	4170	3.5
UD4-1, CD4-1	3560	3900	4060	4180	3.5
UW20-1, CW20-1	3470	3860	4250	4140	4.0
UW20-2, CW20-2	2620	3340	3870	3750	6.0
UD20-1, CD20-1	3820	4060	4520	4590	2.5
UD20-2, CD20-2	2620	3340	3870	3750	6.0
B#X#-1, B#X#-2, B#X#-3, B#X#-4	2860	3270	3570	3510	6.0
B#X#-5, B#X#-6, B#X#-7, B#X#-8, B#-9, B#-10	3820	4060	4520	4290	3.0

Table 3.5: WWF Yield Strength

WWF Style	Yield Strength (ksi)	
	ACI 0.0035 Strain	ASTM 0.0050 Strain
4x8-W4xW4	82.5	88.5
4x8-D4xD4	85.5	92.5
4x16-W20xW10	73.5	75.0
4x16-D20xD10	79.5	82.5
2x8-W20xW11	92.3	103.0
2x8-D20xD11	84.4	91.5

CHAPTER 4

PRESENTATION AND DISCUSSION OF EXPERIMENTAL RESULTS

4.1 Introduction

The experimental results from the pull-out tests and one-way slab tests are presented in terms of load-deflection relationships, and the weld shear test data is presented as the maximum measured weld shear strength. Comparisons are made between coated and uncoated specimens for the pull-out tests and one-way slab tests to examine the effects of epoxy coating on the bond and development of WWF. In addition to the load-deflection relationships for the one-way slab tests, maximum crack sizes and spacing are also given, and the maximum crack sizes are compared to maximum crack sizes estimated using the equations presented in Chapter 2.

4.2 Weld Shear Tests

4.2.1 Presentation of Test Results

The strengths for the weld shear specimens taken from each sheet of WWF are listed in Tables 4.1 through 4.4. Each of these tables groups the data according to the longitudinal wire from which the specimens were taken, and can be seen in Figure 3.1. Four specimens were taken along each longitudinal wire. In addition to showing the measured weld shear strengths, Tables 4.1 through 4.4 also show comparisons of the measured weld shear strength to the minimum weld shear strength of 7000 lbs (35,000 psi x 0.20 in^2) stipulated by ASTM for a fabric with size 20 longitudinal wires.

Four weld shear tests were also conducted on the WWF with size 4 longitudinal wires. Two tests were conducted on the welded plain wire fabric and two on the welded deformed wire fabric. These specimens failed by necking of the longitudinal wire and fracture of the longitudinal wire adjacent to the weld. No further tests were conducted using the small wires since the weld appeared to be able to develop the ultimate strength of the longitudinal wire.

4.2.2 Discussion of Test Results

The weld shear test data in Tables 4.1 through 4.4 shows a definite difference in strength along different longitudinal wires. This can be attributed to the manufacturing process where each weld across the width of a sheet is performed by a different automatic electric resistance welder. The specimens taken from welded plain wire fabric showed much less variation in weld shear strength along a longitudinal wire than did the specimens taken from welded deformed wire fabric. The average percent difference in the maximum and minimum weld shear strengths along a longitudinal wire when compared to the minimum ASTM weld shear strength of 7000 lbs was 9.3% for welded plain wire fabric and 18.1% for welded deformed wire fabric.

The average weld shear strengths along each of the longitudinal wires tested can be seen in Tables 4.1 through 4.4. Only one of these average weld shear strengths did not exceed the minimum ASTM requirement of 7000 lbs. This does not mean that this particular sheet of WWF failed to meet the ASTM standards since ASTM requires that the average of four tests representing the entire width of the sheet of WWF must surpass the minimum weld shear value. The average values shown in Table 4.1 are for a particular longitudinal wire and do not represent the entire width of the sheet of WWF.

Table 4.1: Uncoated 4x16-W20xW10 Measured Weld Shear Strength

Specimen #	Measured Weld Shear Strength (lbs.)	Measured Weld Shear Strength / ASTM Minimum Requirement (7000 lbs)
B2X1-1	8000	1.14
B2X3-1	8280	1.18
B2X4-1	8200	1.17
B2X6-1	7310	1.04
Average	7950	1.14
B4X1-1	9350	1.34
B4X3-1	9530	1.36
B4X4-1	9680	1.38
B4X6-1	9680	1.38
Average	9560	1.37
B6X1-1	7420	1.06
B6X3-1	7770	1.11
B6X4-1	7900	1.13
B6X6-1	7450	1.06
Average	7640	1.09
B12X1-1	9520	1.36
B12X3-1	9190	1.31
B12X4-1	9240	1.32
B12X6-1	9960	1.42
Average	9480	1.35

Table 4.2: Coated 4x16-W20xW10 Measured Weld Shear Strength

Specimen #	Measured Weld Shear Strength (lbs.)	Measured Weld Shear Strength / ASTM Minimum Requirement (7000 lbs)
B2X1-2	8990	1.28
B2X3-2	8970	1.28
B2X4-2	8890	1.27
B2X6-2	8870	1.27
Average	8930	1.28
B4X1-2	8250	1.18
B4X3-2	7890	1.23
B4X4-2	8560	1.22
B4X6-2	8360	1.19
Average	8270	1.18
B6X1-2	8310	1.19
B6X3-2	8750	1.25
B6X4-2	8460	1.21
B6X6-2	7760	1.11
Average	8320	1.19
B12X1-2	7220	1.03
B12X3-2	7700	1.10
B12X4-2	8110	1.16
B12X6-2	7920	1.13
Average	7740	1.11

Table 4.3: Uncoated 4x16-D20xD10 Measured Weld Shear Strength

Specimen #	Measured Weld Shear Strength (lbs.)	Measured Weld Shear Strength / ASTM Minimum Requirement (7000 lbs)
B2X1-3	9370	1.34
B2X3-3	8440	1.21
B2X4-3	8090	1.16
B2X6-3	7690	1.10
Average	8400	1.20
B4X1-3	8370	1.20
B4X3-3	6800	0.97
B4X4-3	7310	1.04
B4X6-3	7160	1.02
Average	7410	1.06
B6X1-3	7430	1.06
B6X3-3	8200	1.17
B6X4-3	8240	1.18
B6X6-3	6910	0.99
Average	7700	1.10
B12X1-3	7210	1.03
B12X3-3	7420	1.06
B12X4-3	7890	1.13
B12X6-3	6920	0.99
Average	7360	1.05

Table 4.4: Coated 4x16-D20xD10 Measured Weld Shear Strength

Specimen #	Measured Weld Shear Strength (lbs.)	Measured Weld Shear Strength / ASTM Minimum Requirement (7000 lbs)
B2X1-4	7480	1.07
B2X3-4	8380	1.20
B2X4-4	8690	1.24
B2X6-4	9010	1.29
Average	8390	1.20
B4X1-4	8240	1.18
B4X3-4	7960	1.14
B4X4-4	7950	1.14
B4X6-4	7480	1.07
Average	7910	1.13
B6X1-4	7720	1.10
B6X3-4	8040	1.15
B6X4-4	8220	1.17
B6X6-4	8940	1.28
Average	8230	1.18
B12X1-4	6650	0.95
B12X3-4	6190	0.88
B12X4-4	6990	1.00
B12X6-4	5910	0.84
Average	6440	0.92

4.3 Pull-Out Tests

4.3.1 Presentation of Results

Pull-out test results are presented in terms of load and slip relationships and can be grouped into two categories; those with no bond along the longitudinal wire and those with 4 in. of bond along the longitudinal wire. The results for the first group, no bond, can be seen in Figures 4.1 through 4.8. The results for the specimens with 4 in. of bond are shown in Figures 4.9 through 4.16. Figures 4.17 and 4.18 show the results from four tests that were run on specimens with no welded cross wire and 4 in. of bond. All of the graphs in Figures 4.1 through 4.18 show the load on the specimen versus the free end slip and loaded end slip. Two separate pull-out specimens are shown in many of the figures because they were taken from the same longitudinal wire in a sheet of WWF. This allowed for an easier comparison between the two specimens. In addition to the pull-out test results, Figures 4.1 through 4.16 also show the weld shear strengths for the weld shear specimens taken from the same longitudinal wire as the pull-out specimens.

The loaded end slip was calculated by subtracting the elongation in the longitudinal wire, over a length of 9 in., from the measured deflection at the loaded end. The elongation was subtracted in order to account for the additional slip at the point of measurement due to the strain in a 9 in. length of the wire from the point of loading to the top of the concrete block. By making this correction, the loaded end slip shown in Figures 4.1 through 4.18 is the amount of slip at the top of the concrete block, not at the point where the load was applied.

