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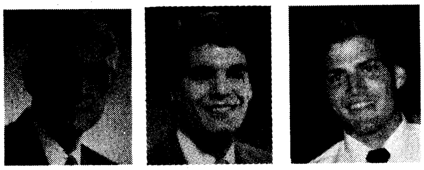
Welded Structural Wire Reinforcement for Columns



by Richard W. Furlong, Gregory L. Fenves, and Eldon P. Kasl

The use of welded structural wire reinforcement as the only reinforcement for concrete columns was demonstrated in a pilot study for which four sets of twin specimens were cast simultaneously and tested under identical conditions. One specimen of each pair was reinforced with conventional reinforcing bars, and its twin was reinforced with the same longitudinal area of welded structural wire reinforcement. Transverse ties in each pair had the same strength per unit of column length.

Stiffness response was virtually identical for twin specimens. All conventionally reinforced specimens and three of the welded structural wire reinforced columns resisted test loads larger than the capacity P_n *determined in accordance with the ACI Building Code (ACI 318-89). The fourth specimen failed at a load equal to 97 percent of its Building Code-predicted capacity* P_n*.*

Keywords: columns (supports); reinforcing steels; stiffness; strength; tests; **tied columns; welded wire fabric.**

Welded structural wire reinforcement (welded wire fabric) can be a cost-effective alternative to conventional tied assemblies of reinforcing bars for installations that involve repetitive bending and extensive tying of bars. Tied columns require both repetitive bends for numerous ties and frequent tying at intersections of longitudinal and transverse bars. Welded structural wire reinforcement is made with its intersections welded together, eliminating further need for field tying of numerous intersections. Furthermore, the welded wire reinforcement is fabricated with equipment that produces uniform shapes for all steel bent in the same pattern. Excellent dimensional control and potential reductions in the cost of field labor may encourage designers to consider welded structural wire reinforcement in lieu of conventional transverse tie reinforcement for columns.

Conventional column reinforcement assemblies and some assemblies of welded structural wire reinforcement for ties are illustrated in Fig. 1 and Fig. 2. The conventionally reinforced assembly reflects the familiar variability among intermediate ties, and it displays a large number of wired tie bar intersections in Fig. 1. The welded structural wire perimeter reinforcement in Fig. 2 offers an arrangement of transverse ties that not only offers better control for uniform thickness of actual concrete cover outside the column steel, but permits placement of longitudinal bars more accurately than is possible with the conventionally tied assembly.

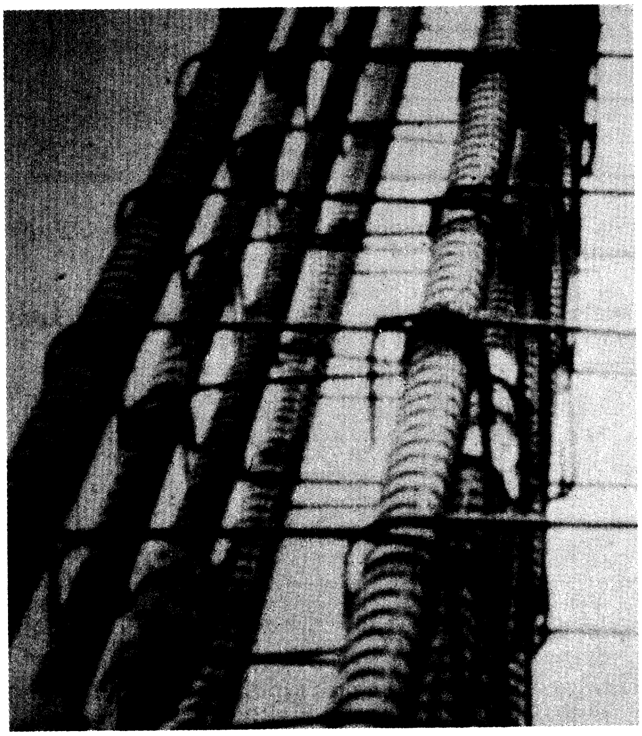


Fig. 1—Conventionally tied reinforcement for columns

Table 1 contains a list of welded structural wire sizes and spacings equivalent to conventional transverse tie sizes and spacings.

