STRENGTHENING OF INFILLED NON-DUCTILE CONCRETE FRAME

COLUMN REINFORCEMENT SPLICES

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DEDICATION

To Mom, Dad, Jennifer,
Jamie and Jason,
for their unwavering support

STRENGTHENING OF INFILLED NON-DUCTILE CONCRETE FRAME COLUMN REINFORCEMENT SPLICES

by

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Randy Bravo
Austin, Texas
December, 1990

ABSTRACT

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Two-thirds scale models of the column splice region of a non-ductile reinforced concrete frame were constructed. The prototype frame was infilled with a structural wall, forcing the columns to act in pure tension and compression when the frame is laterally loaded. The splice was found to be of insufficient length and confinement to develop yield of the spliced bars in tension. The existing splice was strengthened by placing structural steel angles, connected by steel straps, on the corners of the concrete column. The cyclic performance of the strengthened splice was an improvement over the existing splice behavior.

TABLE OF CONTENTS

PAGE

CHAPTER 1 -- INTRODUCTION 1

1.1 Background 1
1.1.1 Need for Seismic Strengthening. 1
1.1.2 Philosophy of Strengthening. 2
1.1.3 Strengthening Criteria. 2

1.2 Research 4
1.2.1 Related Research. 6
1.2.2 Purpose and Scope. 14

CHAPTER 2 -- METHODS FOR STRENGTHENING COLUMN SPLICES 15

2.1 Column Design 15
2.2 Splice Strengthening Techniques 17
2.2.1 Welded Splice. 17
2.2.2 Steel Straps. 19
2.2.2.1 Steel straps, grouted. 26
2.2.2.2 Steel straps, ungrouted. 26
2.2.3 External Ties. 26

CHAPTER 3 -- SPECIMENS AND TEST SETUP 31

3.1 Design of Test Specimens 31
3.1.1 Existing (Unstrengthened) Splice Specimens. 31
3.1.2 Steel Strap, Ungrouted Specimens. 31

3.2 Nomenclature for Specimens 34

3.3 Test Setup 35
3.3.1 Test Frame. 35
3.3.2 Data Acquisition System. 37

3.4 Characteristics of Materials 37
3.4.1 Concrete. 37
3.4.2 Steel. 38
3.4.2.1 Reinforcing bars. 39
3.4.2.2 Strengthening steel. 39

CHAPTER 4 -- EXPERIMENTAL RESULTS 40

4.1 Load History 40

4.2 Response Under Cyclic Loading 40
4.2.1 Unstrengthened Specimens. 41
4.2.2 Strengthened Specimens. 42

CHAPTER 5 -- SUMMARY AND CONCLUSIONS 51

5.1 Summary 51
5.1.1 General. 51
5.1.2 Discussion. 51

5.2 Conclusions 52

REFERENCES 54

LIST OF TABLES

TABLE PAGE

1 Summay of specimen concrete strength 38
2 Test summary 48

LIST OF FIGURES

FIGURE		PAGE
1	Strengthening techniques [8]	3
2	Non-infilled frame vs. infilled frame behavior	5
3	Typical location of column splice	6
4	Rigid block behavior of infilled frame	6
5	Vertical reinforcement schmatic of Gaynor's specimens	8
6	Full infill, Gaynor	8
7	Infill with window, Gaynor	9
8	Infill with door, Gaynor	9
9	Failures of splice in infill wall (Gaynor)	10
10	Vertical reinforcement schematic of Shah's specimen	11
11	Section view of Jimenez's column jacketing scheme [3]	12
12	Column detail before casting [3]	12
13	Full view of reinforcing cage of Jimenez's specimen [3]	13
14	Existing column section	16
15	Welded splice scheme	18
16	Welded splice detail	19

17 Transverse reinforcement. (a) side view of beam; (b) all splitting cracks restrained, $A_{tr} = a_b$; (c) two bars crossing plane of splitting through layer of bars, $A_{tr} = 2a_b/4$ splices[7] 20

18 Steel straps, ungrouted scheme 21

19 Steel straps, grouted scheme 22

20 Details of plate-angle connection 23

21 Sources of bond strength: (a) bond stresses due to friction and chemical adhesion between concrete and steel; (b) free-body diagram of reinforcing bar showing reaction of concrete on ribs; (c) free-body diagram of concrete showing forces exerted by ribs on concrete: radial component of bar reactions acts to produce cracking and spalling of concrete surrounding the bar 24

22 Forces used in flexural check of steel angle 25

23 Elevation view of steel straps scheme 27

24 External ties scheme 29

25 External ties elevation view 30

26 External ties, weld detail 30

27 Full elevation section of existing column specimen 32

28 End connection detail 33

29 End connection detail 34

30 End connection, end view 35

31 Testing apparatus schematic 36

32 Potentiometer setup 38

33 Desired load history 41

34 Response of existing column specimen 1 (U-B-1) 42

35 Failure mode of U-B-1 43

36 Response of existing column specimen 2 (U-B-2) 44

37 Response of steel straps, ungrouted specimen 1
(U-SS6-UG-1) 45

38 Response of steel straps, ungrouted specimen 2
(U-SS6-UG-2) 46

39 Response of steel straps, ungrouted specimen 3
(U-SS6-UG-3) 47

40 Failure mode of specimen U-SS6-UG-1 49

41 Failure mode of specimen U-SS6-UG-2 and
U-SS6-UG-3. 50

CHAPTER 1
INTRODUCTION

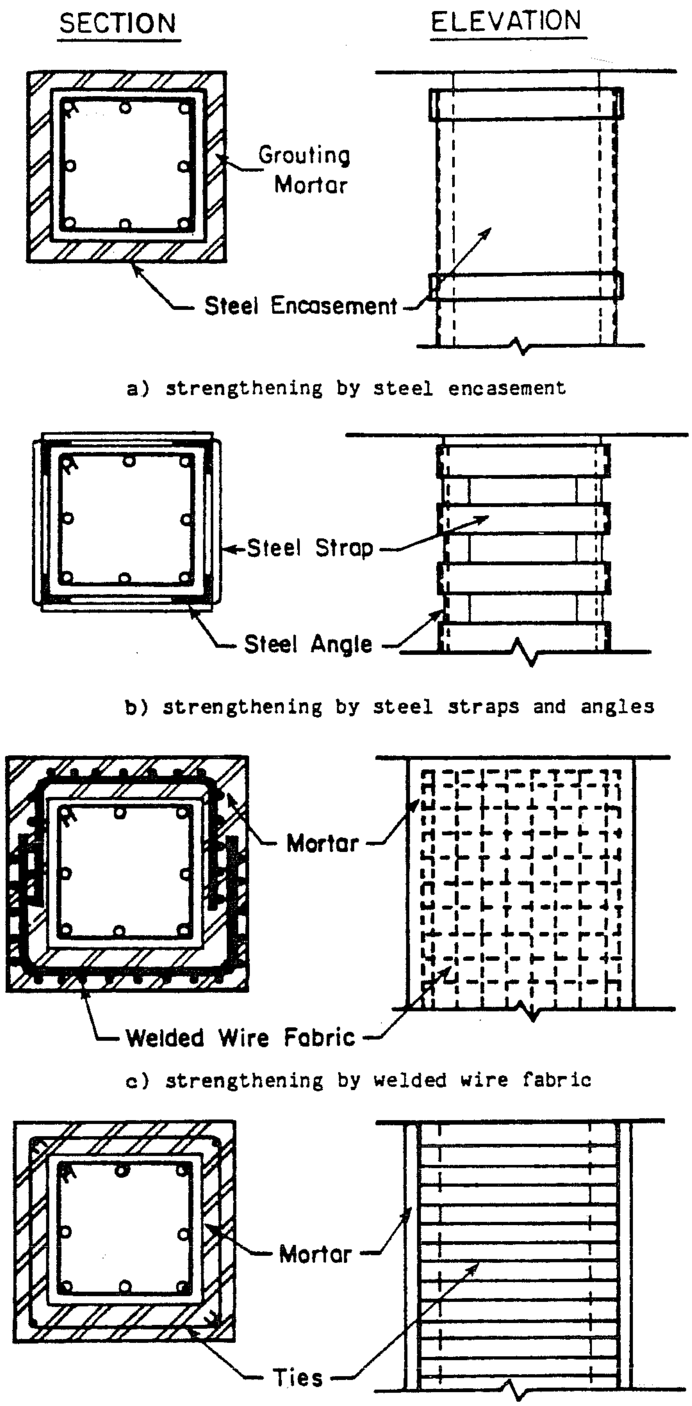
1.1 Background

1.1.1 Need for Seismic Strengthening. The damage incurred by structures as a result of earthquake motion has led to sweeping changes in seismic engineering in the past thirty years. Structures in earthquake prone regions designed by current codes have greater lateral strength and ductility than under previous codes. Unfortunately, many existing structures were built prior to the development of these more comprehensive design practices. If an earthquake of only moderate magnitude were to strike an area with a large number of these potentially inadequate structures, loss of life and property would most likely be high. The need for strengthening of these structures, in order to help reduce life and property losses, is imperative.

Due to the increasing cost of new construction, strengthening existing structures is becoming a more attractive option. The current methods of strengthening are largely based upon engineering judgement, but the effectiveness of a technique is often not known until an earthquake occurs. There is, therefore, a significant need for experimental work to test the effectiveness of schemes prior to implementation.

1.1.2 Philosophy of Strengthening. The engineer must assess the structure from a macroscopic and a microscopic point of view. The structure's overall response to seismic loading must be analyzed to determine load paths and high stress regions. The detailing of the high stress regions must also be studied to determine how the desired levels of strength and ductility can be achieved.

1.1.3 Strengthening Criteria. A retrofitting technique must fulfill the structural performance criteria, as well as be easily constructed, cost effective, and aesthetically pleasant. The strength and ductility of a structural member can be increased by attaching steel plates to the exterior of the member or by jacketing the member, which usually consists of adding longitudinal and/or transverse reinforceing bars (then casting a new concrete cover). To improve the ductility of a member, the confinement of the concrete should be increased and/or the continuity of the reinforcement should be assured. Figure 1 shows a few of the many techniques which can be devised to upgrade the strength and/or ductility of structural members. If it is not possible to strengthen a member, the element might be bypassed completely by adding structural walls or braces. The choice of which technique to use can also be influenced by construction procedures which may cause disruption and/or long term displacement of tenants, require large or complex equipment for installation, or impose heavy labor costs.



a) strengthening by steel encasement

b) strengthening by steel straps and angles

c) strengthening by welded wire fabric

d) strengthening by closely-spaced ties

Figure 1 Strengthening techniques [8].

Aesthetics are also of concern, as the outward appearance of the retrofit must gain public acceptance or no tenants will wish to occupy the building.

1.2 Research

Buildings designed and constructed before the 1960s in seismic zones were designed to carry a much smaller lateral load than is currently used in design. Due to this small design value, the design of a structure was typically controlled by gravity loads. When such a structure is analyzed using current seismic design loads, it may be found to be too flexible and laterally weak. In an effort to stiffen and strengthen the building, a structural wall system may be placed in some of the bays of the frame (see Figure 2). Such a strengthening technique induces significantly higher tensile and compressive stresses in the columns bounding these strong walls than they were originally designed for.

In non-ductile reinforced concrete frames, the longitudinal reinforcement splice in the bounding columns consequently becomes the weak link. The splice length typically specified in pre-1960s construction was of insufficient length (and confinement) to fully develop yield in tension. In addition, the column splices were typically located at the base of the column (see Fig. 3), which, unfortunately, is also the location of the highest tensile forces in the column. As shown in Figure 4, the behavior of an infilled frame is that of a vertical cantilever beam.

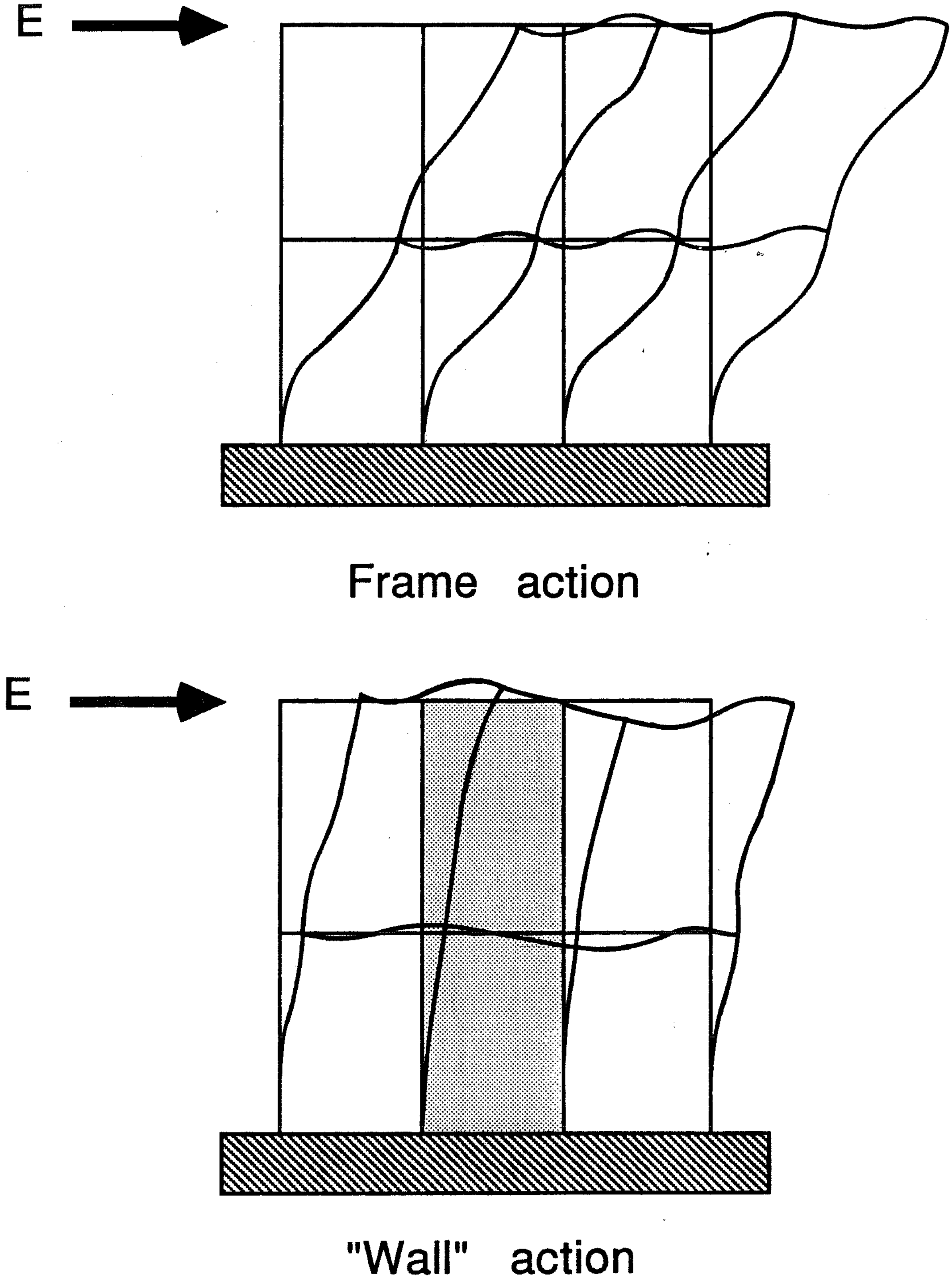


Figure 2 Non-infilled frame vs. infilled frame behavior.

