Research Report

Research Project T9902
Shaft/Column Connections - Experimental Tests

NONCONTACT LAP SPLICES IN BRIDGE COLUMN-SHAFT CONNECTIONS

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16. ABSTRACT

Lap splices in reinforced concrete members typically consist of bars overlapped and placed in contact with each other. In the case of a large-diameter foundation shaft connecting to a smaller-diameter column, it is not possible to have the longitudinal bars be continuous, nor is it possible to provide a standard lap splice; instead an offset or noncontact lap splice of the longitudinal bars is required. With a noncontact lap splice, transfer of forces from one spliced bar to the other occurs through the surrounding concrete, and transverse reinforcement is typically required to provide satisfactory splice performance. Current code provisions on noncontact lap splices are very limited.

This study experimentally investigated the behavior of noncontact lap splices in bridge column-shaft connections. Tests were performed on near full-scale panel specimens, representing a cross-section of a column-shaft connection, and on 1/4 scale column-shaft specimens under both monotonic and cyclic loading. Variables investigated included lap splice length, lapped bar spacing, and spacing of transverse reinforcement. Specimen performance was evaluated in terms of load capacity, failure mechanism, and strength degradation.

Two-dimensional and three-dimensional truss models were developed to predict the behavior of noncontact lap splices. Experimental results supported the proposed behavioral models. Inclined cracks developed in the concrete which defined compression struts running between the offset lapped bars. Transverse reinforcement was required to provide equilibrium to the struts. Tests on specimens detailed based on the proposed models resulted in no strength degradation or slippage of the lapped reinforcing bars even when subjected to cyclic loading. Equations were proposed for the design of noncontact lap splices, including recommendations for required overall lap length and transverse reinforcement.

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EXECUTIVE SUMMARY

Lap splices in reinforced concrete members typically consist of bars overlapped and placed in contact with each other. In the case of a large-diameter foundation shaft connecting to a smaller-diameter column, it is not possible to have the longitudinal bars be continuous, nor is it possible to provide a standard lap splice; instead an offset or noncontact lap splice of the longitudinal bars is required. With a noncontact lap splice, transfer of forces from one spliced bar to the other occurs through the surrounding concrete, and transverse reinforcement is typically required to provide satisfactory splice performance. Current code provisions on noncontact lap splices are very limited.

This study experimentally investigated the behavior of noncontact lap splices in bridge column-shaft connections. Tests were performed on near full-scale panel specimens, representing a cross-section of a column-shaft connection, and on 1/4-scale column-shaft specimens under both monotonic and cyclic loading. Variables investigated included lap splice length, lapped bar spacing, and spacing of transverse reinforcement. Specimen performance was evaluated in terms of load capacity, failure mechanism, and strength degradation.

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strength degradation or slippage of the lapped reinforcing bars even when subjected to cyclic loading. Equations were proposed for the design of noncontact lap splices, including recommendations for required overall lap length and transverse reinforcement.

INTRODUCTION

INTRODUCTION AND BACKGROUND

The construction of reinforced concrete structures often requires the use of lap splices of the reinforcement. Lapped bars are usually placed in contact with each other and lightly wired together. The concrete surrounding the splice provides for the transfer of forces between the lapped bars through bond action between the concrete and the reinforcing bars.

In the case of a large-diameter foundation shaft connecting to a smaller-diameter column, a noncontact, or spaced, lap splice is required due to the abrupt change in diameter from the shaft to the column. The offset, or distance between the bars, in a noncontact lap splice introduces the potential for different failure conditions than are present with conventional lap splices. Current code provisions permit noncontact lap splices provided that the bars are not spaced further apart than the smaller of one-fifth the splice length or 15 cm (6 in.). However, the codes prohibit or discourage lap splices of any kind in regions where yielding of the reinforcement may occur. Both of these code requirements are typically violated with these column-shaft connections.

Adequate transverse reinforcement is critical to the successful performance of a spliced region, particularly for structures subjected to earthquakes. During the 1971 San Fernando earthquake, many structures built in accordance with the codes of the time did not respond as expected, and consequently failed. Of particular interest, several splice connections at the bases of bridge columns failed as a result of pullout of the longitudinal bars, resulting in catastrophic collapse of the bridges. These splice failures were attributed to inadequate reinforcement embedment and insufficient confinement of the splice region.

This research report presents the main results and conclusions from a study investigating the behavior of noncontact reinforcing splices typical of those that would be used for the connections between large-diameter shafts and smaller-diameter columns in bridges. Experimental tests were conducted on near full-scale panel specimens, representing a cross-section of a column-shaft connection, and on 1/4-scale column-shaft specimens under both monotonic and cyclic loading. The effects of several design parameters were investigated, including lap splice length, offset spacing, and amount of transverse reinforcement. Lap splice performance was evaluated in terms of load capacity, failure mechanism and strength degradation. Based on the test results, recommendations are made for the design of column-shaft connections incorporating noncontact lap splices.

RESEARCH OBJECTIVES

The objectives of this study were as follows:

1. to review available literature on the behavior and design of contact and noncontact lap splices;
2. to develop behavioral models that describe the force transfer mechanisms and that provide a rational basis for the design of noncontact lap splices;
3. to experimentally evaluate the performance of noncontact lap splices designed based on the proposed behavioral models; and
4. to develop procedures for the design of noncontact lap splices for application to bridge column-shaft connections.

CURRENT CODE PROVISIONS AND PREVIOUS RESEARCH

Current Development Length and Splice Provisions

Bonding between steel reinforcement and concrete occurs primarily due to a mechanical locking of the ridges on the bars with the surrounding concrete. This interlocking creates both radial and tangential stress components in the concrete due to the reinforcement ridges bearing on the concrete. The radial stresses cause dilation, and potentially subsequent splitting or bursting, of the concrete surrounding the reinforcing bars. These radial stresses can influence the behavior of both contact and noncontact lap splices.

The required splice lengths specified in current codes are based on the development length of reinforcing bars. Depending upon the severity of stress in the spliced regions, lap splices are specified as multiples of the development lengths. In ACI 318-89 (1989), these development lengths are based upon ultimate bond stresses and are a function of concrete compressive strength, f'_c, bar area, A_b, and yield strength of the bar, f_y. ACI Section 12.2.2 specifies the development length in English units for No. 11 bars and smaller as:

$$l_d = \frac{0.04\ A_b f_y}{\sqrt{f'_c}} \qquad \text{(Equation 1)}$$

Equations for the development lengths of No. 14 and No. 18 bars, as well as various modifiers for the development lengths, are also given in ACI Section 12.2.

ACI Section 12.15.1 requires the classification of lap splices in tension as either Class A or Class B. The minimum splice lengths for Class A and Class B splices are given as:

Class A $\quad\quad$ $1.0 l_d$

Class B $1.3l_d$

Section 12.15.2 states that lap splices of deformed bars shall be taken as Class B splices except when Class A requirements are met. Class A splices are allowed when: (a) the area of reinforcement provided is at least twice that required by analysis over the entire length of the splice, and (b) one-half or less of the total reinforcement is spliced within the required lap length. ACI Section 12.14.2.1 prohibits tension splices of bars larger than No. 11. The only provisions for noncontact lap splices are given in ACI Section 12.14.2.3 which stipulates that bars in a noncontact lap splice shall not be spaced apart more than one-fifth the required lap splice length nor 6 in. Implicit in the code provisions is that the design of noncontact lap splices meeting these spacing restrictions is the same as that for regular splices.

The American Association of State Highway and Transportation Officials (AASHTO, 1992) use the basic development length and splice provisions given in the ACI Building Code as their design requirements, with the exception that AASHTO also includes a minimum development length in English units for No. 11 bars and smaller of:

$$l_d = 0.0004\ d_b\, f_y \qquad \text{(Equation 2)}$$

where d_b is the bar diameter. AASHTO also provides a splice length requirement for Class C splices:

Class C $1.7l_d$

Class C splices are defined as high stressed bars with more than 50% of the bars spliced at a given location.

In turn, the Washington State Department of Transportation (WSDOT, 1993) bases their development length and splice design criteria on the AASHTO design requirements. The WSDOT provides design tables specifying development length and splice length values for various bar sizes based on the equations and requirements given by AASHTO.

PREVIOUS RESEARCH ON NONCONTACT SPLICES

Several of the early studies investigating noncontact lap splice performance concluded that there was little or no behavioral difference between contact and noncontact splices. However, this conclusion was based on tests using concrete with compressive strength values that were less than 35 MPa (5000 psi), monotonic loading, limited or no transverse reinforcement, and maximum offset spacing between the splice bars of 3 bar diameters (Sagan, 1988).

Research by Reynolds and Beeby (1982) indicated that there is a good correlation in bond stresses between contact and noncontact lap splices. However, a study by Cairns and Jones (1982) found that there are higher bond stresses with contact lap splices than noncontact lap splices. The reason for this discrepancy can be attributed to the parameters used in the determination of bond stresses, specifically, the length used to represent the transfer of forces. Average bond stress is defined as the tensile force developed in the bar divided by the surface area of the same bar through which the tensile load is transferred. Thus, the average bond stress is dependent on the assumed length of the surface area through which the tensile force is transferred.

A truss model, illustrated in Figure 1, was proposed by Robinson, et al (1974) to explain the behavior of noncontact lap splices. The lapped bars act as the tension chord

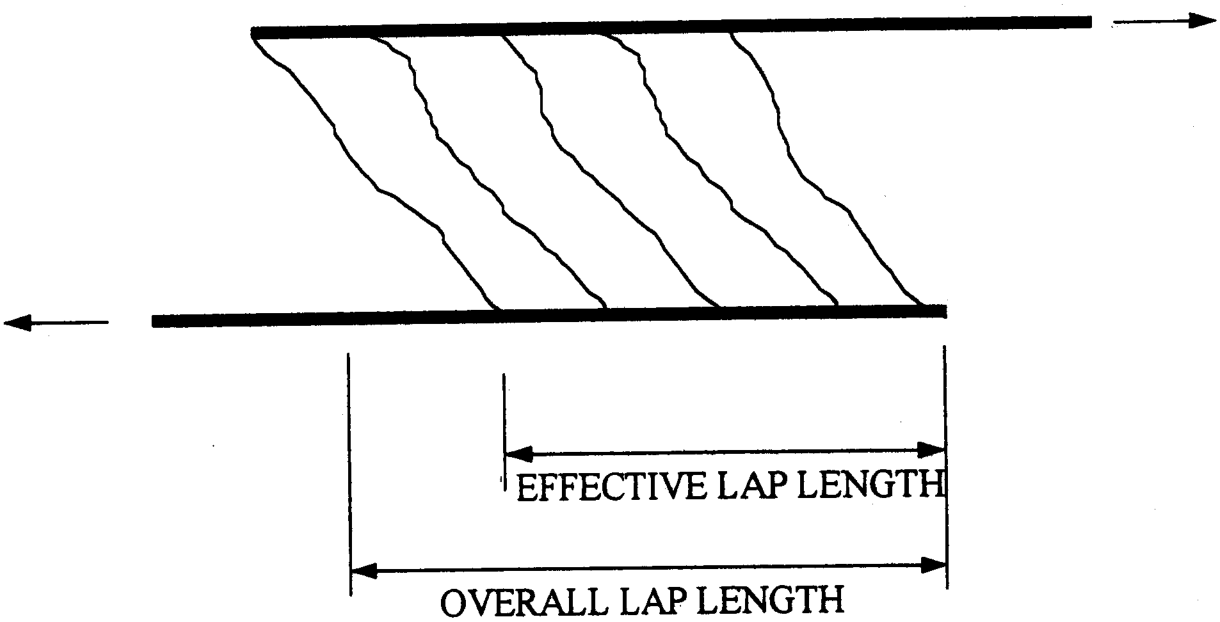


Figure 1 Effective Lap Length (Adapted from Sagan, et al, 1988).

members, the concrete between the lapped bars acts as inclined compression struts, and any transverse reinforcement acts as a tension tie. Because of the inclination of the struts, there is effectively a reduced transfer length with a noncontact lap splice. Thus, with a noncontact lap splice, the overall splice length is composed of the effective splice length plus an additional length that is a function of the bar offset and the strut angle. Diagonal cracking associated with strut formation has been observed to vary between 20° and 70°, but a majority of cracks occur at 45° (Sagan, et al, 1988).

