Suggestions for the Design of R/C Lapped Splices for Seismic Loading

by B. Sivakumar, P. Gergely, and R. N. White

Based on experimental findings and analyses, suggestions are presented for the design of R/C lapped splices in beams and columns to sustain high-intensity cyclic flexural loads with up to 15 cycles into the inelastic range of the spliced bars.

The key aspect of the design is the provision of closely spaced, uniformly distributed stirrup-ties in the splice region. An equation is developed for the spacing of stirrups. Transverse steel requirements for sections with three or more lap splices per layer and for splices with offset bars are also given. An equation is proposed for the splice length as a function of the concrete strength.

Keywords: beams (supports); **bond (concrete to reinforcement);** buckling; columns (supports); ductility; **earthquake resistant structures; lap connections;** reinforced concrete; reinforcing steels, research; **splicing;** stirrups; **structural design.**

Introduction

Current design methods for lapped splices, including the recently suggested versions in Reference 1 by ACI 408-79, are applicable only for monotonic loads with the stress in the reinforcing steel below yield. However, structures in seismic regions may be subjected to severe cyclic forces and, in the event of a major earthquake, some portions of the structure can be subjected to a number of cycles into the inelastic range.

Investigations reported in the literature (summarized in Reference 2) indicate that the strength and ductility of lapped splices are adversely affected by high-intensity cyclic loads. Available documentation in this area is mainly behavior-oriented and little has been done regarding the development of design methods. As a result, most design codes do not permit lapped splices in regions where flexural yielding or severe stress reversals are possible. This is a severe limitation, especially for buildings where the column splices are usually located just above the floor level.

A study was initiated at Cornell University in 1978 to develop guidelines for the seismic design of lapped splices. Beam and "column type" splices under repeated and reversed, high-intensity static cyclic flexural loads have been investigated using 60 full-scale specimens and eight half-scale specimens. Typical full scale beams were 20 in. (508 mm) deep and 21 ft (6.4 m) long, whereas the columns were about 12 in. (300 mm) square and 11 ft (5.2 m) long. The half-scale beams were 10 in. (254 mm) deep and 6 ft (1.82 m) long. A summary of results of this investigation is reported in a companion publication.[2] Based on these findings, design guidelines are developed here.

For lapped splices to safely sustain inelastic deformations under reversing loads, strength as well as ductility are essential design considerations. Due to the progressive nature of bond deterioration and the resultant stiffness degradation under high-intensity cyclic loads, it is feasible to design splices to sustain only a limited number of cycles into the inelastic range. In this study a minimum of 15 to 20 reversing load cycles beyond yield and a maximum reinforcing steel strain in the splice of at least 2.5 times the yield strain were considered as indicative of satisfactory performance.

In most tests higher ductility was not achieved because of stroke limitation on the loading actuators. The maximum reinforcing steel strains were often larger than the design criterion of 2.5 times the yield strain but the failure of strain gages usually precluded the measurement of larger strains.

Development of equation for stirrup spacing

The key aspect of the seismic design of lapped splices is the provision of closely-spaced, uniformly distributed stirrup-ties in the splice region. To determine the required spacing for the stirrups, an equilibrium model of bursting and confining forces was proposed for a corner splice (Fig. 1b).

For the spliced bars to develop adequate anchorage, the bursting forces due to the radial component of the bond stresses have to be effectively resisted. The progressive and often extensive damage to the concrete cover as splice failure is approached suggests that the resistance afforded by the concrete cover is relatively small for high-intensity cyclic loading. Therefore, the cover was disregarded and the radial bond force was assumed to be resisted entirely by the transverse steel.