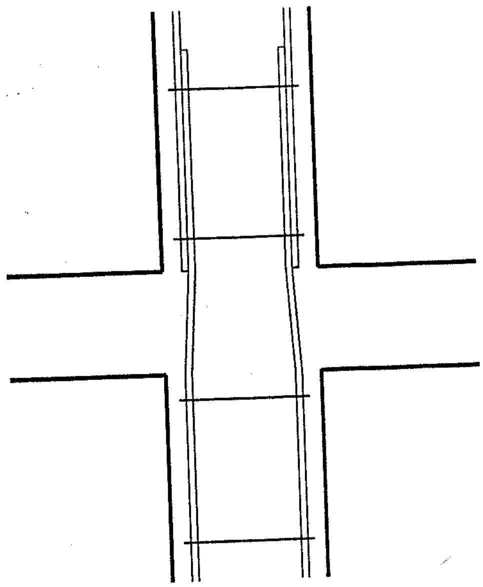


Figure 3 Typical location of column splice.

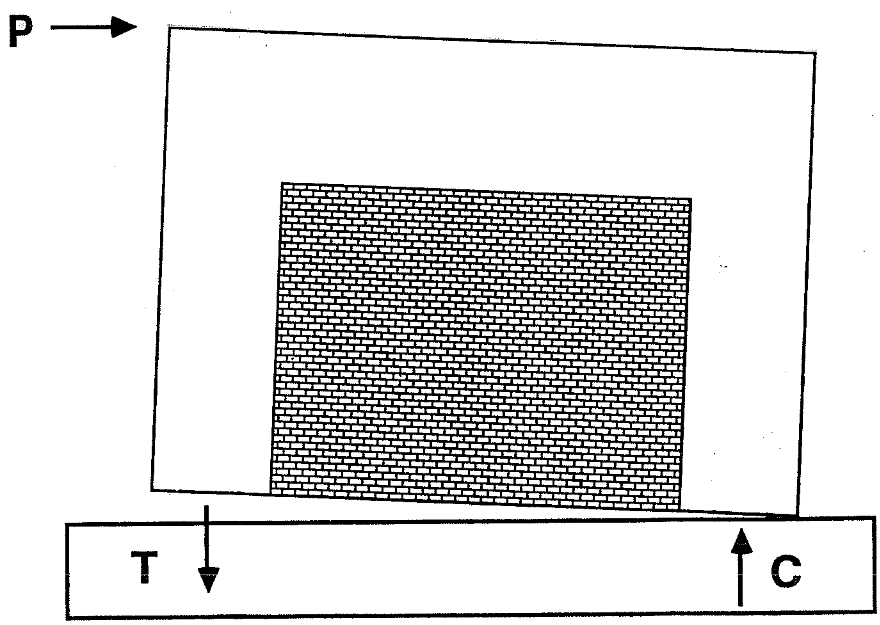


Figure 4 Rigid block behavior of infilled frame.

The columns are subjected to axial tension and compression, with the highest tensile stress occurring at the base of the column.

1.2.1 Related Research. Tests performed by Gaynor [1] indicated just such an inadequacy of the existing splice. Three 2/3-scale one-bay, one-story, non-ductile frames, representative of pre-1960s construction in seismic zones, were built and infilled with shotcrete walls. The column splice was located at the base of each column (see Figure 5). Each infilled frame had a different size and location of an opening (Figures 6, 7, 8) in an attempt to study the effects of openings on the behavior of laterally loaded infilled frames. In each test, however, Gaynor observed that the benefit of the infill wall was limited by the inadequate tensile strength of the splice at the base of the column (see Figure 9).

Shah [2] and Jimenez [3] extended Gaynor's work by developing two different methods to strengthen the column splice,\ in infill frames of designs similar to Gaynor's. Since the damage in Gaynor's bounding frame was localized in the column splice region, Shah removed the infill wall and reused the frame. Shah provided an alternate load path if the existing splice failed by placing three additional #8 bars in the wall next to each column, and casting the infill wall (see Figure 10). The #8 bars were spliced at mid-height, with a splice length of 40.5 inches and closely spaced spiral reinforcement, according to ACI Committee 408 guidelines, around the bars. The spirals were provided to prevent spalling of the

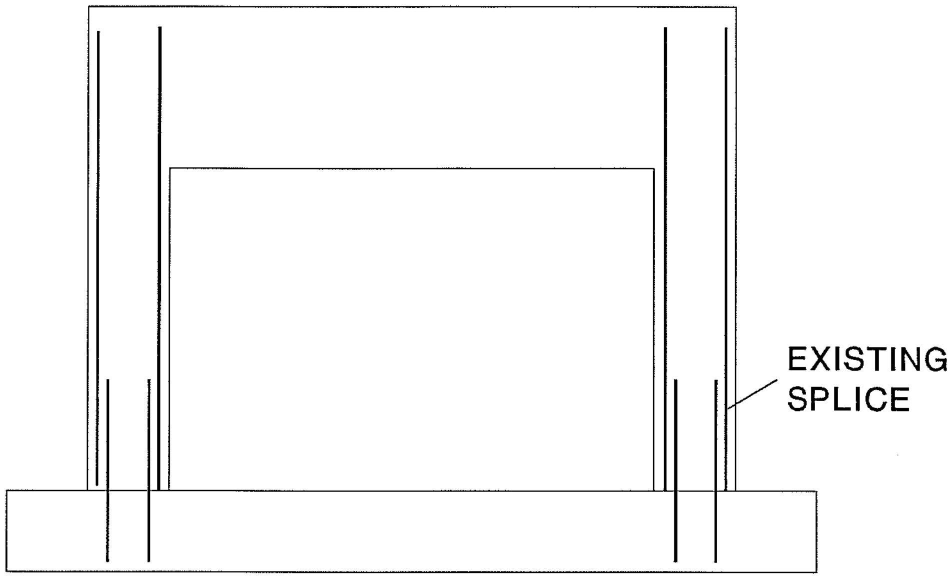


Figure 5 Vertical reinforcement schmatic of Gaynor's specimens.

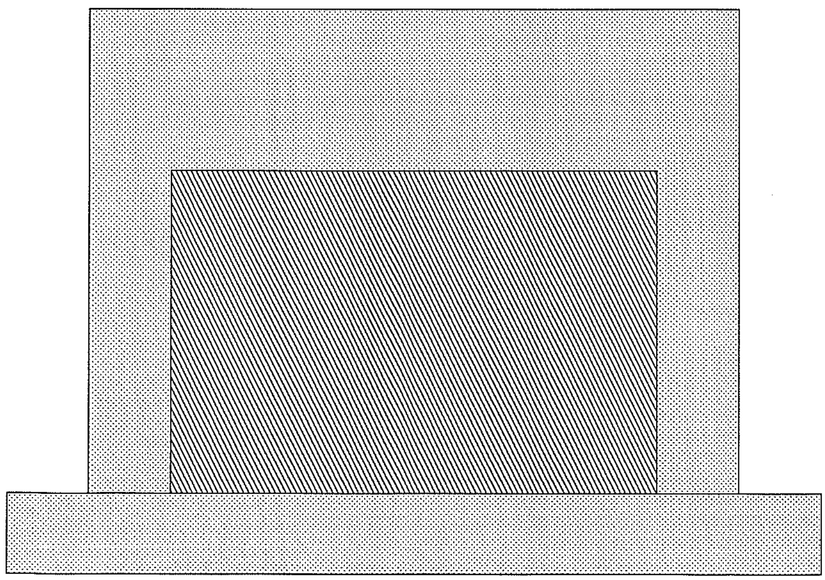


Figure 6 Full infill, Gaynor.

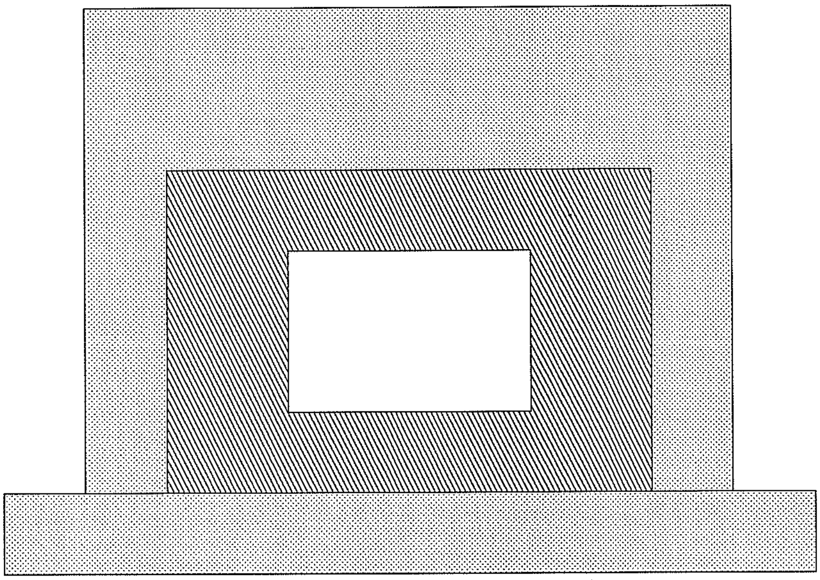


Figure 7 Infill with window, Gaynor.

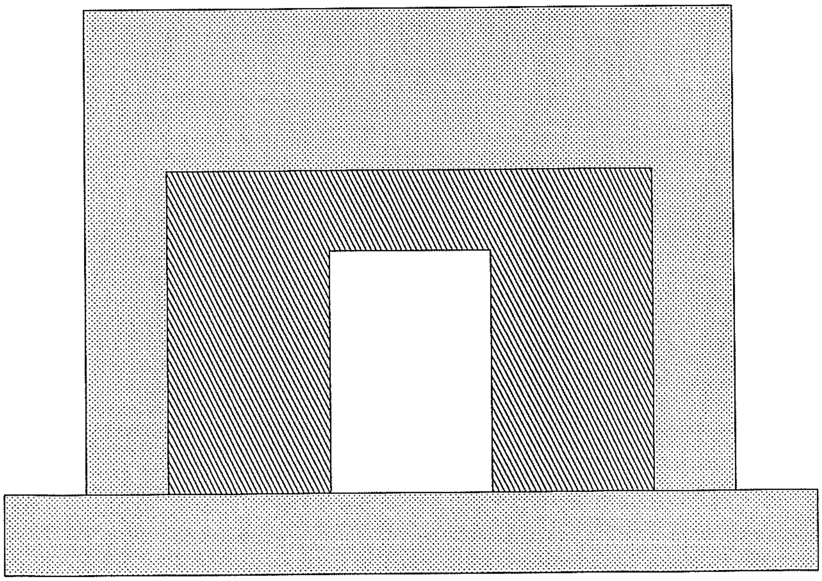


Figure 8 Infill with door, Gaynor.

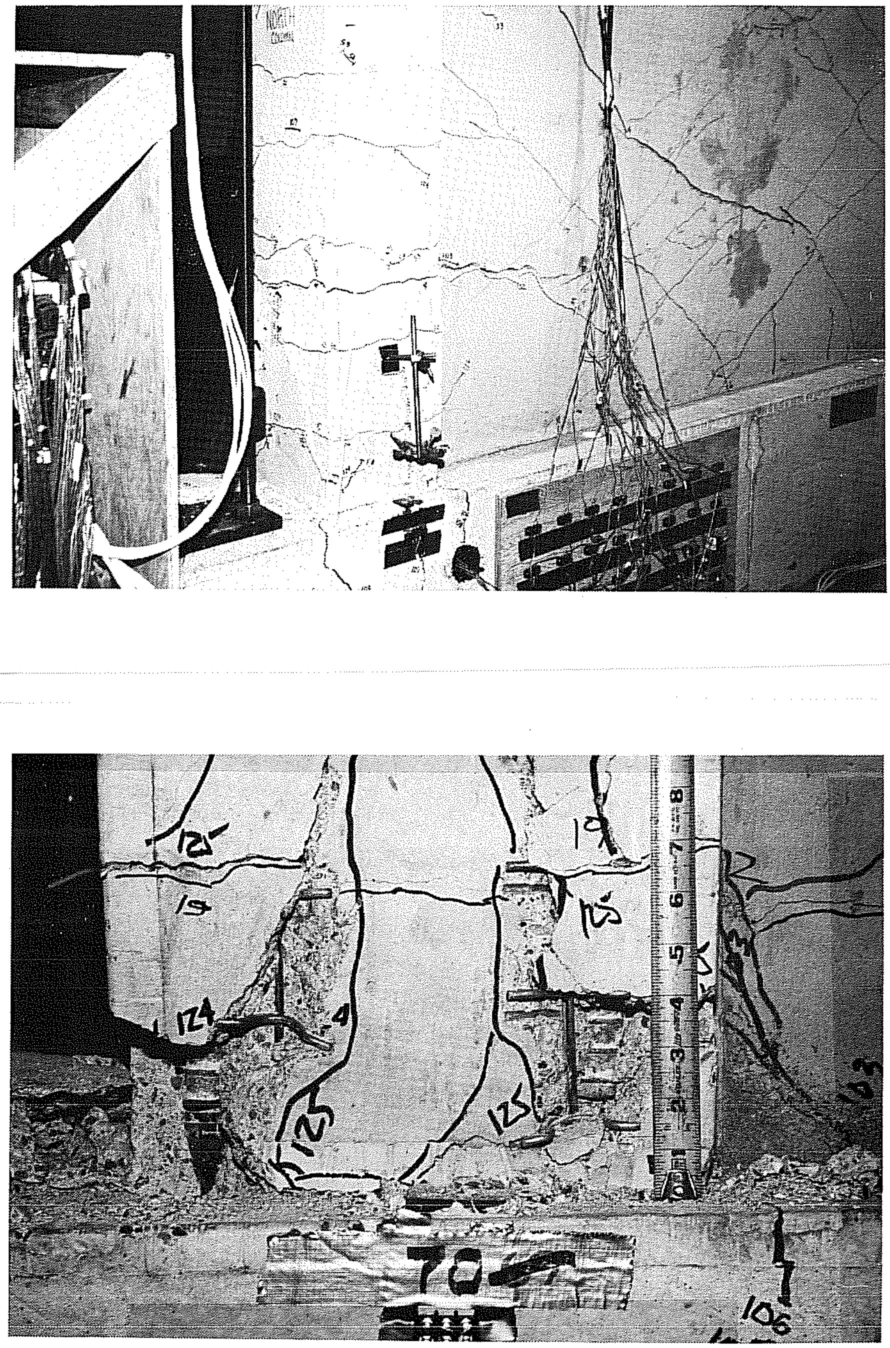


Figure 9 Failures of splice in infill wall (Gaynor).