Cost reductions with welded structural wire column reinforcement would be even greater if longitudinal reinforcing bars were not necessary. Modern equipment can produce welded wire reinforcement with individual wires large enough to provide more than the minimum

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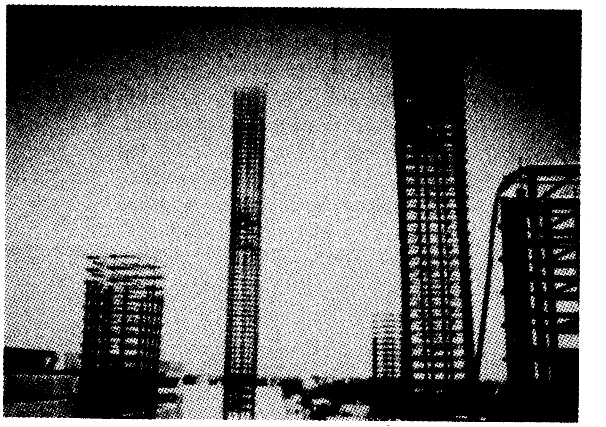

Fig. 2—Welded structural wire perimeter reinforcement for columns

Table 1 — Column tie bars and welded structural wire reinforcement of equivalent area

Tie & spacing	Equivalent wire reinforcement	Tie & spacing	Equivalent wire reinforcement
#3 @ 8″	W8.3 @ 6″ W11 @ 8″	#4 @ 8″	W14.7 @ 6″ W19.6 @ 8″
#3 @ 9″	W7.4 @ 6″ W9.8 @ 8″	#4 @ 9″	W13.1 @ 6″ W17.5 @ 8″
#3 @ 10″	W6.6 @ 6″ W11 @ 10″	#4 @ 10″	W11.8 @ 6″ W19.6 @ 10″
#3 @ 12″	W7.4 @ 8″ W11 @ 12″	#4 @ 12″	W13.1 @ 8″ W19.6 @ 12″
#3 @ 14″	W6.3 @ 8″ W9.5 @ 12″	#4 @ 14″	W11.2 @ 8″ W14 @ 10″
#3 @ 16″	W6.9 @ 10″ W11 @ 16″	#4 @ 16″	W12.3 @ 10″ W19.6 @ 16″
#3 @ 18″	W7.4 @ 12″ W9.8 @ 16″	#4 @ 18″	W10.9 @ 10″ W17.5 @ 16″

*Wire size designations W8.3 to W19.6 used here refer to plain wire. Each size is available also as deformed wire, designated as D8.3 to D19.6.

1 percent reinforcement ratio (ACI 318-89 Section 10.9.1)[1] required for column cross sections. The current tie specification (ACI 318-89 Section 7.10.5.3) that requires intermediate cross ties at alternate vertical bars effectively limits to three the number of vertical wires along one face of a cross section. Within the constraints of the restriction to three bars per face if no interior cross ties are used, 12 D22 wires make an area 2.64 sq. in. (1700 sq. mm), adequate for columns 16 in. (400 mm) square and smaller.

To demonstrate the behavior of columns containing only welded structural wire reinforcement, a pilot study[2] of strength, stiffness, and ductility behavior in such columns was initiated by the Wire Reinforcement Institute. All tests were conducted at the University of Texas Ferguson Structural Engineering Laboratory under the sponsorship and oversight of the Reinforced Concrete Research Council.

STRENGTH AND STIFFNESS TESTS

The specimens in the pilot study were designed to compare directly the performance of conventionally reinforced columns with the performance of welded structural wire reinforced columns. Eight columns were fabricated: four were conventionally reinforced and four were made with structural wire reinforcement. Fig. 3 contains sketches of the eight cross sections used in the study. The square columns with conventional reinforcement contained only four corner bars and No. 3 ties located 10 in. (25 mm) apart. The corresponding equivalent welded structural wire reinforcement consisted of vertical bars that had the same total area as that of the four corner bars. Transverse ties compatible with the vertical wires (to achieve reliable welds at intersections, the smaller wire must have an area not less than 40 percent of the area of the larger wire) were spaced to produce the same transverse steel area as that of the No. 3 ties at 10-in. (25-mm) centers. Longitudinal "bars" of wire fabric assemblies cannot be located at corners, as transverse wires cannot be bent at welds.