Assuming a uniform distribution of bond stresses at failure and bond forces acting at 45 deg to the bar axis, it can be shown[4] that for splices located in a constant moment region the unit radial bond force resultants F_1 and F_2 for each bar of the splice are:

$$F_1 = F_2 = \frac{f_y d_b^2}{4\ell_s} \quad (1)$$

Vertical equilibrium of forces in Fig. 1b gives:

$$(F_1 + F_2)s = A_{tr} f_{st} \quad (2)$$

where f_{st} is the tensile stress in the transverse reinforcement and A_{tr} is the area of transverse reinforcement normal to the plane of splitting for each splice. At the design level, yield penetrated about 20 percent along the splice at the end where the spliced bar continued. Using a $0.80\ell_s$ effective splice length and substituting Eq. (1) into Eq. (2), the following expression for stirrup-tie spacing is obtained.

$$s = 1.6 \frac{A_{tr}\ell_s}{d_b^2} \frac{f_{st}}{f_y} \quad (3)$$

Examination of strain data showed that #4 (12.7 mm) size stirrups exhibited lower strains at failure than #3 (9.5 mm) size stirrups, even when equal amounts of transverse steel were provided in the two cases. The more closely spaced #3 (9.5 mm) stirrups were more efficient in resisting bursting forces since the zone of influence of a stirrup is small in comparison to stirrup spacing. Hence, the total bursting force resisted by the #3 (9.5 mm) stirrups was higher than for the larger stirrups, thus making the strain level higher in the smaller stirrups. A design stress for the stirrups was chosen to closely model the average stresses observed near failure for #3 (9.5 mm) and #4 (12.7 mm) size stirrups.[3] The function chosen was:

$$f_{st} = \frac{40{,}000}{\pi\sqrt{A_{tr}}} \text{ (psi)} \quad \left(\frac{275}{\pi\sqrt{A_{tr}}} \text{ (MPa)}\right) \quad (4)$$

This function also predicts a reasonable design stress level for #5 (16 mm) stirrups. Using this relationship, Eq. (3) can be simplified to the following expression for reinforcement with f_y = 60 ksi (Grade 60 reinforcement):

$$s = k \frac{A_{tr}\ell_s}{d_b^2} \quad (5)$$

$$\text{where } k = \frac{3/8}{\text{stirrup diameter}} \quad \left(\frac{9.5 \text{ mm}}{\text{stirrup diameter}}\right)$$

When #3 size (9.5 mm) stirrups are used, Eq. (5) becomes:

$$s = \frac{A_{tr}\ell_s}{d_b^2} \quad (6)$$

The stirrup spacing given by Eq. (5) is for splices in corner bars located in a constant moment region. For splices in single curvature under a moment gra-

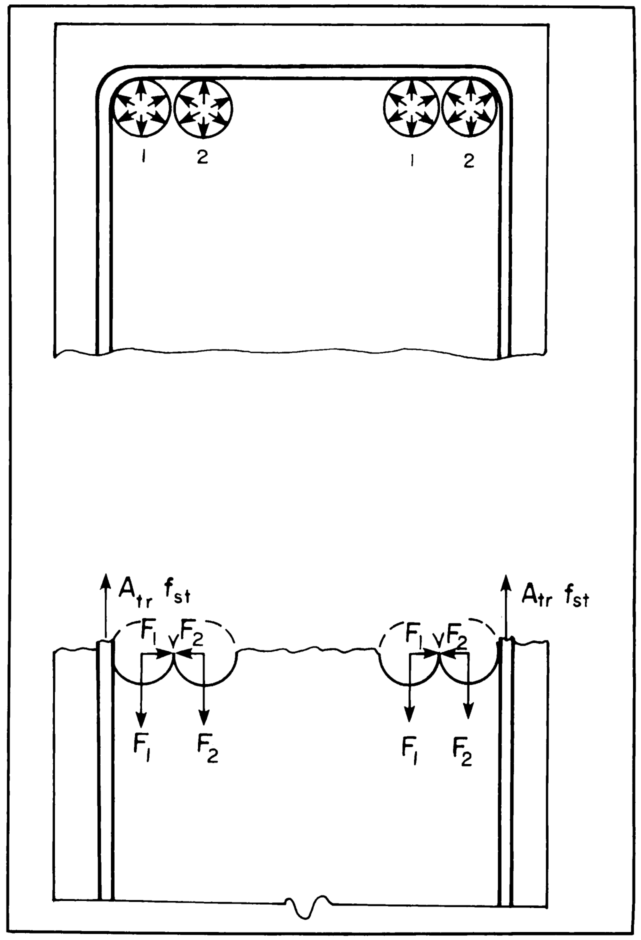


Fig. 1 — Equilibrium model of corner splices.

