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16. Abstract

In Report 242-1, "The Influence of Casting Position on Development and Splice Length of Reinforcing Bars," and Report 242-2, "The Influence of Shear on Lapped Splices in Reinforced Concrete," the details and results of the experimental investigation on the influence of casting position and shear on splice and development length were presented.

Based on the test results, suggestions are made for revising specifications for "top reinforcement" development and splice length as a function of casting position and concrete slump characteristics. The influence of shear and transverse reinforcement geometry is discussed and suggestions made for design applications.

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INFLUENCE OF CASTING POSITION AND SHEAR ON DEVELOPMENT AND SPLICE LENGTH--DESIGN RECOMMENDATIONS

by

J. O. Jirsa and J. E. Breen

Research Report No. 242-3F

Research Project No. 3-5-78-242

Influence of Casting Position and of Shear on the
Strength of Lapped Splices

Conducted for

Texas
State Department of Highways and Public Transportation

In Cooperation with the
U. S. Department of Transportation
Federal Highway Administration

by

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

November 1981

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the views or policies of the Federal Highway Administration. This report does not constitute a standard, specification or regulation.

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

P R E F A C E

In this report, the final phase of a study on "The Influence of Casting Position and of Shear on Strength of Lapped Splices" is presented. The objective of the project was to review existing data and to conduct an experimental program for determining the effect of casting position and shear on reinforcement development and splice strength and to suggest modification, if needed, for design codes. In this report, tests to determine the influence of casting position and shear on development and splice length of reinforcing bars are reviewed and suggestions for changes in design specifications are presented.

The work was sponsored by the Texas State Department of Highways and Public Transportation and the Federal Highway Administration, and administered by the Center for Transportation Research at The University of Texas at Austin. Close liaison with the State Department of Highways and Public Transportation has been maintained through Mr. Melvin C. Jackson, James C. Wall, and Warren K. Sandberg, who served as contact representatives during the project, and with Mr. William Dallas of the Federal Highway Administration.

The project was conducted in the Phil M. Ferguson Structural Engineering Laboratory located at the Balcones Research Center of The University of Texas at Austin. The authors are particularly indebted to Messrs. B. S. Hamad, J. J. Luke, S. Neumann, and A. J. Zekany, Assistant Research Engineers at the Ferguson Laboratory for their major contributions to the overall study.

S U M M A R Y

In Report 242-1, "The Influence of Casting Position on Development and Splice Length of Reinforcing Bars," and Report 242-2, "The Influence of Shear on Lapped Splices in Reinforced Concrete," the details and results of the experimental investigation on the influence of casting position and shear on splice and development length were presented.

Based on the test results, suggestions are made for revising specifications for "top reinforcement" development and splice length as a function of casting position and concrete slump characteristics. The influence of shear and transverse reinforcement geometry is discussed and suggestions made for design applications.

IMPLEMENTATION

The results of this study will permit further refinement of recommendations resulting from Project 3-5-72-154. The recommendations included in that study have been the subject of considerable discussion in appropriate committees of the American Concrete Institute. The Committee on Bond and Development (ACI 408) has made a recommendation to the Building Code Committee of the American Concrete Institute (ACI 318) for changes in the provisions for development length and splices. The proposed changes are based largely on the work carried out under Project 154. Because AASHTO provisions are based primarily on ACI design recommendations, it is likely that the changes in ACI 318 will eventually appear in AASHTO Specifications. To provide a design recommendation which handles all aspects of development and splice length of reinforcement, including the effect of casting position and shear, the research conducted under Project 242 will further improve design recommendations.

Current design specifications contain confusing, often anomalous, statements which are difficult for designers to apply in design situations. The implementation of the results from this program should help to clarify the role of casting position, shear, and properties of fresh concrete on the strength of anchored bars.

C O N T E N T S

FIGURES

T A B L E S

CHAPTER 1

INTRODUCTION

1.1 Introduction and Objective

The transfer of stress from steel to concrete in a reinforced concrete structure is essential for satisfactory performance of the structure. For deformed bars, the stress transfer is largely the result of interlocking (or bearing) of the bar deformations (lugs) against the concrete. A number of factors may influence the stress transfer phenomenon. In this program, the influence of casting position and shear on bar development length and splices has been studied. Results of a literature survey and an experimental investigation conducted during the program are described in Refs. 1 and 2. The objective of this report is to (1) briefly review current design procedures, (2) review the test results in light of existing design provisions, and (3) propose modifications in existing procedures to reflect the test data obtained.