Specimens of the same size and reinforcement area were cast simultaneously from the same batch of transit mix concrete. All column specimens were in a vertical position during placement of concrete.

Specimens 8 in. (200 mm) thick and 15 in. (380 mm) wide were used in the study to provide some information about the influence of omitting the interior tie for welded structural wire reinforcement that contained more than three vertical bars along the 15-in. (380-mm) width of the compression face. In one pair of 15 x 8-in. (380 x 200-mm) specimens, the longitudinal reinforcement ratio was 1.55 percent. The conventional section had six No. 5 bars, three along each 15-in. (380-mm) face, and interior transverse J-bars through the section to connect the center bars. The corresponding welded structural wire section also employed three D18.6 wires along the 15-in. (380-mm) faces and two D15.6 wires along each 8-in. (25-mm) face. No interior J-bar tie through the section was used at the center longitudinal wire in the 15-in. (380-mm) face.

The other pair of 15 x 8-in. (380 x 200-mm) specimens contained reinforcement ratios of 2.0 percent. The conventional section had four No. 7 corner bars, whereas the corresponding welded wire reinforced section had twelve D19.6 wires, five along each 15-in. (380-mm) face and one at each 8-in. (200-mm) face. No J-bar interior transverse ties were used on any of these columns. Concrete for all specimens was made with ¾-in. (20-mm) maximum size coarse aggregate, sand, Type I cement, and water in the proportions 1780/1600/425/252 in lb/yd^3 (1056/949/252/150 in kg/m^3). The strength reported for each specimen is the average value from three cylinders tested within 24 hr of the specimen test.