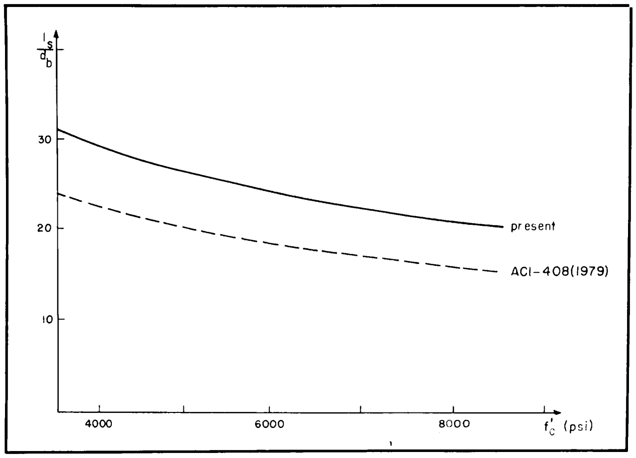


Fig. 2 — Comparison of suggested splice length requirements as a function of concrete strength (1000 psi = 6.89 MPa)

dient (constant shear) the stirrup spacing may be increased by multiplying by the factor in Eq. (7), as derived in Reference 4.

$$1.0 \leqslant \frac{1}{(1 - \ell_s/2z)} \leqslant 2.0 \qquad (7)$$

where z is the distance to the point of contraflexure from the high moment end of the splice. However, this factor can be conservatively omitted in cases where z is difficult to establish.

Design implications

(a) Splice length: In using Eq. (5) the designer is free to adopt a suitable splice length, provided the minimum splice length requirements are met. Test results showed that the $30d_b$ minimum splice length for columns required by Appendix A of ACI 318-77[5] was adequate for splices with a clear cover of at least $1.5d_b$ and for concrete strengths of at least 3500 psi (24 MPa). Test results and an analysis[4] show that shorter splice lengths can be used for higher strength concretes. Shorter splice lengths and correspondingly smaller stirrup spacings are normally preferred for higher strength concretes, which are stiffer and more brittle than weaker concretes and hence exhibit poorer bond stress redistribution properties. For bottom cast splices, the splice length requirements for Grade 60 reinforcement with a clear cover of at least $1.5d_b$ can be expressed as:

$$\ell_s \geqslant \frac{1860}{\sqrt{f'_c}} d_b \geqslant 20d_b \qquad \left(\frac{22{,}320}{\sqrt{f'_c}} d_b \text{ mm}\right) \qquad (8)$$

The mimimum splice lengths required by the above expression are about 30 percent longer than that suggested by ACI 408[1] for monotonic loads, with maximum amount of effective transverse steel (Fig. 2).

(b) Stirrup spacing — splices only in corners: The stirrups should be uniformly distributed along the splice length at a spacing not exceeding that given by Eq. (5), or its increased value in the presence of shear given by the amplification factor in Eq. (7). It is instructive to observe, from Eq. (5), that for given rebar and stirrup sizes the total number of stirrups required for the splice region (equal to ℓ_s/s) is independent of the splice length. This, in effect, means that a shorter splice length requires proportionally smaller stirrup spacing. The factor k in Eq. (5) reflects the better efficiency achieved by using smaller closely spaced stirrups as compared to larger, more widely spaced stirrups.

Tests show that the zone of influence of each stirrup along the splice is quite small. It is recommended that the combination of splice length and stirrup size be chosen such that the required stirrup spacing is not more than 6 in. (150 mm). This spacing should also be continued to a distance d outside the high moment end to prevent premature failure resulting from shear or buckling of rebars near the splice end.[2]

(c) Stirrup spacing — three or more splices per layer: For splices with a clear spacing of not less than $4d_b$ in sections with three or more splices per layer, the spacing of stirrups-ties for the corner splices are as stated in (b), and apply without modification. Tests show that as a result of the wide spacing between splices the interior splices can be freed of the transverse steel requirements of (b). However, in order to prevent premature buckling of the interior splice bars in compression, it is required that all interior splices be laterally restrained by supplementary ties, distributed along the splice length and also continued to a distance d outside the high moment end. A tie spacing not exceeding the larger of 6 in. (150 mm) or $6d_b$ is suggested.