The reason for the apparent discrepancy in reported bond stresses from one researcher to another can be explained by the fact that Reynolds and Beeby (1982) determined bond stress based on the effective lap length. Cairns and Jones (1982) used overall lap length in their calculations, and thus they obtained lower values for bond stress.

Sagan, et al (1988) investigated the performance of noncontact lap splices under monotonic and repeated inelastic loading. Reinforced concrete panel specimens incorporating noncontact lap splices were constructed and tested in tension. The study examined the effects of various parameters on the overall performance of noncontact lap splices, including splice bar spacing, concrete compressive strength, splice bar size, lap length, and transverse reinforcement.

The overall splice length used in Sagan's study was determined by adding the effective lap length and the longitudinal component of compressive strut length. The longitudinal component of a strut with an assumed angle of 50° (i.e., tan 50°) is approximately equal to 1.2. Thus, the overall splice length, l_s, was determined to be the effective lap length, l_{eff}, plus 1.2 times the offset spacing, s:

$$l_s = l_{eff} + 1.2s \qquad \text{(Equation 3)}$$

Transverse reinforcement in their specimens was based on recommendations developed by Sivakumar (1983) for contact lap splices. The spacing of transverse reinforcement, s_{tr}, was determined as:

$$s_{tr} = k \frac{A_{tr}\, l_s}{d_b^2} \qquad \text{(Equation 4)}$$

with $k = 0.375/d_{tr}$;

A_{tr} = transverse bar area;

d_{tr} = transverse bar diameter; and

d_b = spliced bar diameter.

Sagan, et al observed that the noncontact lap splices behavior can be represented as a plane truss using the overall lap length in the analysis. The transfer of forces between longitudinal bars occurred as a result of the development of compression struts in the concrete between the spliced bars. Transverse reinforcement was found to be critical for successful performance of the noncontact lap splices. Tests conducted on specimens constructed based on Equations 3 and 4 performed satisfactorily and achieved yielding of the spliced bars. However, Sagan, et al recommended that a strut and tie model be used for noncontact lap splices with bar offsets greater than 12 bar diameters, or 12 in., under monotonic loading, and bar offsets greater than 8 bar diameters, or 8 in., under repeated loading.

RESEARCH APPROACH AND PROCEDURES

PROPOSED BEHAVIORAL MODELS

Based on a review of existing research, conceptual models were developed to describe the observed physical behavior of noncontact lap splices. These models were based on the truss analogy to represent force transfer mechanisms within the splices. The proposed models provided insight into the parameters influencing splice behavior and served as the basis for designing the test specimens of this study.

Two-dimensional Model

For equilibrium to be satisfied in lap splices, load applied to one bar must in some way be transferred to the other. For contact lap splices, since the bars are wired together and touching, transfer of forces occurs directly from one bar to another through concrete bond stresses over the standard splice length. Noncontact lap splices must transfer load from one splice bar through the surrounding concrete via compression struts to the other splice bar.

Based on the truss analogy, Figure 2 illustrates a proposed representation of the transfer of forces between noncontact longitudinal bars. The compressive struts, C, between the bars represent the transfer of forces from the inner bar to the outer bar through the surrounding concrete at some angle θ. If the bars were in contact, the splice length, l_s, would be equal to some multiple of the development length, l_d; i.e., l_s would be based on the criteria for a normal lap splice. For noncontact lap splices spaced apart a distance s, the total noncontact splice length, l_{ns}, necessary for satisfactory force transfer would be equal to the standard required splice length, l_s, the length over which the bond stresses develop, plus the

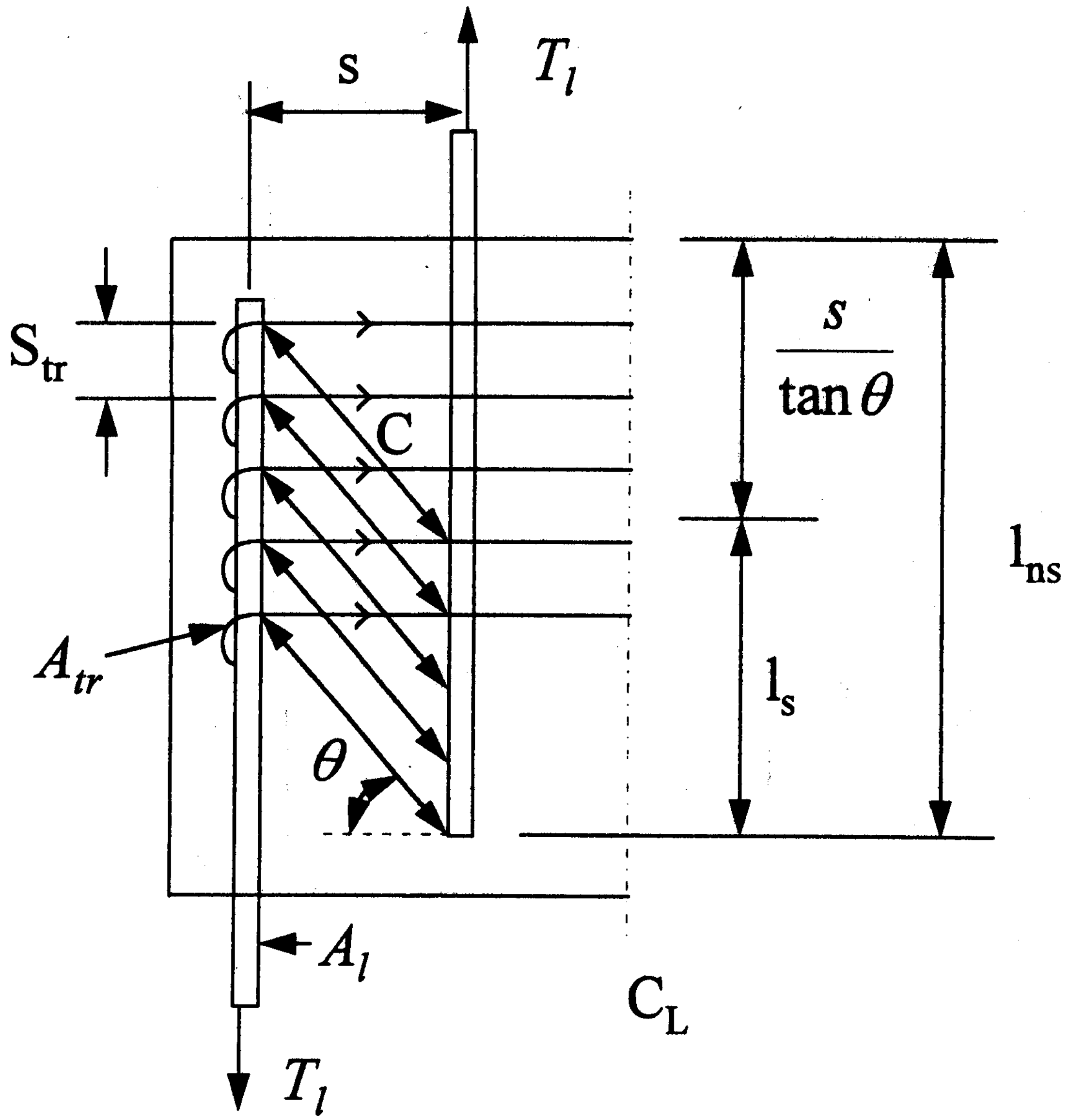


Figure 2 Proposed Two-Dimensional Behavioral Model for Noncontact Lap Splices.

vertical component of the strut length, $s/\tan\theta$. If the angle θ is approximated as 45°, then the total splice length for noncontact lap splices should be taken as:

$$l_{ns} = l_s + s \qquad \text{(Equation 5)}$$

From equilibrium requirements between forces in the longitudinal reinforcement and the transverse reinforcement, and with an angle θ equal to 45°, then the force in the longitudinal bars, T_l, is equal to force in the transverse bars, T_{tr}:

$$T_l = T_{tr} \qquad \text{(Equation 6)}$$

Equation 6 can be rewritten as:

$$A_l f_{yl} = \frac{A_{tr} f_{ytr} l_s}{s_{tr}} \qquad \text{(Equation 7)}$$

where A_l & A_{tr} = area of longitudinal and transverse reinforcement, respectively;

f_{yl} & f_{ytr} = yield strength of longitudinal and transverse reinforcement, respectively;

l_s = standard required splice length ; and

s_{tr} = spacing of the transverse reinforcement.

Rearranging Equation 7, the spacing of transverse reinforcement can be determined from the following equation:

$$s_{tr} = \frac{A_{tr} f_{ytr} l_s}{A_l f_{yl}} \qquad \text{(Equation 8)}$$

In order to develop the full capacity of the longitudinal bars, Equation 8 can be rewritten in terms of the ultimate strength of the bars, f_{ul}, as:

$$s_{tr} = \frac{A_{tr} f_{ytr} l_s}{A_l f_{ul}} \qquad \text{(Equation 9)}$$

Three-Dimensional Model

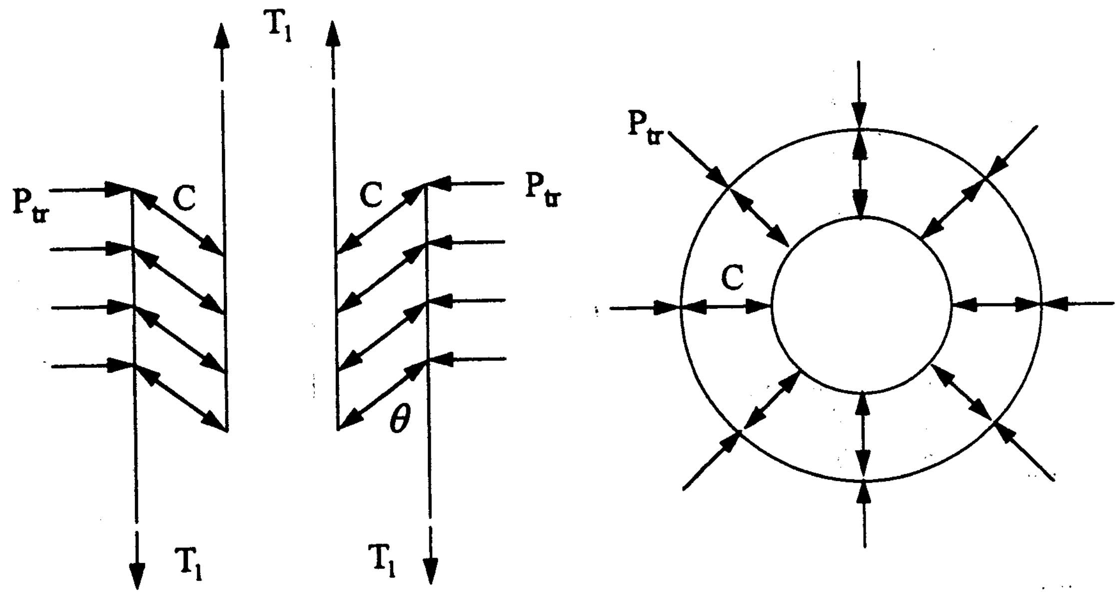
The two-dimensional model can be expanded to develop a model to represent three-dimensional behavior. For the three-dimensional model, as for the two-dimensional model, with an assumed strut angle of 45°, force in the longitudinal direction is equal to force in the transverse direction. However, whereas the two-dimensional model was developed on a per bar relationship, the three dimensional model is developed in terms of the total force in the longitudinal and transverse (radial) directions. Figure 3 shows the three-dimensional model.