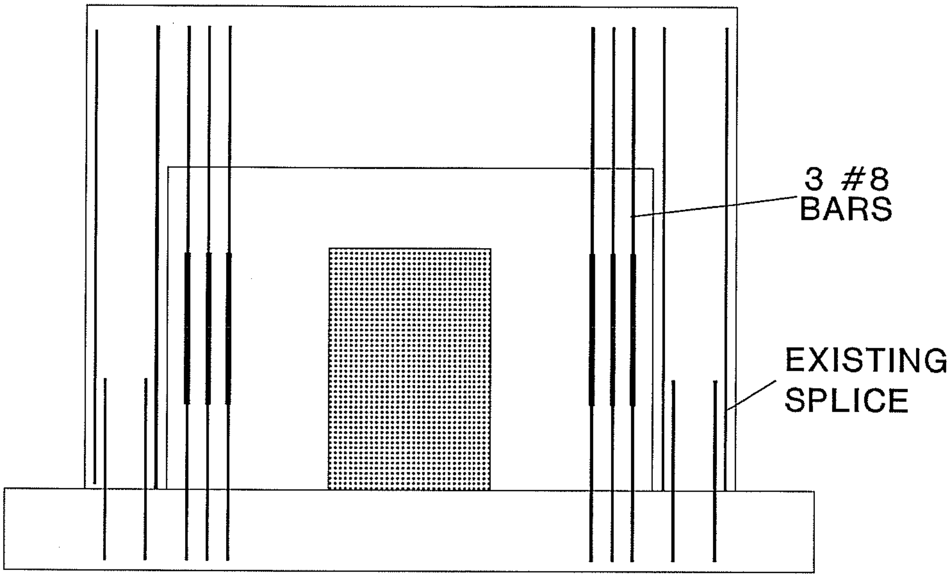


Figure 10 Vertical reinforcement schematic of Shah's specimen.

wall and failure of the splice. This strengthening technique was effective for a preventing a brittle failure of the wall when the column splice failed in tension. Jimenez[3] strengthened the existing splice by jacketing the entire column, increasing the confinement around the existing splice, then adding a wall similar to Gaynor's (see Figures 11, 12, 13). This technique also prevented brittle splice failure in the columns.

Both Shah and Jimenez implemented techniques which prevented tensile failure of the column splice, but such techniques may be impractical in the field.

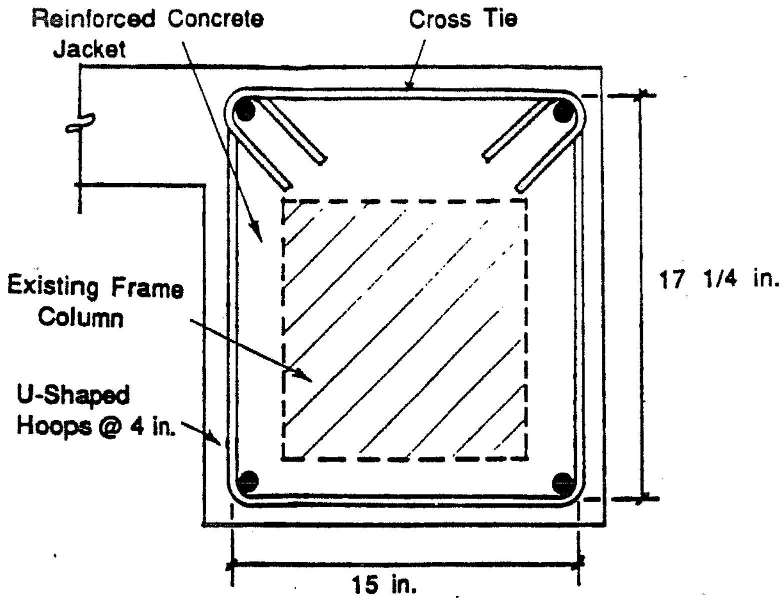


Figure 11 Section view of Jimenez's column jacketing scheme [3].

Figure 12 Column detail before casting [3].

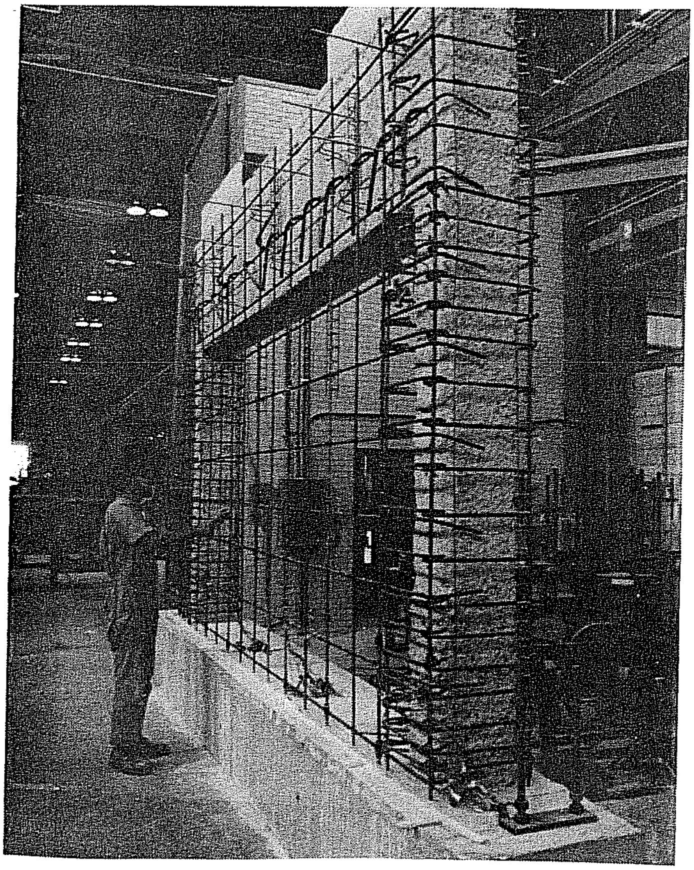


Figure 13 Full view of reinforcing cage of Jimenez's specimen [3].

Removing a wall (or even part of a wall) would require displacement of tenants, incur high labor and/or equipment costs, and be noisy. Jacketing the full length of the column just to strengthen the splice would incur higher labor and materal costs than necessary. Jimenez's jacketing technique also enlarges the column, which reduces the leasable floor space available to the building owner. A jacketing scheme which minimizes the change in cross-sectional dimensions and the length of column which needs to be altered would be an attractive alternative.

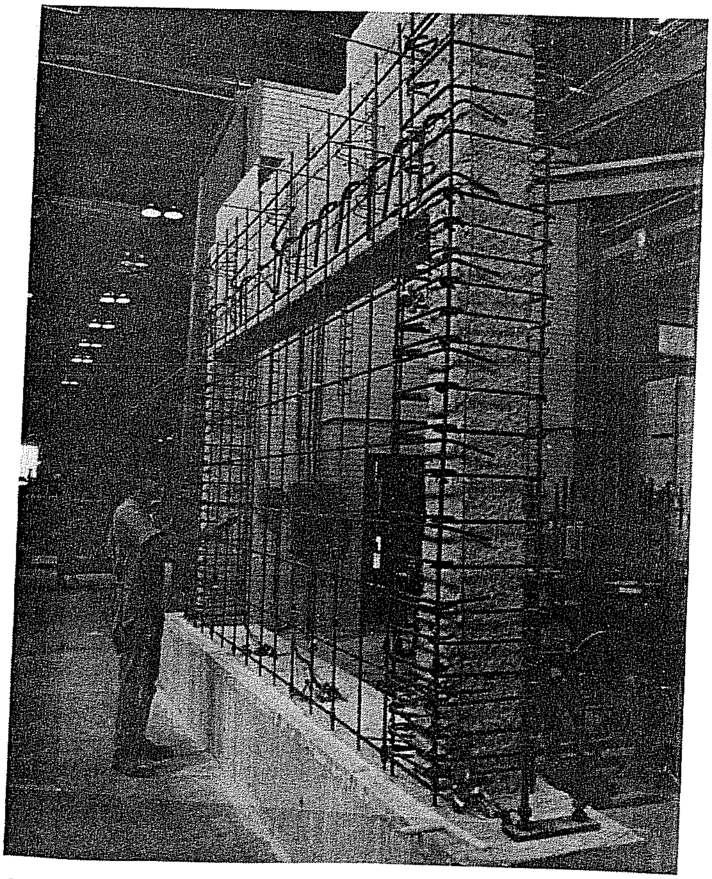


Figure 13 Full view of reinforcing cage of Jimenez's specimen [3].

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1.2.2 Purpose and Scope. Reported herein is part of a continuing project undertaken to observe the response of existing splices in non-ductile reinforced concrete frames subjected to cyclic axial tensile and compressive loading. The splice specimen's design and construction was representative of pre-1960s practice in seismic zones. The goal was to develop yield of the spliced bars in tension. The strengthening techniques implemented were specified to limit the change in column dimensions, as well as to only strengthen the splice region (assuming this is the only location of inadequacy in the column).

CHAPTER 2
METHODS FOR STRENGTHENING COLUMN SPLICES

2.1 Column Design

The existing column cross-section shown in Figure 14 was designed in accordance with ACI 318-56[4]. This 1956 code specified the column splice length to be 24 longitudinal bar diameters (18 inches for #6 bars) and a tie spacing of 12 inches. The ties were fabricated with 90° ends, which was the conventional practice at the time (135° bends are required today).

The strength of the existing splice was estimated using an equation developed by Orangun, et al.[5]. The equation provides a means of calculating the bond stress at failure of a lap splice in tension, considering tie spacing and size, as well as bar spacing, as the primary factors.

$$U'_{calc} = U_{calc} + U_{tr}$$

$$= \left[1.2 + \frac{dC}{d_b} + \frac{50d_b}{\ell_s} + \frac{A_{tr}f_{yt}}{500sd_b}\right]\sqrt{f'_c} \qquad \text{[Ref. (5), Eq. 6]}$$

and

$$\frac{A_{tr}f_{yt}}{500sd_b} \leq 3$$

where

U'_{calc} = single longitudinal bar bond stress with transverse reinforcement (psi)

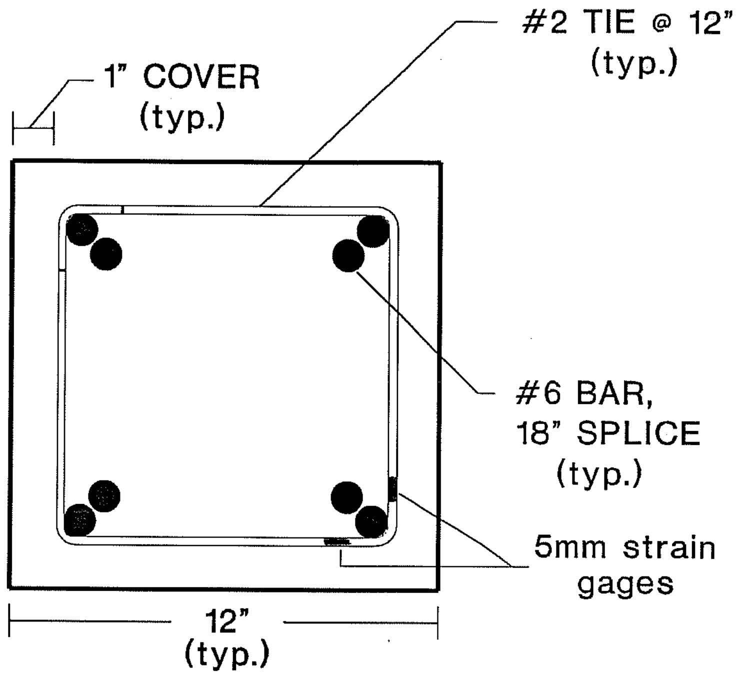


Figure 14. Existing column section.

C = smaller of clear cover and 1/2 clear spacing between spliced bars (in.)

d_b = longitudinal bar diameter (in.)

ℓ_s = splice length (in.)

A_{tr} = area of tie crossing splitting-crack plane (ie. 2 x cross-sectional area of ties ÷ number of splices in splitting plane) (in.) (See Figure 17)

f_{yt} = yield stress of ties

s = spacing of ties (in.)

f'_c = concrete compressive strength (psi)

For the existing splice shown in Figure 14,

C = 1.25 in.

d_b = .75 in.

ℓ_s = 18 in.

A_{tr} = 0.49 in.^{2} (cross-sectional area of #2 tie)

f_{yt} = 71000 psi (actual #2 tie yield stress)

s = 12 in.

f'_c = 3930 psi (See Section 3.4.1 of this report)

U'_{calc} = 568 psi

Predicted failure load = 4 bars X single bar perimeter X U'_{calc} per bar X ℓ_s

= 4 X (πx.75") X 568 psi X 18"

= 96 kips

Load to produce yield in longitudinal bars = 123^k

In the following discussion, various splice strengthening techniques to be evaluated in the report and the continuing project will be presented.

2.2 Splice Strengthening Techniques

2.2.1 Welded Splice. A lap splice is formed by extending bars past each other far enough to permit the force in one bar to be transferred by bond stress through the concrete into the second bar. A lap splice indirectly transfers force between the bars, but a positive connection (such as a weld) directly transfers the force. If the bars can be welded to assure yield, welding would be an obvious choice when selecting a strengthening scheme. Frequently, however, column bars in the field are not well tied (or tied at all) prior to the concrete being cast, and a gap between the bars results. Welding of the splice can still be accomplished, however, by welding a short bar between the spliced bars to bridge the gap and provide an intermediate path by which the force can be transferred (see Figure 15).