If the clear spacing between splices is less than $4d_b$, then all splices including interior splices, should be confined by corners of stirrup-ties satisfying the requirements of (b), as in the case of sections with cor-

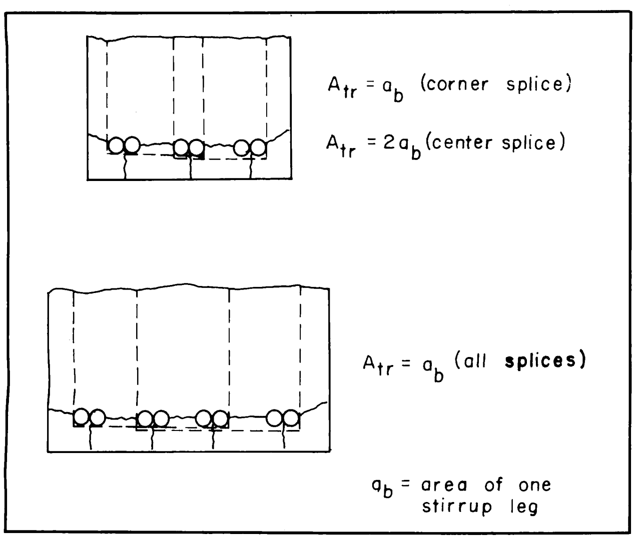


Fig. 3 — Definition of transverse reinforcement A_{tr} for splices with clear spacing less than $4d_b$.

ner splices only. A_{tr} in Eq. (5) for three or more splices in a layer is the area of transverse steel for each splice normal to the plane of splitting that could develop through the layer of splices (Fig. 3).

(d) Additional stirrup requirements for offset bars: If the main bars are offset at the end of the splice, as is often done in columns, additional transverse reinforcement of at least the amount required by ACI 318-77[5] should be placed at the splice end near the bend location. This results in double stirrup-ties being provided near the bend location (including the one required for splice confinement), or the double stirrup-tie could be replaced by a single stirrup-tie of a larger size bar.

(e) Other factors and limitations: The validity of Eq. (5) and (8) has been demonstrated for bar sizes up to and including #10 (32 mm) for normal strength concretes up to 4000 psi (27.6 MPa). The tests with concrete strengths up to 9000 psi (62 MPa) had small (#6, 19 mm) bars and the combination of high strength concretes with larger bars has not been studied. The case where the cover over the splices is lost; for example, at flexural hinges, has not been studied. The splice length should be calculated from the edge of hinge regions.

The stirrup spacing given by Eq. (5) is applicable both for spliced bars lapped side-by-side and for those lapped one-above-the-other. (Expressed with respect to the axes of bending.)[2]

Tests indicate that for moderate levels of shear up to 3 $\sqrt{f_c'}$ (psi) (0.25 $\sqrt{f_c'}$ MPa), the stirrups provided for bond are also effective in shear. The interaction of bond and high levels of shear was not evaluated and remains an area not well understood at this time. Also, the effect of direct axial compression on splice behavior was not studied in this investigation. The maximum compressive stresses recorded in the splice bars under flexural loads did not exceed $0.5f_y$. In this range, the splices designed for tension performed at least as well in compression. Further research is needed to study the effect of combined high axial compression and flexural load reversals on splice behavior.

Summary

Lapped splices can be designed to safely sustain high-intensity cyclic loads with at least 15 to 20 cycles into the inelastic range. The following suggestions are made for the design:

(1) The splice length required for Grade 60 reinforcement with a clear cover of at least $1.5d_b$ shall be taken as:

$$\ell_s \geqslant \frac{1860}{\sqrt{f_c'}} d_b \geqslant 20 d_b \qquad \left(\frac{22{,}320}{\sqrt{f_c'}} d_b \text{ SI}\right)$$

For f_c' of 3800 psi (26 MPa) or greater, ℓ_s may conservatively be taken as $30d_b$. The splice length should be increased in accordance with code recommendations for top cast horizontal reinforcement.