With an assumed strut angle θ of 45°, the total splice length required for the noncontact lap splice is again given by Equation 5 as:

$$l_{ns} = l_s + s \qquad \text{(Equation 5)}$$

Also, the total force applied in the vertical direction must be equal to the total force in the transverse direction, represented by Equation 6. However, the transverse force is now provided by a continuous spiral cage instead of individual transverse bars, and can be defined as:

$$T_{tr} = \pi\, d_c\, p_{tr}\, l_s \qquad \text{(Equation 10)}$$



Column-Shaft
Transfer Mechanism

Column-Shaft
X-section

Figure 3 Proposed Three-Dimensional Behavioral Model for Noncontact Lap Splices.

with the pressure resulting from the transverse spiral, p_{tr}, being defined from basic reinforced concrete design equations (e.g., MacGregor, 1992) as:

$$p_{tr} = \frac{2\ A_{sp}\ f_{ytr}}{d_c\ s_{tr}} \qquad \text{(Equation 11)}$$

where d_c = diameter of the core (out-to-out of the spiral); and

A_{sp} = area of the spiral reinforcement.

Using the equation for tension in the longitudinal direction being equal to tension in the transverse direction, substituting in the equation for pressure, and rearranging, the spacing for transverse reinforcement is given as:

$$s_{tr} = \frac{2\ \pi\ A_{sp}\ f_{ytr}\ l_s}{A_l\ f_{yl}} \qquad \text{(Equation 12)}$$

Equation 12 can be rewritten in terms of the ultimate strength of the longitudinal bars as:

$$s_{tr} = \frac{2\ \pi\ A_{sp}\ f_{ytr}\ l_s}{A_l\ f_{ul}} \qquad \text{(Equation 13)}$$

The spacing determined from Equation 13 is the required pitch of spiral reinforcement for the shaft region incorporating the noncontact lap splice of a column-shaft connection in order to fully develop the column reinforcing bars.

EXPERIMENTAL TESTING PROGRAM

Test Specimens and Parameters

In this study, experimental tests were performed on near full-scale panel specimens, representing a cross-section of a column-shaft connection, and on 1/4-scale column-shaft specimens under both monotonic and cyclic loading. A summary of the specimen details and test parameters is given here. More detailed information is provided in Smith (1995).

Panel Specimens. The main panel tests were conducted on fifteen panels subjected to tensile loading. The selection of specimen details was based on obtaining information on splice design parameters not addressed in current code provisions and on the results of preliminary testing. Parameters investigated in the panel testing included variation of the overall splice length, l_{ns}, offset spacing distance, s, and spacing of transverse reinforcement, s_{tr}. Dimensions and splice arrangement for the panel specimens are shown in Figure 4.

The offset spacings used in this study were selected to be representative of reinforcing offsets that might exist in the connection regions of large-diameter shafts and smaller-diameter columns. Offset spacings of 15, 23, 31 and 38 cm (6, 9, 12 and 15 in.) were used. The standard splice length was based on WSDOT design charts (WSDOT Bridge Design Manual, 1993). The basic development length, l_{db} of a No. 8 bar in tension for f_y = 414 MPA (60 ksi) and f'_c = 28 MPa (4 ksi) is 76 cm (30 in.) for other than top bars. Splices with high stressed bars with more than 50% spliced are classified as Class C splices, and the basic development length is multiplied by 1.7. Thus, the normal (contact) required splice length, l_s, for the bars and materials used in the panel specimens is 130 cm (51 in.).

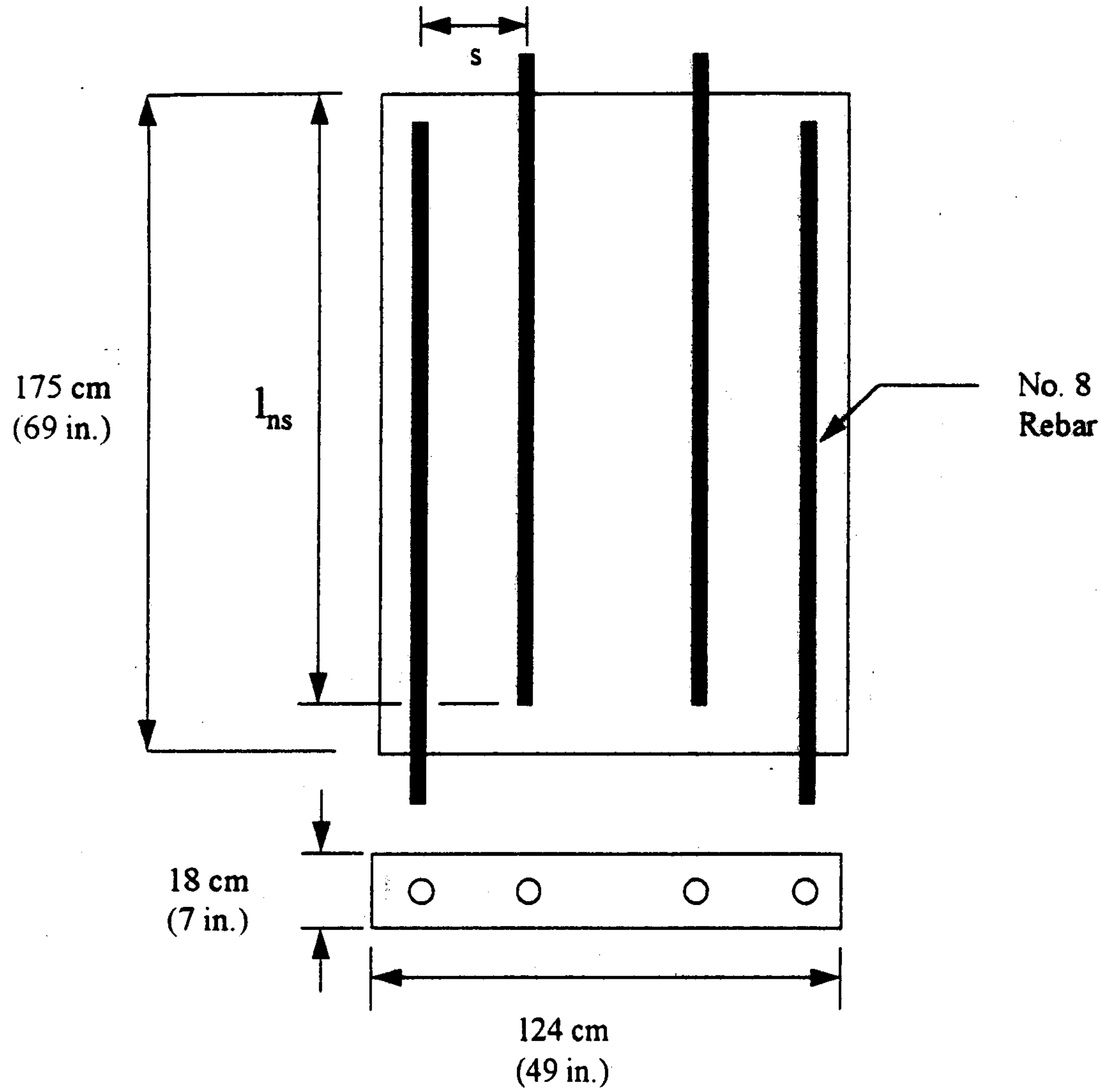


Figure 4 Panel Specimen Details.

A summary of the panel specimen parameters is given in Table 1. Of the fifteen panel specimens tested, the first three did not contain transverse reinforcement and were considered to be preliminary specimens. Eleven of the remaining twelve specimens had various parameters selected based on the previously discussed proposed two-dimensional behavioral model. Specimen No. 12 had only top bars for the transverse reinforcement.

The panel specimens were constructed using concrete with a target 28-day compressive strength of 21 MPa (4000 psi). The longitudinal splice bars were No. 8 bars that incorporated at one end a special upset thread for attachment to the loading system. The measured yield and ultimate strengths of the longitudinal bars were approximately 470 MPa (68 ksi) and 650 MPa (95 ksi), respectively. The transverse bars were No. 3 bars, with hooks at each end and with a measured yield strength of 480 MPa (70 ksi).

<u>Column-Shaft Specimens</u>. Two column-shaft specimens were constructed and tested to verify the results of the panels tests and the applicability of the proposed three-dimensional behavioral model. These specimens were designed as 1/4-scale representations of actual column-shaft connections. One specimen was designed to be tested in tension, similar to the panel specimens, and the other designed to be tested in flexure. Figures 5 and 6 provide details for the two column-shaft specimens.

For the specimens, the shaft diameter was 61 cm (24 in.) and the column diameter was 30 cm (12 in.). Longitudinal reinforcement consisted of No. 4 bars and the spiral reinforcement was 4-gage wire. The measured yield and ultimate strengths of the No. 4 bars were approximately 470 MPa (68 ksi) and 620 MPa (90 ksi), respectively, and the wire had

TABLE 1 Summary of Panel Specimen Parameters

Specimen No.	s (cm)	l_{ns} (l_s=130 cm)	Transverse Steel	Loading
Preliminary 1	15	l_s	None	mono tension
Preliminary 2	15	l_s+2s	None	mono tension
Preliminary 3	30	l_s+s	None	mono tension
1	15	l_s+s	per Equation 9	mono tension
2	23	l_s+s	per Equation 9	mono tension
3	30	l_s+s	per Equation 9	mono tension
4	38	l_s+s	per Equation 9	mono tension
5	15	l_s	per Equation 9	mono tension
6	38	l_s	per Equation 9	mono tension
7	15	l_s+s	per Equation 9	cyclic tension
8	38	l_s+s	per Equation 9	cyclic tension
9	15	l_s+s	per Equation 9	mono tension
10	15	l_s+s	per Equation 9	mono tension
11	30 & 46	l_s+s (s=38)	per Equation 9	mono tension
12	30	l_s+s	top mat only	mono tension