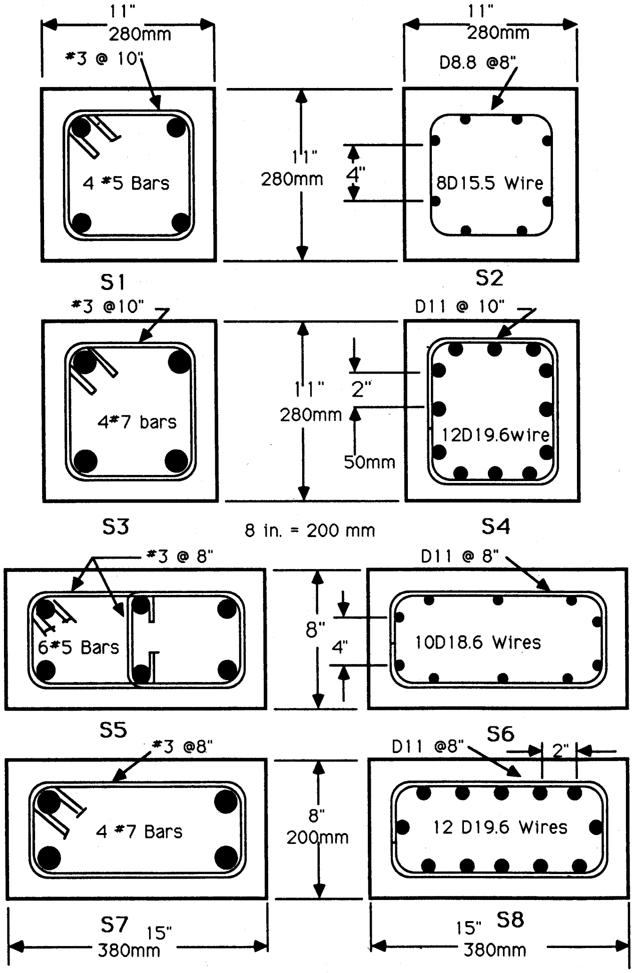

Fig. 3—Test specimen section details

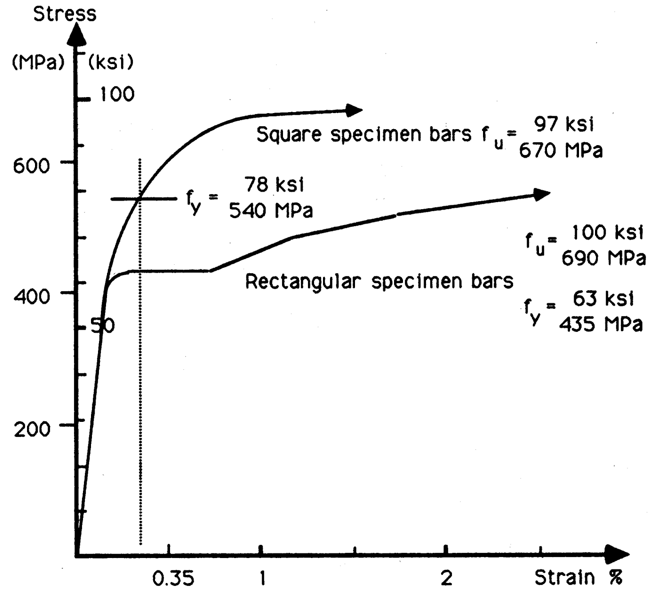

Fig. 4(a)—Stress-strain diagrams for deformed bars

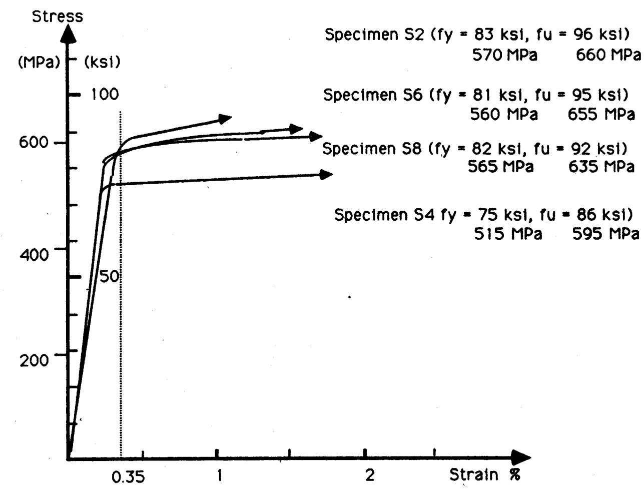

Fig. 4(b)—Stress-strain diagrams for deformed wires

Reinforcing bars of billet steel meeting ASTM A 615-85 (Reference 3) were used for the bar-reinforced specimens. Stress-strain curves for bar samples cut from bars that were to be used in the specimens are shown in Fig. 4(a). Bars used in the square specimens had no well-defined yield point, a proportional limit near 50 ksi (345 MPa), and tension strength near 75 ksi (517 MPa) at 0.3 percent strain.

Welded wire fabric meeting the requirements of ASTM A497-86 (Reference 4) was used for the wire-reinforced specimens. Stress-strain curves for samples from longitudinal wires used in each specimen are shown in Fig. 4(b). All wire deformations were cold-formed indentations. The D19.6 wires of Specimen S4 had a yield strength of 75 ksi (517 MPa), and all other deformed wire showed yield strengths near 82 ksi (565 MPa). Each welded wire fabric "cage" consisted of one sheet that was machine-bent along each corner to form a four-sided assembly.

The test loading was applied through spherical bearings against thick plates at enlarged ends of column specimens. The bearings were set along the middle of the column width and at an eccentricity of about 1/15th of the column thickness. A diagram of a specimen prepared for the loading machine is given in Fig. 5. The eccentric test loads were used not only to keep the largest force needed to cause failure within the 600-kip (2700-kN) capacity of available equipment, but also to control the face of maximum compression and to produce some curvature for comparisons of flexural stiffness response. The load was applied by a hydraulic 300-ton (2700-kN) testing machine at a rate of 0.1 to 0.2 kips/sec (0.5 to 1 kN/sec).

