THE BEHAVIOR OF MULTIPLE LAP SPLICES IN WIDE SECTIONS

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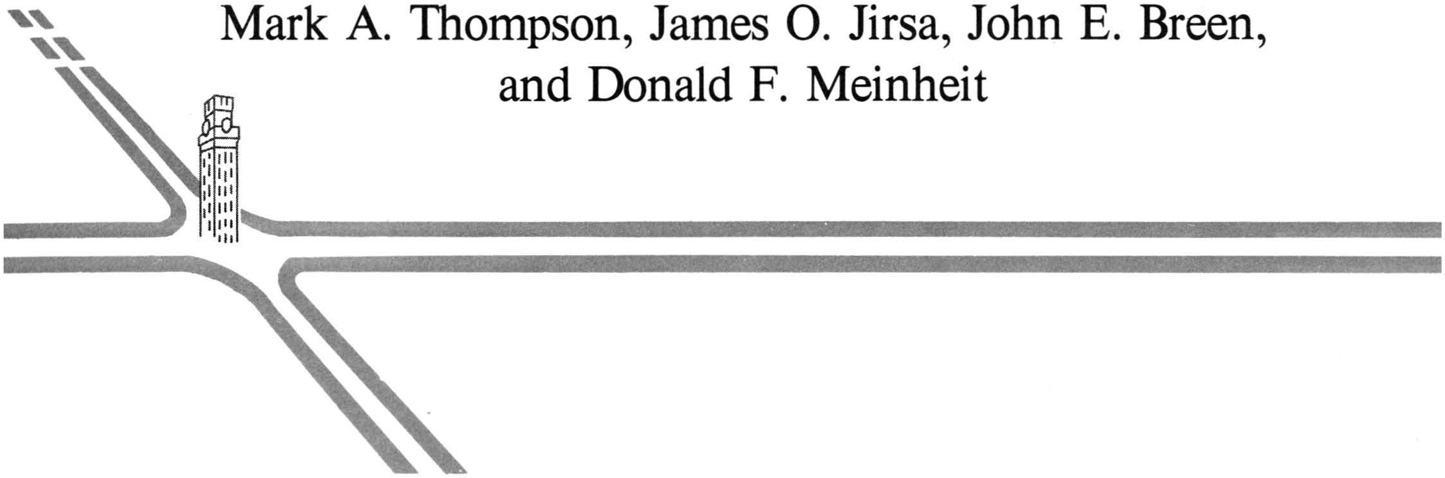

SUMMARY REPORT 154-1 (S)

SUMMARY OF
RESEARCH REPORT 154-1

PROJECT 3-5-72-154

COOPERATIVE HIGHWAY RESEARCH PROGRAM
WITH TEXAS HIGHWAY DEPARTMENT
AND
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Introduction

For a typical cantilever retaining wall, construction procedures normally require lap splicing of the reinforcing steel at the junction of the wall and the base. This junction is a region of peak moment in the wall, and a splice failure in such a situation would mean failure of the wall, since the structure has no redundancy. Therefore, an understanding of the behavior of lapped splices is essential to the design of a retaining wall.

The performance of lap splices in narrow beam sections has been the subject of extensive investigation. Tests indicate that the failure of spliced sections may initiate at an edge. The behavior patterns of wall splices have been studied but not completely bounded, due to limitations on the number of bars spliced and size of the specimens tested. Available test data seem to indicate that a splice in a wide section could be considered stronger than a similar splice in a beam containing only a small number (one or two) of spliced bars. The added strength of splices in a wall could be attributed to the fact that a smaller percentage of splices in a wall section are edge splices.

Therefore, the basic questions are: (1) What are the behavior patterns of wall-type spliced sections? (2) Would the alteration or elimination of the edge splices in such a section lead to a significant increase in strength of the wall splice? (3) How much would transverse reinforcement in the splice region of a wide section affect the strength of the section? (4) How well do splice strength equations predict the performance of wall section splices?

Test Program

Twenty-five tests were conducted to study reinforcing bar splice behavior in wide sections. The tests were proportioned to simulate a cantilever retaining wall. Figure 1 compares the prototype and the model. In a typical retaining wall, the splice would be vertically cast and would be subjected to a moment gradient with only one end of the splice subjected to maximum moment. All test specimens were subjected to a two-point loading to produce a constant moment along the splice. With uniform moment the splice is subject to a stress condition as severe as that in the prototype.

Of the twenty-five specimens tested, one contained #6 bars, ten had #8 bars, ten had #11 bars, and four contained #14 bars. The loading was applied 6 in. from the end of the specimen. The overall specimen length for the #14 bar specimens was 21 ft., with smaller bar specimens having an overall length of 17 ft. The reactions were located 12 in. outside the ends of the splices. With varying splice lengths (ℓ_s) and constant specimen lengths, the shear span was varied between specimens. Fig. 2 shows a typical cross section for a specimen with all bars spliced.