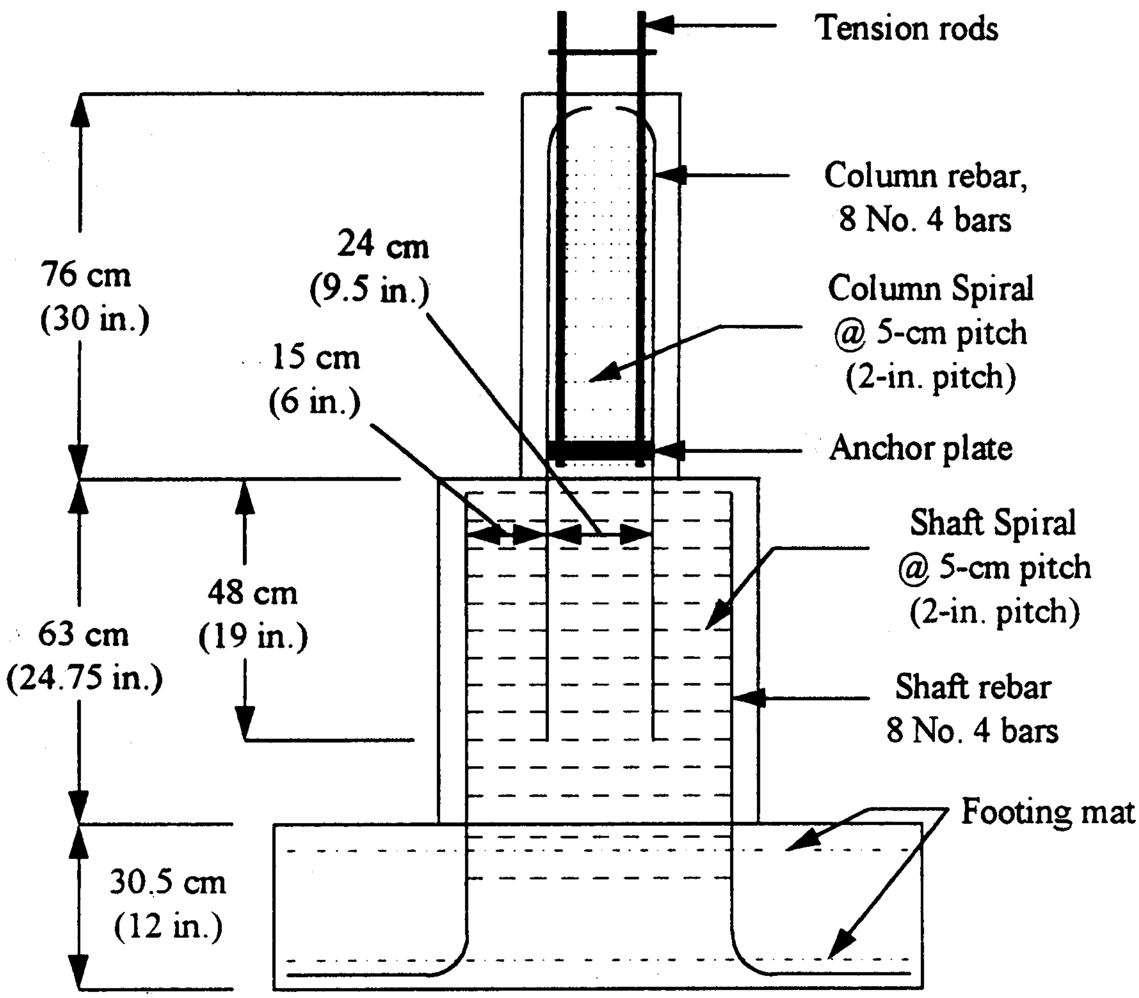


Figure 5 Column-Shaft Tension Specimen Details.

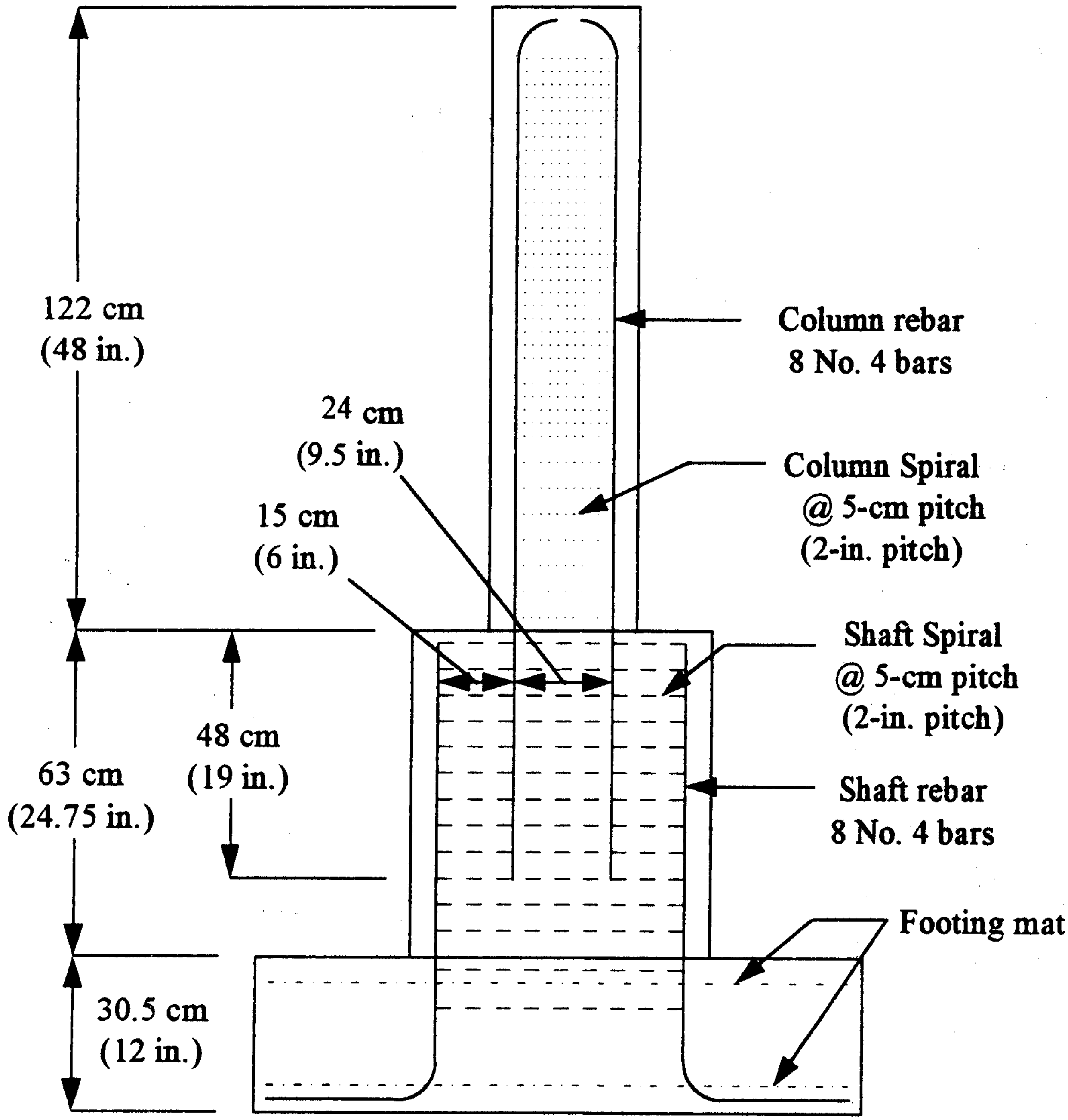


Figure 6 Column-Shaft Flexural Specimen Details.

a measured yield strength of approximately 610 MPa (89 ksi). The concrete used in the specimens had a target 28-day compressive strength of 21 MPa (4000 psi).

A splice bar offset of 15 cm (6 in.) was used in both specimens. The standard splice length was based on AASHTO design equations (AASHTO, 1992), with the exception that minimum development length provisions were not used. The basic development length, l_{db}, of a No. 4 bar in tension for f_y = 414 MPa (60 ksi) and f'_c = 28 MPa (4 ksi) is 19 cm (7.6 in.) for other than top bars. For a Class C splice, the basic development length is multiplied by 1.7. Thus, the normal (contact) required splice length, l_s, for the bars and materials used in the shaft-column specimens is 33 cm (13 in.). For both specimens, the provided overall lap length, l_{ns}, was determined from Equation 5 as 48 cm (19 in.). The spacing of the transverse reinforcement in the shaft was determined using Equation 13, resulting in a spiral pitch of 5 cm (2 in.).

Test Setup and Procedures

Panel Tests. The panel test setup was designed to apply tension loading to the exposed ends of the spliced bars of the panel specimens. A reaction frame was constructed to test the panel specimens in the vertical position. The longitudinal reinforcement was connected to high strength threaded rods via mechanical splice couplers, and the panels were then anchored to the loading system using washer plates and nuts at the ends of the threaded rods. The loading system consisted of two hydraulic jacks which were connected to a pump such that equal pressure was supplied to each jack, resulting in equal force to each rod connected to the specimen. A pressure gage on the pump and a load cell were used to determine the force being exerted on the bars. Figure 7 shows the test setup for the panel tests.

Loading of the panel specimens was controlled manually. Loading was temporarily stopped and observations were made when large crack propagation occurred and when the steel began to yield. Loading was resumed and the process continued until failure of a longitudinal bar by slippage or fracture occurred, or until extensive yielding of longitudinal bars developed.

Column-Shaft Tests. For the column-shaft tension test, two threaded rods were cast into the column so that hydraulic jacks could load the system in tension, similar to the panel tests. A steel plate was anchored to the bottom of the rods and cast into the concrete. Testing of this specimen was executed in the same manner as the panel tests. Figure 8 shows the basic testing setup for the column-shaft tension specimen.

The column-shaft flexure test was designed similarly to the tension test, with the exception of a longer column section. Metal plates were clamped to the top of the column and provided a means by which to attach a hydraulic actuator to the specimen for flexural loading. Figure 9 shows a photograph of the test setup. The column-shaft flexural specimen was loaded in flexure using a servo-controlled hydraulic actuator. The specimen was subjected to a series of increasing displacement cycles, beginning with applying 1.3 cm (0.5 in.) of displacement in one direction, returned to zero displacement, and then applying a displacement of -1.3 cm (-0.5 in.) in the opposite direction. Increasing displacements were applied, ranging from ±1.3 cm (±0.5 in.) to ±8.9 cm (±3.5 in.) in 1.3 cm (0.5 in.) increments.