Longitudinal strains were monitored throughout each test with linear voltage transducer devices (LVDT) reacting against platforms attached to each specimen at 17.5 and 30 in. (445 and 760 mm) above and below midheight of each specimen along the 60-in. (1500-mm) test length of the columns. Lateral displacements were measured and recorded.

TEST RESULTS

Typical specimen behavior during the tests involved linear deformations early in the tests, and nonlinear deformations as test loads neared capacity levels. Searches were made continuously for any cracking of concrete prior to failure. The specimens with conven-

Table 2 — Summary of test results

Specimen	Reinforcement	Section size, in. x in.	ρ_{long}	f_y, ksi	f'_c, ksi	e, in.	P_{max}, kips	ϵ_u, in./in.	P_{ACI}, kips	$\frac{P_{max}}{P_{ACI}}$	s/D
S1	4#5, #3 @ 10"	11 x 11	1.0	62	4.26	0.75	523	0.0030	433	1.21	16
S2	8 D15.5, D8.8@8"	11 x 11	1.0	74	4.26	0.75	509	0.0024	446	1.14	18
S3	4#7, #3 @ 10"	11 x 11	2.0	62	3.83	0.75	510	0.0024	447	1.14	11.4
S4	12D19.6, D11@10"	11 x 11	1.9	74	3.83	0.75	482	0.0031	468	1.03	20
S5	6#5, #3 @ 8"	15 x 8	1.5	62	4.01	0.50	497	0.0033	449	1.11	12.8
S6	10D18.6, D11@8"	15 x 8	1.5	74	4.01	0.50	461	0.0023	450	1.02	16.4
S7	4#7, #3 @ 8"	15 x 8	2.0	62	3.22	0.50	443	0.0028	389	1.14	9.1
S8	12D19.6, D11@8"	15 x 8	2.0	74	3.22	0.50	395	0.0024	406	0.97	16.0

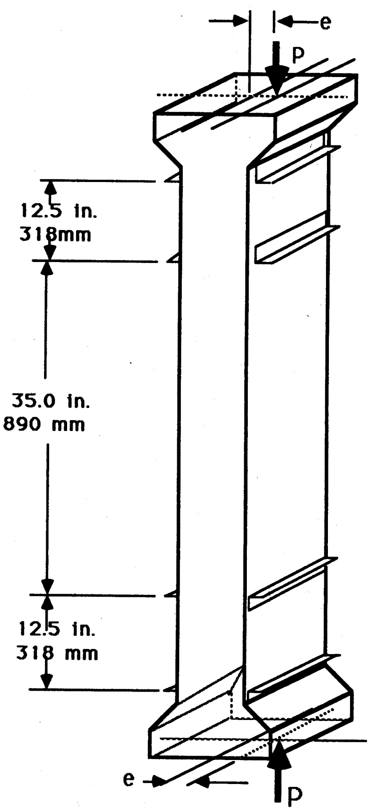

Fig. 5—Test arrangement and displacement meter locations

tional reinforcement displayed fewer longitudinal cracks than those observed on companion welded structural wire reinforced specimens. The failure region for every column was above midheight and appeared to initiate at one of the corners, although the entire compression face of the failure region generally split off as longitudinal bars buckled. In Specimen 7 with No. 7 vertical bars, the concrete spalled off before the No. 7 bars appeared to have yielded.

All longitudinal compression face bars buckled as soon as the stabilizing concrete spalled away. The spalled compression face concrete from welded structural wire reinforced columns generally was broken more extensively than that from the conventionally reinforced sections, possibly reflecting a tendency to form a zone of lamination with both vertical and horizontal reinforcement more closely spaced. After the loss of the compression face at a failure zone, curvatures increased dramatically, and resistance to load diminished quickly.

A summary of test results is given in Table 2. The value of compressive strength f'_c tabulated for each pair of columns is the average of values for both. Maximum loads and surface strains measured at the load stage just before the final "failure" load was applied are listed for every specimen. Each conventionally reinforced specimen resisted loads larger than those resisted by companion specimens that were reinforced with welded structural wire. Wire reinforcement had a yield strength higher than that of conventional reinforcement. Since conventionally reinforced specimens developed strengths higher than wire-reinforced specimens, apparently the full yield strength of wire did not develop before concrete spalled at failure. Just before failure forces were reached, the impression that wire became unstable without yielding before concrete failed was suggested also from observations of more vertical cracking in wire-reinforced columns than in companion conventionally reinforced columns. The "early" failure of structural wire specimens may have been due to the absence of corner bars or longitudinal spacing of transverse ties too large to stabilize longitudinal bars, or a combination of both influences.