1.2 Current Specifications

1.2.1 Casting Position

Development Lengths. The only aspect of casting position covered by the current AASHTO Bridge Specification or the ACI Building Code is the influence of height of casting on the bond strength of horizontal bars. No difference in bond behavior is indicated for vertical bars compared to horizontal bars and the implication is given that the "top bar" effect is only relevant for horizontal bars. In the 1951 ACI Code, "top bars" were defined as "horizontal bars so placed that more than 12 in. of concrete is cast in the member below the bar." The allowable unit bond stress of top bars was limited to 0.7 of bottom bars.

The 1951 ACI Code provision for top bars was heavily influenced by work done by Clark [7,8]. Clark assumed that the loaded end slip of a bar in a pullout test corresponded to one-half the crack width that would develop in a beam at the same bar stress. A crack width of 0.02 in. would then correspond to a loaded end slip of 0.01 in. Based on serviceability requirements, the 0.02 in. crack width was taken as an upper limit on the permissible crack widths in beams at working loads. Clark's studies provided the basis on which ACI Committee 208, Bond Stress, proposed a set of allowable unit stresses for bond which were later adopted by Committee 318 for the 1951 ACI Code [3].

The ACI Code specifications concerning allowable unit bond stress of a top bar remained unchanged in the 1957 and 1963 ACI Codes [4]. With the introduction of ultimate strength design requirements in the 1971 ACI Code [4], the limitation on unit bond stress was dropped in favor of development length, ℓ_d, requirements.

$$u_{top} = 0.7\, u_{bottom} \qquad \text{(ACI 318-51)}\ \text{(ACI 318-57)}\ \text{(ACI 318-63)}$$

and $$\ell_d \propto 1/u$$

therefore, $$\ell_{d_{top}} = (1/0.7) \times \ell_{d_{bottom}}$$

$$= 1.4\ell_{d_{bottom}} \qquad \text{(ACI 318-71)}\ \text{(ACI 318-77)}$$

Even though the ACI Code switched to ultimate strength from working stress design criteria, the limitations on development length were still tied to values of unit stress that were based on serviceability requirements. The development length specifications of the 1971 ACI Code (ACI 318-71) were adopted by the 1979 AASHTO (American Association of State Highway and Transportation Officials) Interim Specifications for Bridges, and more recently the 1977 ACI Code (ACI 318-77) [4] has accepted this specification unchanged. It is of interest to note that the ACI Code defines a top bar -- "12 in. of concrete cast in member below the bar." The 1975 AASHTO Specifications include the same definition, however, subsequent versions omit the definition, so that a "top" bar is not specified.

In 1978, ACI-ASME Technical Committee on Concrete Pressure Components for Nuclear Service [6] proposed that the 1.4 factor for "top bars" not be applied to horizontal or diagonal wall bars. The committee suggested that in walls the effect of rising water and air is lessened by the depth of the member and by the multiple runs of horizontal bars, which tend to distribute any excess air and water. The committee also suggested that the higher hydrostatic head of the wet concrete in walls would minimize the bond degradation under bars. No experimental evidence was given to support the proposal.

It is evident that provisions for top reinforcement based on a serviceability limitation for the crack width (0.02 in.) in beams were retained in the current AASHTO Specification and ACI Code, even though ultimate strength design is specified. However, tests show that reinforcement tends to show increasing resistance to slip at higher levels of loading before failure.

Splice Length. The current ACI 318-77 Building Code [4] and AASHTO 1979 Interim Bridge Specifications [5] determine splice length by applying factors to the basic development length. The factors depend on (1) the percentage of steel spliced within the lap length, and (2) the ratio of area of reinforcement provided to the area of reinforcement required by analysis. For example, for Class C tension lap splices (defined in ACI 318-77, Section 12.16.2 [4] and in AASHTO 1979 Specifications, Section 1.5.22 [5], the required splice length is 1.7 times the development length.

$$\ell_{s\,(bottom)} = 1.7 \cdot \ell_d$$

$$\ell_{s\,(top)} = 1.7 \times 1.4\ \ell_d$$

In a Class C splice, more than 50 percent of the bars are spliced at a given section and the bar stresses at the section are more than 0.5 f_y.