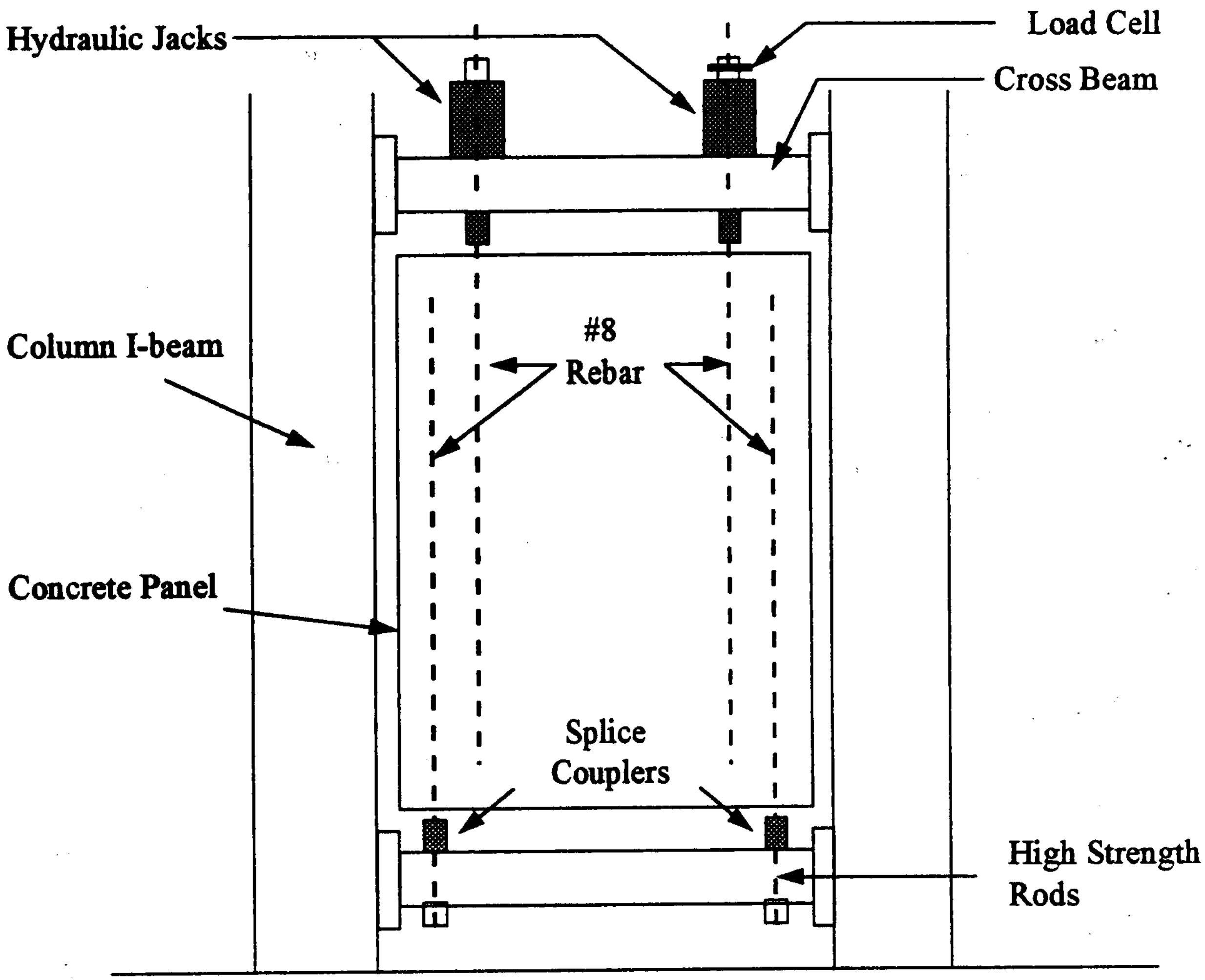


Figure 7 Panel Testing Setup.

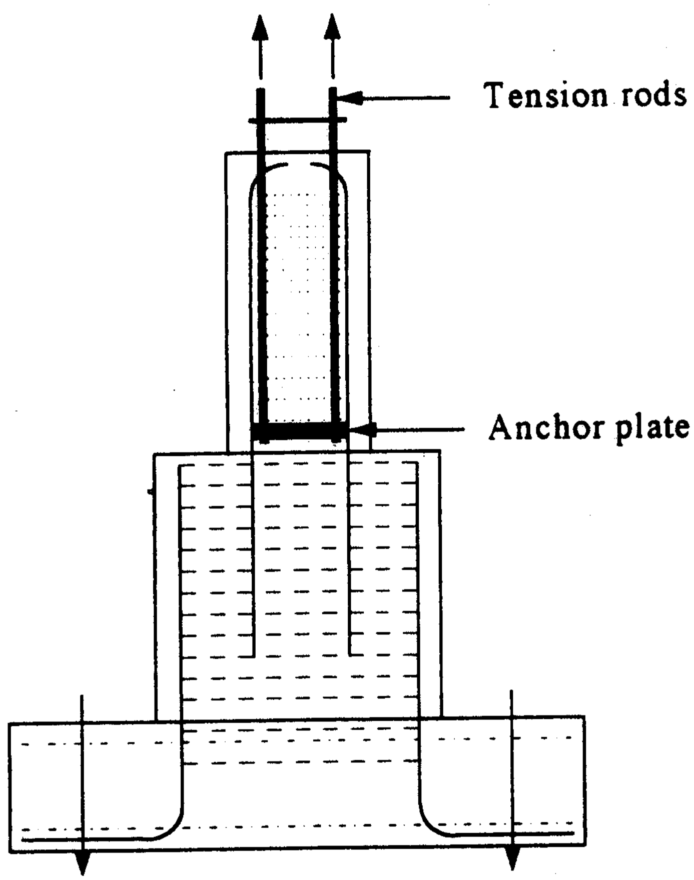


Figure 8 Column-Shaft Tension Testing Setup.

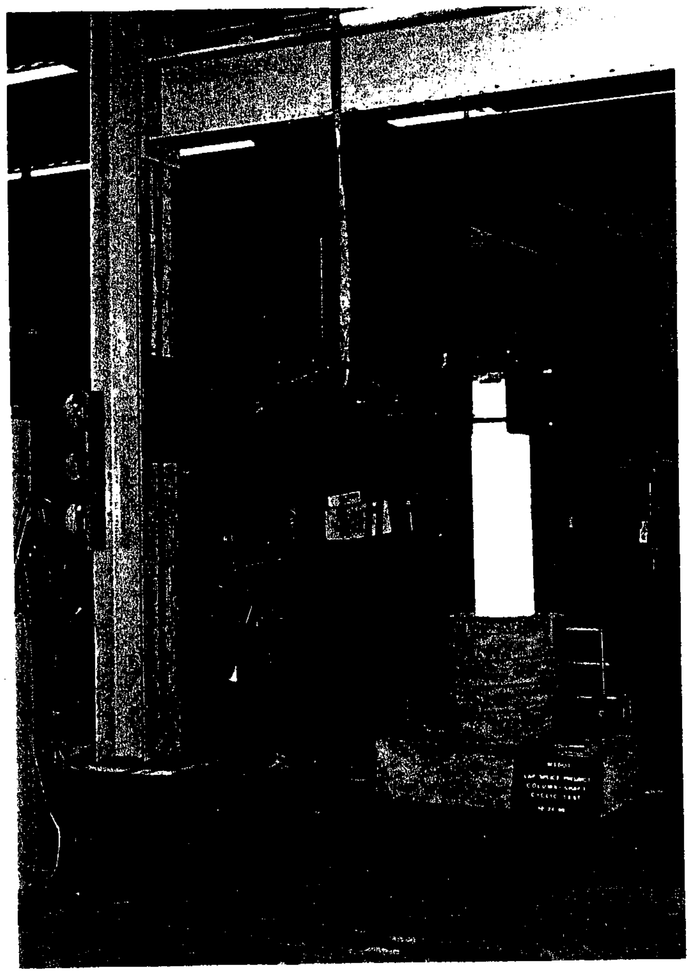


Figure 9 Column-Shaft Flexural Testing Setup.

RESEARCH FINDINGS AND DISCUSSION

PANEL TESTS

Preliminary Tests

The purpose of the tests on the preliminary specimens was to determine if it was possible to develop full capacity in a noncontact lap splice without the presence of transverse reinforcement. Splice offset distances of 15 and 30 cm (6 and 12 in.) were investigated, along with provided total splice lengths, l_{ns}, ranging from the standard splice length, l_s, to the standard splice length plus two times the offset distance, l_s+2s.

Failure in the preliminary specimens occurred when the applied loads reached values of approximately 40-60% of the bar yielding force, with a trend of increasing load capacity with increasing overall splice length. All of the preliminary specimens failed as a result of tension cracking of the concrete perpendicular to the spliced bars. This tension cracking was a result of in-plane flexural bending of the panel due to the offset of the spliced bars. Figure 10 shows a specimen that has failed in this manner. Even when a relatively long lap length was used (i.e., l_s+2s), the lack of transverse reinforcement resulted in the inability of the lap splices to develop the full bar force.

Monotonic Loading

Lap Splice Length Equal to Standard Length Plus Offset Distance. Specimen No.'s 1 through 4, 9 and 10 in Table 1 were constructed with provided noncontact splice lengths, l_{ns}, equal to the standard lap length, l_s, plus the offset distance, s, and were subjected to monotonic tensile loading. Transverse reinforcement was provided in accordance with Equation 9.

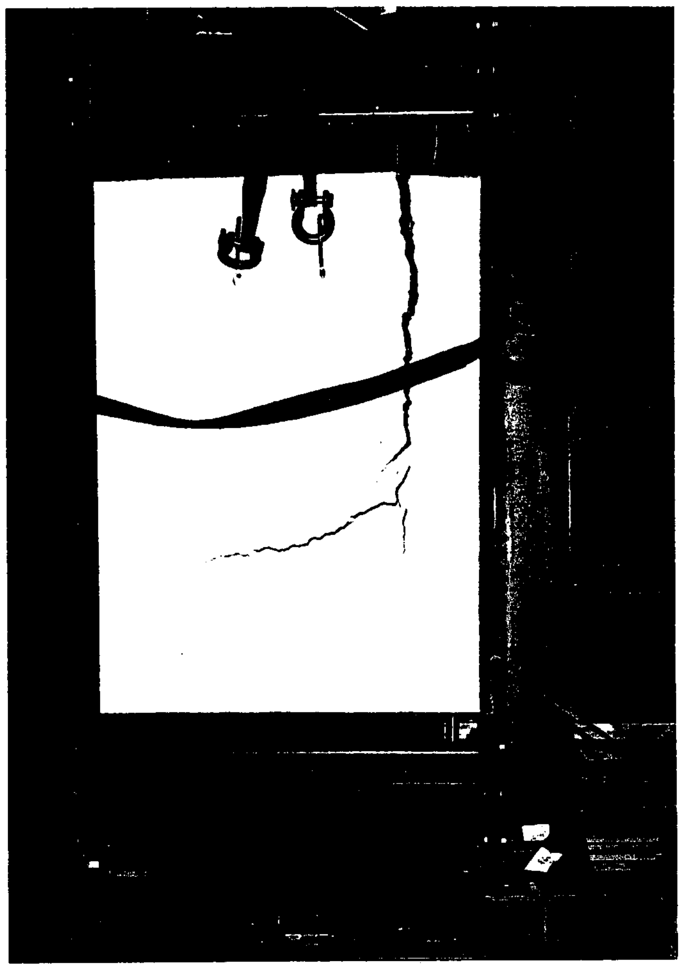


Figure 10 Tensile Cracking Failure in the Preliminary Panel Specimens.

Specimen No.'s 1 and 2 were successful in achieving fracture of the longitudinal reinforcement, instead of failure by tensile cracking as was observed in the preliminary tests. The fracture loads were 342 kN (76.8 k) for Specimen No. 1, and 336 kN (75.6 k) for Specimen No. 2. Specimen No.'s 3, 4, 9, and 10 were loaded so as to only produce extensive yielding and strain hardening of the longitudinal reinforcement in order to prevent damage to the testing setup. The average ultimate load applied to these specimens was 320 kN (72 k), corresponding to a stress level in the bars of 630 MPa (91 ksi).

All of these specimens exhibited the same general pattern of cracking. Horizontal cracks developed first across the middle of the specimen due to the tensile loading applied to the entire panel. Continued loading precipitated the presence of small vertical cracks developing along the inner longitudinal bars at the edge of the panel. Soon after the vertical cracks formed, diagonal cracks developed between the spliced bars, in some cases outlining compression struts running between the spliced bars. In general, the greater the offset spacing, the greater the amount and extent of cracking in the specimens. Figure 11 illustrates typical cracking in these specimens.

Strains gages were applied to the longitudinal bars of Specimen No.'s 9 and 10 along the length of the splice as well as to several of the transverse bars. Figures 12 and 13 show plots of strain on the longitudinal bars versus depth of embedment into the panel for various levels of applied load. The observed trends in the strain patterns can be explained in terms of the transfer of forces via the truss mechanism. For both the case of an offset distance of 15 cm (6 in.) and 38 cm (15 in.), the strains along the inner bars decreased with depth as the force was transferred through compressive struts in the concrete and over to the outer bars.