ACI Section 3.5.3.5 requires that welded structural wire with a specified yield strength of 60 ksi (414 MPa) attain at least that stress at a strain no greater than 0.35 percent. The largest measured failure strain at the compression surface of any specimen was 0.33 percent in Specimen 5 with J-bars between the center longitudinal bars, and the lowest value was 0.23 percent for its companion Specimen 6 without such bars. The average of maximum compression strains was 0.29 percent before concrete spalled from conventionally reinforced specimens, and it was 0.26 percent before concrete spalled from welded structural wire specimens.

A closer spacing of transverse ties may have helped structural wire reach its actual 81-ksi (562-MPa) strength before concrete failed. If an effective stiffness near yield were taken as 80 percent of a nominal value of 28,000 ksi (195,000 MPa) for steel, and the buckling stress is set equal to the nominal yield strength f_y, the buckling length of longitudinal bars (and a spacing limit for transverse ties) can be determined with the classical compression strut stability relationship[5]

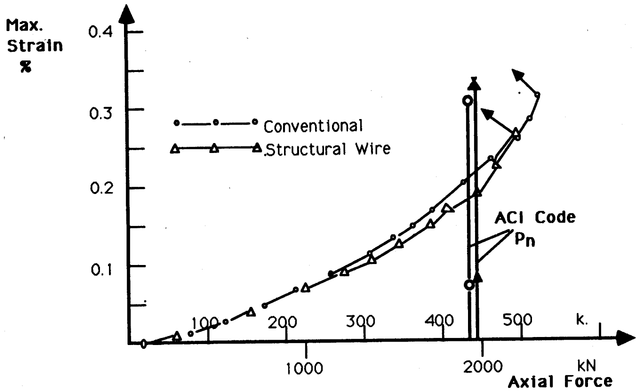

Fig. 6(a)—Square Specimens S1 and S2 with 1 percent longitudinal reinforcement

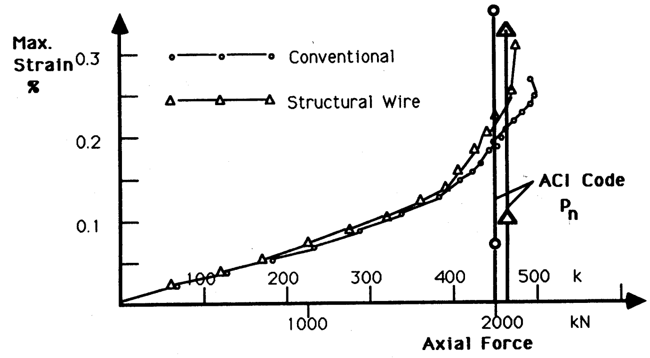

Fig. 6(b)—Square Specimens S3 and S4 with 2 percent longitudinal steel

$$P_{cr} = \pi^2 EI/l^2$$

or

$$P_{cr}/A = f_y = \pi^2 (0.8E)A/(l/r)^2$$

for which P_{cr} = buckling force

E = steel nominal modulus of elasticity
= 28,000 ksi (195,000 MPa)
I = strut cross section moment of inertia
= Ar_2

with

A = strut cross section area
r = strut cross section radius of gyration = $D/4$
D = strut diameter
l = buckling length of strut