1.2.2 <u>Shear</u>. The influence of shear on development or splice length is not included directly in the computations for ℓ_d or ℓ_s. However, AASHTO and ACI provisions require that the bar diameter be selected so that

$$\ell_d \leq \frac{M_n}{V_u} + \ell_a$$

where M_n = nominal moment strength assuming all reinforcement at the section to be stressed to the specified yield strength f_y

V_u = factored shear force at the section

ℓ_a = additional embedment length at support or at point of inflection

The equation is based on the flexural bond stress requirements contained in previous codes. Along a beam subjected to a moment gradient (shear), there will be bond stresses developed along the bar.

Present AASHTO and ACI provisions ignore the effect of transverse reinforcement on the development of bars and splices, except for the confining effect of spirals.

CHAPTER 2

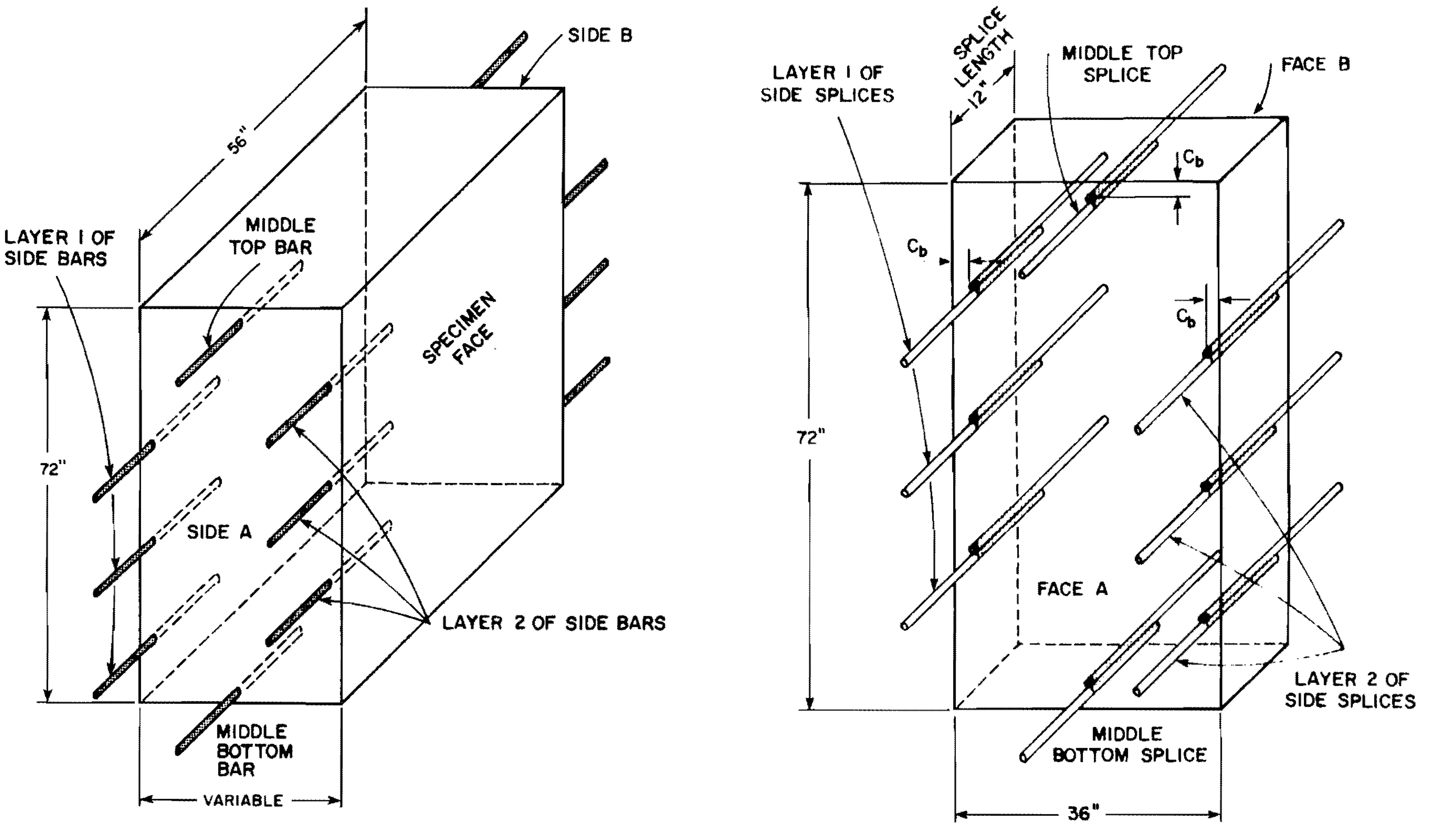
REVIEW OF TEST RESULTS

2.1 Tests to Evaluate Influence of Casting Position

2.1.1 Specimens. The test specimens consisted of large blocks of concrete with anchored or spliced bars cast in the block with relatively thin side cover as in walls, as shown in Fig. 2.1. The test bars were arranged so that the depth of concrete placed in the form beneath the bar was varied. Other variables considered included concrete strength, concrete consistency (slump), concrete cover, and bar or splice orientation. The length of the anchored bar or splice was chosen to ensure that failure would occur as a result of concrete splitting and not steel yielding. The embedded lengths were 12 in. for #7 and #9 bars and 20 in. for #11 bars.

In Specimens D1, D2, and D3, sixteen bars were tested. The primary variable was the casting position. Specimens D1 and D2 were cast with concrete having a 3 in. slump but for Specimen D3 the design slump was increased to 8-1/2 in. In Specimen D4, the behavior of horizontal and vertical bars was studied.

Two specimens were tested to determine the influence of casting position on the behavior of horizontal lapped splices. In Specimen S1, the splices were oriented so that the plane of the splice (the plane containing the line of tangency and the axes of the two horizontal bars) was perpendicular relative to the bottom of the formwork (stacked splices). Splices at the top and bottom surface of the concrete were perpendicular to the face of the concrete, while the side splices were parallel to the concrete face. These orientations are referred to as face-perpendicular and face-parallel splices, respectively (Fig. 2.2). To determine whether the splice orientation relative to the concrete face or relative to the bottom of the formwork would dominate in determining the mode of failure for splices cast at different heights in the specimen, Specimen S2



(a) Specimens D1, D2, and D3

(b) Specimen S1 (stacked splices)