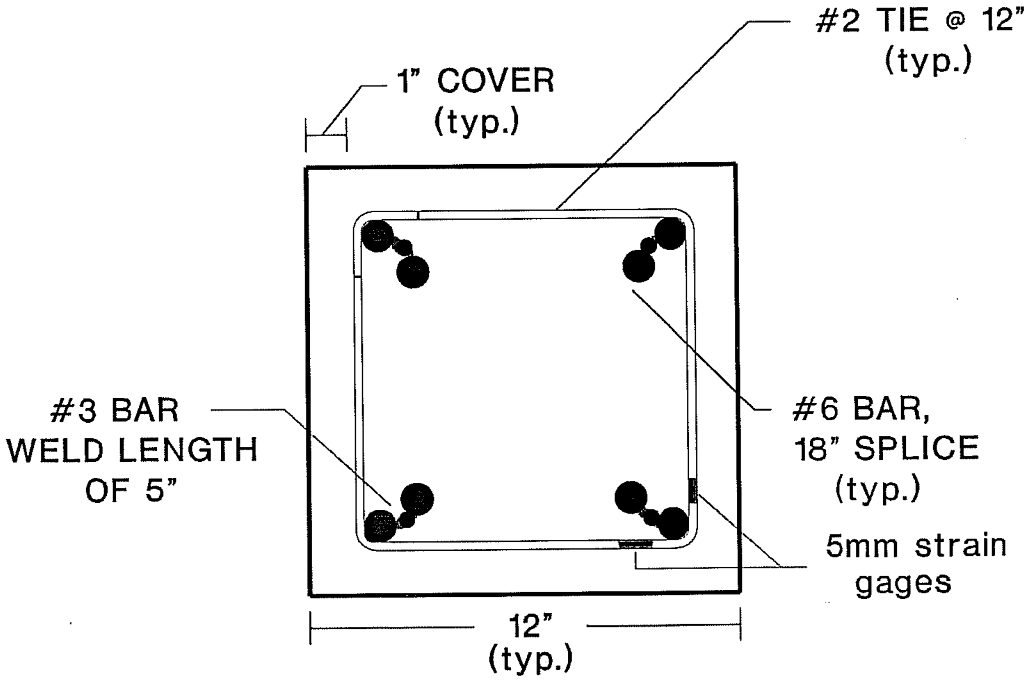


Figure 15. Welded splice scheme.

Though welding spliced bars offers the distinct advantage of creating a positive connection, the disadvantages are also significant. First, the scheme

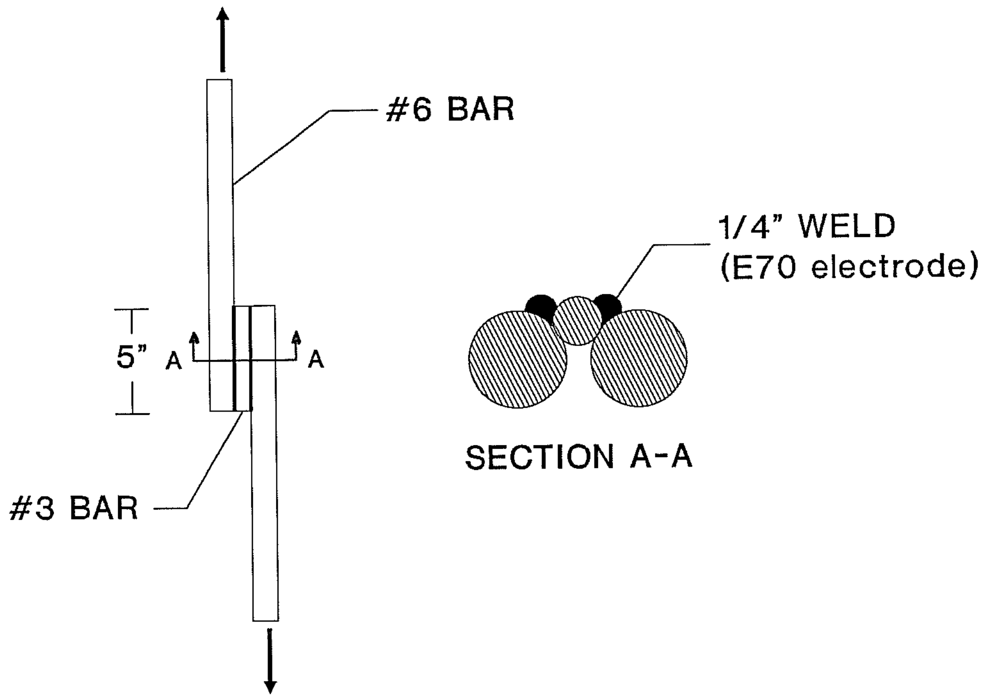


Figure 16 Welded splice detail.

requires a large amount of concrete demolition in order to gain access to the splice. Such demolition is extremely time-consuming, and therefore may be expensive. Secondly, when a small bar is welded between the spliced bars, the weld is only placed along the top face of the bars (see Figure 16). The tensile performance of such a welding scheme is not known. Finally, the chemistry of reinforcing steel in the United States today (and probably 30 years ago as well) is not well controlled and is difficult to weld correctly. Due to the variability of chemistry, the welds may fracture at low load levels and be useless.

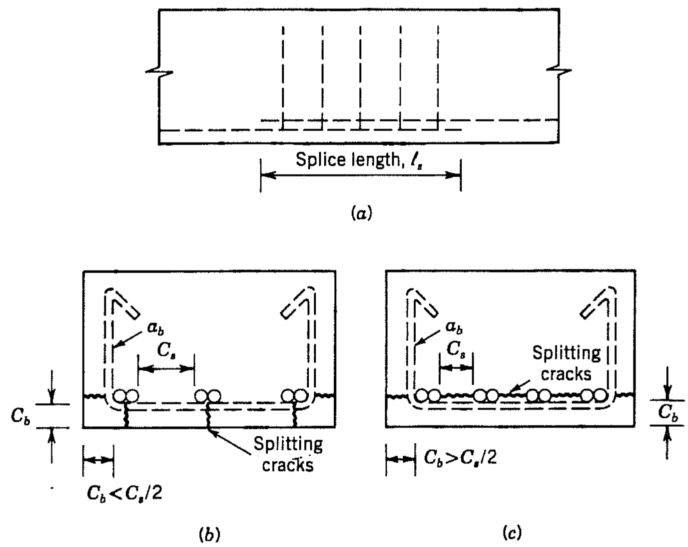


Figure 17 Transverse reinforcement. (a) side view of beam; (b) all splitting cracks restrained, $A_{tr} = a_b$; (c) two bars crossing plane of splitting through layer of bars, $A_{tr} = 2a_b/4$ splices[7].

2.2.2 Steel Straps. A repair technique that was popular in Mexico City following the 1985 earthquake was that of placing structural steel angles on each column corner, then welding steel plates between them (See Figure 1b). This steel acts as supplemental transverse reinforcement. The technique is easy to install as it requires no demolition, but has never been tested as to its value in improving splice performance. One method is to simply clamp the steel angles to the column corners, then weld the plates between the angles (see Figure 18). Another possibility consists of placing a spacer underneath the angle, welding the

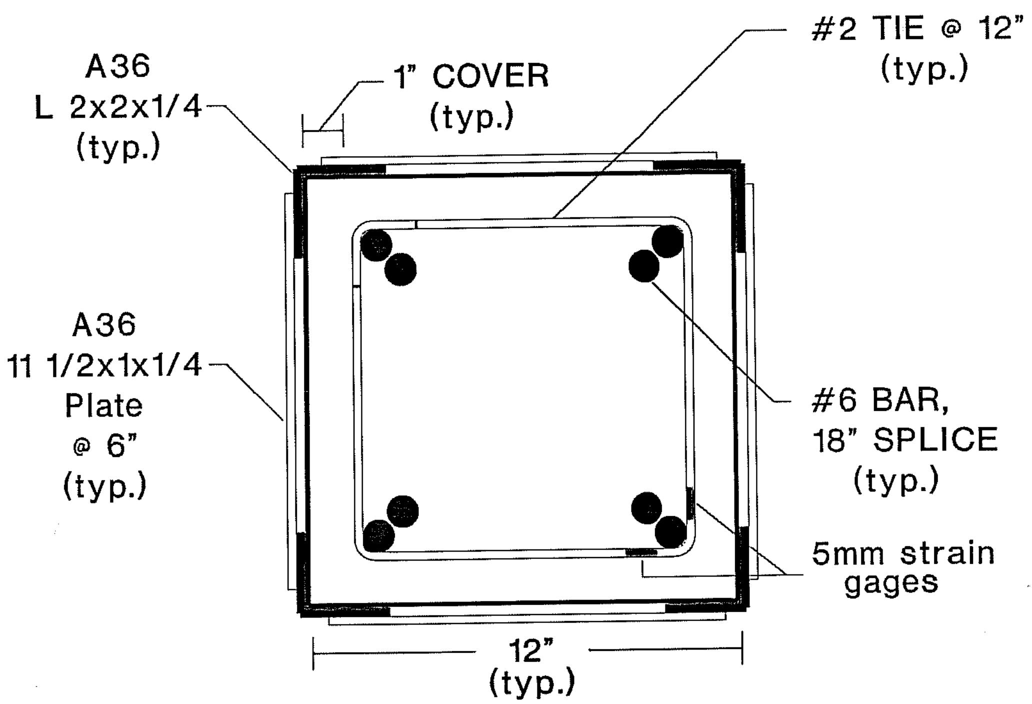


Figure 18. Steel straps, ungrouted scheme.

plates, then placing grout between all the angles and plates and the column surface (see Figure 19).

When tension is applied to a rebar splice in concrete, a crack forms at the end of each bar, followed by longitudinal crack formation along each bar, followed by bar pullout when the concrete cover spalls. The transfer of force from the bar to the concrete is shown in Figure 21. In order to determine the cross-sectional dimensions of the plates, the portion of the equation (5) from Orangun, et al. [5] which provides an estimate of the maximum contributions of transverse reinforcement to bond strength was used as follows:

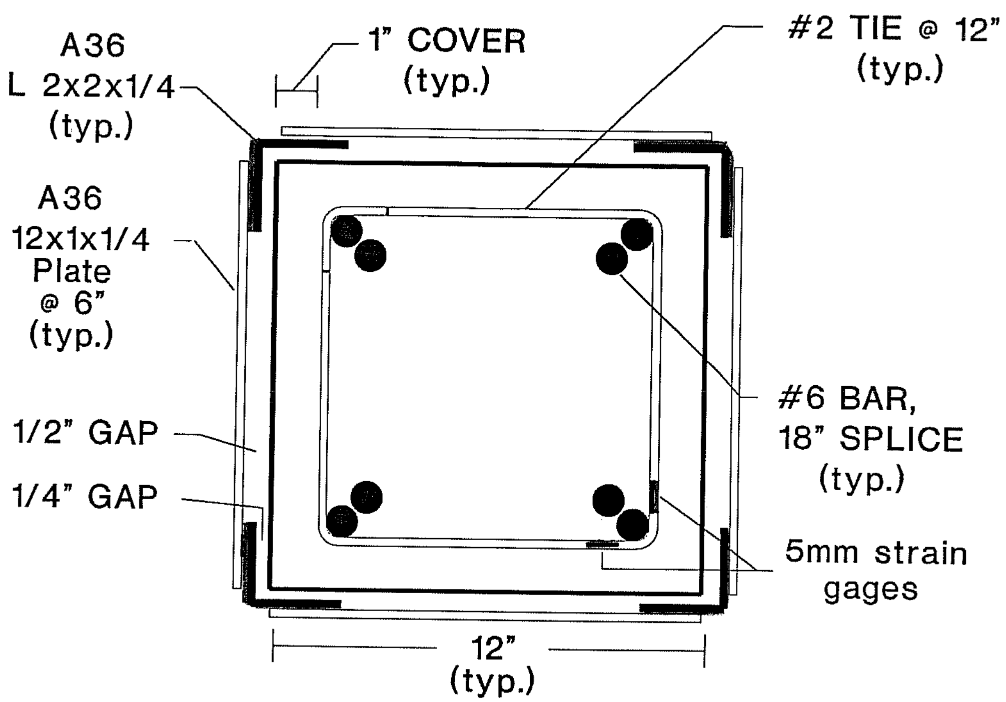


Figure 19. Steel straps, grouted scheme.

$$\frac{A_{tr} f_{yt}}{500 s d_b} \le 3 \qquad \text{[Ref. 5, eq. (5)]}$$

where:

A_{tr}	=	A_{plate} perpendicular to plane of splitting
f_{yt}	=	60 ksi for ties (used nominal yield for this design)
s	=	6 in.
d_b	=	.75 in.

A_{plate} x 3600 psi $\le$ 3 X 500 X 6 in. X .75 in.