Taking the effective buckling length as the tie spacing $s = 1$

$$f_y = [0.80\ \pi^2 E\ ]/[(4s/D)^2] \quad (1)$$

$$\text{and a limit for } s/D = \sqrt{13{,}800/f_y} = 118/\sqrt{f_y} \text{ in ksi} \quad (2a)$$

$$= \sqrt{96{,}200/f_y} = 310/\sqrt{f_y} \text{ in MPa} \quad (2b)$$

The maximum spacing of transverse ties for Grade 60 reinforcing bars would be 15.2 bar diameters, and the maximum spacing for wire, with a yield strength of 71.5 ksi (493 MPa), becomes 14 bar diameters. That limit was exceeded in this study for all of the specimens with welded structural wire reinforcement as indicated with s/d ratios in Table 2. The ACI Building Code restricts tie spacing to 16 times the diameter of longitudinal bars, 48 diameters of the tie, or the least lateral dimension of the cross section, whichever is smallest. The 16 bar diameter limit was exceeded for the wire-reinforced square specimens and virtually matched for the rectangular wire-reinforced specimens. Possibly, tie spacing did not affect test results, as all longitudinal bars were observed to buckle as soon as concrete spalled away. These limited test observations could not establish whether impending buckling of bars caused concrete to split longitudinally and then to spall, or whether longitudinal splitting of concrete removed lateral restraint to the bars to allow them to buckle. Certainly, confining concrete will stabilize steel bars, but the concrete appeared to become unstable at compressive strains above 0.2 percent but less than the 0.3 percent failure strain assumed for flexural strength of concrete. Under compressive forces at very low eccentricity, more research is needed to establish whether or not current tie spacing rules are adequate to insure the development of limit strength both from the concrete and from reinforcing steel when the yield strain of steel is greater than 0.2 percent.

Failure forces P_n were determined for the applied eccentricities (ACI 318-89, Sections 10.2.3 and 10.2.7). No second-order deformation was considered. Values of P_n are listed in Table 2. All except the column labeled S8 resisted applied forces larger than the ACI Building Code procedures would have indicated. Specimen S8 reached 97 percent of its "code" capacity. The absence of a transverse tie through the core of Specimen 8 to hold the middle vertical wires may be cited in addition to the 16D tie spacing and the absence of corner longitudinal bars to explain why concrete spalled before 100 percent of its ACI-determined strength was attained.

Graphs that display measured compressive axial strain and recorded compressive axial force for companion specimens are given in Fig. 6 and Fig. 7. Conventionally reinforced specimen response graphs are made with hollow circles, and welded structural wire specimen graphs are made with hollow triangles. The graphs of Fig. 6 for square specimens indicate almost identical stiffness response in the "elastic" range as

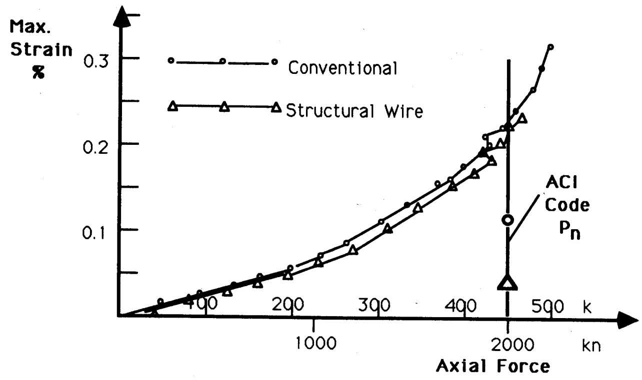

Fig. 7(a)—Rectangular Specimens S5 and S6 with 1.5 percent steel

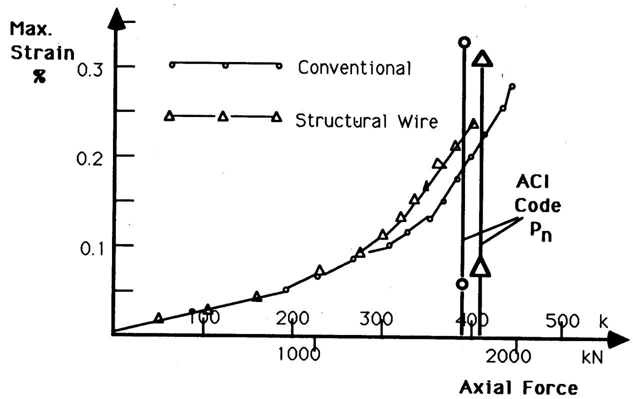

Fig. 7(b)—Rectangular Specimens S7 and S8 with 2 percent steel

well as in the nonlinear range for each pair reinforced with the same areas of longitudinal steel. For the lower reinforcement ratio of 1 percent, the conventionally reinforced Specimen S1 developed a greater maximum strain than did the structural wire Specimen S2. In contrast, with 2 percent reinforcement ratio, the conventionally reinforced Specimen S3 in Fig. 6(b) was less ductile than the structural wire Specimen S4. Values of limit strength P_n derived from ACI 318-89 procedures are shown on the graphs of Fig. 6 and 7.