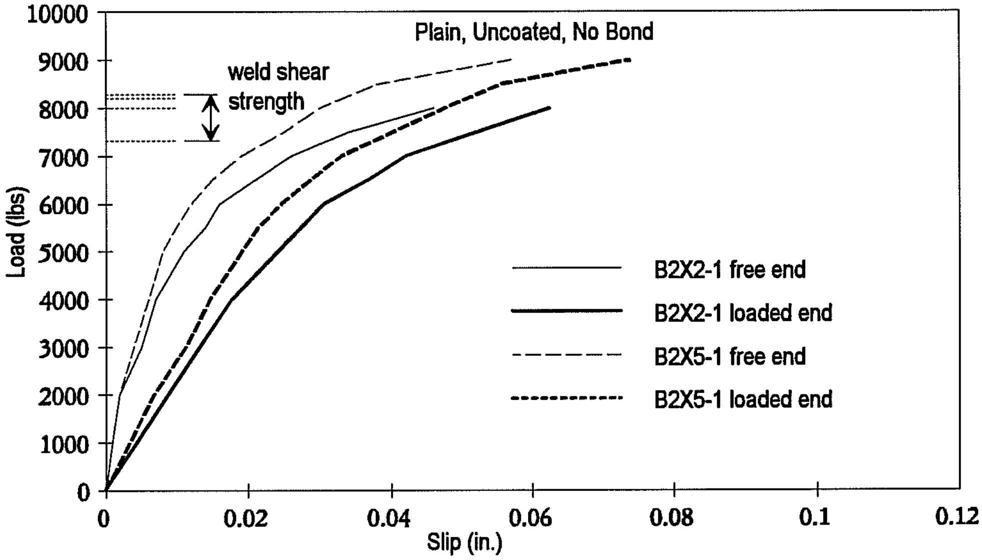


Figure 4.1: Pull-Out Specimens B2X2-1 and B2X5-1

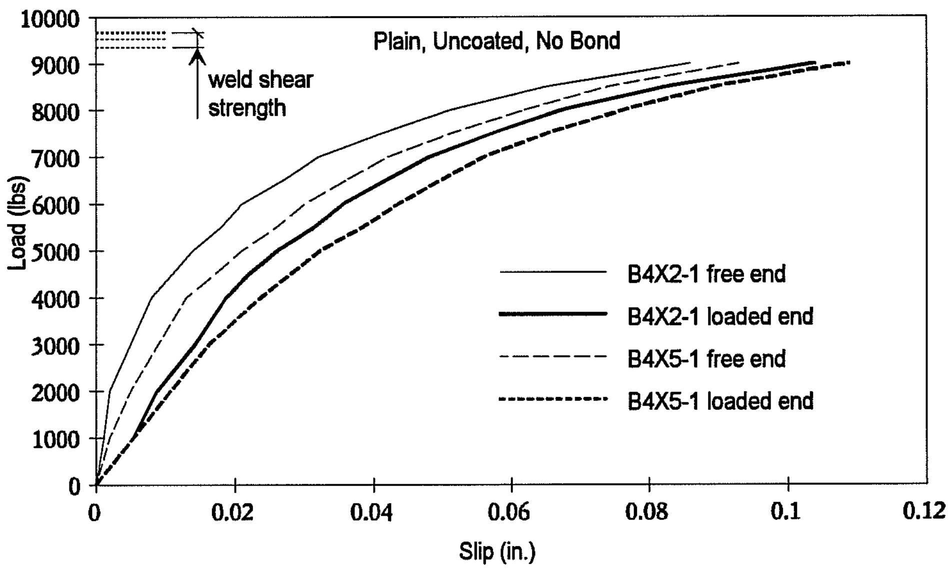


Figure 4.2: Pull-Out Specimens B4X2-1 and B4X5-1

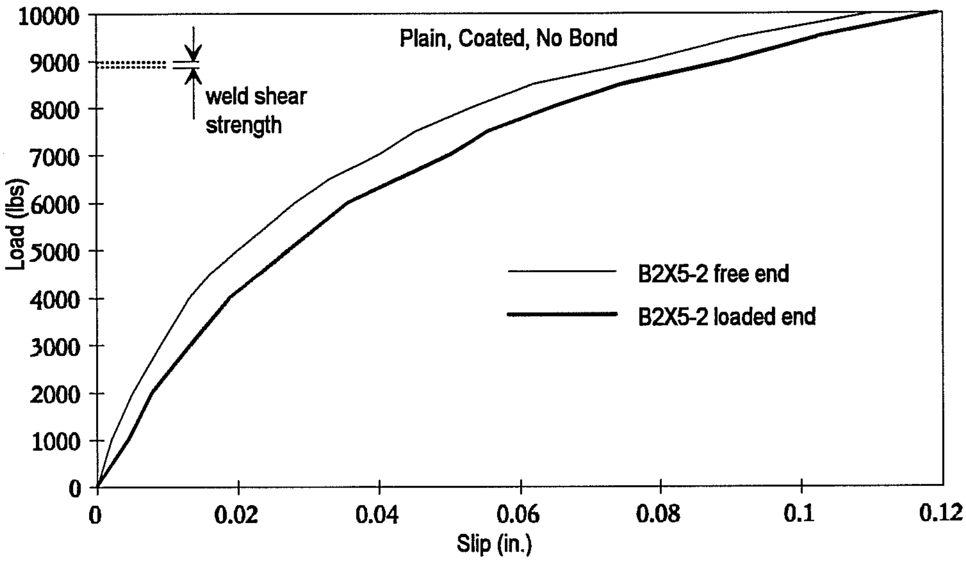


Figure 4.3: Pull-Out Specimen B2X5-2

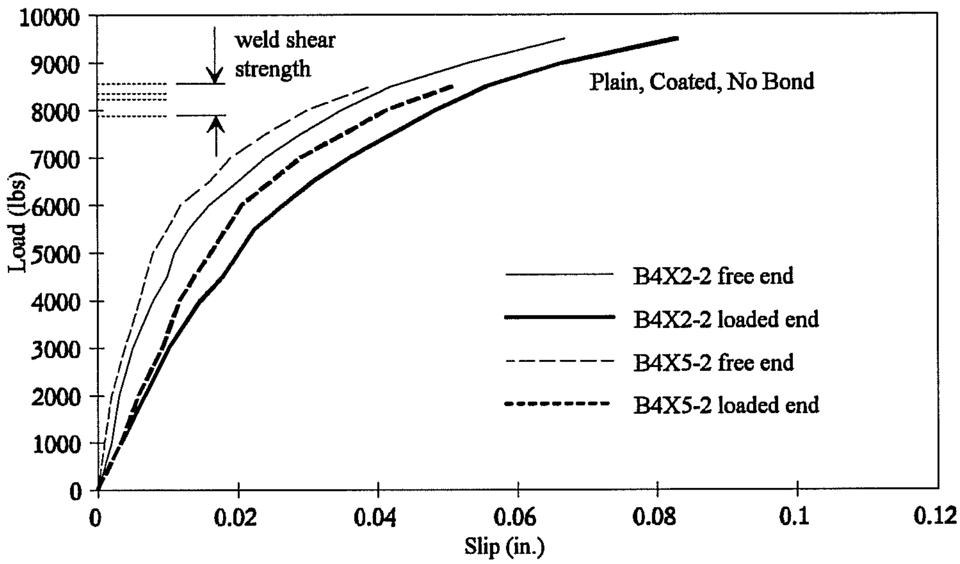


Figure 4.4: Pull-Out Specimens B4X2-2 and B4X5-2

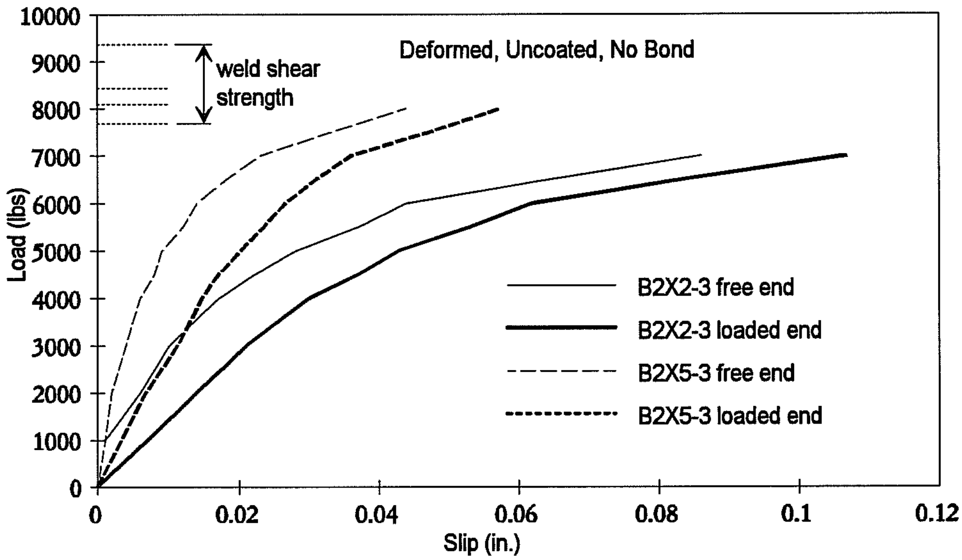


Figure 4.5: Pull-Out Specimens B2X2-3 and B2X5-3

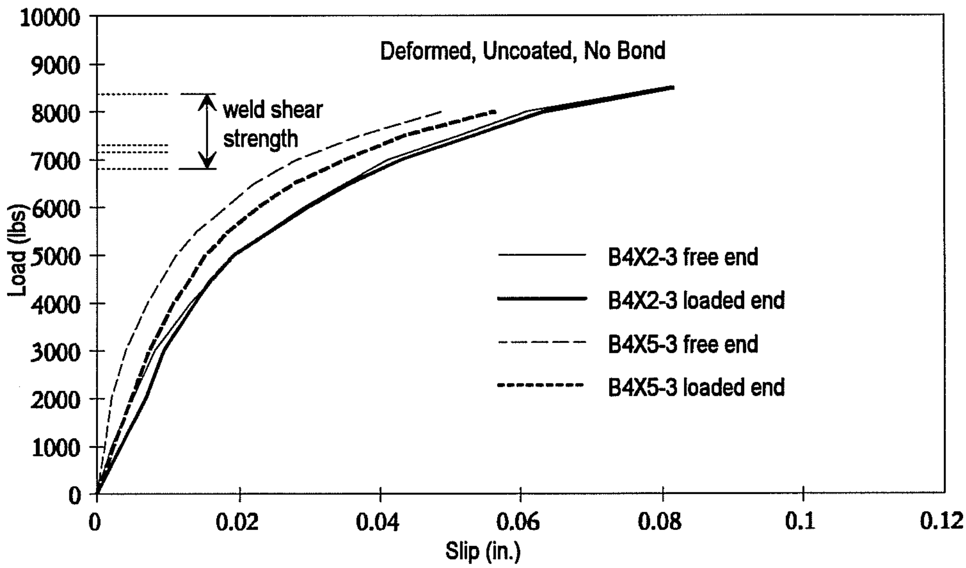


Figure 4.6: Pull-Out Specimens B4X2-3 and B4X5-3

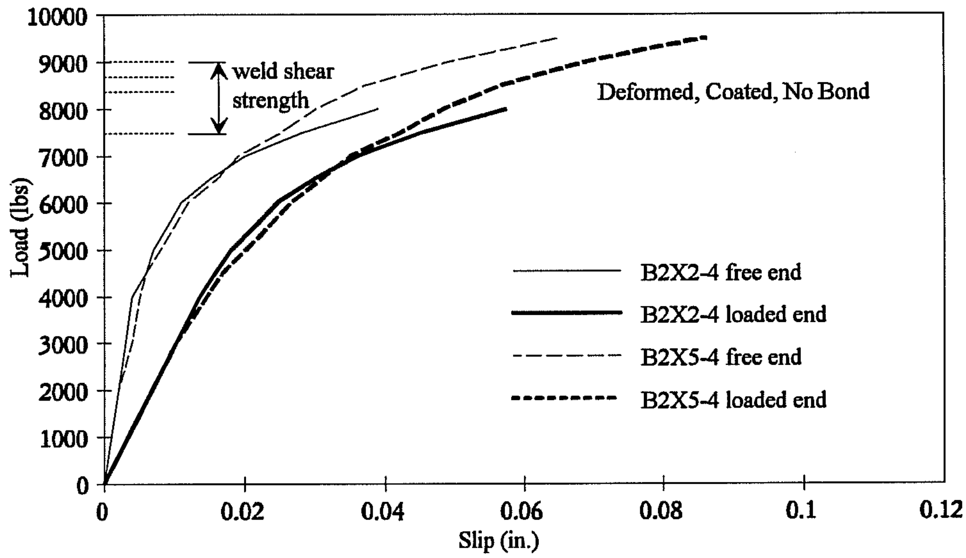


Figure 4.7: Pull-Out Specimens B2X2-4 and B2X5-4

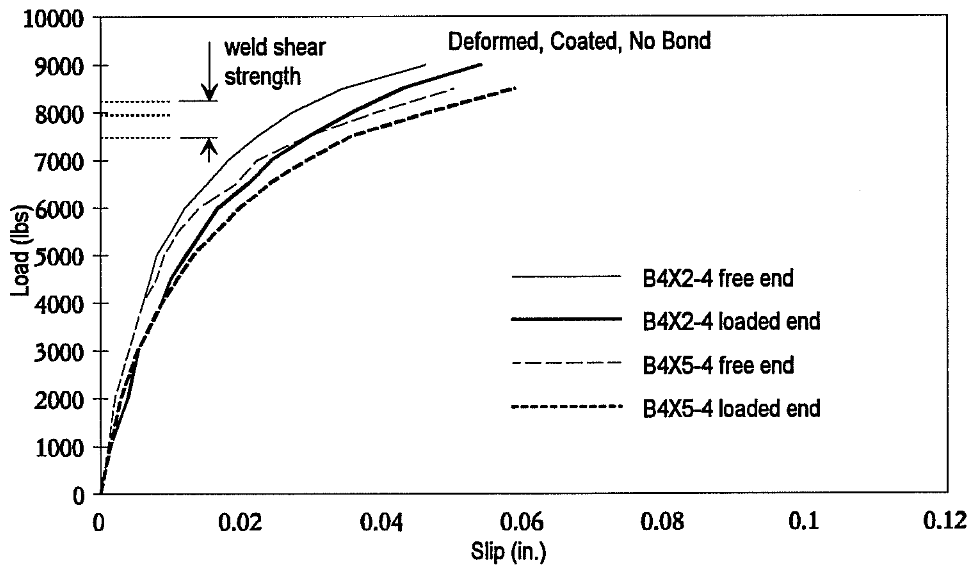


Figure 4.8: Pull-Out Specimens B4X2-4 and B4X5-4

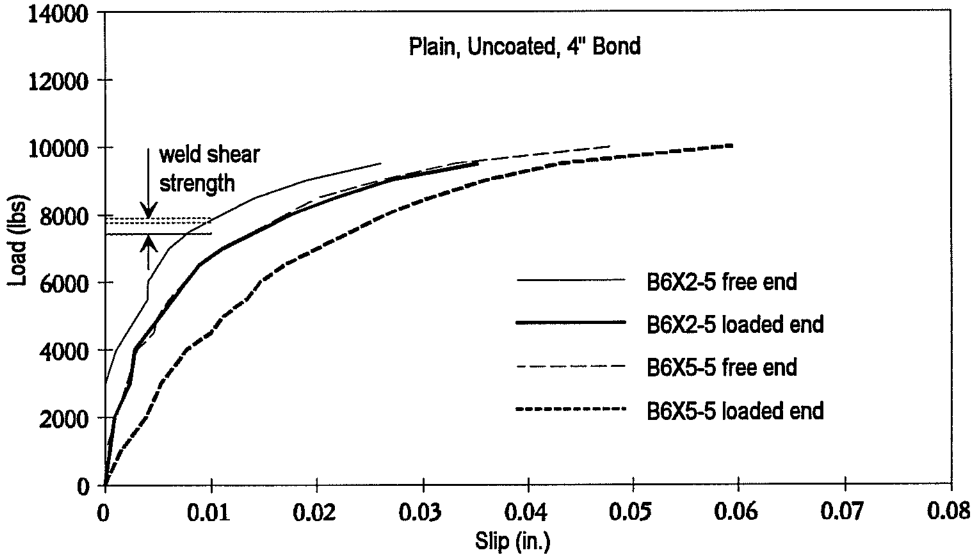


Figure 4.9: Pull-Out Specimens B6X2-5 and B6X5-5

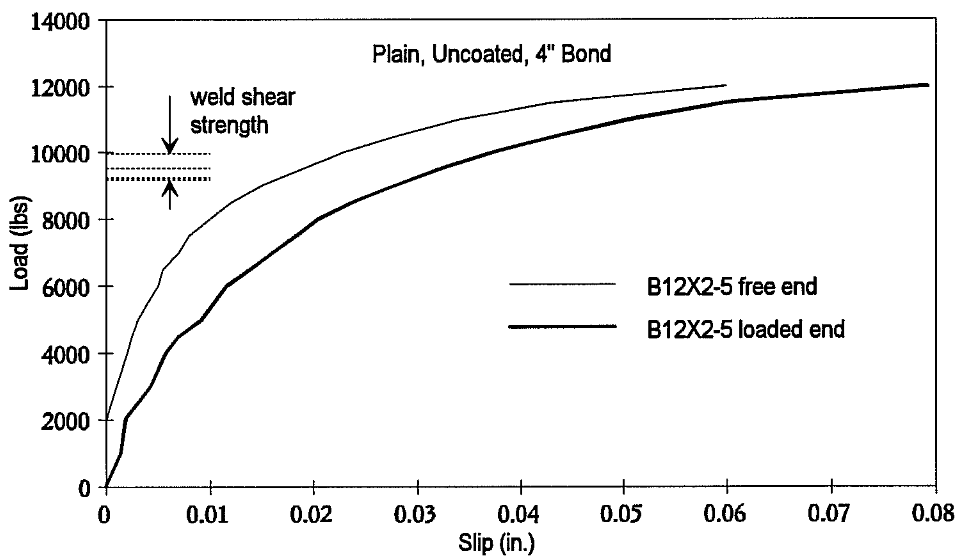


Figure 4.10: Pull-Out Specimen B12X2-5

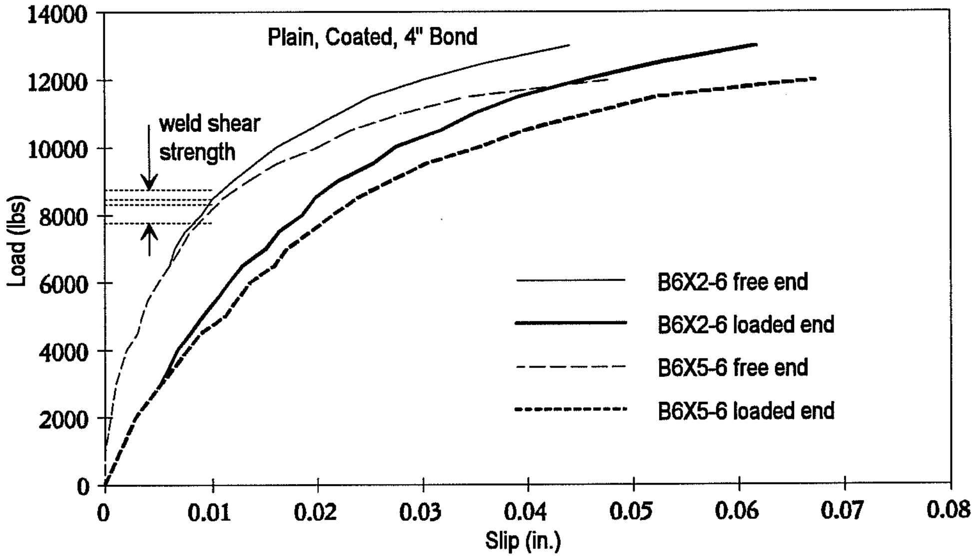


Figure 4.11: Pull-Out Specimens B6X2-6 and B6X5-6

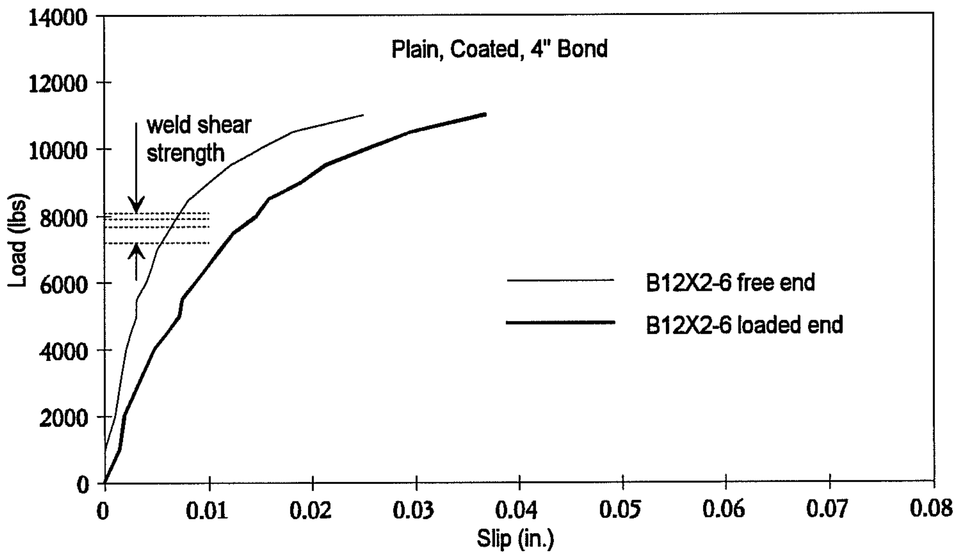


Figure 4.12: Pull-Out Specimen B12X2-6

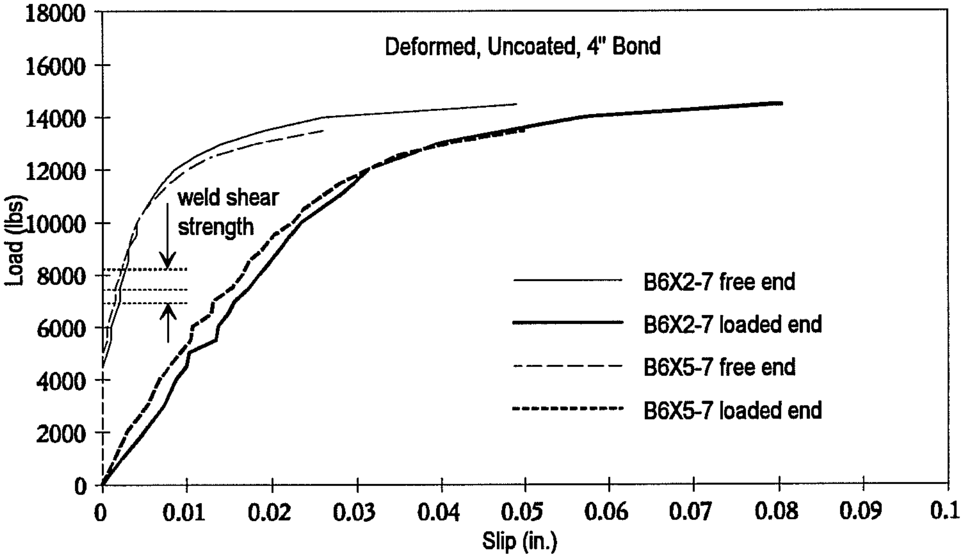


Figure 4.13: Pull-Out Specimens B6X2-7 and B6X5-7