$A_{plate} \ge .188 \text{ in.}^2$

<u>USE 1 X 1/4 CROSS SECTION</u> (A_{plate} = 0.25 in.^{2})

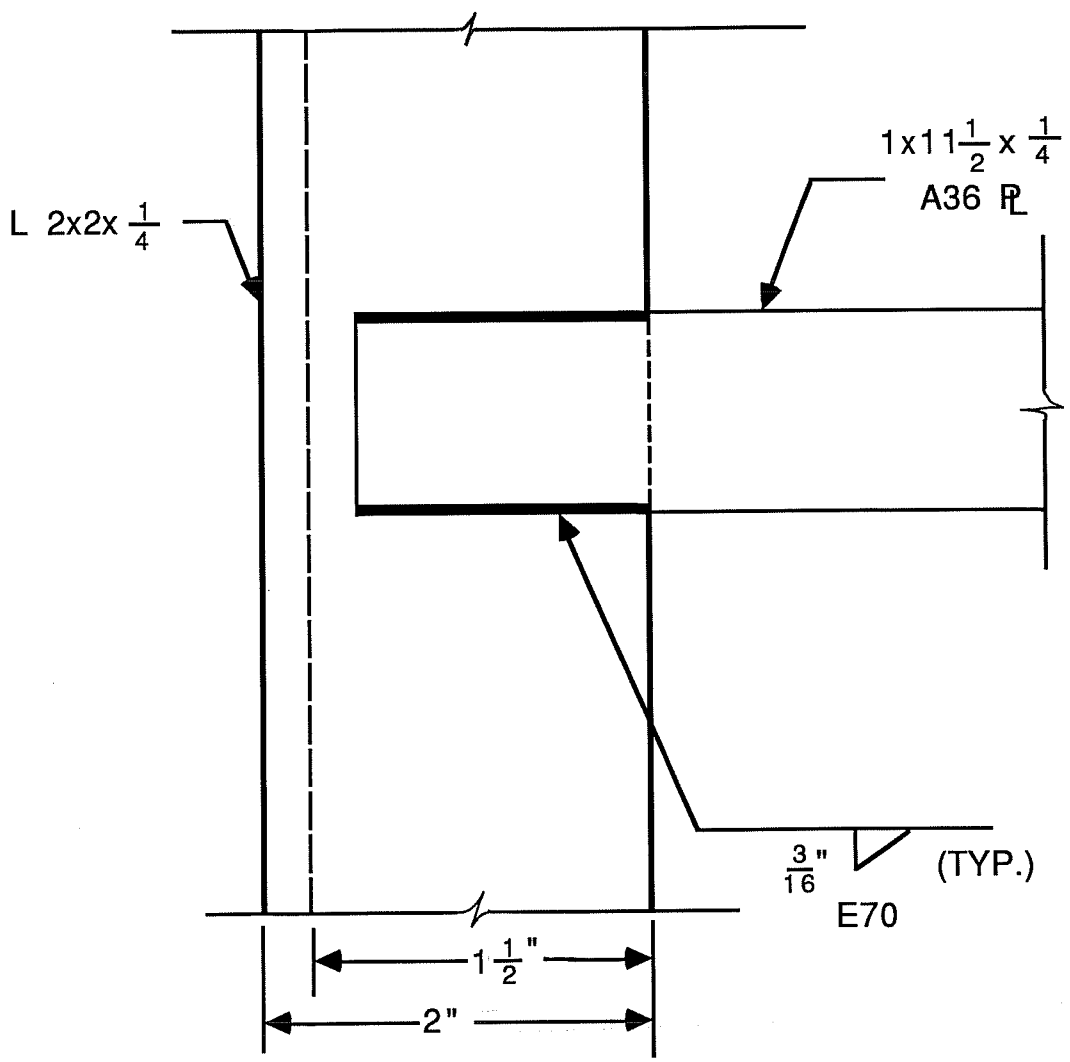


Figure 20. Details of plate-angle connection.

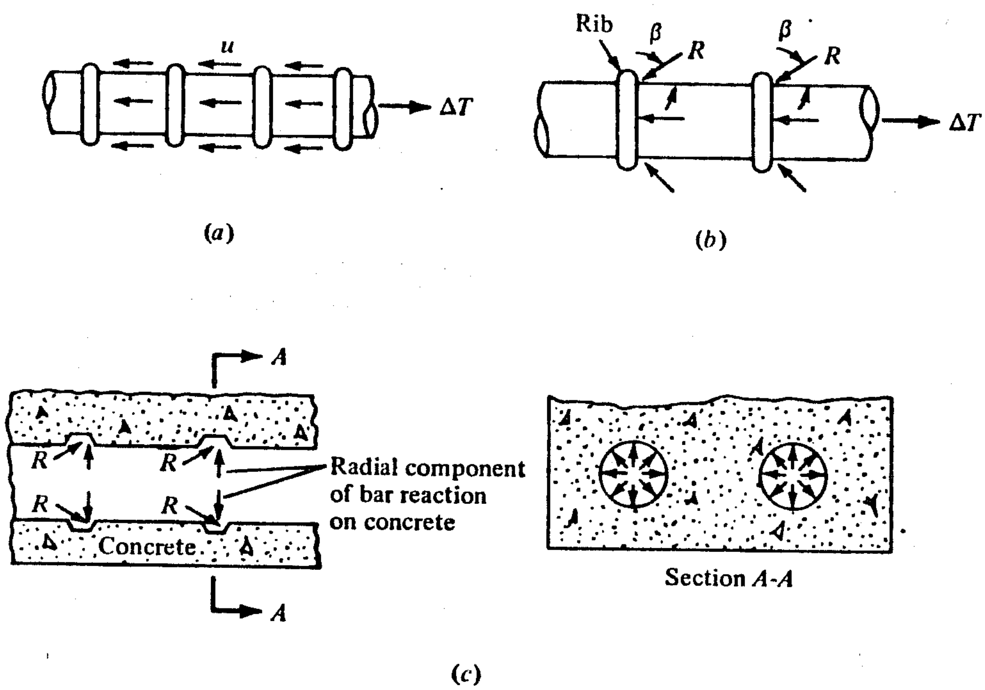


Figure 21. Sources of bond strength: (a) bond stresses due to friction and chemical adhesion between concrete and steel; (b) free-body diagram of reinforcing bar showing reaction of concrete on ribs; (c) free-body diagram of concrete showing forces exerted by ribs on concrete: radial component of bar reactions acts to produce cracking and spalling of concrete surrounding the bar.

The steel angle size was chosen arbitrarily (what might be practical in the field), then checked to assure the angle will not yield in flexure. Figure 22 shows the force W as a reaction load to the maximum developed in the plates, $A_{pl}f_y$. The steel angle is treated as a series of fixed end beams carrying a uniform load of W/S (plate spacing). For the details of the test columns, the angle size

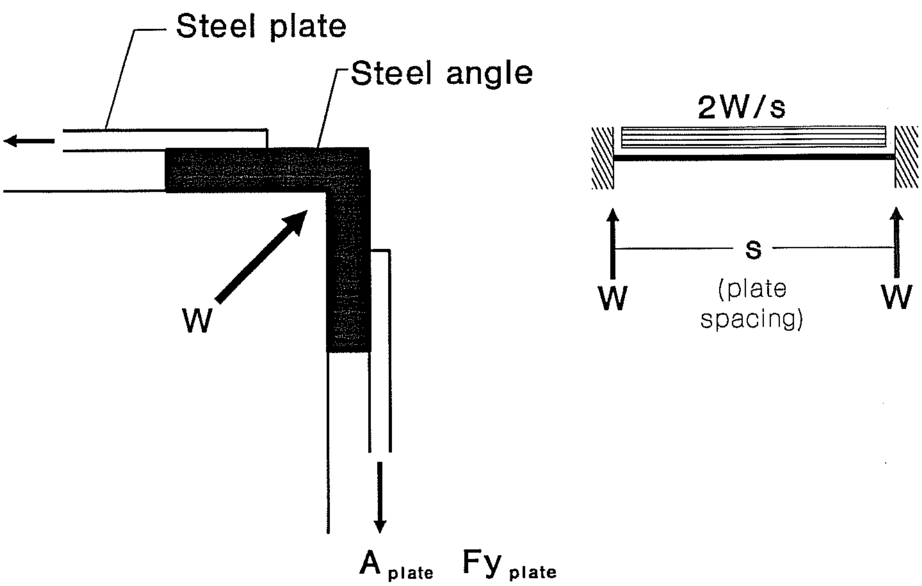


Figure 22. Forces used in flexural check of steel angle.

required was calculated as follows:

$$A_{pl}\,f_y \leq 3 \text{ X } 500 \text{ X } S \text{ X } d_b$$
$$= 1500 \text{ X } 6 \text{ in. X } .75 \text{ in.}$$
$$= 6750 \text{ lb.}$$

$$W = \sqrt{(A_{pl}\,f_{yt})^2 + (A_{pl}\,f_{yt})^2} = \sqrt{2\,(6750\text{lb.})^2}$$
$$= 9545 \text{ lb. (is equivalent concentrated load)}$$
$$W/S = 9545 \text{ lb/6 in.}$$
$$= 1591 \text{ lb./in. (transformed W into uniform load)}$$

$$M = \frac{(W/S) \text{ X } (S)^2}{12}$$
$$= \frac{1591 \text{ lb./in. X } (6 \text{ in.})^2}{12}$$
$$= 4.77 \text{ kips/in.}$$

For L 2 X 2 X 1/4,

$I = r^2A$
$= (.391 \text{ in.})^2 (.938 \text{ in.}^2)$
$= .143 \text{ in.}^4$
$C =$ Location of fiber furthest from centroid of steel angle
$= .805$ in.

$$MAXIMUM\ FLEXURAL\ STRESS = \sigma_{MAX} = \frac{M_c}{I}$$

$$\sigma_{MAX} = \frac{4.77 \text{ kips/in.} \times .805 \text{ in.}}{.143 \text{ in.}^4}$$
$$= 26.8 \text{ ksi} < f_y$$

<u>USE L 2 X 2 X 1/4</u>

<u>2.2.2.1 Steel straps, grouted.</u> The plate sizes and spacings and angle size (shown in Figure 23) are the same for the grouted and ungrouted techniques. A dry-pack grout (an expansive grout mixed with minimal water) can be used to assure contact between the steel and concrete. The grout provides positive contact around the column perimeter as well as along the length of the strengthened region at the corners. However, hand-packing the grout is a time-consuming process, and therefore may be impractical for field installation. Placing or injecting a grout cover over the entire strengthened region might be more practical and economical. Placing the cover provides the same contact between the steel and concrete, and may be easier to accomplish. Such a cover is visually appealing, and provides corrosion and fire protection, if needed.

<u>2.2.2.2 Steel straps, ungrouted.</u> The installation of this technique consists only of slightly chipping the column corners (so that the steel angle fits properly),

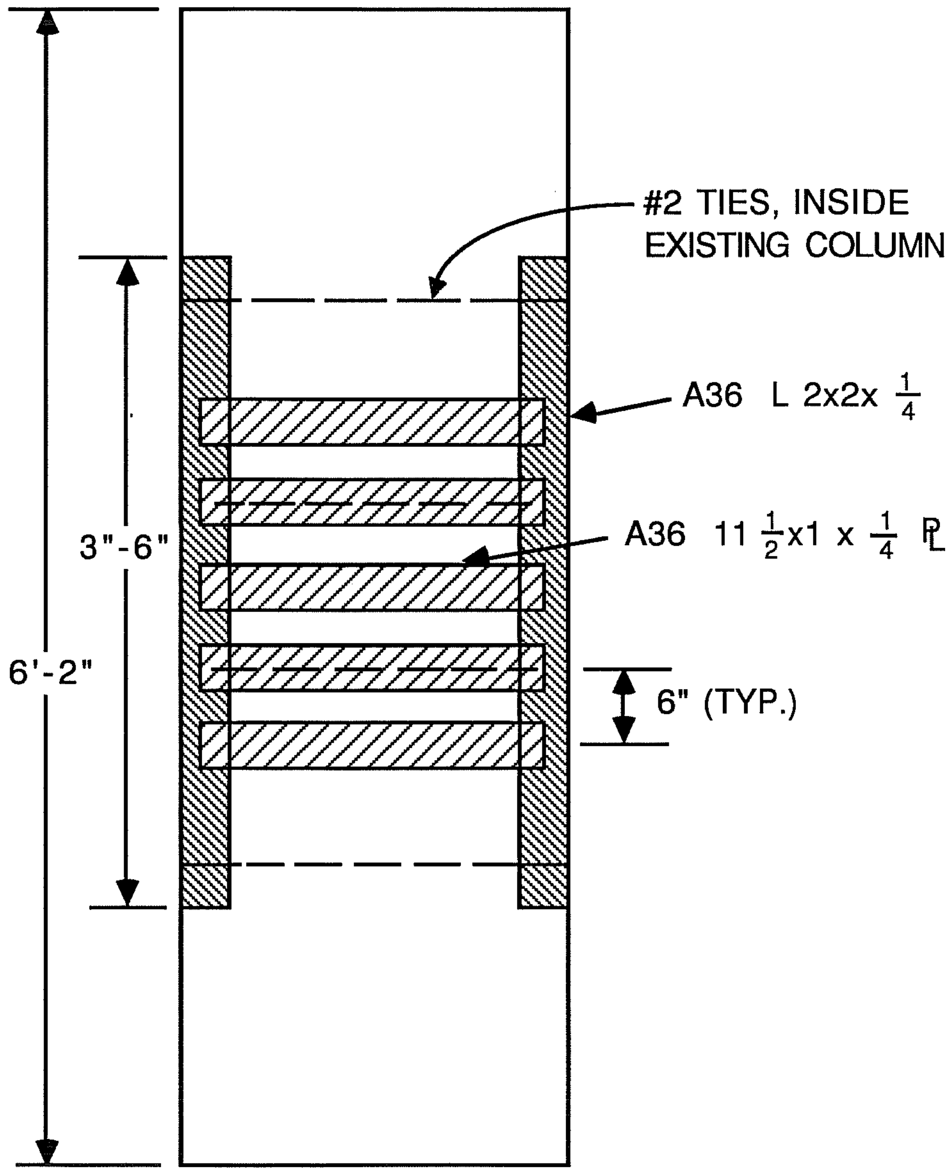


Figure 23. Elevation view of steel straps scheme.

clamping the angles to the corners and welding the plates. Since it is easy to construct, and therefore requires fewer man-hours to install, it should be economically attractive.

2.2.3 External Ties. This technique of increasing confinement consists of adding sufficient ties to satisfy ACI 318-89, Section 21.4.4, provisions for a boundary element of a structural wall. The tie size and spacing should also be checked against the Orangun equation in section 2.2.2 of this report. For Eq. (5) of Orangun, et al,

$$A_{tr} = .196 \text{ in.}^2,\ f_{yt} = 6000 \text{ psi},\ S = 3 \text{ in.},\ d_b = .75 \text{ in.}$$

Since

$$A_{tr} = .196 \text{ in.}^2 > \frac{3 \text{ X } 500 \text{ s X } d_b}{f_{yt}}$$

$$= \frac{1500 \text{ X } 3 \text{ in. X } .75 \text{ in.}}{60000 \text{ psi}} = .0563 \text{ in.}$$

$$A_{tr} \text{ provided} = 0.20\ (\#4) > .0563 \text{ in.}^2$$

U-shaped stirrups are placed at specified locations, then the overlapping legs are welded (see Figures 24, 25, 26). In order to obtain contact with the column surface around the perimeter, the ties must be bent to close tolerances. For aesthetic reasons, corrosion and fire protection, a grout cover may be cast.