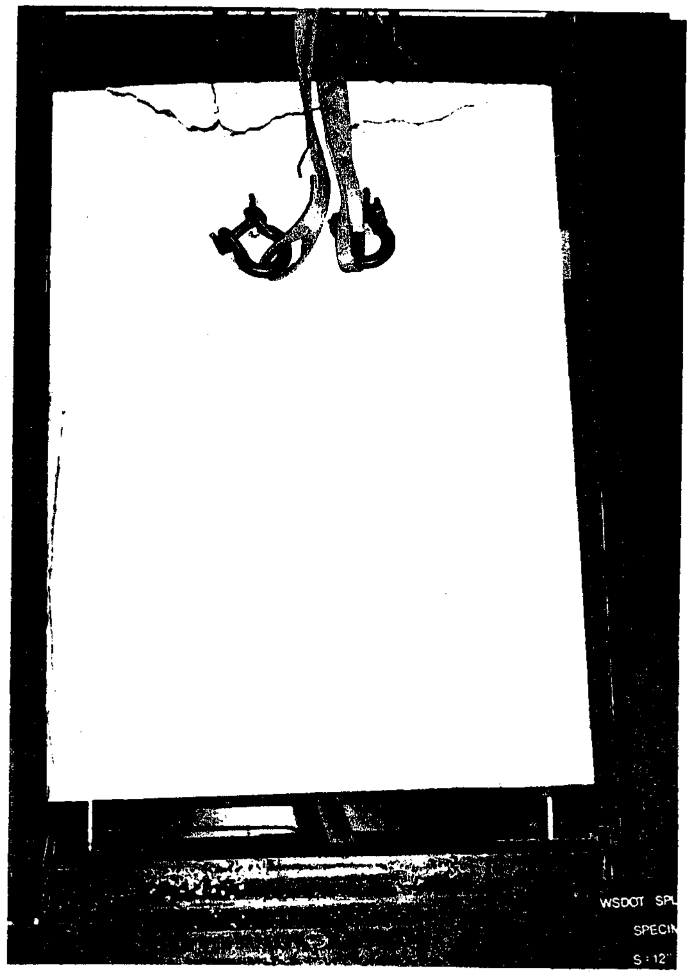
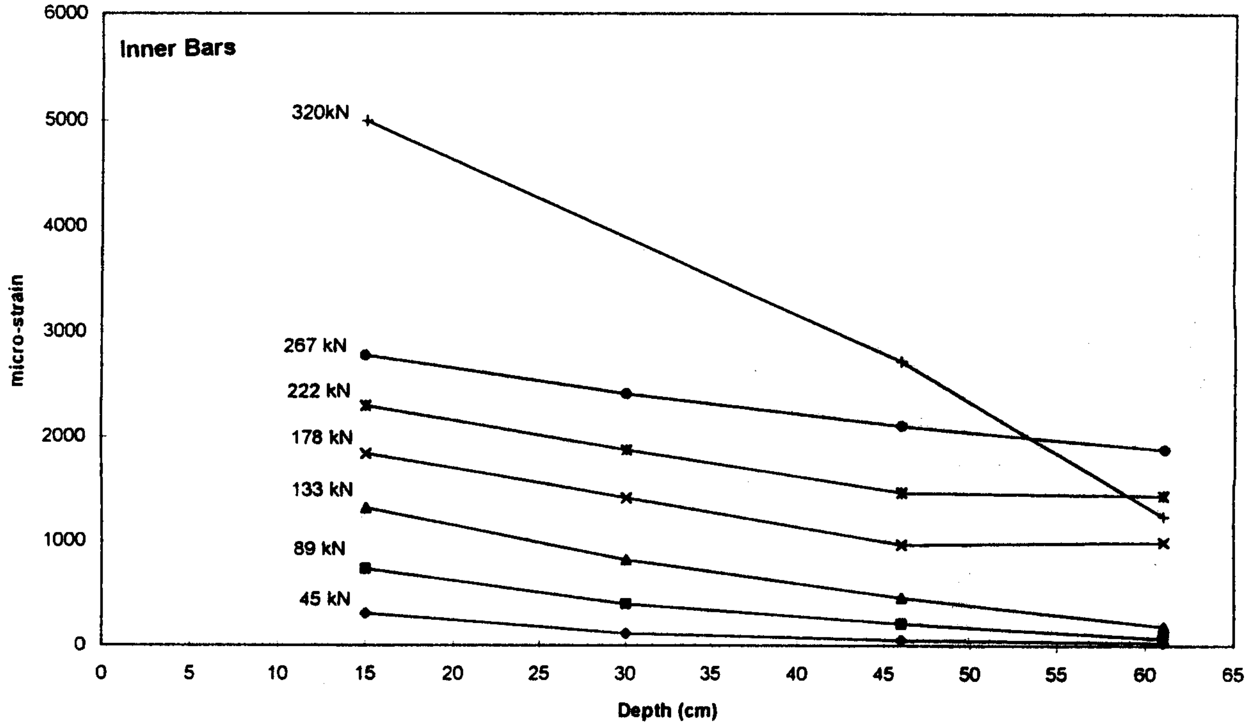


Figure 11 Cracking Pattern in Panel Specimens.



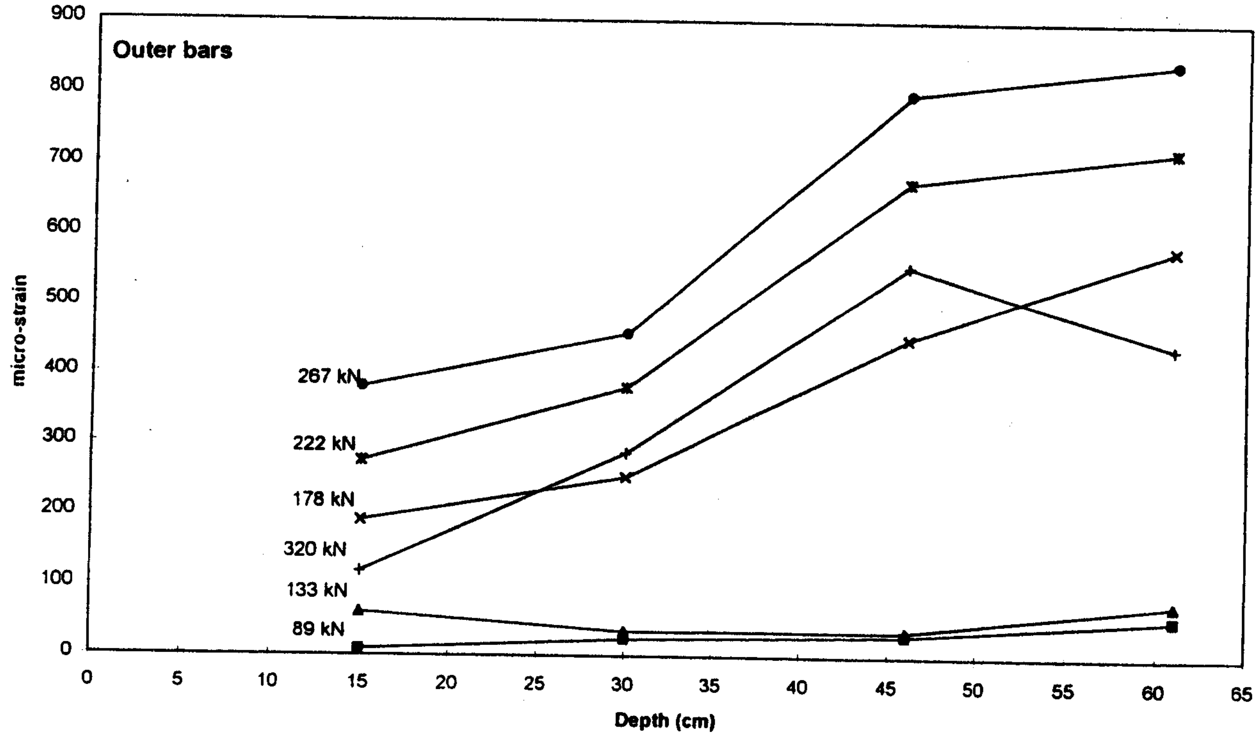


Figure 12 Longitudinal Bar Strains for an Offset Spacing of 15 cm (6 in.).

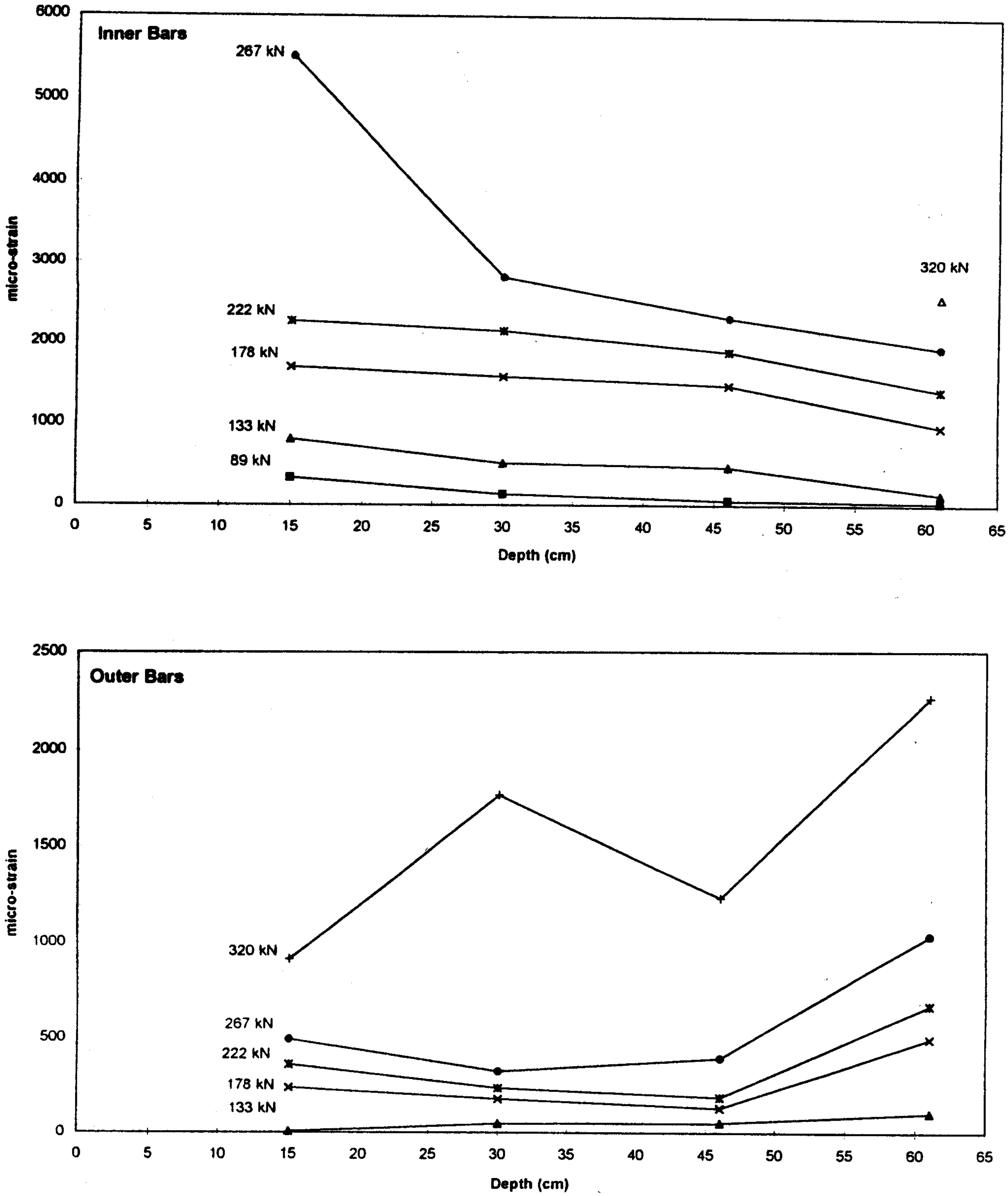


Figure 13 Longitudinal Bar Strains for an Offset Spacing of 38 cm (15 in.).