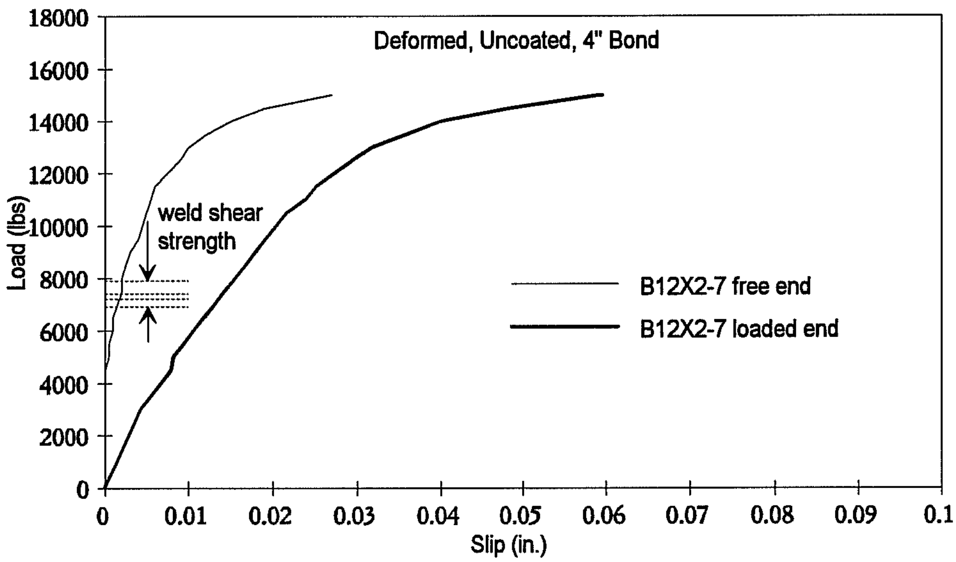


Figure 4.14: Pull-Out Specimen B12X2-7

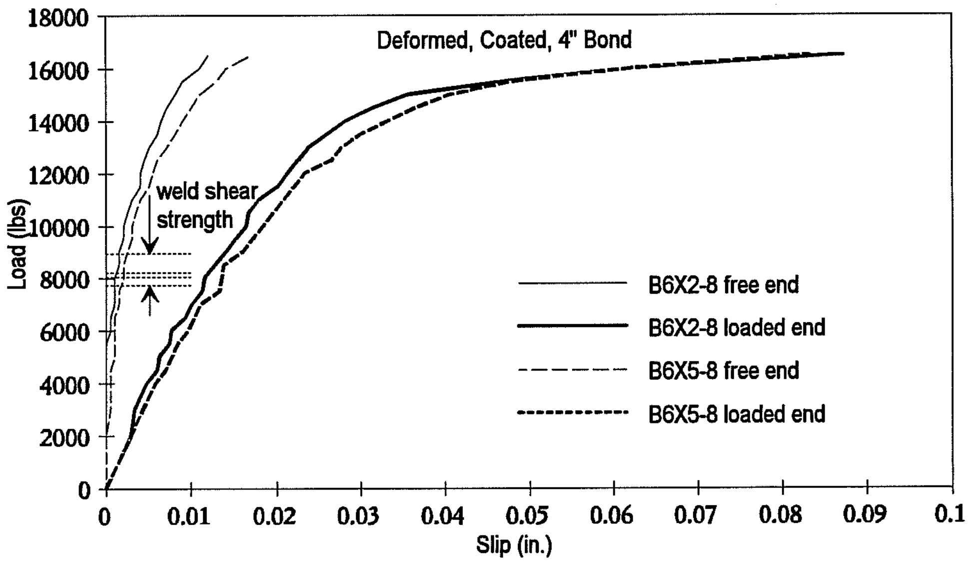


Figure 4.15: Pull-Out Specimens B6X2-8 and B6X5-8

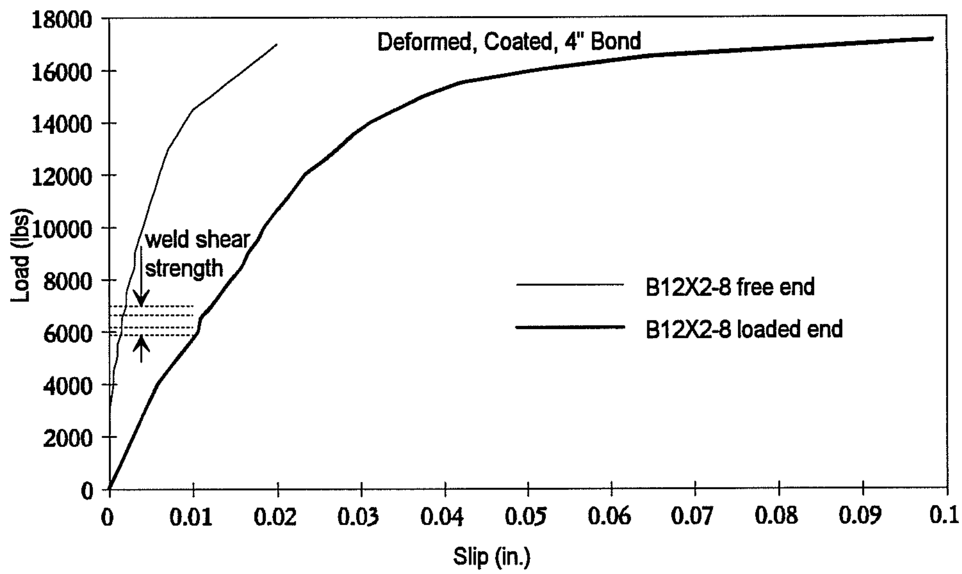


Figure 4.16: Pull-Out Specimen B12X2-8

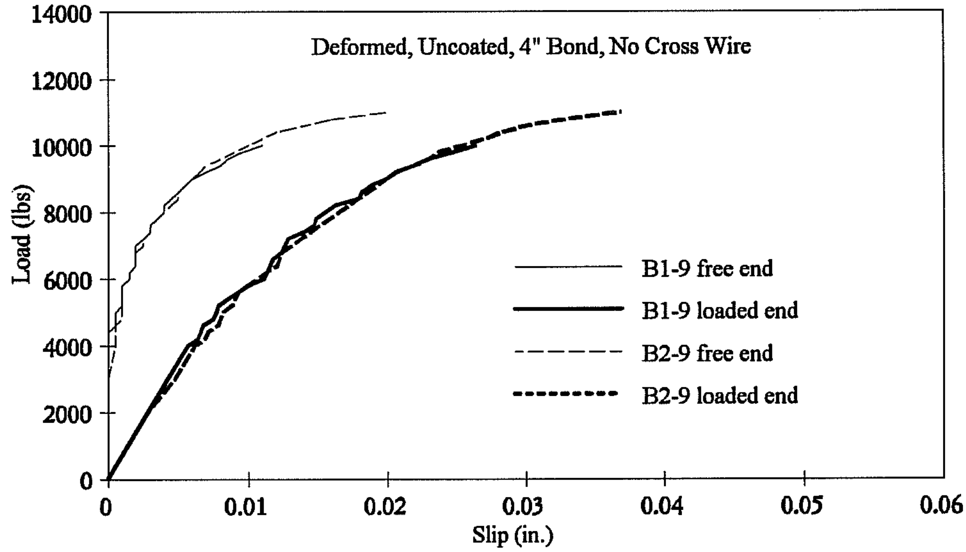


Figure 4.17: Pull-Out Specimens B1-9 and B2-9

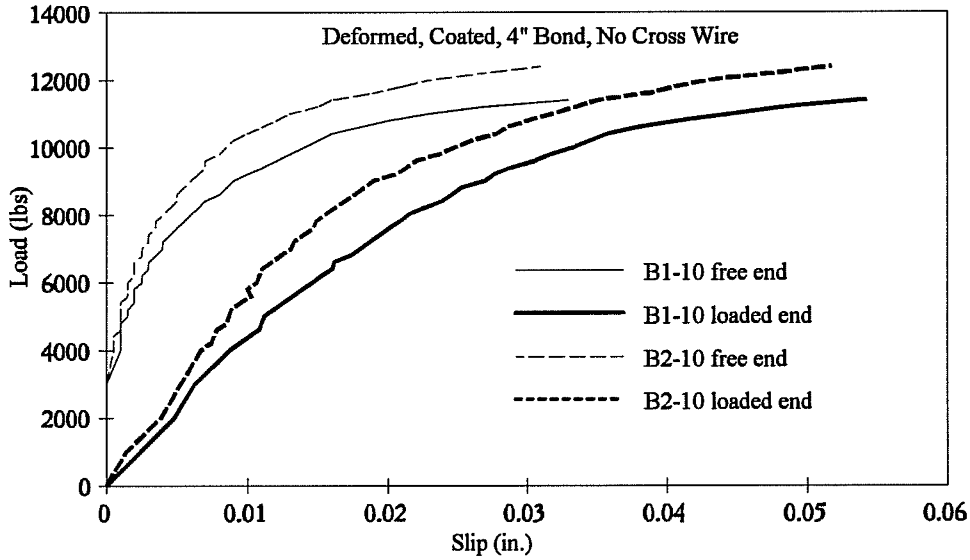


Figure 4.18: Pull-Out Specimens B1-10 and B2-10

4.3.2 Discussion of Test Results

The pull-out test data can be examined by comparing the loaded end and free end slip at various applied load to average weld shear strength ratios, where the average weld shear strength was determined from the same longitudinal wire used for the pull-out specimens. With this type of comparison the effect of variation in weld shear strength between different longitudinal wires in the same sheet of WWF could be minimized. Tables 4.1 through 4.4 show the difference in scatter for weld shear strength along a given wire and the averages for different wires. With this method the effect of coated or uncoated transverse wires and coated or uncoated longitudinal wires on the bond and development of the WWF could be more easily established.