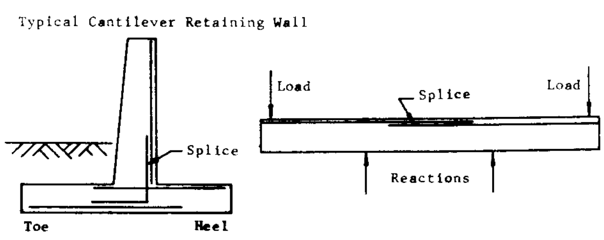


Fig 1. Prototype and model

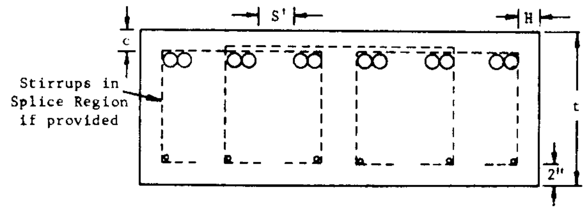


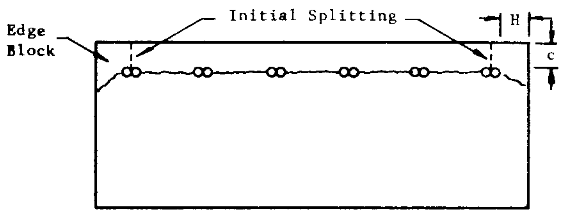
Fig 2. Typical cross section of #6, #8, and #11 bar specimens with all bars spliced (shown in testing position)

Variables. (1) Splice length and bar diameter. Splice length varied from 12 in. for a #6 splice to 60 in. for #14 splices.
(2) Ratio of clear bottom cover to clear spacing of splices. The clear spacing between the spliced bars was maintained at 4 in. for all tests, except for one specimen with #11 bars which had a clear spacing of 6 in. The clear bottom cover was varied from 1 in. for several of the #8 and #11 bar specimens, to 3 in. for two of the #8 bar specimens. The ratio of cover to clear spacing, C/S', varied from 0.25 to 0.75.
(3) Edge condition. Previous studies [1] have indicated that in the case where a number of bars are spliced at the same section, the failure of the splice may be initiated by splitting near the edge or outside splice. In a wide wall section, the edge splices may be a small percentage of the total number of splices. For the wide specimens tested in this program, the number of edge splices to total number of splices was either 2 to 5 or 2 to 6. In specimens with all bars spliced, the clear edge cover was equal to one-half the clear spacing in seventeen tests and in four tests was equal to the clear spacing. In four tests, the outside edge bar was continuous.
(4) Transverse reinforcement. Seven specimens contained transverse reinforcement in the splice region, six with U-stirrups and one with spiral reinforcement.
(5) Casting position. Current design recommendations require that for top cast bars with greater than 12 in. of concrete cast below, the splice length must be increased by 40 percent. In a typical retaining wall, the splice is cast in a vertical position. Vertical casting of test specimens was ruled out because of size and handling problems. Bottom cast specimens are likely to match the bond characteristics of a vertically cast splice in a wall quite closely; however, two specimens were cast with more than 12 in. of concrete below the bars in order to bound the problem.

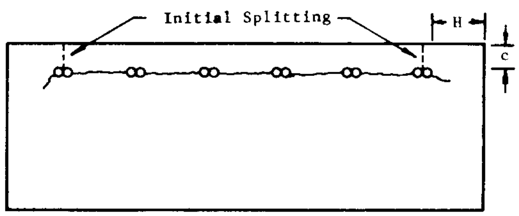
Test Procedure. The loading was applied incrementally until failure occurred. All deflection and strain data were recorded after each load increment. The widths of flexural cracks at the splice region were measured with a crack-measuring microscope at various load levels. After failure, crack patterns were photographed.

Test Results

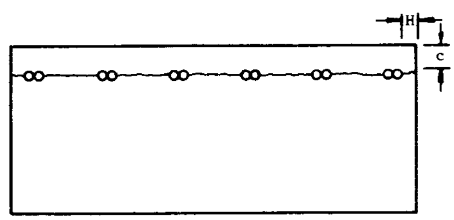
Cracking Patterns and Failure Modes. Splitting cracks in the vicinity of the splice led to the failure of the specimens. The following modes of failure (shown in Fig. 3) were observed in the specimens tested this program.
(1) Face and side split—Initial splitting occurred in the clear cover over the edge splices. As splitting cracks developed on the sides, the edge "block" would tend to break loose, destroying the bond along the outside edge splices. The remaining interior splices



Face and Side Split Failure Mode ($c \leq H \leq 2c$)



Confined Face Split Failure Mode ($H > 2c$)



Face Split Failure Mode ($H < c$)

Fig 3. Failure modes

failed in a face split mode (cover over the bars split in a plane through bars).