(2) For splices in corner bars, uniformly distributed closed-hoop stirrups shall be provided over the splice length at a spacing not exceed:

$$s \leqslant \frac{A_{tr}\ell_s}{d_b^2} \leqslant 6 \text{ in.} \qquad (150 \text{ mm})$$

The stirrup spacing should be multiplied by the following applicable factor or factors for:

stirrup sizes other than #3 (9.5 mm)

$$\frac{3/8}{\text{stirrup diameter}} \quad (9.5 \text{ mm})$$

splices subjected to a moment gradient

$$1.0 \leqslant \frac{1}{\left(1 - \frac{\ell_s}{2z}\right)} \leqslant 2.0$$

It is conservative to omit the second factor.

(3) Transverse steel requirements are given for sections with three or more splices per layer; these depend on the clear spacing of the splices and on stability considerations of the splice bars in compression.

(4) The stirrup spacing provided for the splice region should be continued to a distance d outside the high moment end of the splice.

(5) If the main bars are offset at the end of the splice by bending, additional transverse steel of at least the amount required by ACI 318-77[5] should be provided at the splice end near the bend location.

(6) For moderate levels of shear up to 3 $\sqrt{f_c'}$ (psi) (0.25 $\sqrt{f_c'}$ MPa) the stirrups provided for bond are also effective in shear.

(7) Because of the high demands placed on first-story columns, it is recommended that splices should not be used close to ground level where hinges cannot be forced into beams framing into the columns.

Acknowledgments

This research has been supported by the Problem-Focused Research Program and the Earthquake-Hazard Mitigation Program of the National Science Foundation. The contributions of former graduate research assistants F. Fagundo, A. Tocci, and K. Lukose are gratefully acknowledged.

References

1. ACI Committee 408, "Suggested Development, Splice, and Standard Hook Provisions for Deformed Bars in Tension," (ACI 408.1R-79), American Concrete Institute, Detroit, 1979, 3 pp.

2. Lukose, K.; Gergely, P.; and White, R. N., "Behavior of Reinforced Concrete Lapped Splices Under Inelastic Cyclic Loading," ACI JOURNAL, *Proceedings* V. 79, No. 5, Sept.-Oct. 1982, pp. 355-365.

3. Tocci, A. D.; Gergely, P.; and White, R. N., "The Behavior and Strength of Lapped Splices in R/C Beams Subjected to Cyclic Loading," *Report* No. 81-1, Department of Structural Engineering, Cornell University, Ithaca, 1981, 256 pp.

4. Sivakumar, B.; White, R. N.; and Gergely, P., "Behavior and Design of R/C 'Column-Type' Lapped Splices Under Inelastic Cyclic Loading," *Report* No. 82-11, Department of Structural Engineering, Cornell University, Ithaca, 1982, 201 pp.

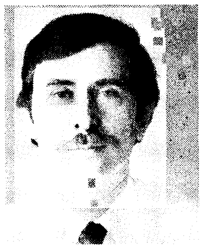
5. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-77)," American Concrete Institute, Detroit, 1977, 102 pp.

Notation

A_{tr} = area of transverse reinforcement normal to the plane of splitting for each splice (in.^{2})
d = effective depth of flexural member (in.)
d_b = diameter of spliced bar (in.)
f_c' = concrete compressive strength (psi)
f_{st} = stress in stirrup leg (psi)
f_y = yield strength of main bar (psi)
k = factor defined as $\dfrac{3/8 \text{ in.}}{\text{stirrup diameter}}$
ℓ_s = splice length (in.)
s = stirrup spacing over splice (in.)
z = distance to the point of contraflexure from the high-moment end of the splice (in.)

Received and reviewed under Institute publication policies.

B. Sivakumar, an ACI member, is presently a structural engineer with N. H. Bettigoli Engineers, Paramus, N.J. He formerly was a graduate student at Cornell University and is a graduate of the Indian Institute of Technology in Madras, India.



***Peter Gergely,** a Fellow of the Institute, is professor of structural engineering at Cornell University where he has been on a faculty since 1963. A member of several technical committees, he is currently chairman of Committee 445, Shear and Torsion, and is immediate past chairman of Committee 408, Bond and Development of Reinforcement.*

***Richard N. White,** also a Fellow of ACI, is professor and director of the School of Civil and Environmental Engineering, Cornell University. A member of several ACI committees, he has been at Cornell since receiving his PhD from the University of Wisconsin in 1961.*