Figures 4.19 through 4.26 show the loaded end slip and free end slip for specific pull-out specimens at various applied load to average weld shear strength ratios. Figure 4.19 shows the loaded end slip and Figure 4.20 shows the free end slip for the pull-out specimens that were constructed with welded plain wire fabric and no bond along the longitudinal wire. The specimens with a "-1" extension were uncoated and those with a "-2" extension were coated. Tables 4.5 and 4.6 give the values of the average loaded end and free end slip for the uncoated and coated specimens at the various ratios of applied load to average weld shear strength. In addition, these tables also show the range of values used to determine each average slip. From Tables 4.5 and 4.6 it can be seen that the average slip for the uncoated specimens was consistently larger than the slip for the coated specimens. Figures 4.21 and 4.22, and Tables 4.7 and 4.8 for the specimens constructed with welded deformed wire fabric and no bond, also show the same trend. The specimens with a "-3" extension were uncoated and those with a "-4" extension were coated.

From the results shown in Figures 4.19 through 4.22 and Tables 4.5 through 4.8, it can be concluded that epoxy-coating on the transverse wire does not detrimentally affect the weld shear performance of either welded plain wire fabric or welded deformed wire fabric. Even though both the coated welded plain wire fabric and coated welded deformed wire fabric showed less slip at both the loaded end and free end for the applied load to average weld shear strengths considered, it is believed that there is too much scatter in the data, as shown in Tables 4.5 through 4.8, to conclude that epoxy-coating on the cross wire alters the weld shear performance of WWF.

Figures 4.23 and 4.24 show the loaded end slip and free end slip for the pull-out specimens constructed with weld plain wire fabric and 4 in. of bond along the longitudinal wire. The specimens with a "-5" extension were uncoated and corresponded to the weld shear specimens with a "-1" extension, and the specimens with a "-6" extension were coated and corresponded to the weld shear specimens with a "-2" extension. Tables 4.9 and 4.10 give the values of the average loaded end and free end slip for the uncoated and coated specimens at the various ratios of applied load to average weld shear strength. In addition, these tables also show the range of values used to determine each average slip. From Tables 4.9 and 4.10 it can be seen that the average slip for the uncoated specimens was generally greater than or equal to the slip for the coated specimens. Figures 4.25 and 4.26, and Tables 4.11 and 4.12 for the pull-out specimens constructed with welded deformed wire fabric and 4 in. of bond, also show the same trend. The specimens with a "-7" extension were uncoated and corresponded to the weld shear specimens with a "-3" extension, and the specimens with a "-8" extension were coated and corresponded to the weld shear specimens with a "-4" extension.

From Figures 4.23 through 4.26, and Tables 4.9 through 4.12, it can be concluded that epoxy-coating on both the transverse wire and longitudinal wire does not detrimentally affect the bond characteristics of either welded plain wire fabric or welded deformed wire fabric. Even though the results tend to show that the average slip for the coated specimens was less than the average slip for the uncoated specimens, it is believed that there is too much scatter in the data to conclude that epoxy-coating on the longitudinal wire alters the bond performance of WWF.

Figure 4.27 shows both the loaded end slip and free end slip at various applied loads for the four pull-out specimens that were constructed with deformed wires, 4 in. of bond, and no cross wire. The specimens with a "-9" extension were uncoated and those with a "-10" extension were coated. Tables 4.13 and 4.14 give the values of the average loaded end and free end slip for the uncoated and coated specimens at the various load levels. In addition, these tables also show the range of values used to determine each average slip. From Table 4.13 it can be seen that the largest difference between the uncoated and coated average free end slip was 0.003 in. at a load of 10000 lbs. At this load the average slip of the coated wires was 10% higher than the average slip of the uncoated wires. It is believed that the differences in slip shown in Figure 4.27, and Tables 4.13 and 4.14 are so small that epoxy-coating on the longitudinal wire did not alter the bond performance of WWF.

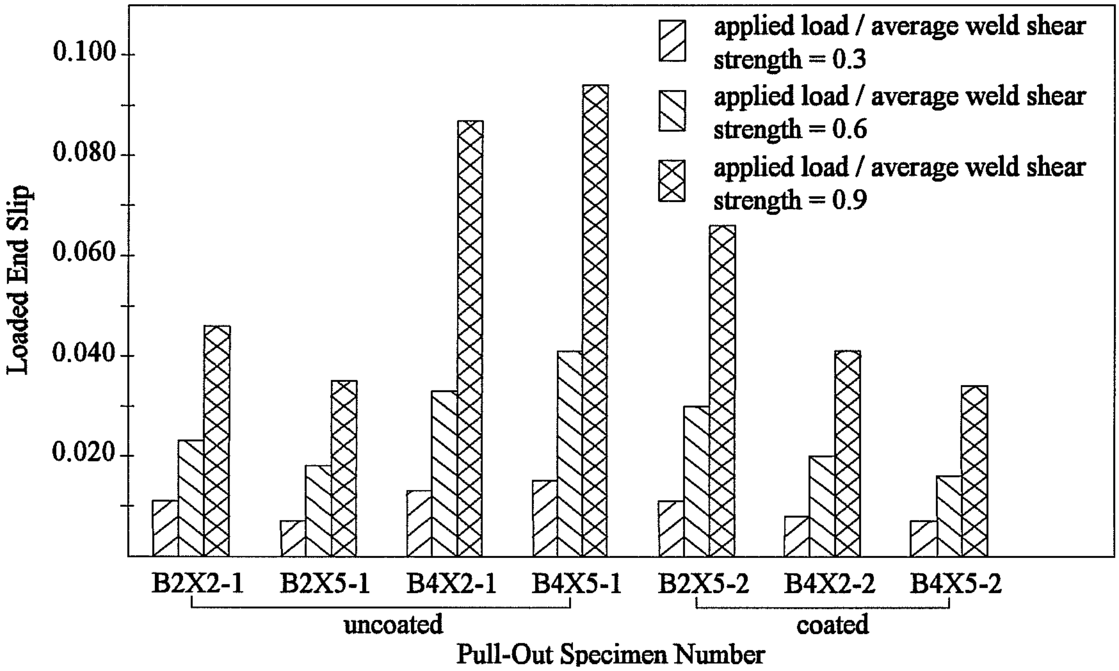


Figure 4.19: Loaded End Slip for Welded Plain Wire Fabric Pull-Out Specimens With No Bond

Table 4.5 Average Loaded End Slip for Uncoated and Coated Welded Plain Wire Fabric Pull-Out Specimens With No Bond

Applied Load / Average Weld Shear Strength	Uncoated Wire Average Slip (in.)	Range of Slip Values for Uncoated Wires	Coated Wire Average Slip (in.)	Range of Slip Values for Coated Wires
0.3	0.012	0.009 to 0.015	0.009	0.007 to 0.011
0.6	0.029	0.018 to 0.041	0.022	0.016 to 0.030
0.9	0.066	0.035 to 0.094	0.047	0.034 to 0.066

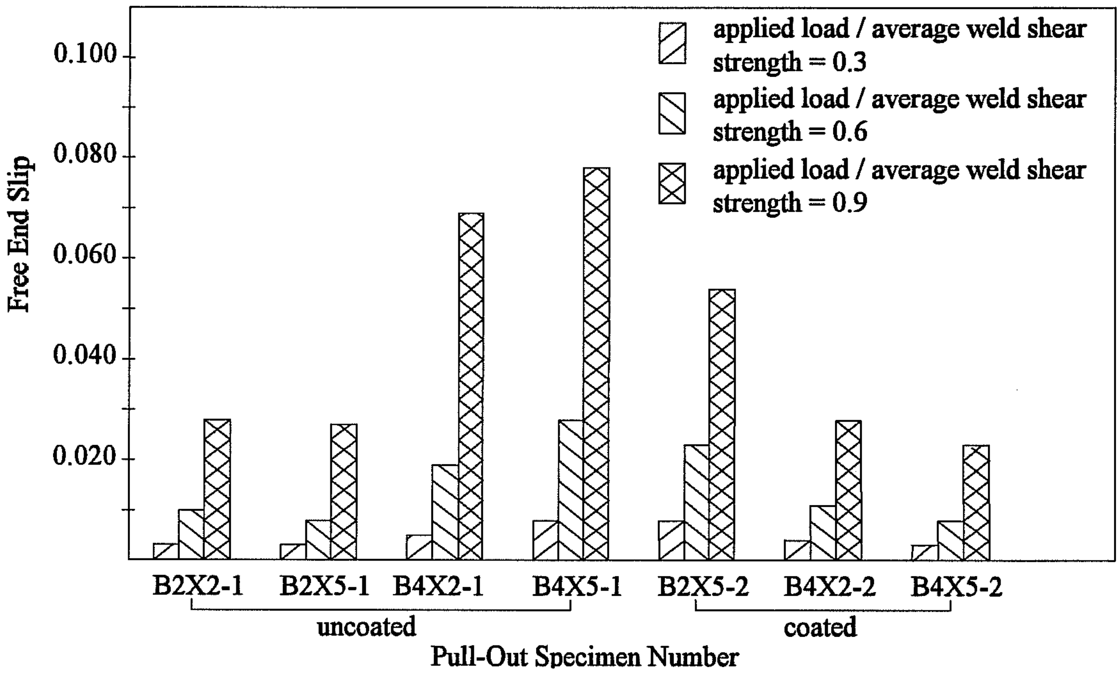


Figure 4.20: Free End Slip for Welded Plain Wire Fabric Pull-Out Specimens With No Bond

Table 4.6: Average Free End Slip for Uncoated and Coated Welded Plain Wire Fabric Pull-Out Specimens With No Bond

Applied Load / Average Weld Shear Strength	Uncoated Wire Average Slip (in.)	Range of Slip Values for Uncoated Wires	Coated Wire Average Slip (in.)	Range of Slip Values for Coated Wires
0.3	0.005	0.003 to 0.008	0.005	0.003 to 0.008
0.6	0.016	0.008 to 0.028	0.014	0.008 to 0.023
0.9	0.051	0.027 to 0.078	0.035	0.023 to 0.054

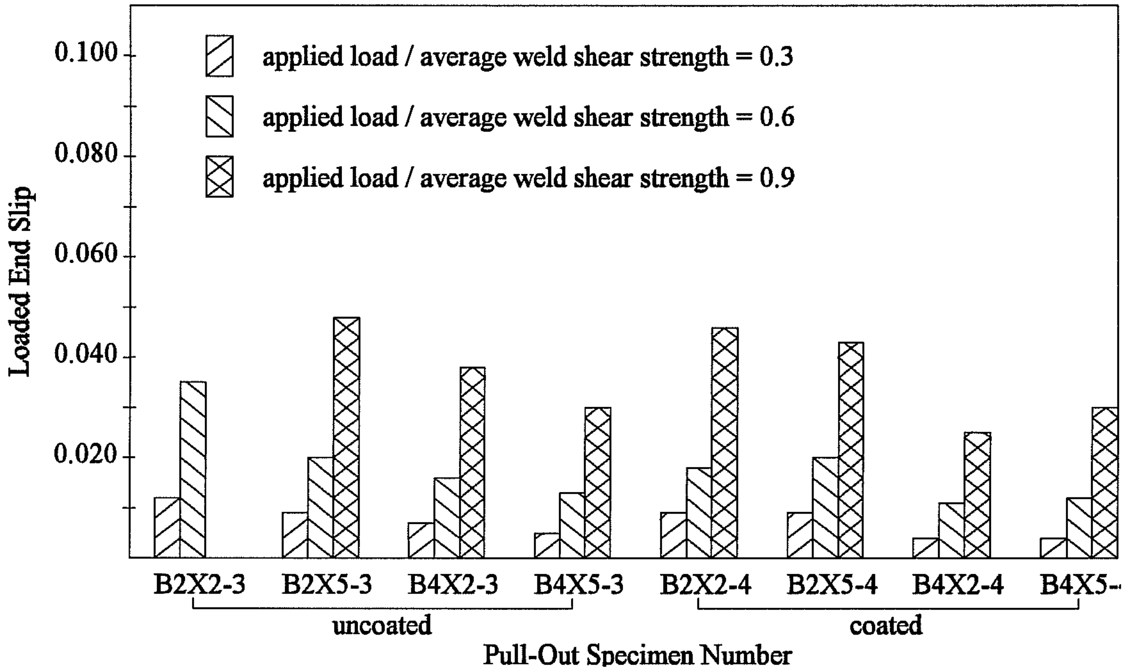


Figure 4.21: Loaded End Slip for Welded Deformed Wire Fabric Pull-Out Specimens With No Bond

Table 4.7: Average Loaded End Slip for Uncoated and Coated Welded Deformed Wire Fabric Pull-Out Specimens With No Bond

Applied Load / Average Weld Shear Strength	Uncoated Wire Average Slip (in.)	Range of Slip Values for Uncoated Wires	Coated Wire Average Slip (in.)	Range of Slip Values for Coated Wires
0.3	0.009	0.005 to 0.014	0.007	0.004 to 0.009
0.6	0.021	0.013 to 0.035	0.015	0.011 to 0.020
0.9	0.039	0.030 to 0.048	0.036	0.025 to 0.046