(2) Confined face split mode—Specimens with large edge cover or continuous edge bars failed in the confined face split mode. The first splitting cracks in the splice region appeared over the edge splices. Because the edge cover was wide and relatively stiff, splitting cracks did not form on the sides of a specimen. Failure resulted in a lifting of the clear cover between the edge splices.

(3) Face split mode—For specimens in which the edge cover was less than the clear cover, the first sign of distress appeared as side splitting cracks when failure was imminent.

Bar Strains across End of Splice. The strain distributions related directly to the failure patterns described. Generally for the face and side split mode of failure, the edge splices showed the greatest cracking and splitting distress. The loss of capacity associated with splitting is evident in the strain distributions, because the edge splices were at lower strains (or stresses) than the interior splices. The presence of transverse reinforcement, stirrups or spirals, in the splice region did not seem to affect the distributions of steel strain. This correlates with the observation that the addition of transverse reinforcement in the splice region did not seem to alter the failure mode of a specimen.

Bar Strains along Splice. Since the rate of variation of bar stress (or strain) along the splice length is proportional to the local bond stress along the bar, the rate of change of the strain along the lap length represents the bond stress developed along the splice length. A study of the strain distributions along the exterior and interior splices gave additional insight into splice behavior. At loads below the failure load, the rate of change of the strain along the edge bars was generally equal to or greater than the rate of change along the interior splices. As failure of the specimen was approached, the rate of change of strain along the exterior bars tended to decrease, indicating a drop in bond stress along these bars. This was verified by the cracking patterns observed for specimens with all bars spliced in which the cracking prior to failure was concentrated around the edge splices. The strain distributions along interior splices exhibited a fairly constant slope near failure. It was noted previously that there was little cracking at interior splices prior to failure.

Strains in Transverse Reinforcement. At cracking of the concrete in the plane of the splice, the rate of change of strain in the transverse steel increased, indicating that the stirrups were picking up a larger amount of stress (or strain) per unit increase of stress (or strain) in the longitudinal reinforcement. For a given strain in the longitudinal steel, the strain was lower in stirrups located further from the splice end. For edge splices the strain in the transverse steel increased greatly prior to failure of the specimen and indicates edge splice failure precedes failure of the entire section.

Average Crack Widths. The widest flexural cracks in the constant moment region occurred at the ends of the splice. The average crack width across the splice end at a working stress level in the longitudinal steel of approximately 36 ksi ranged from 0.007 to 0.24 in.

Evaluation of Test Results

In all tests, the splice section was a region of high tensile stress in the bars ($f_s > 0.5f_y$) and more than half the bars were spliced at a section. Under these conditions the splices are classified as Class C splices following current ACI [3] and AASHTO [4] specifications for splices. Provisions for determining splice length are essentially the same in both codes.

Using code provisions [3,4] stresses were calculated for each of the twenty-five specimens. A ratio of the measured average steel stresses at failure to the calculated stresses for each of the specimens ranged from 1.51 to 3.80. The predicted strength, using ACI and AASHTO specifications for splices was always underestimated for the wide splice sections tested in the program. This apparent conservatism is due to the fact that current design equations do not reflect all of the parameters which have been shown to be critical to splice strength. As a result, a modification of design procedures may result in considerable economy without sacrificing safety.

Conclusions

Based on the test results obtained in this study, the following conclusions can be made:

(1) Increased edge cover or the use of continuous edge bars in a wide section may provide up to about a 10 percent increase in total splice section strength. In general, the strength of a section seems to be governed by the capacity of the interior bars, such that a modification of edge conditions does not appear warranted for design.

(2) Prevailing ACI and AASHTO code provisions for length of splices were overly safe when applied to the wide specimens tested in this program. The strength of the specimens as tested ranged from between 1.5 to 3.8 times the strength predicted using current provisions. The large difference between the predicted and the measured strength can be attributed to the omission from current provisions of many parameters shown to be critical to splice strength. Because current splice design provisions appear to greatly underestimate splice strength, a reevaluation of splice

design was undertaken in an accompanying phase of this project and is reported in Ref. 5.

References

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KEY WORDS: lap splices, behavior, retaining wall, test, failure.

The full text of Research Report 154-1 can be obtained from Mr. Phillip L. Wilson, State Planning Engineer, Planning & Research Division, File D-10, State Department of Highways and Public Transportation, P.O. Box 5051, Austin, Texas 78763.