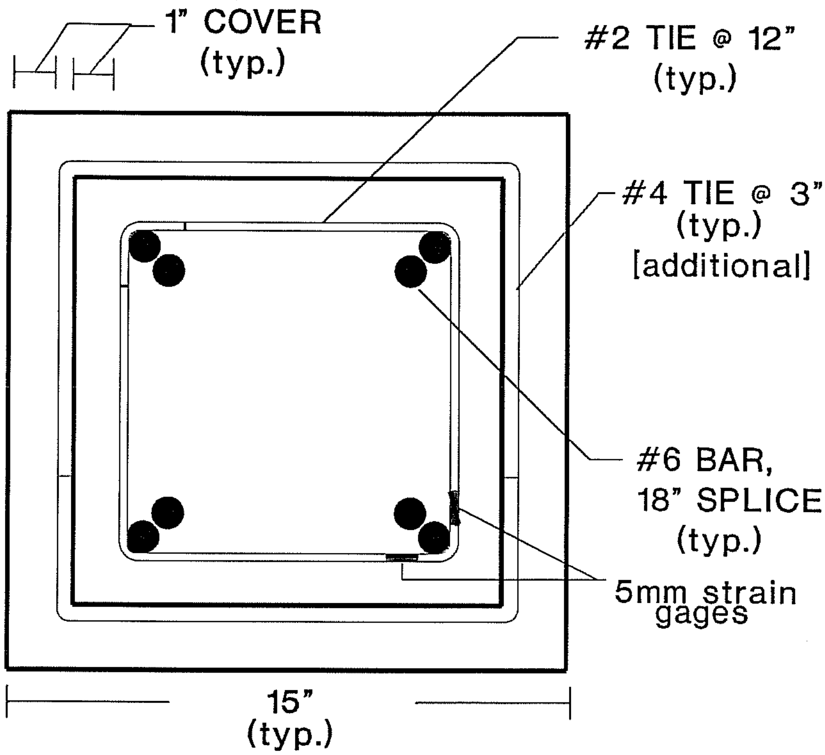


Figure 24. External ties scheme.

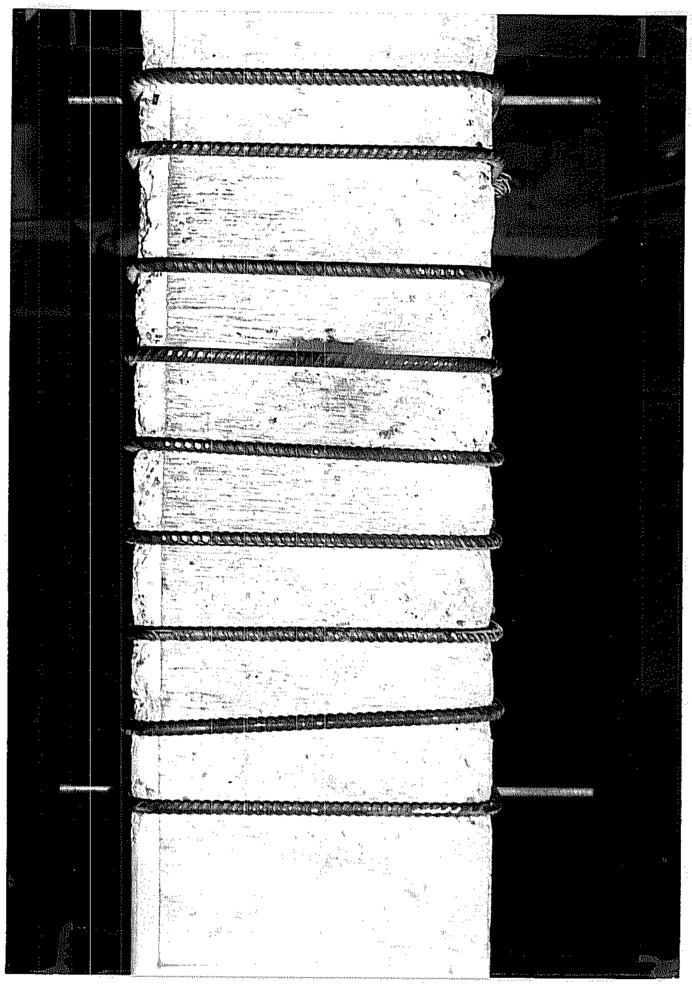


Figure 25. External ties elevation view.


Figure 26. External ties, weld detail

CHAPTER 3
SPECIMENS AND TEST SETUP

3.1 Design of Test Specimens

3.1.1 Existing (Unstrengthened) Splice Specimens. The prototype member that was considered was an 18-in. x 18-in. reinforced concrete column with #9 longitudinal bars and #3 ties that was part of a laterally loaded infilled frame. The 2/3-scale model of the prototype is shown in Figure 27. Only the splice region of the column was modeled due to the fact that only pure axial loads are imposed on the column in this configuration. Tension was applied to each specimen by means of the end connections shown in Figures 28, 29 and 30. The tensile force was transferred to the splice by means of the nut-plate assembly shown. The plate was designed not to deform appreciably under the design load.

Two unstrengthened specimens and three specimens strengthened with ungrouted steel straps were tested. The other techniques will be tested in a subsequent phase of the project.

3.1.2 Steel Strap, Ungrouted Specimens. The plates were located over the existing #2 ties in the specimens (see Figure 23) because, in the field, tie locations will not necessarily be known. Since large transverse reinforcement at wide spacing less effectively confines cracks than small transverse reinforcement

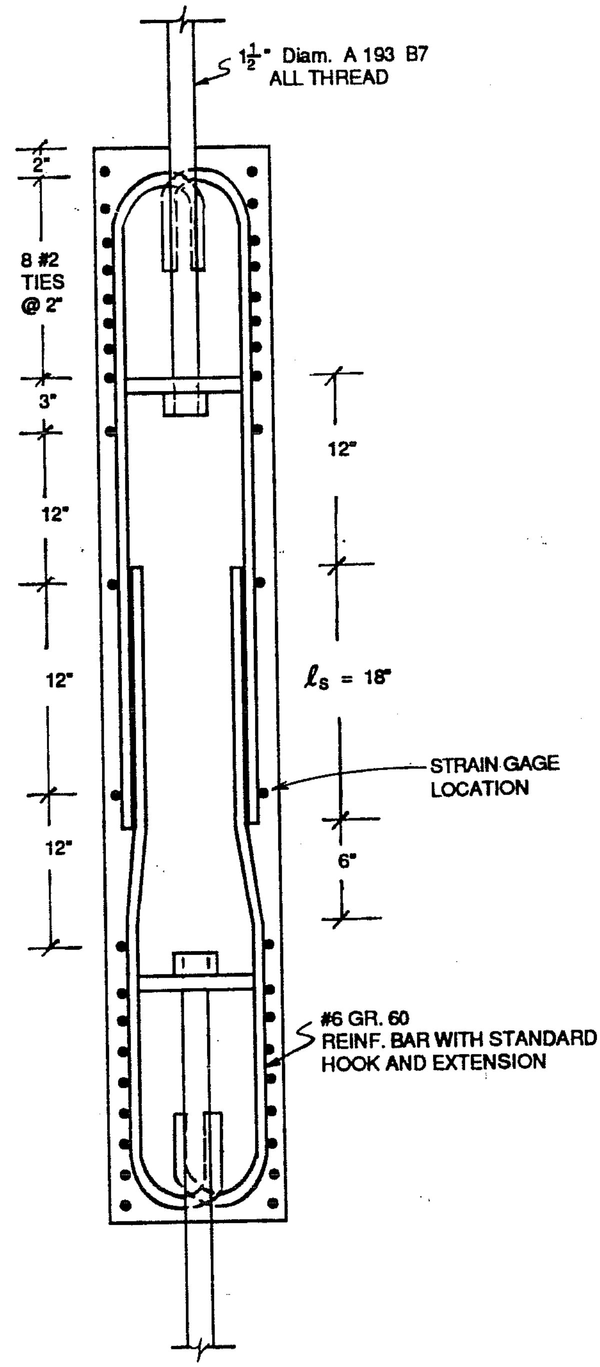


Figure 27 Full elevation section of existing column specimen.

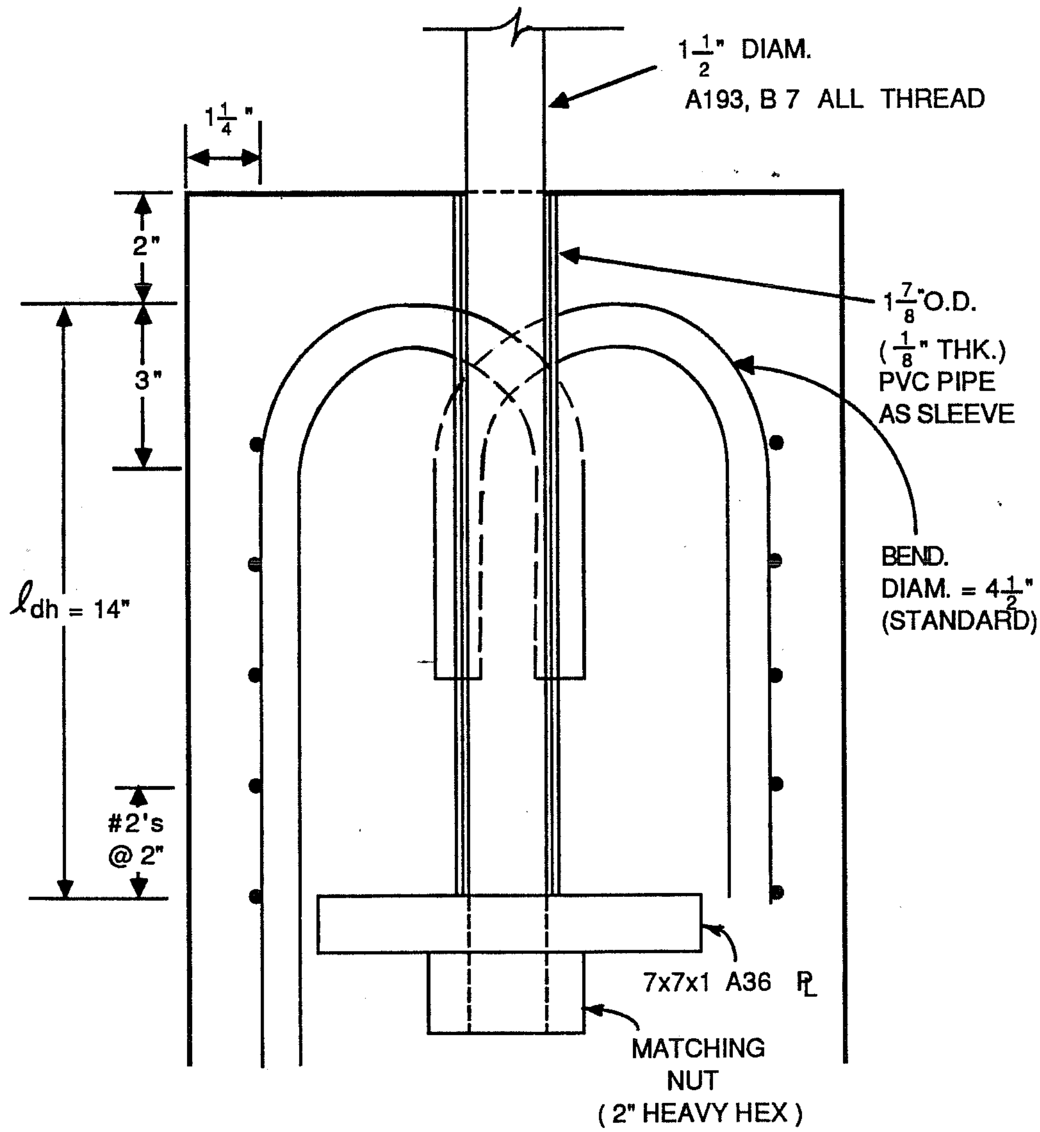


Figure 28. End connection detail.

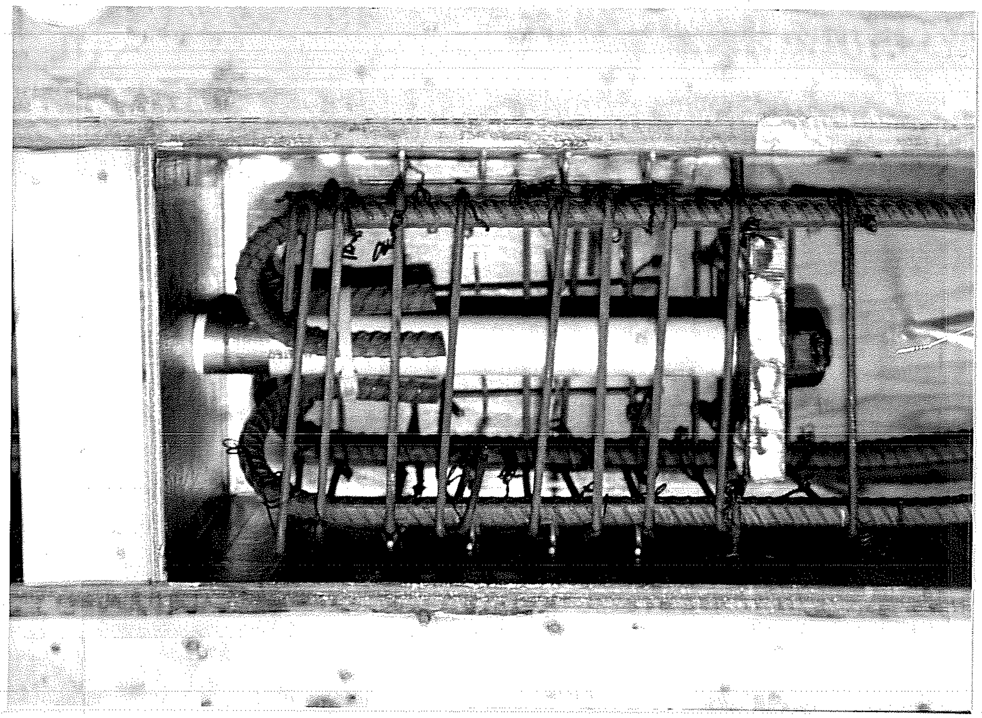


Figure 29. End connection detail.

at small spacings, placing the plates directly over existing ties gives the worse case possible in the field.