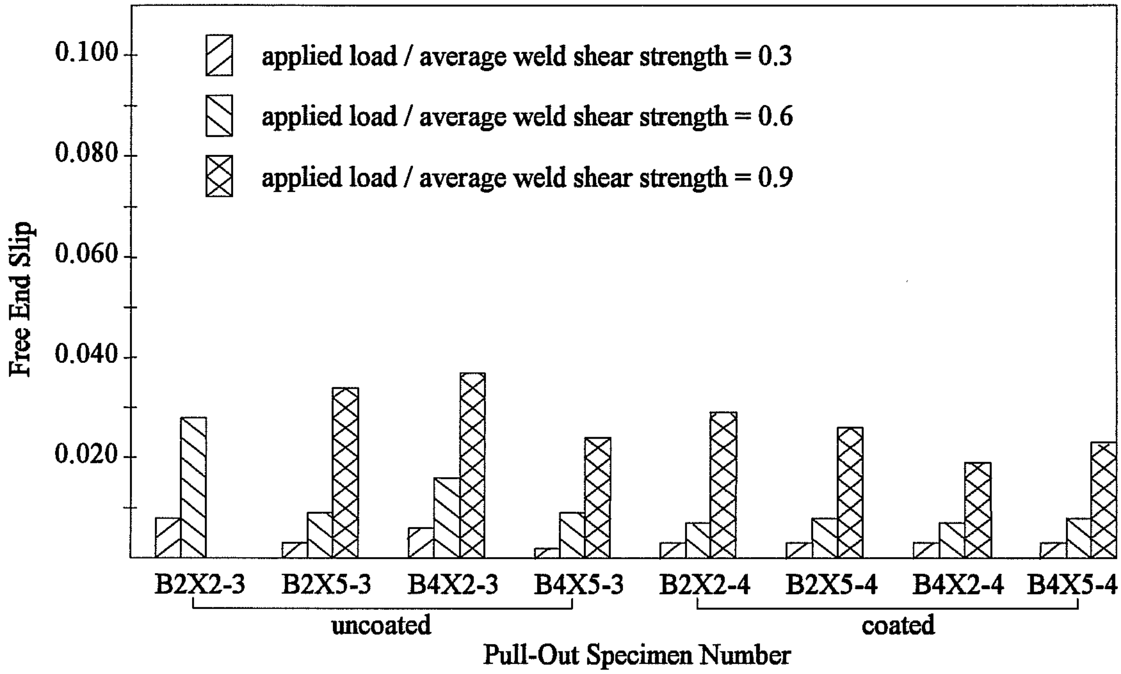


Figure 4.22: Free End Slip for Welded Deformed Wire Fabric Pull-Out Specimens With No Bond

Table 4.8: Average Free End Slip for Uncoated and Coated Welded Deformed Wire Fabric Pull-Out Specimens With No Bond

Applied Load / Average Weld Shear Strength	Uncoated Wire Average Slip (in.)	Range of Slip Values for Uncoated Wires	Coated Wire Average Slip (in.)	Range of Slip Values for Coated Wires
0.3	0.005	0.002 to 0.008	0.003	0.003 to 0.003
0.6	0.016	0.009 to 0.028	0.008	0.007 to 0.008
0.9	0.032	0.024 to 0.037	0.024	0.019 to 0.029

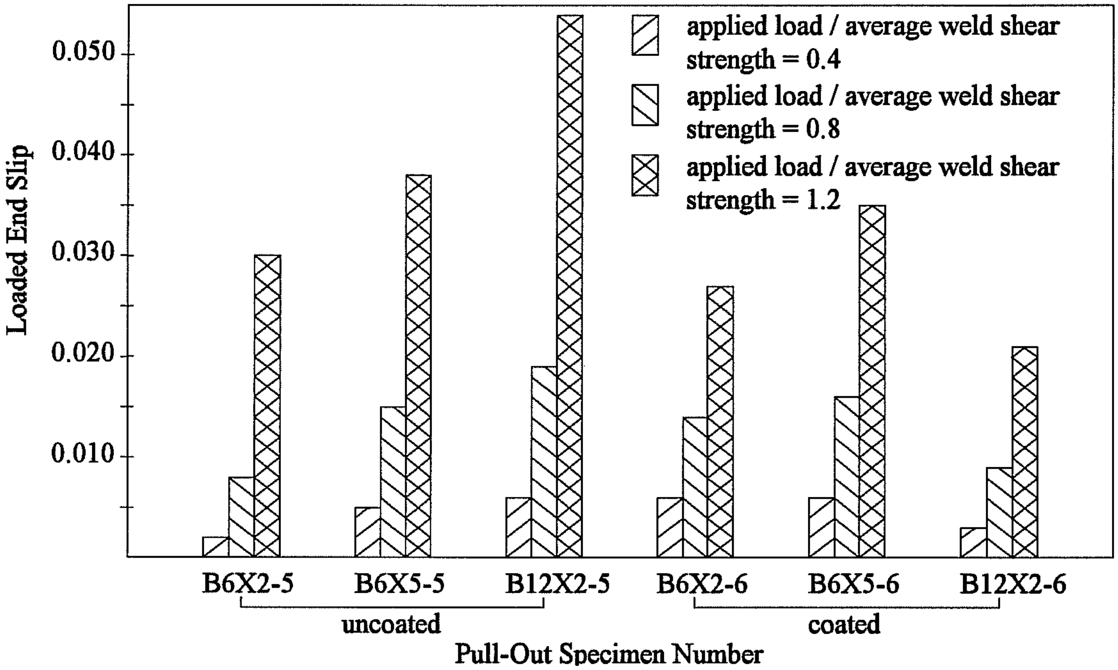


Figure 4.23: Loaded End Slip for Welded Plain Wire Fabric Pull-Out Specimens With 4 in. of Bond

Table 4.9: Average Loaded End Slip for Uncoated and Coated Welded Plain Wire Fabric Pull-Out Specimens With 4" of Bond

Applied Load / Average Weld Shear Strength	Uncoated Wire Average Slip (in.)	Range of Slip Values for Uncoated Wires	Coated Wire Average Slip (in.)	Range of Slip Values for Coated Wires
0.4	0.004	0.002 to 0.006	0.005	0.003 to 0.006
0.8	0.014	0.008 to 0.019	0.013	0.009 to 0.016
1.2	0.042	0.030 to 0.058	0.028	0.021 to 0.035

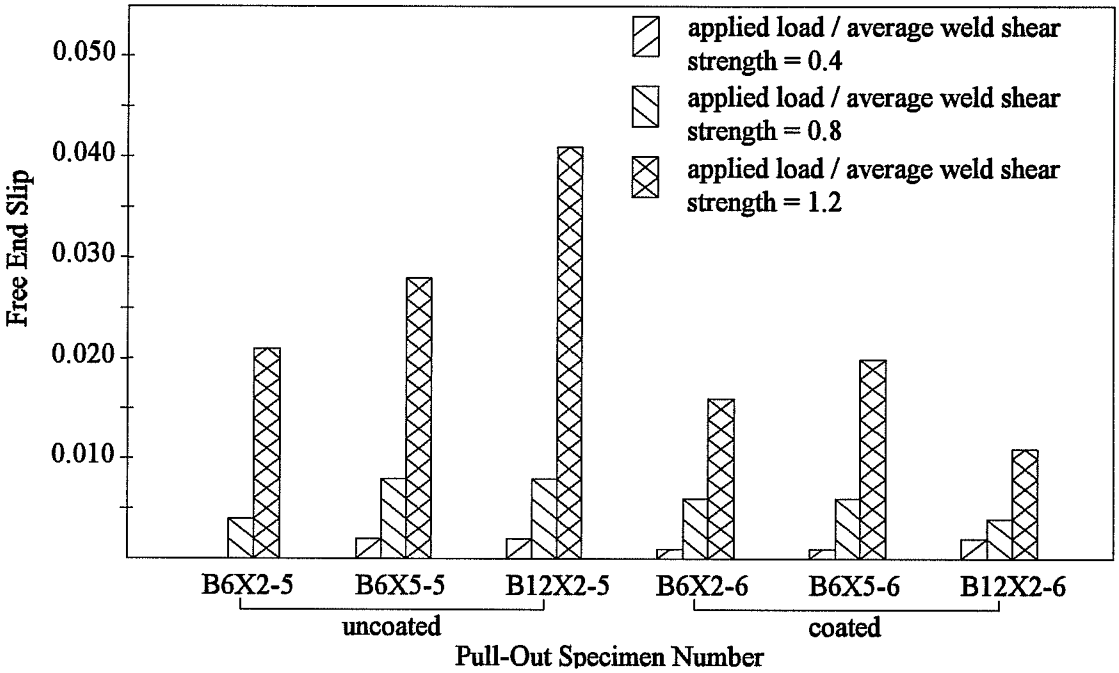


Figure 4.24: Free End Slip for Welded Plain Wire Fabric Pull-Out Specimens With 4 in. of Bond

Table 4.10: Average Free End Slip for Uncoated and Coated Welded Plain Wire Fabric Pull-Out Specimens With 4 in. of Bond

Applied Load / Average Weld Shear Strength	Uncoated Wire Average Slip (in.)	Range of Slip Values for Uncoated Wires	Coated Wire Average Slip (in.)	Range of Slip Values for Coated Wires
0.4	0.001	0.000 to 0.002	0.001	0.001 to 0.002
0.8	0.007	0.004 to 0.008	0.005	0.004 to 0.006
1.2	0.030	0.021 to 0.041	0.016	0.011 to 0.020

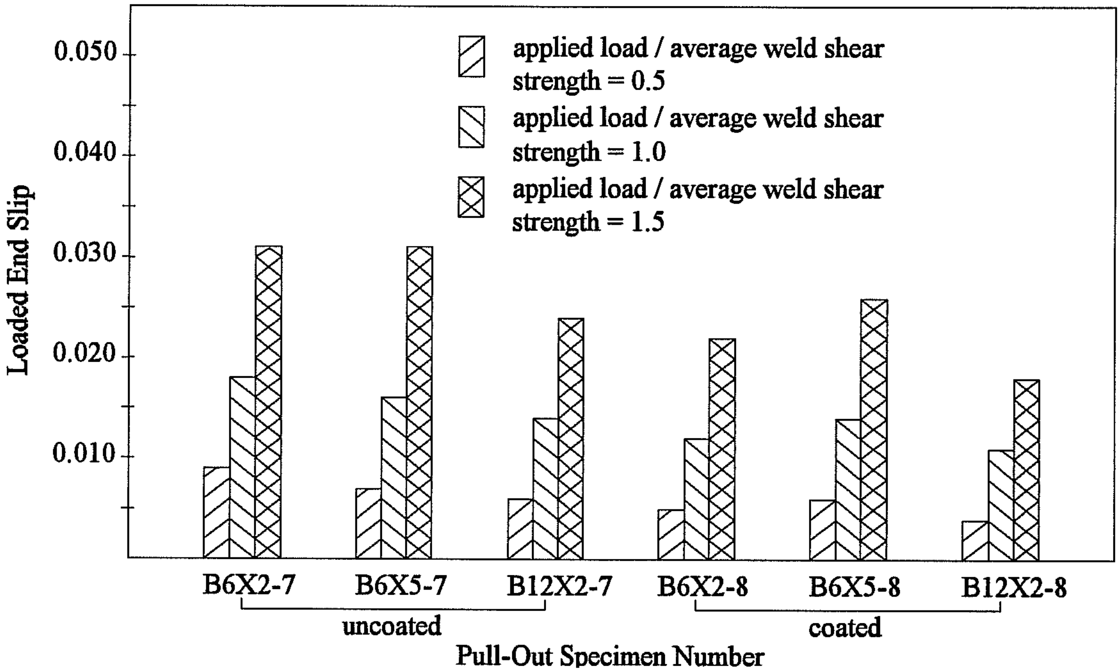


Figure 4.25: Loaded End Slip for Welded Deformed Wire Fabric Pull-Out Specimens With 4 in. of Bond

Table 4.11: Average Loaded End Slip for Uncoated and Coated Welded Deformed Wire Fabric Pull-Out Specimens With 4 in. of Bond

Applied Load / Average Weld Shear Strength	Uncoated Wire Average Slip (in.)	Range of Slip Values for Uncoated Wires	Coated Wire Average Slip (in.)	Range of Slip Values for Coated Wires
0.5	0.007	0.005 to 0.009	0.005	0.004 to 0.006
1.0	0.016	0.012 to 0.018	0.012	0.011 to 0.014
1.5	0.029	0.022 to 0.031	0.022	0.018 to 0.026

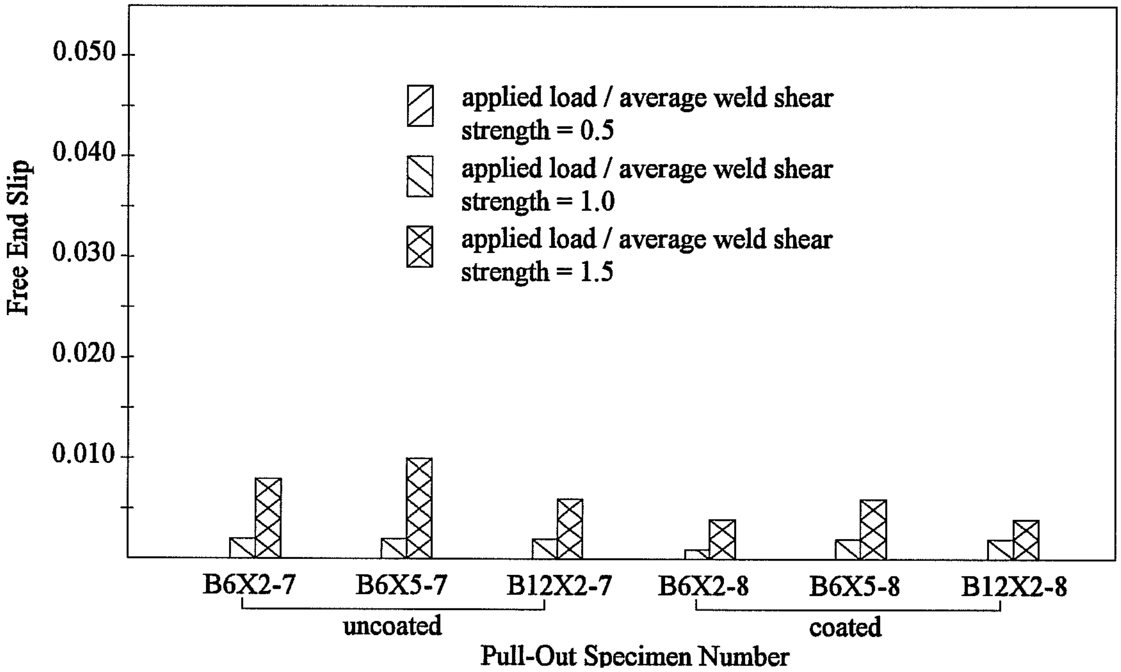


Figure 4.26: Free End Slip for Welded Deformed Wire Fabric Pull-Out Specimens With 4 in. of Bond