Almost the same similarity of stiffness and relative ductility among the wide-face specimens is reflected in the graphs of Fig. 7, although the change in reinforcement ratio from 1.5 for Fig. 7(a) to 2 percent for Fig. 7(b) was smaller than was the difference in ratios for the square columns. Strength values P_n derived from ACI 318-89 procedures are shown for each specimen. These graphs suggest that stiffness response of concrete columns cannot indicate whether the column contains conventional or welded structural wire reinforcement.

CONCLUSIONS

A pilot study was conducted to compare stiffness and strength responses between columns conventionally reinforced and columns made with equal areas of welded structural wire reinforcement. The tests indicated that:

1. No initial stiffness response differences were discernible from the measurements made in the pilot study, although three of the four specimens conventionally reinforced sustained before failure higher average strains than those observed for the wire reinforced specimens.
2. Columns reinforced with welded structural wire reinforcement resisted forces larger than 97 percent of the forces that would be predicted with the rectangular stress block strength analysis procedures defined by the ACI Building Code (ACI 318-89).
3. The one specimen that failed to resist more than its nominal value P_n determined in accordance with the rectangular stress block and 0.3 percent limit strain procedures of ACI 318-89 was the only specimen that did not contain interior ties through the core to stabilize "alternate longitudinal bars" on the compression face as required by the ACI Building Code. Interior ties appear to be needed.
4. Columns subjected to limit loads at low eccentricity can experience crushing-spalling deterioration at surface strains less than 0.3 percent.
5. The ratio P_{test}/P_n was 1.15 for specimens conventionally reinforced, and the ratio P_{test}/P_n was 1.04 for specimens reinforced only with welded wire fabric.

SUGGESTED SUBSEQUENT RESEARCH

This pilot study shows that 8 to 11 in. (200 to 280 mm) thick columns reinforced with wire fabric alone can support the same limit loads as columns of the same size reinforced with conventional deformed bars. With minimum concrete cover, the small size of specimens in this study produced maximum compression strains of steel smaller than those that would develop in large specimens. Additional tests of larger columns reinforced only with welded wire fabric should be made to observe the reliability of deformed wire at larger compression strains before concrete spalls.

Current tie spacing rules should be reviewed with tests of columns reinforced with bars of Grade 75 and higher. If columns of relatively low concrete strength (f_c' less than 4 ksi or 28 MPa) indicate bar buckling before f_y develops, required concrete quality should be increased until f_y can develop.

The influence of the absence of interior ties should be studied thoroughly with additional specimen tests. Similarly, the significance of corner bars should be studied with some tests with longitudinal bars only in the center of each face, simply to document more thoroughly the amount (if any) of strength reduction and ductility reduction without corner bars.

Load reversal ductility of wire-reinforced concrete columns should be studied both for applications in earthquake-resistant structures and for fatigue characteristics.

NOTATION

A = area of cross section
D = diameter of reinforcing bar
E = stiffness, Young's modulus of elasticity
e = eccentricity of axial force
f'_c = compressive strength of concrete as determined from tests of standard cylinders
f_u = ultmate strength of reinforcement
f_y = yield strength of reinforcement
I = moment of inertia for cross section
P = axial force
P_{cr} = axial force to create elastic buckling
P_{max} = maximum value of axial force reached in test
P_n = nominal strength of reinforced concrete cross section with procedures acceptable according to Section 10.2.7 of the ACI Building Code
r = radius of gyration of cross section
s = longitudinal spacing of transverse ties
ϵ_u = limit strain below which concrete is assumed to resist applied stress

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