3.2 Nomenclature for Specimens

U - B - 1

U: Unspaced splice (tight lap splice, no space between spliced bars)

B: (base) Existing splice

1: Specimen #1 (2 specimens tested)

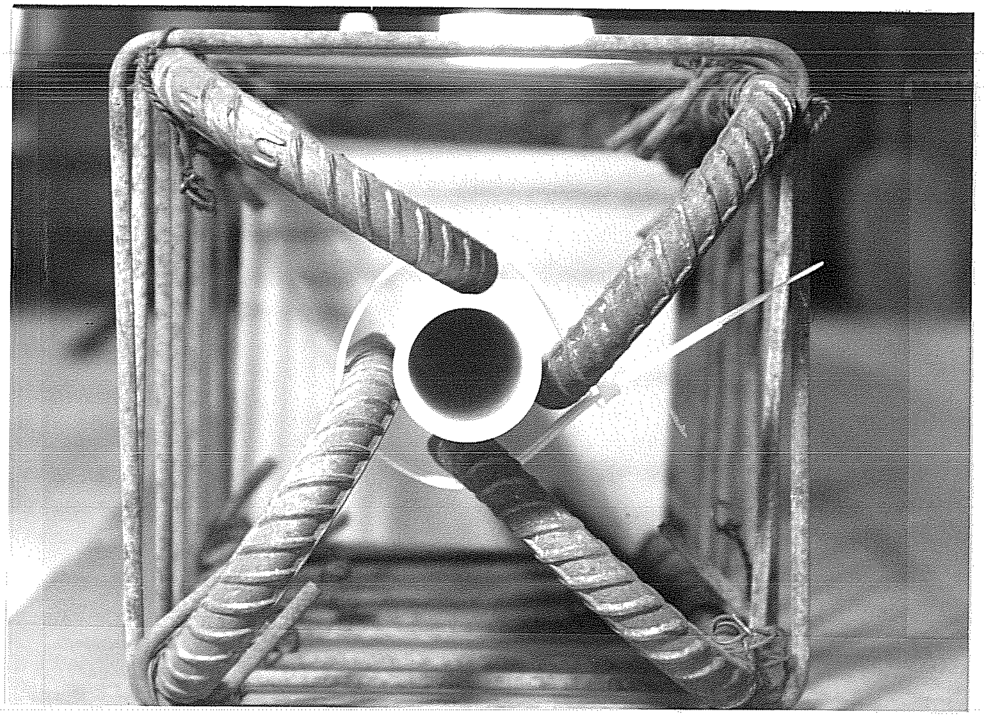
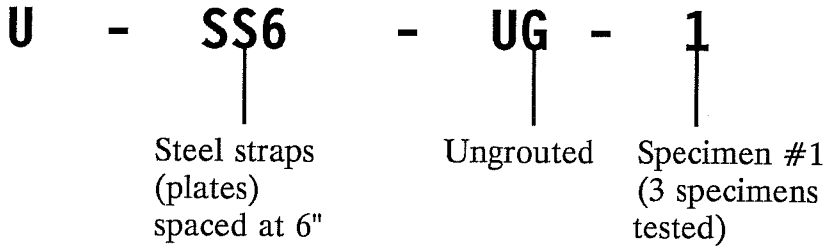


Figure 30. End connection, end view.



The above nomenclature was used to identify the specimens tested.

3.3 Test Setup

3.3.1 Test Frame. The frame was designed to be a self-reacting apparatus (see Figure 31). To apply tension to the specimen, a 1-1/2 in. diameter ASTM 193 all-thread rod was connected to the embedded nut-plate assembly (discussed in Section 3.3.2) in each end of the specimen. Rams then

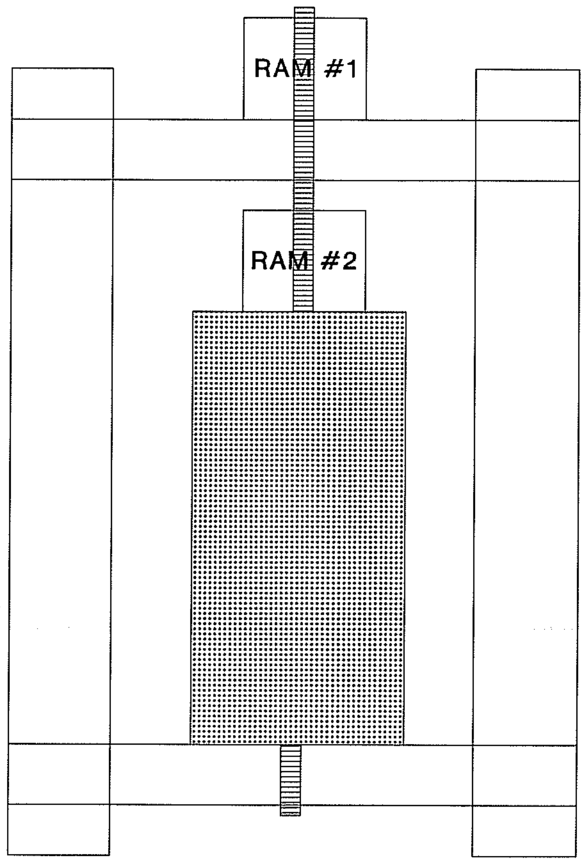


Figure 31. Testing apparatus schematic.

acted on the rod. To apply compression, Ram 2 reacted against the bottom of the top beam in the frame. When one ram was acting, the other ram's piston was withdrawn to assure that only a single ram would act on the specimen. Each ram had a 100-ton capacity.

3.3.2 Data Acquisition System. A 100-ton capacity load cell was placed under each hydraulic ram to measure the load throughout the test. A pressure transducer was also used, in order to verify the load exhibited by the load cell.

Linear potentiometers were used to measure displacement across the splice region (Figure 32). Load and displacement were continuously monitored by an X-Y plotter.

Strain gages were placed on a single reinforcing tie at the critical section in each specimen (see Figures 14, 18, 27). Also, in one of the steel strap specimens, strain gages were placed on the plates at the critical section, to measure stress in plates.

A PC-based data acquisition unit was used to acquire the data. The computer scanned all channels at approximately 20,000 pound (20 kip) increments, and data was recorded on both a printout and a floppy disk.

3.4 Characteristics of Materials

3.4.1 Concrete. The nominal 28-day concrete strength was 3000 psi for each specimen. All specimens were cast at the same time from the same batch of ready-mix concrete. The maximum size of the coarse aggregate was 3/8-in. (2/3 scale of 5/8"). The mix was a 4-sack, 5-in. slump mix. The concrete compressive strengths are shown in Table 1.

Figure 32. Potentiometer setup.

TABLE 1

TIME	(psi)	
7 days	2680	
28 days	3510	
44 days First specimen tested	3850	Average 3930
92 days Last specimen tested	4010	

3.4.2 Steel.

3.4.2.1 Reinforcing bars. Grade 40 was typically used in pre-1960's buildings. However, as Grade 40 steel is extremely difficult to obtain today, Grade 60 steel was used for all rebar. The #2 column ties had a yield stress of 71 ksi, while the #6 longitudinal bars possessed a yield stress of 69 ksi.

3.4.2.2 Strengthening steel. Conventional A36 steel was used for the steel straps and angles. The 2 x 2 x 1/4 steel angles' yield stress was 53.6 ksi, while the 2 x 1/4 steel plate yield was 53.2 ksi.

CHAPTER 4
EXPERIMENTAL RESULTS

4.1 Load History

The bond strength of a reinforcing bar deteriorates more rapidly when loaded cyclically than when loaded monotonically, due to slip reversal. Each of the specimens tested was subjected to at least the first two cycles of the nominal load history shown in Figure 33. The specimens were loaded through cycles 1 and 2, then loaded in tension to yield of the spliced bars (if possible).

4.2 Response Under Cyclic Loading

The tests performed served primarily to discover flaws in the test setup, but some useful information can still be gleaned from the results. Each of the five specimens tested exhibited similar behavior throughout cycles 1 and 2, though at higher load levels the similarities were few. Once the first cracks were opened in tension in cycle 1, a relatively constant axial stiffness (slope of axial load versus deformation response curve) was observed through the remainder of cycles 1 and 2. Normally the first cracks formed over the ties in the splice region. These cracks sometimes progressed into splitting cracks.

In each of the axial load versus deformation response curves, tensile loads and deformations are shown as positive and compressive loads and deformations are negative.

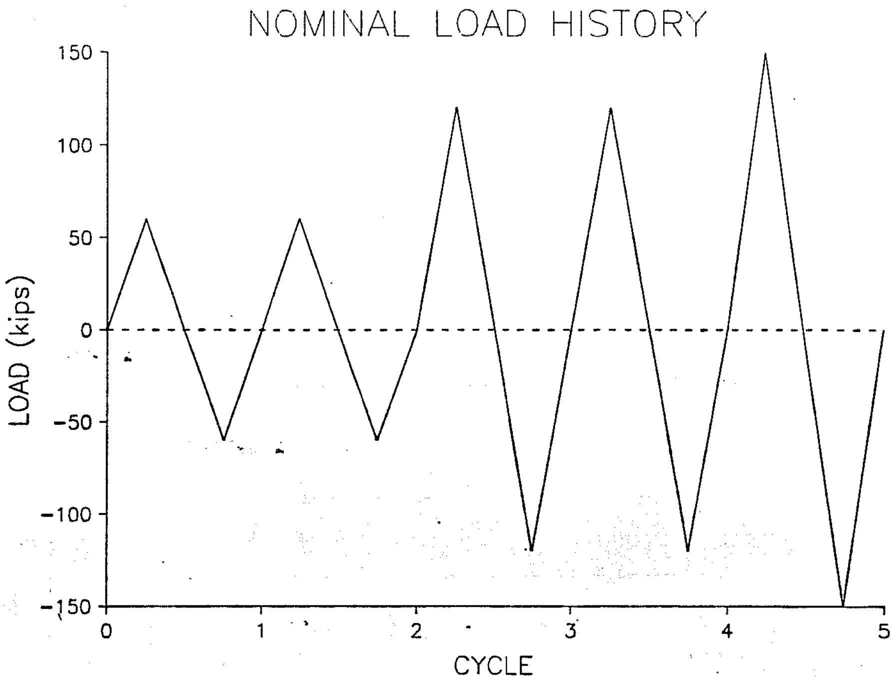


Figure 33. Desired load history.

4.2.1 Unstrengthened Specimens. Two unstrengthened specimens were tested. The response of Specimen U-B-1 is shown in Figure 34. The splice failed suddenly in tension at approximately 85 kips (about .7 F_y) in cycle 3. The initial cracks formed over the ties in the splice region and then tensile splitting cracks formed along the splice as shown in Figure 35. The failure occurred at the two locations shown in Figure 35. Following tensile failure, the reserve compressive strength of the specimen was approximately 60 kips. All tie strain gages in this specimen were found to be inoperative. Specimen U-B-2 exhibited a similar

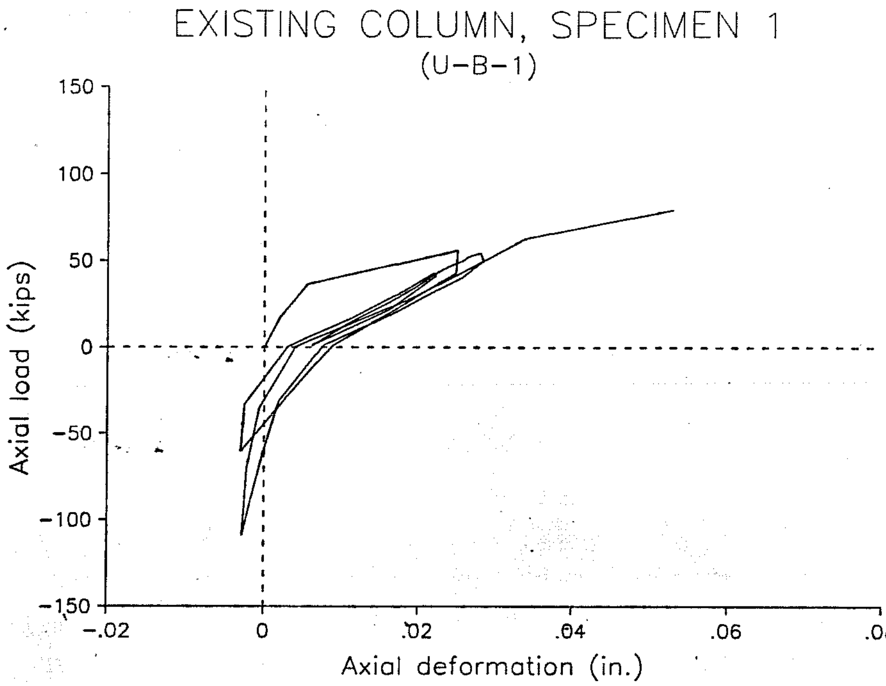


Figure 34. Response of existing column specimen 1 (U-B-1).

behavior to that of U-B-1 (see Figure 35), but a nut in one end connection failed at approximately 80 kips. This premature failure prevented further testing.

The actual clear cover over the spliced bars of U-B-1 was 1-1/2 in. where failure occurred. Using this value for C in equation (6) of Orangun, et al., presented in Section 2.1 of this report, the theoretical failure load is 108 kips, 25% greater than the actual failure load of approximately 85 kips.

4.2.2 Strengthened Specimens. Three specimens were strengthened using the steel strap, ungrouted technique. The strengthened specimens exhibited a larger tensile capacity than the unstrengthened specimen, but the degree of improvement varied. Specimen U-SS6-UG-1 carried a maximum tensile load of

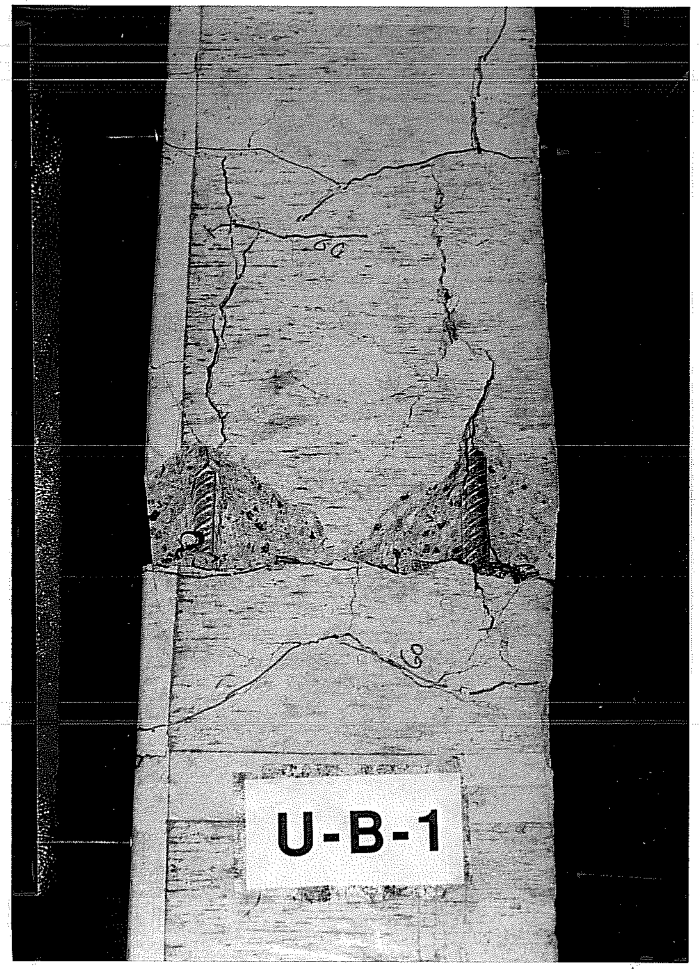


Figure 35. Failure mode of U-B-1.