Table 4.12: Average Free End Slip for Uncoated and Coated Welded Deformed Wire Fabric Pull-Out Specimens With 4 in. of Bond

Applied Load / Average Weld Shear Strength	Uncoated Wire Average Slip (in.)	Range of Slip Values for Uncoated Wires	Coated Wire Average Slip (in.)	Range of Slip Values for Coated Wires
0.5	0.000	0.000 to 0.000	0.000	0.000 to 0.000
1.0	0.002	0.002 to 0.002	0.002	0.001 to 0.002
1.5	0.008	0.006 to 0.010	0.005	0.004 to 0.006

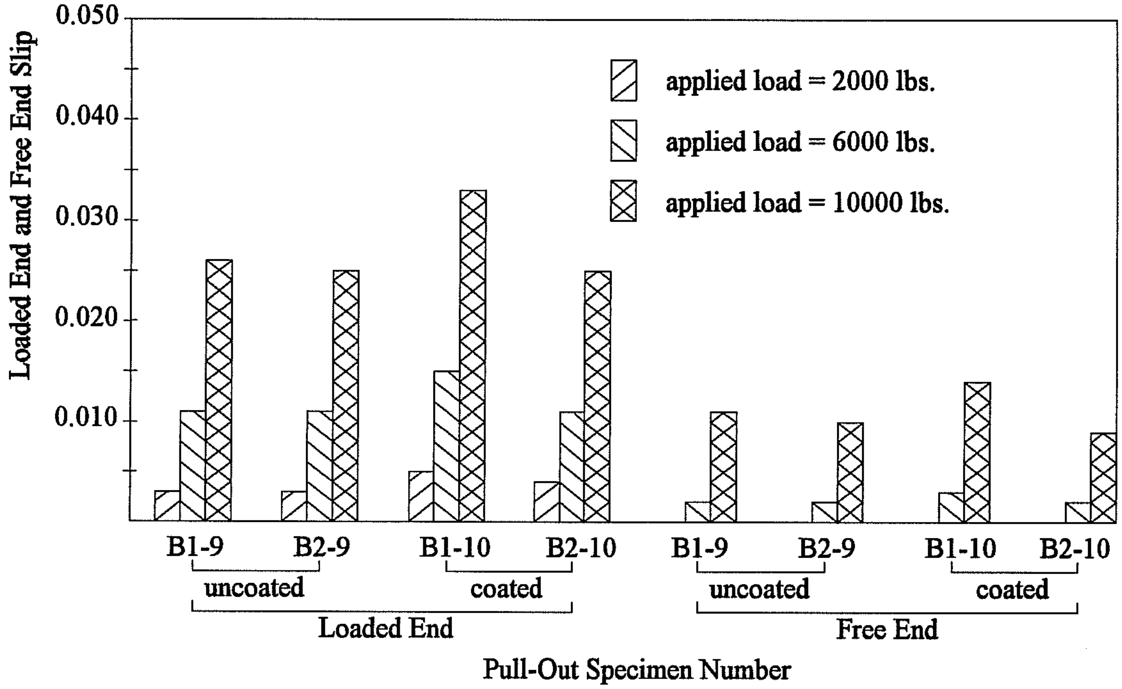


Figure 4.27: Loaded End and Free End Slip for Deformed Wire Pull-Out Specimens With 4 in. of Bond

Table 4.13: Average Loaded End Slip for Uncoated and Coated Deformed Wire Pull-Out Specimens With 4 in. of Bond

Applied Load (lbs)	Uncoated Wire Average Slip (in.)	Range of Slip Values for Uncoated Wires	Coated Wire Average Slip (in.)	Range of Slip Values for Coated Wires
2000	0.003	0.003 to 0.003	0.005	0.004 to 0.005
6000	0.011	0.011 to 0.011	0.013	0.011 to 0.015
10000	0.026	0.026 to 0.026	0.029	0.025 to 0.033

Table 4.14: Average Free End Slip for Uncoated and Coated Deformed Wire Pull-Out Specimens With 4 in. of Bond

Applied Load (lbs)	Uncoated Wire Average Slip (in.)	Range of Slip Values for Uncoated Wires	Coated Wire Average Slip (in.)	Range of Slip Values for Coated Wires
2000	0.000	0.000 to 0.000	0.000	0.000 to 0.000
6000	0.002	0.002 to 0.002	0.003	0.002 to 0.003
10000	0.011	0.010 to 0.011	0.012	0.009 to 0.014

4.4 One-Way Slab Tests

4.4.1 Presentation of Test Results

The results of the slab tests are presented in terms of load-deflection relationships, crack widths, and crack spacing at selected load levels. Figures 4.28 through 4.33 show plots of the ratio of the test moment to the ultimate moment capacity versus an equivalent beam center span deflection for the 12 one-way slab tests. The equivalent beam center span deflection was determined by adding the measured center span deflection to the average of the two end span deflections. This was done so that the slab deflection represents an element simply supported at the ends. The ultimate moment capacity for each of the slabs was calculated with the following equation by using the concrete strength at the time of testing and the measured wire yield strength as defined by ACI 318-89 and given in Table 3.5:

$$M_{ult.} = A_s f_y \left(d - \frac{A_s f_y}{2\,(0.85) f_c' b}\right) \qquad (18)$$

where: A_s = *total area of longitudinal steel, in*^{2}.
f_y = *measured steel yield strength, ksi.*
d = *distance from extreme compression fiber to tensile steel, in.*
f_c' = *measured concrete compressive strength, ksi.*
b = *width of slab, in.*

The calculated ultimate moment capacities are given in Table 4.15.

The results for the slab specimens are presented in Figures 4.28 through 4.33. In each figure, both specimens were identical except that one was constructed with uncoated WWF and one with coated WWF. As can be seen in Figures 4.32 and 4.33, specimens UW20-2, CW20-2, UD20-2, and CD20-2 failed before reaching the yield strength of the slabs. The failure occurred in the splice region and a photograph of a typical failure can be seen in Figure 4.34. This figure shows that failure occurred by splitting of the concrete between the outer transverse wires in the lap and by pulling out the longitudinal wires extending beyond the last transverse wire.

Table 4.16 shows the crack data for slabs UW4-1, CW4-1, UD4-1 and CD4-1. Both the average crack spacing in the constant moment region and the maximum crack width at the extreme tensile fiber are shown for a number of different applied moments. In addition, the stress in the steel at these applied moments is also shown. The steel stress was calculated by assuming a triangular concrete stress distribution and by assuming that the steel had not yet yielded. An applied moment of 108 in-kips is greater than the yield moment and thus the stress in the steel is indicated to be at or greater than the yield stress. The average crack spacing was calculated by dividing the length of the constant moment region by the number of cracks extending across the slab in this region.

The same average crack spacing and maximum crack width data is shown in Table 4.17 for slabs UW20-1, CW20-1, UD20-1, and CD20-1 and in Table 4.18 for slabs UW20-2, CW20-2, UD20-2, and CD20-2.

Table 4.15: Calculated Ultimate Moment Capacities for One-Way Slab Specimens

Specimen Number	Calculated Ultimate Moment Capacity (in-kips)
UW4-1, CW4-1	99
UD4-1, CD4-1	103
UW20-1, CW20-1	860
UW20-2, CW20-2	746
UD20-1, CD20-1	932
UD20-2, CD20-2	703

Table 4.16: Crack Spacing and Width for Slabs UW4-1, CW4-1, UD4-1, and CD4-1

Specimen Number	Applied Moment = 79.2 in-kips f_s = 67.9 ksi		Applied Moment = 108 in-kips $f_s \geq f_y$	
	Average Crack Spacing (in.)	Max. Crack Width ($x10^{-3}$ in.)	Average Crack Spacing (in.)	Max. Crack Width ($x10^{-3}$ in.)
UW4-1	36	30	9.0	60
CW4-1	14	40	10	50
UD4-1	14	16	6.0	25
CD4-1	10	26	6.5	30

Table 4.17: Crack Spacing and Width for Slabs UW20-1, CW20-1, UD20-1, and CD20-1

Specimen Number	Applied Moment = 360 in-kips f_s = 31.0 ksi		Applied Moment = 540 in-kips f_s = 46.5 ksi		Applied Moment = 792 in-kips f_s = 68.1 ksi	
	Average Crack Spacing (in.)	Max. Crack Width ($\times10^{-3}$ in.)	Average Crack Spacing (in.)	Max. Crack Width ($\times10^{-3}$ in.)	Average Crack Spacing (in.)	Max. Crack Width ($\times10^{-3}$ in.)
UW20-1	9.0	13	6.5	25	6.5	50
CW20-1	8.0	20	6.5	30	6.5	50
UD20-1	6.5	3	5.1	13	4.8	25
CD20-1	5.1	4	5.1	10	4.8	20

Table 4.18: Crack Spacing and Width for Slabs UW20-2, CW20-2, UD20-2, and CD20-2

Specimen Number	Applied Moment = 294 in-kips f_s = 31.1 ksi		Applied Moment = 420 in-kips f_s = 44.4 ksi	
	Average Crack Spacing (in.)	Max. Crack Width ($\times10^{-3}$ in.)	Average Crack Spacing (in.)	Max. Crack Width ($\times10^{-3}$ in.)
UW20-2	5.5	9	4.6	20
CW20-2	6.0	10	*	*
UD20-2	4.3	7	4.0	20
CD20-2	5.0	9	4.3	16

* Specimen failed before this applied moment was reached.