It can also been seen in the figures that, as more force was transferred from the inner bars to the outer bars over the splice length, the strains in the outer bars increased with depth.

Plots of strain versus depth of embedment for the transverse reinforcement are shown in Figure 14 and 15. The plots show that transverse strains are insignificant for small loads. For larger loads, with the larger offset spacing of 38 cm (15 in.) the transverse strains are much larger and develop along a greater length of the splice than is observed for the offset spacing of 15 cm (6 in.). Thus, with the smaller offset distance, the contributions from the transverse reinforcement are small. Conversely, with the larger offset distance, the transverse bars are more heavily loaded and more critical to the successful performance of the splice.

Splice Lengths Equal to the Development Length: Specimen No.'s 5 and 6 were constructed to observe the behavior of noncontact lap splices with shorter lap lengths ($l_{ns} = l_s$). As a result of the shorter lap lengths, closer spaced transverse reinforcement is required based on the proposed behavioral model (the required transverse reinforcement in the specimens was determined using Equation 9, but with the substitution of "l_s-s" for "l_s" in the equation to reflect the shorter effective lap length). Effectively, these tests investigated the ability of the proposed behavioral model to account for the interplay between embedment length and transverse reinforcement requirements.

Specimen No.s 5 and 6 failed in a similar manner to the $l_{ns} = l_s + s$ specimens, but with somewhat more cracking in the surrounding concrete. These specimens were taken to 320 kN (72 kips) and were able to achieve extensive yielding and strain hardening of the longitudinal bars.

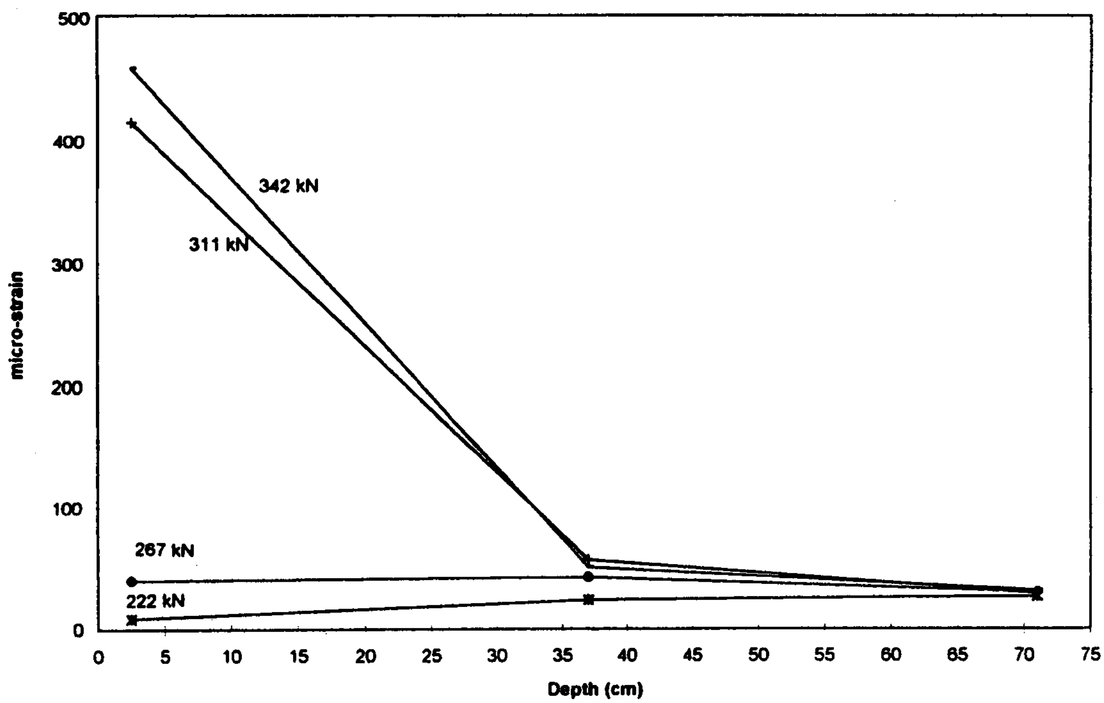


Figure 14 Transverse Bar Strains for an Offset Spacing of 15 cm (6 in.).

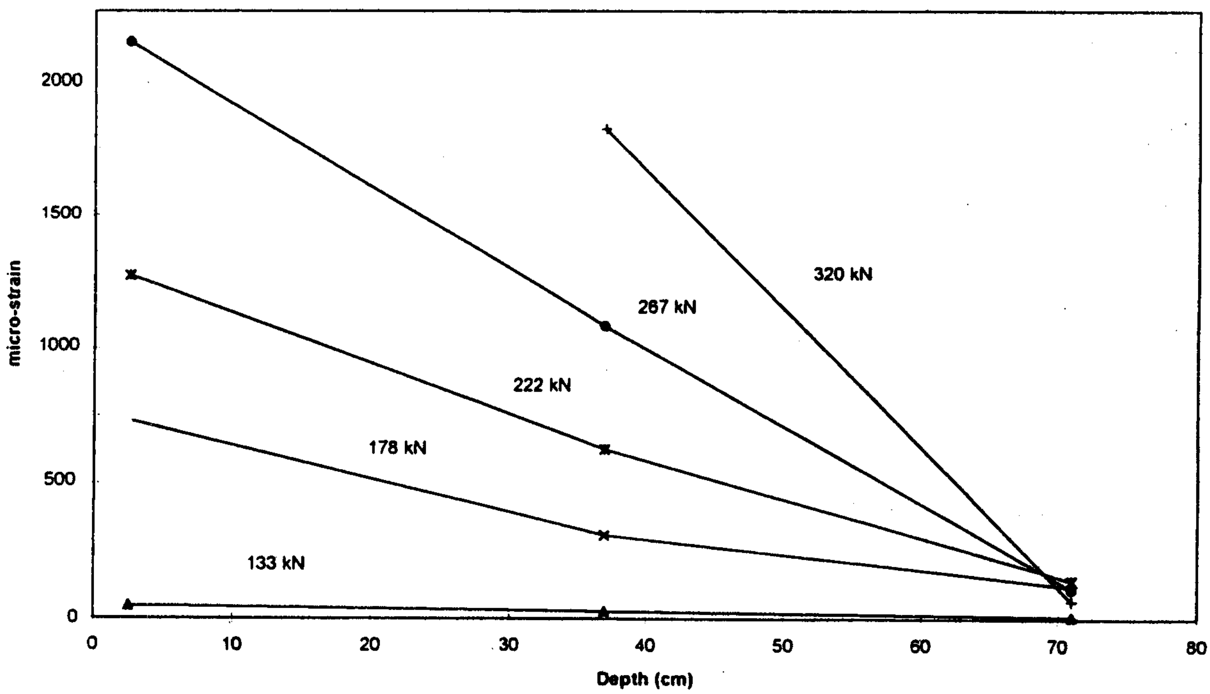


Figure 15 Transverse Bar Strains for an Offset Spacing of 38 cm (15 in.).

Unequal Offset Distances: Specimen No. 11 was designed and tested to observe the effects of unequal offset spacings, as may occur due to placement errors in construction. One offset was placed at 30 cm (12 in.) and the other at 46 cm (18 in.). The specimen was constructed with a provided noncontact splice length, l_{ns}, equal to the standard lap length, l_s, plus the average offset distance, s. Transverse reinforcement was determined using Equation 9 based on the average offset of 38 cm (15 in.).

Specimen No. 11 exhibited the same general behavior as the previous specimens, but due to the larger offset spacing, more cracking of the concrete was present along the bar with the larger offset. This specimen was loaded to 320 kN (72 kips) and was able to develop extensive yielding and strain hardening of the longitudinal bars.

Only Top Transverse Reinforcement: To determine if the lap splices would perform satisfactorily when only the top region of the splice was confined with transverse reinforcement, Specimen No. 12 was constructed with 6 transverse bars spaced evenly over the first 20 cm (8 in.) of the lap splice. This specimen failed by loss of bond of a longitudinal bar, as a result of vertical splitting of the concrete along the bar, at a load of approximately 270 kN (60 kips). This test indicated the need to distribute transverse steel over the lap length in order to control cracking and to achieve full development of the longitudinal bars.

Cyclic Loading

Specimen No.'s 7 and 8 were subjected to cyclic tensile loading to determine the durability of lap splices to repeated loading within the yielding range. Both specimens were constructed with provided noncontact splice lengths, l_{ns}, equal to the standard lap length, l_s, plus the offset distance, s, and transverse reinforcment based on Equation 9. Specimen No.s 7 and 8 incorporated offset distances of 15 and 38 cm (6 and 15 in.), respectively.

The specimens were loaded to approximately 245 kN (55 kips) to obtain yielding of the longitudinal bars and then released. This process was repeated for 35 cycles, observing the behavior of the specimens during testing. No progressive damage was apparent. For the last cycle, both specimens were loaded to 320 kN (72 kips) to achieve extensive yielding and strain hardening of the longitudinal bars. The pattern of cracking followed that of the previous tests. The results from these tests showed that the lap splices designed according to the proposed equations could withstand repeated cyclic tension loading with minimal damage and no degradation in load-carrying capacity.

COLUMN-SHAFT SPECIMENS

Tension Test

A tension test was performed on a column-shaft specimen to investigate the performance of offset splices in a three-dimensional application. The full tension loading on the splice represents a more severe loading condition than is likely to be experienced by actual bridge column-shaft connections (i.e., it is very unlikely that the entire column would ever be taken to its capacity in tension). The column-shaft specimen was constructed with a provided noncontact splice length, l_{ns}, equal to the standard lap length, l_s, plus the offset distance, s. Transverse reinforcement in the shaft, in the form of a continuous spiral, was determined using Equation 13.

During testing, cracks developed in this specimen at the base of the column and along the top of the shaft extending from the column to the shaft, as shown in Figure 16. The cracks developed due to the presence of bursting forces resulting from the column longitudinal steel force being transferred to the surrounding concrete and over to the shaft longitudinal steel. The spiral reinforcement in the shaft was critical in restricting the growth of the bursting cracks. Loading was taken to extensive yielding and some strain hardening of the longitudinal bars and then stopped due to a shear failure occurring in the specimen footing.

A peak load of 534 kN (120 kips) was achieved during testing, corresponding to a stress level in the longitudinal bars of approximately 520 MPa (75 ksi). While failure occurred prematurely in this specimen, due to the footing failure, no signs of distress were observed in the column-shaft region up through extensive yielding of the spliced bars.

Strain gages were placed on the longitudinal bars in both the column and the shaft to observe the variation of strain along the length of the noncontact lap splice. The profiles of strain versus depth were similar to those obtained for the panel tests. The strains on the inner bars decreased with depth and the strains on the outer bars increased with depth as load was transferred from the inner bars to the outer bars.