90 kips (see Figure 37) while U-SS6-UG-2 and U-SS6-UG-3 reached 120 kips (see Figures 38 and 39). The end connection hooks failed at 120 kips (the yield load). Specimen U-SS6-UG-1 failed in tensile splitting while the confining steel in Specimens U-SS6-UG-2 and U-SS6-UG-3 was tight enough on the column to hold the splitting cracks closed. Though U-SS6-UG-1 failed in a different mode, the strengthening technique still improved the splice performance by preventing

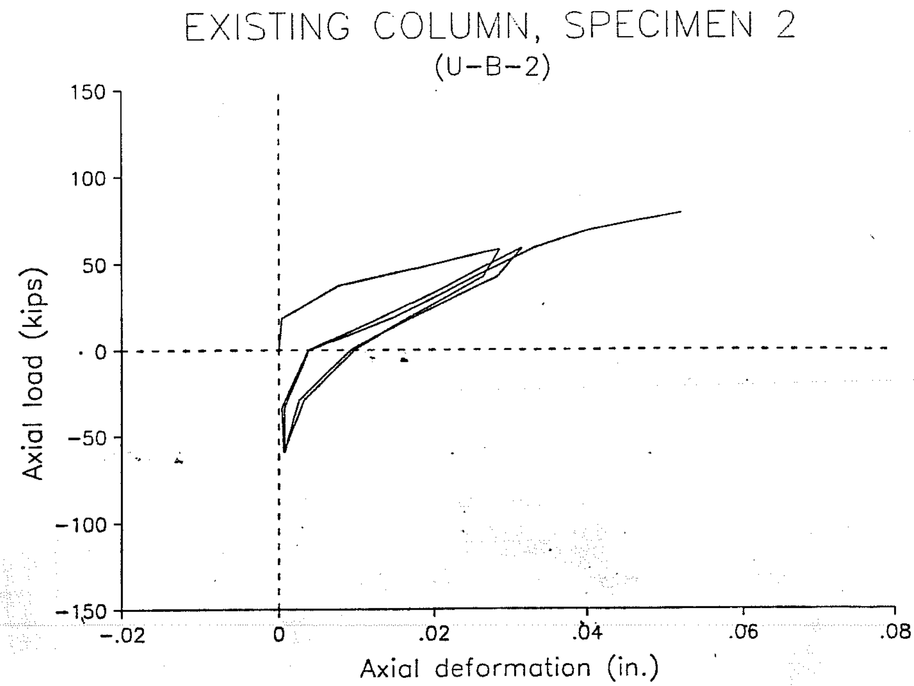


Figure 36. Response of existing column specimen 2 (U-B-2).

the concrete from spalling off when even large cracks opened.

Table 2 gives a summary of the test results.

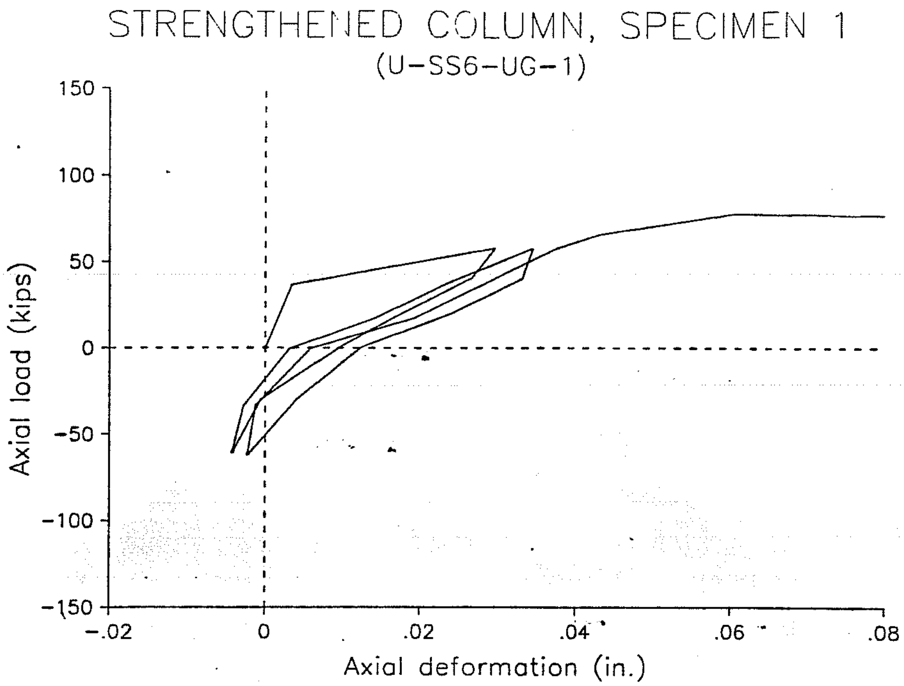


Figure 37. Response of steel straps, ungrouted specimen 1 (U-SS6-UG-1).

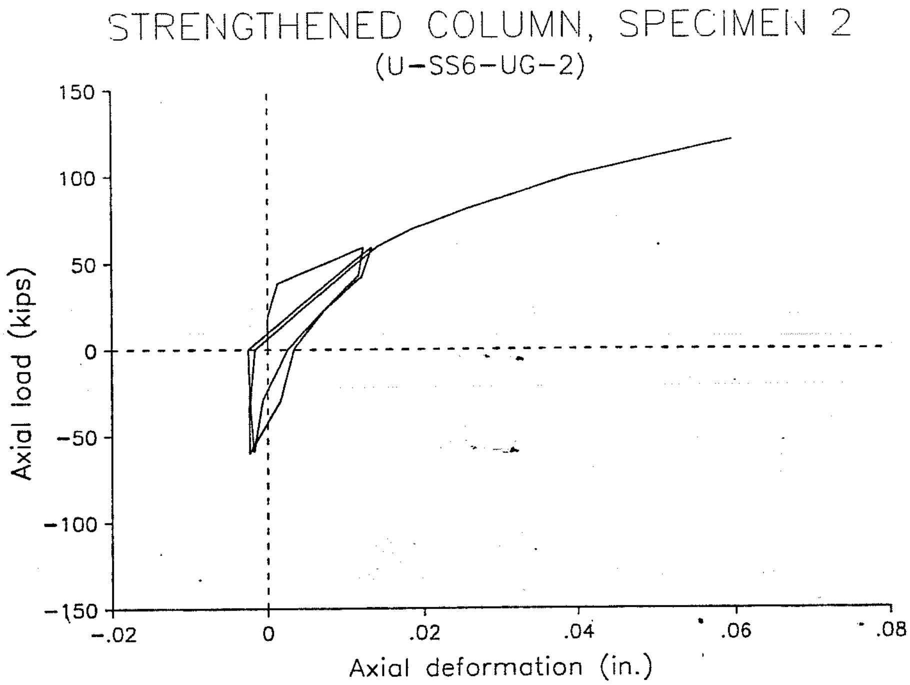


Figure 38. Response of steel straps, ungrouted specimen 2 (U-SS6-UG-2).

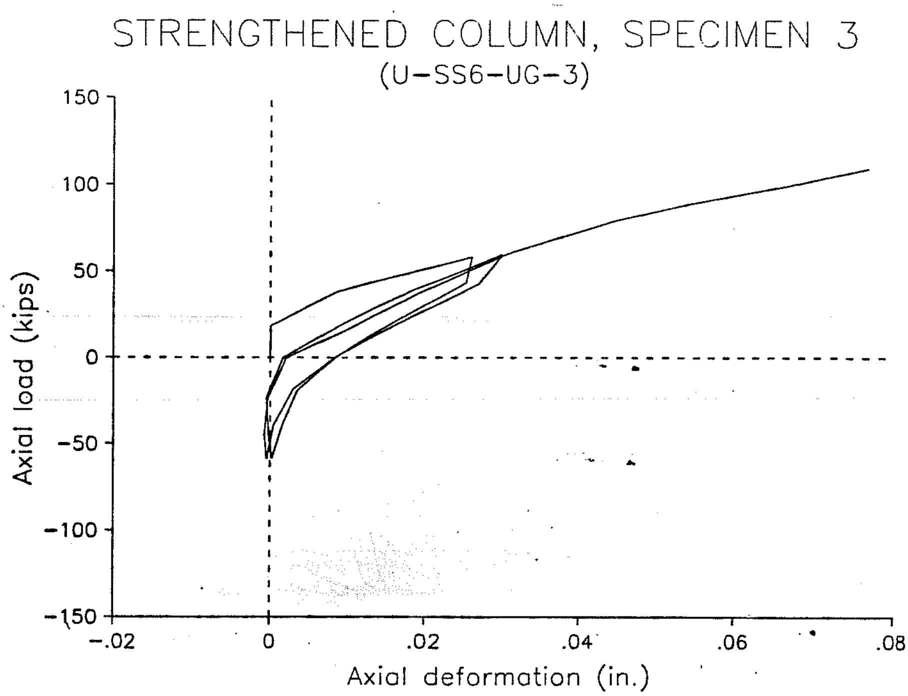


Figure 39. Response of steel straps, ungrouted specimen 3 (U-SS6-UG-3).

TABLE 2 TEST SUMMARY			
SPECIMEN	MAXIMUM LOAD (kips)	TENSILE FRACTION OF F_y	FAILURE MODE
U-B-1	85	.71	Brittle, tensile splitting
U-B-2	80	.67	Nut failure in end connection
U-SS6-UG-1	90	.75	Tensile splitting
U-SS6-UG-2	120	1.0	Hook failure in end connection
U-SS6-UG-3	120	1.0	Hook failure in end connection

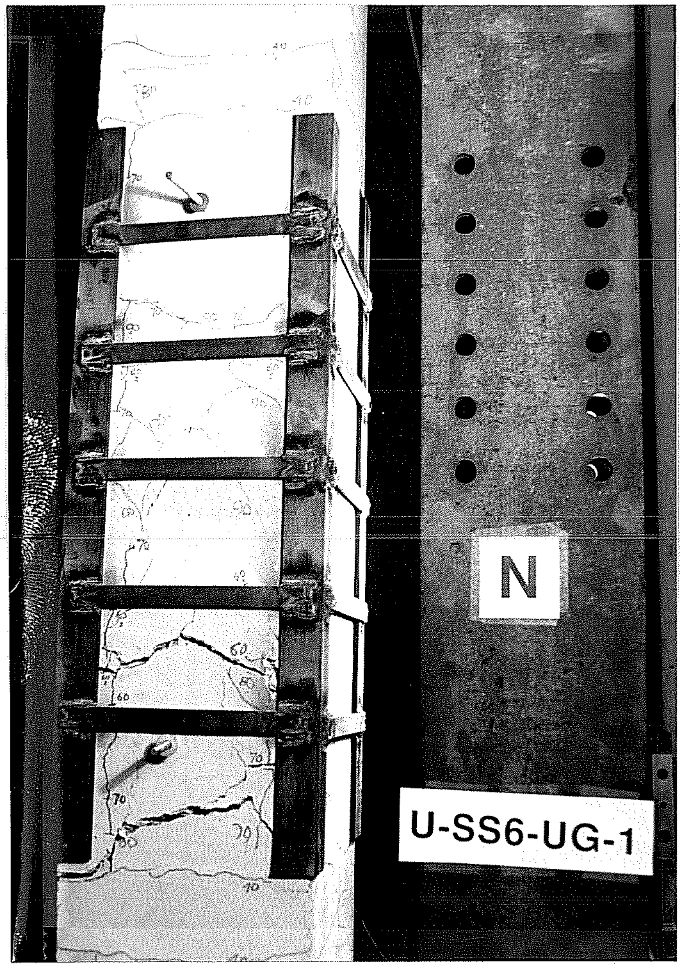


Figure 40. Failure mode of specimen U-SS6-UG-1.

Figure 41. Failure mode of specimen U-SS6-UG-2 and U-SS6-UG-3.

CHAPTER 5
SUMMARY AND CONCLUSIONS

5.1 Summary

5.1.1 General. Five specimens were tested. Each specimen was a 2/3-scale model of a column splice in a non-ductile reinforced concrete frame strengthened by an infill wall. The ungrouted steel strap strengthening technique applied to the specimens improved the cyclic behavior of the splice.

5.1.2 Discussion. Though the steel strap ungrouted technique was easily built, the need for contact with the concrete along the length of each steel angle was difficult to consistently provide. The control of the quality of construction was difficult to provide for the following reasons:

- The concrete surface near the corners must be square (90°) and smooth, in order to provide good contact with the steel angle;
- The steel angles must be tight on both sides of each corner, which is difficult to control due to the following possibilities:
  - the steel angle is not straight along its length and/or not square due to fabrication;
  - the column surface is not square and/or smooth due to fabrication and/or weathering.

Though the results are by no means extensive, for each specimen tested (unstrengthened and strengthened), the splice appears to have a reliable and consistent performance at .5 F_y and lower in tension.

The ungrouted steel strap confinement technique may actually increase the tensile capacity of the plain concrete. In conventional reinforced concrete, transverse (and longitudinal) reinforcement carried no load until the concrete cracks. Cracking normally occurs, however, outside the confined core. By using the steel strap confinement, the steel angles and plates hamper small cracks from growing and the concrete from spalling.

Also, failure in each specimen occurred on the same face of the column. The specimens were cast horizontally. No top bar effect had been anticipated since there was less than 12" of concrete below the bars (see ACI 318-83), but each specimen failed at the top face.

5.2 Conclusions

1. Column reinforcement lap splices with insufficient splice length and ties at wide spacings will most likely fail in tension in a brittle manner.

2. Using ungrouted steel straps to strengthen the splice improved the performance though the degree of improvement is uncertain.

3. In order to prevent failure of the end connection of the specimen, an external confinement scheme should be applied to the connections.

4. Since each specimen showed signs of failure on the "top" face, similar specimens in the future should be cast vertically. Also, to minimize eccentricities that would aggravate such a top-bar effect, bracing of the specimen in both directions should be used during testing.

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