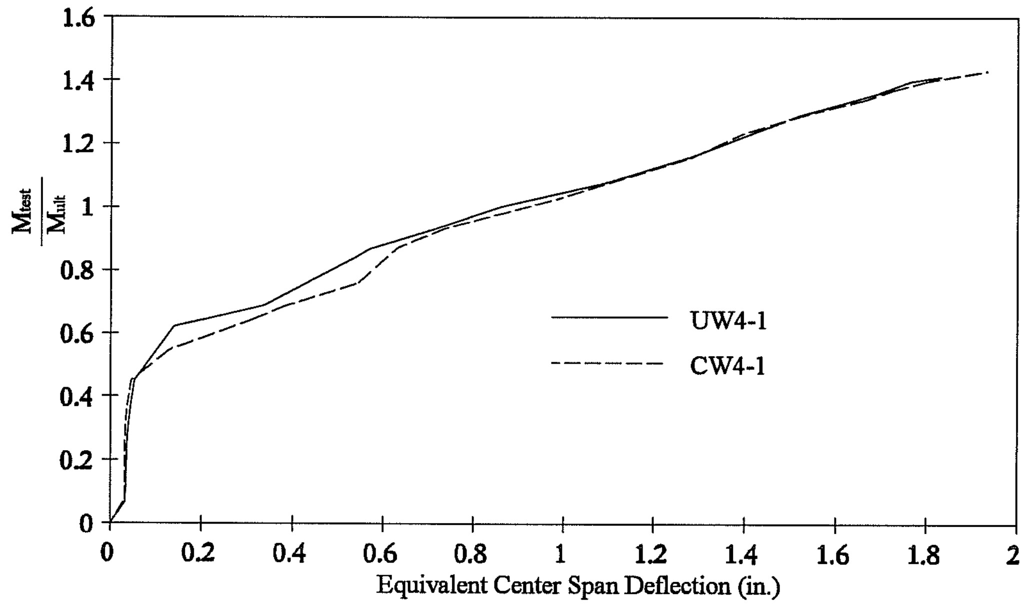


Figure 4.28: One-Way Slabs UW4-1 and CW4-1

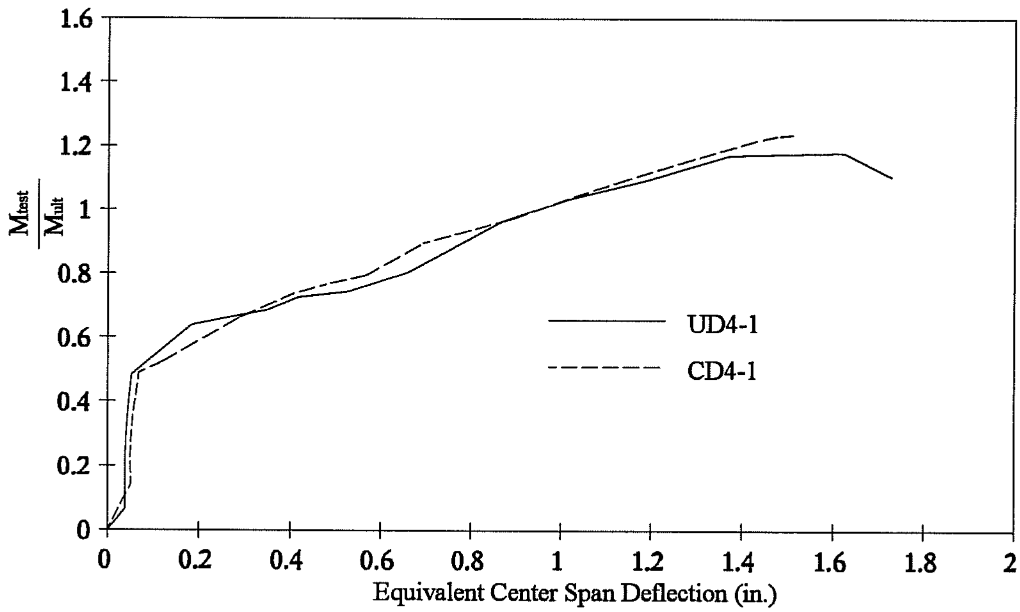


Figure 4.29: One-Way Slabs UD4-1 and CD4-1

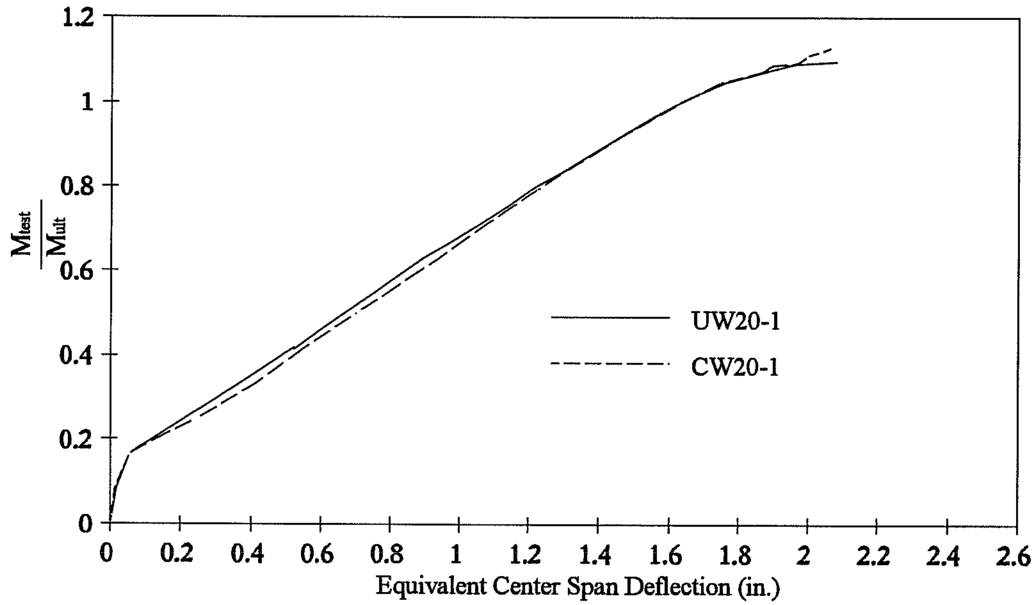


Figure 4.30: One-Way Slabs UW20-1 and CW20-1

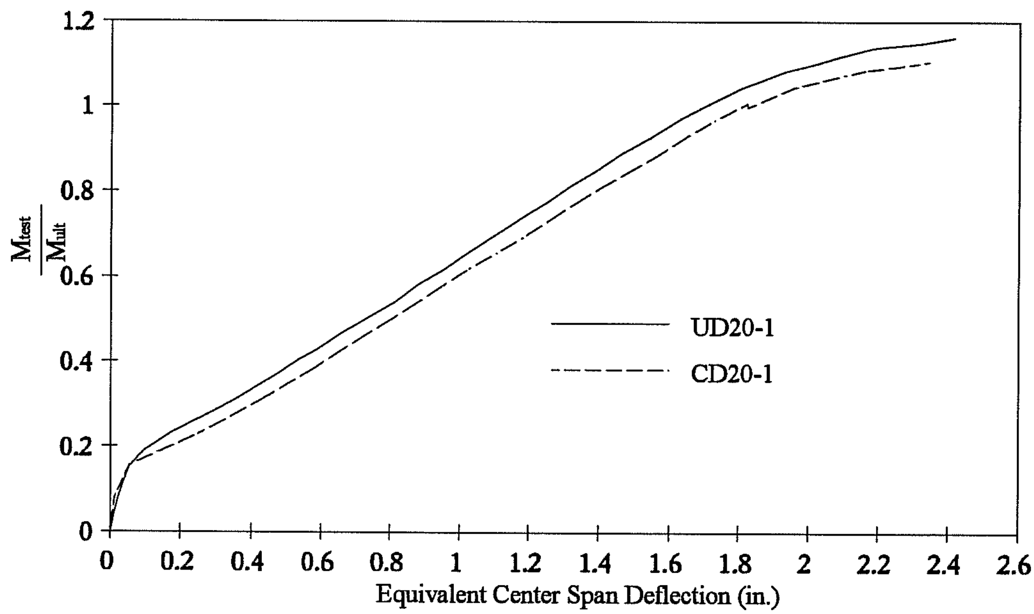


Figure 4.31: One-Way Slabs UD20-1 and CD20-1

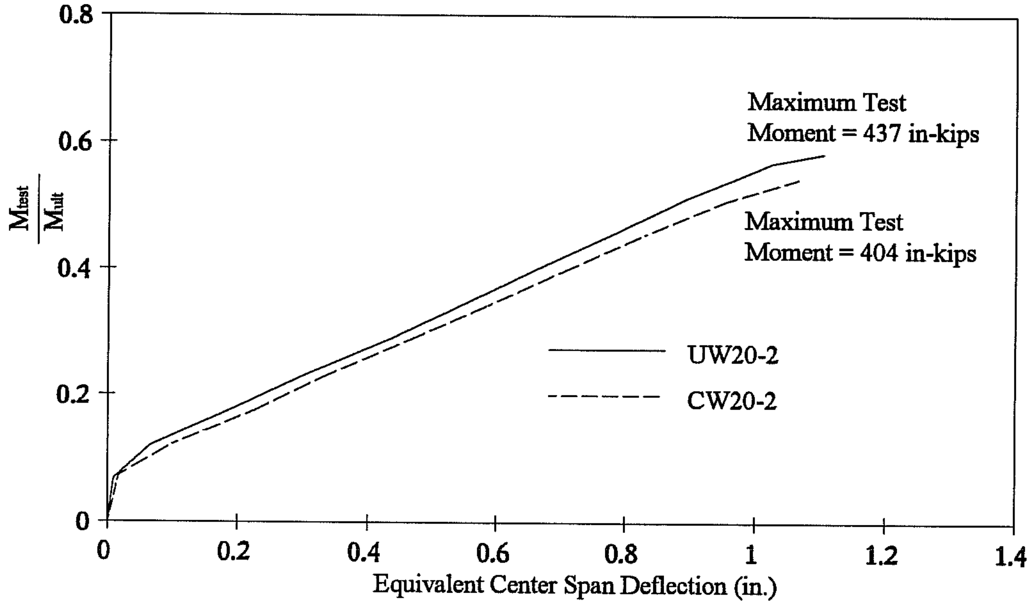


Figure 4.32: One-Way Slabs UW20-2 and CW20-2

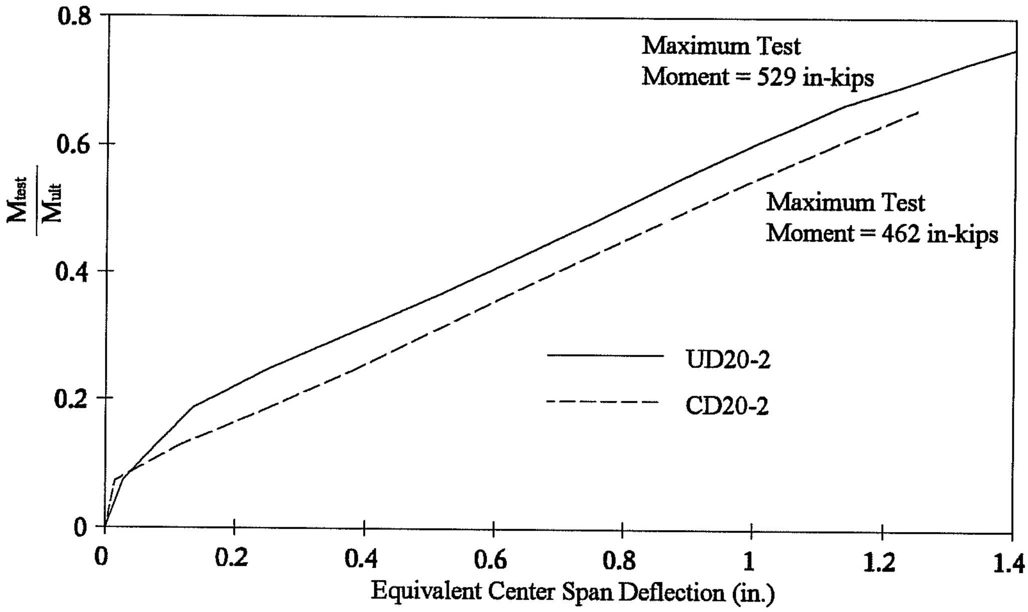


Figure 4.33: One-Way Slabs UD20-2 and CD20-2

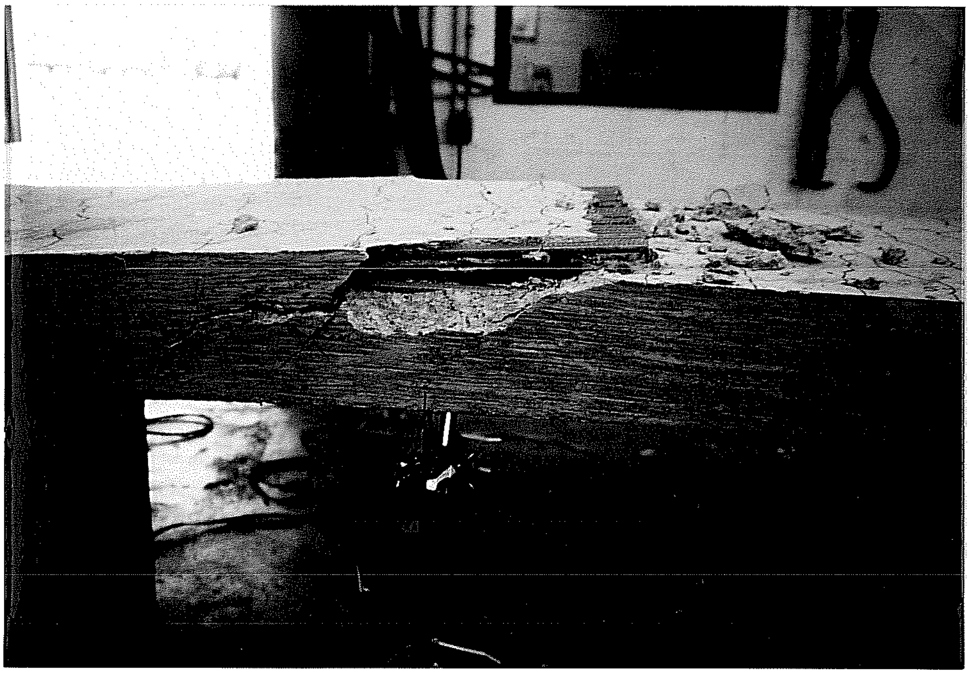


Figure 4.34: Photograph of Typical Failure for Slabs UW20-2, CW20-2, UD20-2, and CD20-2

4.4.2 Discussion of Test Results

4.4.2.1 Load Versus Deflection Curves

The results from the one-way slab tests can be used to support the conclusions developed from the pull-out tests. Slabs UW4-1, CW4-1, UD4-1, CD4-1, UW20-1, CW20-1, UD20-1, and CD20-1 were all able to develop the full yield strength of the longitudinal wires before a splice failure occurred. In fact, none of these slabs failed in the splice region, and the tests were concluded because it was apparent that the longitudinal wires were yielding. As can be seen in Figures 4.28 through 4.31 each pair of coated and uncoated slabs behaved nearly identically under load. This reinforces the conclusion that there is no difference in bond characteristics between epoxy-coated and non epoxy-coated WWF. In the case of coated versus uncoated reinforcing bars, the load-deflection curves were quite different as can be seen in Figure 2.3.

Slabs UW20-2, CW20-2, UD20-2, and CD20-2, which failed in the splice region prior to reaching the yield strength of the slabs, can be used to examine the effects of epoxy-coating on the bond of WWF when a bond failure occurs. The other slab specimens did not reach a splice failure so the maximum bond strength was not reached.

Figure 4.32 shows the ratio of test moment to ultimate moment capacity versus the equivalent center span deflection for slabs UW20-2 and CW20-2. Both of these slabs behaved nearly the same with only a 7.5% difference in their maximum strengths. After failure, the average cover on the longitudinal wires was measured and it was found that the average cover for slab UW20-2 was 7/16 in. and the average cover for slab CW20-2 was ½ in. These average covers are less than the minimum ¾ in. cover allowed by ACI 318-89 and a lower maximum slab strength may have resulted because of the smaller cover. Even though the small cover may have caused a lower slab strength, this is not seen as a reason for variations in strength between the two slabs because both had nearly identical amounts of cover.