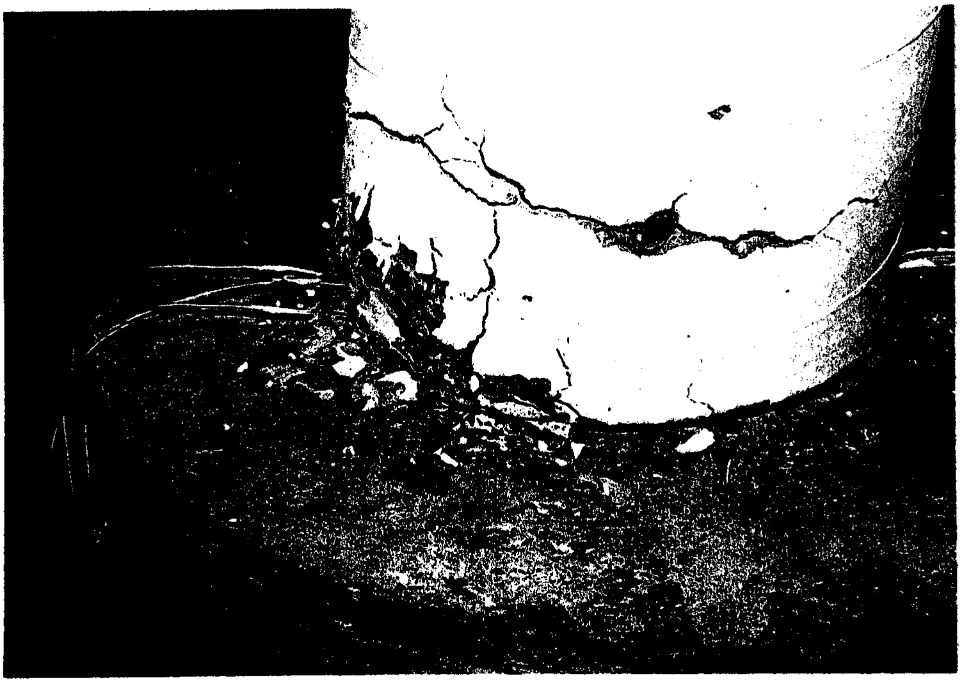


Figure 16 Cracking Patterns in Column-Shaft Specimen.

Flexural Test

The flexural test was the final test performed to investigate the behavior and suitability of the equations developed for noncontact lap splices. Splice details for this specimen were the same as those used for the tension column-shaft specimen.

The cracking pattern in the flexure specimen was similar to the cracking pattern exhibited in the column-shaft tension specimen. Cracks radiated outward from the column to the shaft and down the sides of the shaft. Again, the spiral reinforcement in the shaft controlled the propagation of the cracks and maintained the integrity of the splice.

Loads and displacements of the column were recorded during testing of the flexure specimen. The specimen performed very well under the imposed loading regime, with no distress being observed in the column-shaft connection. Figure 17 shows the load-deflection hysteresis curves obtained from the column-shaft flexure test. The curves show no loss of strength due to bar slip or concrete spalling, indicating that the splice length and spacing of transverse reinforcement enabled the specimen to exhibit constant load-carrying capacity through extensive deformations.

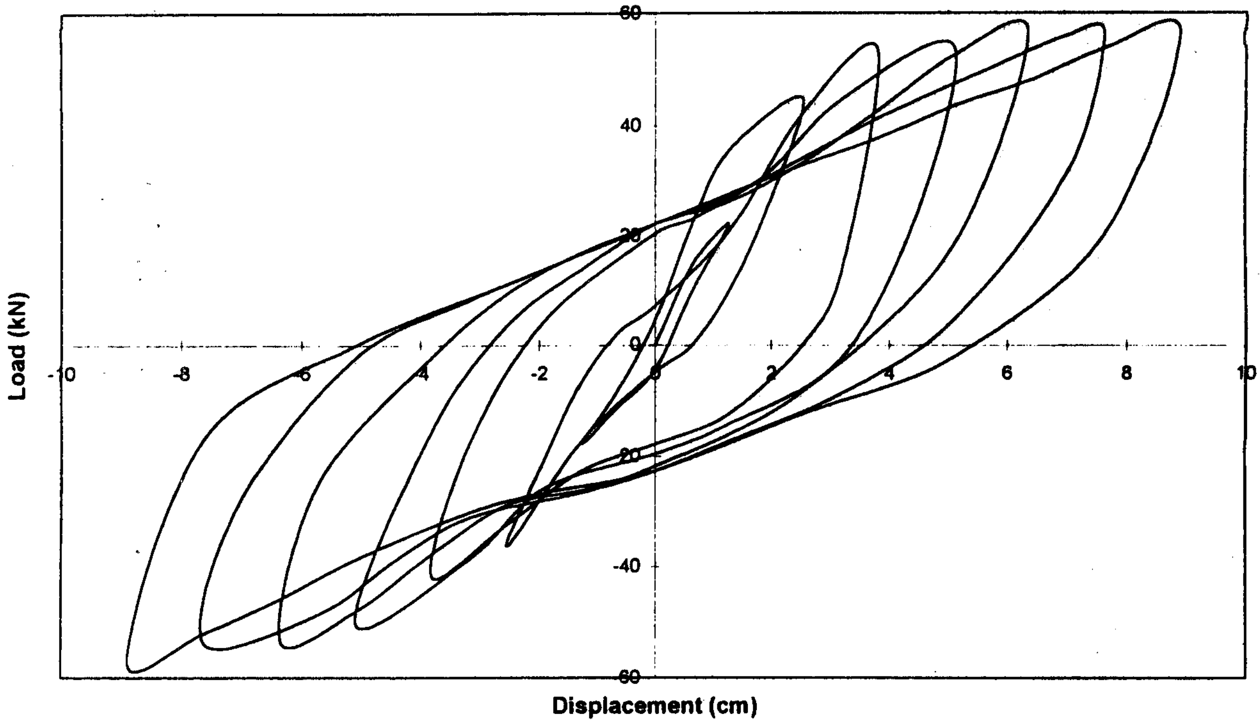


Figure 17 Load vs. Displacement Hysteresis Curves for the Column-Shaft Flexural Specimen.

CONCLUSIONS

On the basis of the results of this experimental study, the following conclusions are made:

1. Transverse reinforcement is necessary, even with long splice lengths, for the full development of noncontact lap splices. All specimens in this study without transverse reinforcement failed as a result of brittle tensile cracking of the concrete.

2. The observed behavior of noncontact lap splices can be represented with a truss analogy. The lapped bars act as the tension chord members, the concrete between the lapped bars acts as inclined compression struts, and transverse reinforcement, acting as a tension tie, is required to provide equilibrium to the struts. This truss analogy was used to develop proposed design equations for noncontact lap splices.

3. Noncontact lap splices with splice lengths equal to the standard splice length or the standard splice length plus the offset distance performed well when the proposed equations were used to design the transverse reinforcement.

4. Noncontact lap splices designed using the proposed equations were able to withstand repeated cyclic tension loading and cyclic flexural loading with no degradation in strength.

5. All specimens designed using the proposed equations for splice length and spacing of transverse steel performed well, developing either fracture or extensive yielding and strain hardening of the longitudinal bars.

RECOMMENDATIONS/APPLICATIONS/IMPLEMENTATION

The results of this study provide a basis for the design of noncontact lap splices in the connection regions between large-diameter foundation shafts and smaller-diameter bridge columns. Behavioral models, based on a truss analogy, were developed in this study to represent the force transfer mechanisms within noncontact splices and served as the basis for developing design recommendations. The design recommendations were verified through experimental tests of both two-dimensional panel specimens and three-dimensional column-shaft specimens incorporating noncontact lap splices.

When designing a noncontact lap splice in a shaft-column connection region, the total noncontact lap splice length to be provided should be based on the standard required splice length plus the offset distance:

$$l_{ns} = l_s + s \qquad \text{(Equation 5)}$$

This provision is intended to result in bond stresses in the noncontact lap splice that are similar to those in a properly designed contact lap splice. Note that tension lap splices of bars greater than No.11 are prohibited by many current codes, and no information is provided by this project to support the use of lap splices with No.14 and No.18 bars.

In the shaft region encompassing the noncontact lap splice, spiral reinforcement should be provided with a maximum pitch determined from :

$$s_{tr} = \frac{2\,\pi\,A_{sp}\,f_{ytr}\,l_s}{A_l\,f_{ul}} \qquad \text{(Equation 13)}$$

This requirement for the spiral reinforcement pitch may be met when satisfying other requirements for the transverse reinforcement in the shaft. Also, the end of the spiral reinforcement should be properly anchored at the top of the shaft so as to prevent the possibility of the spiral reinforcement unwinding if spalling should occur in the shaft. However, the intent of the recommendations for the design of the shaft-column connections is to force any yielding and spalling to occur in the column section, away from the noncontact lap splice.

The tests of this study investigated the performance of noncontact lap splices in which the lapped bars were of the same size. To be conservative in the design of a lap splice in the case where the lapped bars are of different sizes, it is recommended that the standard splice length for the larger bar be used in determining the overall noncontact lap length. Additionally, if the larger bars are in the shaft, then standard 90° hooks at the top of the shaft bars may improve the connection performance.

For two-dimensional noncontact lap splices, the provided lap length should again be based on the standard required splice length plus the offset distance:

$$l_{ns} = l_s + s \qquad \text{(Equation 5)}$$

Spacing of transverse reinforcement in a two-dimensional noncontact lap splice should be determined from:

$$s_{tr} = \frac{A_{tr} f_{ytr} l_s}{A_l f_{ul}} \qquad \text{(Equation 9)}$$

ACKNOWLEDGMENTS

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REFERENCES

American Association of State Highway and Transportation Officials (1992). *Standard Specifications for Highway Bridges*, 15th Ed.

American Concrete Institute (1989). *Building Code and Commentary*, ACI 318-89, Detroit, Mich.

Cairns, J., and Jones, K. (1982). "Bond Performance of Ribbed Reinforcing Bars in Lapped Joints," *Bond in Concrete*, Applied Science Publishers, London, pp. 342-352.

MacGregor, James G. (1992). *Reinforced Concrete Mechanics and Design*, 2nd Edition, Prentice Hall, Inc.

Reynolds, G. C., and Beeby, A. W. (1982). "Bond Strength of Deformed Bars," *Bond in Concrete*, Applied Science Publishers, London, pp. 434-445.

Robinson, J.R., Zsutty, T.C., Guirogodze, G., Lima, L.J. and Hoang Long Ung und Villatoux (1974). "La Couture des Jonctions par Adherence," Bulletin d'Information des CEB, Nr. 99, Marz.

Sagan, V. E., Gergely, P., and White, R. N. (1988). "Behavior and Design of Noncontact Lap Splices Subjected to Repeated Inelastic Tensile Loading," Report No. 858-5, Department of Structural Engineering Research, Cornell University, Ithaca, August.

Sivakumar, B., Gergely, P., and White, R. N. (1983). "Suggestions for the Design of R/C Lapped Splices for Seismic Loading," *Concrete International: Design & Construction*, V.5, No. 2, February, pp. 46-50.

Smith, Carol Louise (1995). "Noncontact Lap Splices in Bridge Column-Shaft Connections," M.S. Thesis, Washington State University, 80 pp.

Washington State Department of Transportation (1993). *Bridge Design Manual*.