Figure 4.33 shows the ratio of the test moment to ultimate moment capacity versus the equivalent center span deflection for slabs UD20-2 and CD20-2. As can be seen in this figure, slab CD20-2 had approximately 15% larger deflections for each test moment to ultimate moment capacity ratio above the cracking load. Even though the deflections were larger for CD20-2, the difference was small and the stiffness of the two slabs appeared to be the same. Slab UD20-2 reached a 13% higher maximum moment than slab CD20-2, but this can be accounted for by differences in average cover between the two slabs. The average cover for specimen UD20-2 was 11/16 in. while that for CD20-2 was ½ in. Using an equation proposed by Jirsa, Lutz, and Gergely[18], shown below, this additional 3/16 in. cover could result in a 20% increase in the bond stress. This could explain why slab CD20-2 failed at a lower moment than slab UD20-2.

$$u = [1.2 + \frac{3\,C}{d_b} + \frac{50\,d_b}{l}]\sqrt{f_c'} \tag{19}$$

where: u = *bond stress, psi.*
C = *amount of cover, in.*
d_b = *diameter of bar, in.*
l = *splice length, in.*
f_c' = *concrete strength, psi.*

The test results from slabs UW20-2, CW20-2, UD20-2, and CD20-2 again tend to confirm the conclusion that epoxy-coating does not detrimentally affect the bond characteristics of either welded plain wire fabric and welded deformed wire fabric.

4.4.2.2 Cracking Patterns

The effects of epoxy-coating on the bond of WWF were also evaluated using the cracking patterns for the one-way slab specimens. Tables 4.16 through 4.18 present the average crack spacing and maximum crack width at different values of steel stress for each of the one-way slab specimens.

The crack data for slabs UW4-1, CW4-1, UD4-1, and CD4-1 are given in Table 4.16. For the steel stress of 67.9 ksi, there appears to be a large difference in the average crack spacing and maximum crack width between the slabs reinforced with coated and uncoated welded plain wire fabric and welded deformed wire fabric. This difference was most likely due to the low percentage of reinforcement and the relatively small difference between the cracking moment and ultimate moment capacity for these slabs. The applied moment at which this data was taken was very close to the cracking moment of the slabs, therefore the concrete may not have been fully cracked causing large differences in the steel stress along the length of the constant moment region. At a higher steel stress, above the yield stress, the average crack spacing and maximum crack width corresponded very closely for each pair of slabs.

Tables 4.17 and 4.18 show the average crack spacing and maximum crack size for slabs UW20-1, CW20-1, UD20-1, CD20-1 and UW20-2, CW20-2, UD20-2, CD20-2 respectively. At each steel stress there appears to be a very close correlation in the average crack spacing and maximum crack width between the coated and uncoated slabs reinforced with either welded plain wire fabric or welded deformed wire fabric. This indicates that there is little or no difference in the crack width and spacing between the slabs reinforced with coated and uncoated WWF. In the case of coated versus uncoated reinforcing bars, the crack widths were very different as shown in Figure 2.4.

One aspect of the crack data in Tables 4.16 through 4.18 that should be pointed out is the difference in cracking characteristics between the slabs reinforced with welded plain wire fabric and those reinforced with welded deformed wire fabric. Welded deformed wire fabric tended to do a better job of limiting the maximum crack width by providing better bond along the bar and reducing the average crack spacing. The slabs reinforced with welded plain wire fabric showed larger cracks spaced further apart, while the slabs reinforced with welded deformed wire fabric had more cracks spaced closer together and smaller in width.

The crack widths measured at various values of steel stress for each of the slabs were compared to the values obtained using the equations presented in Chapter 2. Table 4.19 shows the measured and estimated crack widths for the slabs reinforced with welded plain wire fabric. As can be seen in this table, the maximum crack widths estimated by Equation 17 were typically smaller than the measured maximum crack widths. This same general result was found for the slabs reinforced with welded deformed wire fabric as shown in Table 4.20. The estimated maximum crack widths in this table were determined using Equation 15. In general, the crack widths were underestimated using Equations 15 and 17 except at very low steel stress values where the estimates were reasonably close.

Table 4.19: Estimated Maximum Crack Width for Slabs Reinforced With Welded Plain Wire Fabric

Specimen Number	Steel Stress (ksi)	Measured Max. Crack Width (in. x 10^{-3})	Estimated Max. Crack Width* (in. x 10^{-3})
UW4-1	67.9	30	22
CW4-1	67.9	40	22
UW20-1	31.0	13	12
	46.5	25	18
	68.1	50	27
CW20-1	31.0	20	12
	46.5	30	18
	68.1	50	27
UW20-2	31.1	9	11
	44.4	20	16
CW20-2	31.1	10	11

*Estimated Maximum Crack Width by Equation 17

Table 4.20: Estimated Maximum Crack Width for Slabs Reinforced With Welded Deformed Wire Fabric

Specimen Number	Steel Stress (ksi)	Measured Max. Crack Width (in. x 10^{-3})	Estimated Max. Crack Width* (in. x 10^{-3})
UD4-1	67.9	16	11
CD4-1	67.9	26	11
UD20-1	31.0	3	6
	46.5	13	8
	68.1	25	12
CD20-1	31.0	4	6
	46.5	10	8
	68.1	20	12
UD20-2	31.1	7	5
	44.4	20	7
CD20-2	31.1	9	5
	44.4	16	7

*Estimated Maximum Crack Width by Equation 15

CHAPTER 5

DESIGN CONSIDERATIONS

5.1 ACI 318-89 Requirements for Epoxy Coated WWF

The design requirements currently included in ACI 318-89 for the development of welded deformed wire fabric in tension require that the basic development length, defined in Section 12.7, be multiplied by the modification factors from Sections 12.2.3 through 12.2.5. The modification factor for epoxy-coated reinforcement is given in Section 12.2.4.3. Section 12.2.4.3 states that a modification factor of either 1.5 or 1.2 be used for bars depending on the amount of cover and clear spacing. The tests conducted in conjunction with this investigation have indicated that epoxy-coated welded wire fabric has essentially the same development and splice strength as uncoated welded wire fabric. This leads to the recommendation that a modification factor of 1.0 be applied to epoxy-coated WWF which can be accomplished by adding the following statement to ACI 318-89 Section 12.2.4.3:

Welded wire fabric with cross wires within the development length and cross wires lapped by at least 2" 1.0

In addition to this requirement, it is recommended that the following paragraph be added to the Commentary R12.2.4.3:

When welded wire fabric is used for reinforcement, the cross wires provide *anchorage to the wire being developed and tests indicate that epoxy-coating does not influence the development or splice strength. (Ref. XX)*

Ref. XX: This Study.

The stipulation that a factor of 1.0 can only be used for WWF when cross wires are present within the development length is important since this investigation did not directly consider the development of epoxy-coated wire. A number of pull-out tests using epoxy-coated and uncoated wire were conducted as a part of this experimental program and by Schmitt and Darwin[16], but it is not believed that there is enough data in these investigations to make a definite conclusion about the effects of epoxy-coating on the development of wire.

In order to specifically include deformed wire in the ACI 318 requirements for epoxy-coated reinforcement and to eliminate confusion about the modification factor for epoxy-coated deformed wire, it is recommended that the first statement in ACI 318-89 Section 12.2.4.3 be modified to include deformed wire as follows:

Bars and deformed wires with cover less than $3d_b$ or clear spacing between bars less than $6d_b$ 1.5

In order to clarify the provisions in ACI 318-89 for epoxy coated WWF it would be desirable to include an additional statement to the Commentaries R12.7 and R12.8 as follows:

Tests have indicated that epoxy-coated welded wire fabric has essentially the same development and splice strength as uncoated welded wire fabric. Therefore, an epoxy-coating factor of 1.0 is included for the development or splice of epoxy-coated welded wire fabric with cross wires within the development or splice length.

Sections 12.7 and 12.8 define the required development length for welded deformed wire fabric and welded plain wire fabric respectively. The development of epoxy-coated welded deformed wire fabric is covered explicitly since Section 12.7 requires the use of the modification factors in Sections 12.2.3 through 12.2.5. For the case of welded deformed wire fabric, the additional statement above for the Commentary R12.7 is not necessary, but it is desirable. Since the use of the modification factors in Sections 12.2.3 through 12.2.5 are not required for the development length of welded plain wire fabric, the user is not given any direction in Section 12.8 about how epoxy-coating affects the development length of welded plain wire fabric. For the case of welded plain wire fabric it is desirable to include the above statement in the Commentary R12.8 so that it is specifically stated that no modification in the development length needs to be made for epoxy-coating.

5.2 ACI 318-89 Development Length Equations for Welded Deformed Wire Fabric

The current equation in ACI 318-89 for determining the development length of welded deformed wire fabric in tension is based upon outdated material requirements. ASTM A 497 currently requires a minimum weld shear strength of 35,000 psi times the wire cross section area. The equation shown in Section 12.7.2 uses the past ASTM weld shear requirement of 20,000 psi times the wire cross section area to determine the development length of welded deformed wire fabric. The development length determined by using this equation may be overly conservative and consideration should be given to replacing the 20,000 psi with 35,000 psi for consistency with current ASTM requirements.

CHAPTER 6

CONCLUSIONS

6.1 Conclusions

From the experimental program carried out in conjunction with this investigation, it can be concluded that epoxy-coating has little or no effect on the bond and development of welded plain wire fabric and welded deformed wire fabric. This conclusion is based on a series of pull-out tests and one-way slab tests conducted on epoxy-coated and uncoated welded plain wire fabric and welded deformed wire fabric. Load versus deflection curves for both types of tests showed that epoxy-coating did not have a detrimental effect on the bond between the WWF and the concrete. In addition, the one-way slab tests demonstrated that there is no significant difference in the cracking behavior of slabs reinforced with epoxy-coated and uncoated WWF, although there was a significant difference in cracking behavior between slabs reinforced with welded plain wire fabric and welded deformed wire fabric.

Current ACI 318 requirements for the development of epoxy-coated reinforcement are based upon experimental results obtained from deformed bars. From the experimental results obtained in this investigation, it can be concluded that a change needs to be made in the code provisions for the development of epoxy-coated WWF. It has been found that no additional modification factor needs to be applied to the development length of WWF to account for the effects of epoxy-coating when anchorage is basically due to cross wires..

6.2 Topics for Future Consideration

As a result of this investigation, a number of additional areas for future consideration have been identified. One area for future consideration is the bond and development of epoxy-coated wire. Through the limited number of pull-out tests associated with this investigation and those conducted by Schmitt and Darwin[16], the epoxy-coating does not appear to have a detrimental effect on bond of wire in tension, but more tests need to be conducted in this area before a final conclusion can be made.

Another aspect of the bond and development of WWF that needs further consideration is the equation used in Section 12.7.2 of ACI 318-89 to determine the basic development length of welded deformed wire fabric. This equation is based on the old ASTM weld shear strength requirement of 20,000 psi times the wire cross section area. ASTM A 497 currently requires a minimum weld shear strength of 35,000 psi times the wire cross section area. The current equation in Section 12.7.2 may require overly conservative development and splice lengths, therefore a change may be desirable for consistency with current ASTM requirements.

BIBLIOGRAPHY

1. Podolny, Walter Jr., "Structural Fabric Reinforcement in Concrete Slabs," Journal of the American Concrete Institute, Proceedings V. 65 No. 10, October 1968, pp. 877-884.

2. Manual of Standard Practice for Structural Welded Wire Fabric, Wire Reinforcement Institute, 1992.

3. ACI Committee 318, Building Code Requirements for Reinforced Concrete (ACI 318-89), American Concrete Institute, Detroit, 1989.

4. ASTM A 82 REV A-90, "Standard Specification for Steel Wire, Plain, for Concrete Reinforcement," American Society for Testing and Materials.

5. ASTM A 496 REV A-90, "Standard Specification for Steel Wire, Deformed, for Concrete Reinforcement," American Society for Testing and Materials.

6. ASTM A 185 REV A-90, "Standard Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement," American Society for Testing and Materials.

7. ASTM A 497 REV B-90, "Standard Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement," American Society for Testing and Materials.

8. ASTM A 884 REV B-91, "Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement", American Society for Testing and Materials.

9. Treece, Robert A., and Jirsa, James O., "Bond Strength of Epoxy-Coated Reinforcing Bars," Phil M. Ferguson Structural Engineering Laboratory, Department of Civil Engineering / Bureau of Engineering Research, The University of Texas at Austin, No. 87-1, January 1987.

10. Hamad, Bilal S.; Jirsa, James O.; and d'Abreu d'Paolo, Natalie I., "Effect of Epoxy Coating on Bond and Anchorage of Reinforcement in Concrete Structures," Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin, Research Report 1181-1F, Project No. 3-5-88/0-1181, December 1990.

11. Elstner, R. C., "Test Investigation of Splices in Continuously Reinforced Concrete Pavement," Wiss Janney and Associates, October 1962.

12. Atlas, Amos; Siess, Chester P.; Bianchini, Albert C.; and Kesler, Clyde E., "Behavior of Concrete Floor Slabs Reinforced with Welded Wire Fabric," T&AM Report No. 260,

University of Illinois, August 1964.

13. Lloyd, John P., "Splice Requirements for One-Way Slabs Reinforced with Smooth Welded Wire Fabric," Publication No. R(S) 4, Civil Engineering, Oklahoma State University, June 1971.

14. Lloyd, John P., and Kesler, Clyde E., "Behavior of One-Way Slabs Reinforced with Deformed Wire and Deformed Wire Fabric," T&AM Report No. 323, University of Illinois, April 1969.

15. Sur, L. M.; Jensen, R. S.; Reynolds R. J.; and Kesler C. E., "WRI Study of Deformations on Wires," Mimeographed Paper, Theoretical and Applied Mechanics Department, University of Illinois, July 1965.

16. Schmitt, Tony R., and Darwin, David, "Bond of Epoxy-Coated Wire to Concrete," Structural Engineering and Engineering Materials SL Report 92-5, The University of Kansas Center for Research, Inc., November 1992.

17. Lee, S. L.; Mansur, M. S.; Tan, K. H.; and Kasiraju, K., "Cracking Behavior of One-Way Slabs Reinforced with Welded Wire Fabric," ACI Structural Journal, V. 86, No. 2, March-April 1989, pp. 208-216.

18. Jirsa, James O.; Lutz, LeRoy A.; and Gergely, Peter, "Rationale for Suggested Development, Splice, and Standard Hook Provisions for Deformed Bars in Tension," Concrete International, July 1979, pp. 47-61.

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