Fig. 2.1 Test specimens--casting position

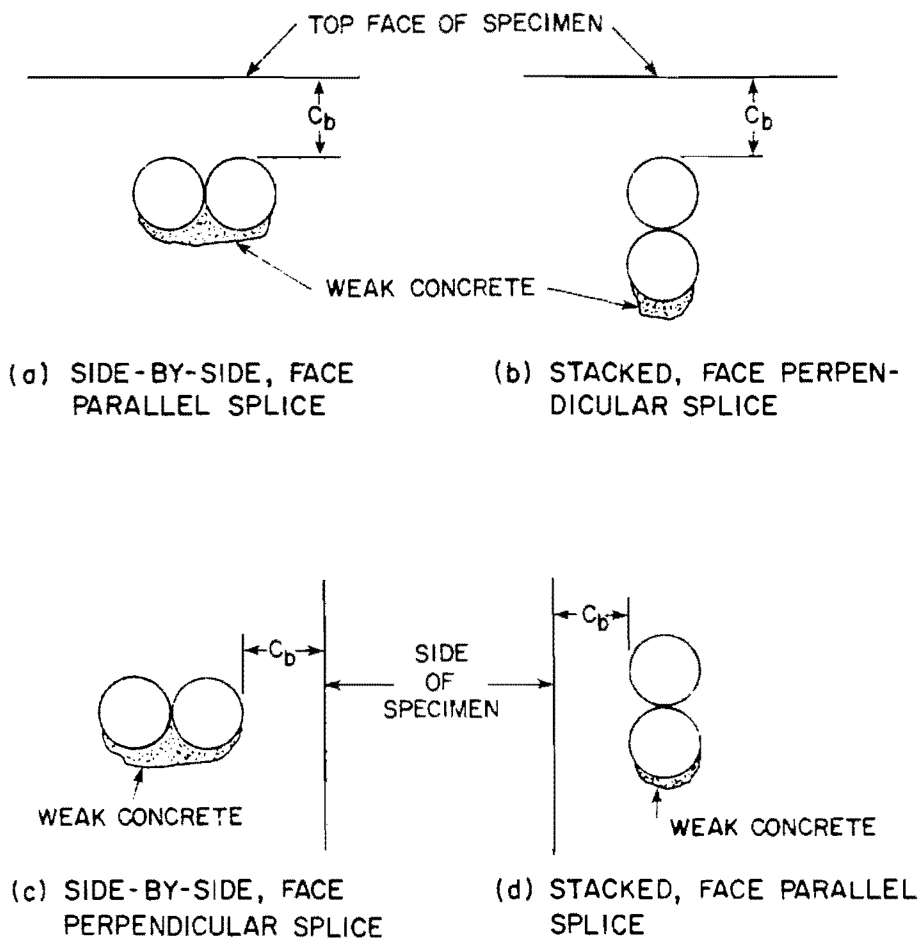
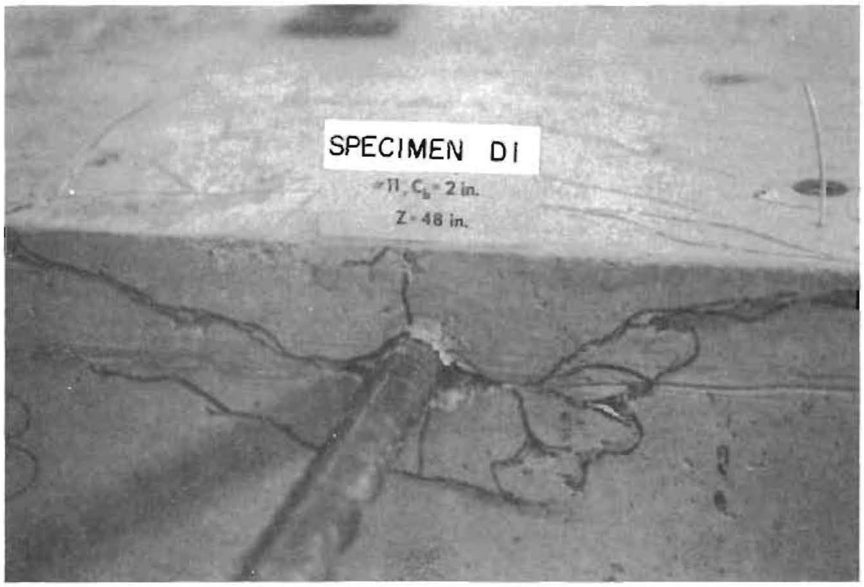


Fig. 2.2 The effect of splice orientation on accumulation of inferior concrete

contained all horizontal bars spliced side-by-side, with the top and bottom splices oriented face-parallel and the side splices face-perpendicular.

2.1.2 Behavior. In almost all tests the slip at the loaded end started at an early stage of loading. Bond splitting started over the bar at the loaded end and progressed toward the unloaded free end. When testing the bars, the loading was normally halted when splitting spread over the entire concrete cover along the anchorage length of the reinforcing test bar. The type of failure which describes the crack pattern of these bars is a V-notch failure. Figure 2.3 illustrates the V-notch failure for the side bars in Specimens D1 and D2. Transverse reinforcement prevented cracking from spreading into the zone of adjacent bars but did not affect the cover directly over the test bar.

2.1.3 Test Results. Typical stress-slip curves for bars in concrete with low (3 in.) slump are shown in Fig. 2.4. Tables 2.1 and 2.2 list the ultimate steel stresses for Specimens D1 and D2. Bond efficiency ratios, relative to the adjusted ultimate stress of the middle bottom bar, are listed for the different casting positions. Similar bond efficiency ratios were calculated for stresses at 0.01 in. loaded end slip for Specimens D1 and D2. In Fig. 2.5, bond efficiency ratios at ultimate and at 0.01 in. loaded end slip are plotted. The results indicate a drop in bond strength with increase in the height of the bar above the bottom of the form. Figure 2.5 shows that up to a height of 48 in. the reduction in bond strength at ultimate is less than 10 percent for #7 and #11 bars and less than 15 percent for #9 bars. The ACI Code and AASHTO specify a 30 percent reduction in bond strength (or a 40 percent increase in development length) for a "top bar" with more than 12 in. of concrete cast below the bar. Figure 2.5 shows that the reduction in bond strength with height is generally greater at 0.01 in. loaded end slip than at ultimate. The larger reduction provides some justification for the current specification values for "top bar" when based on a 0.01 in. loaded end slip. However, even using stresses at 0.01 in. loaded end slip, the 30 percent



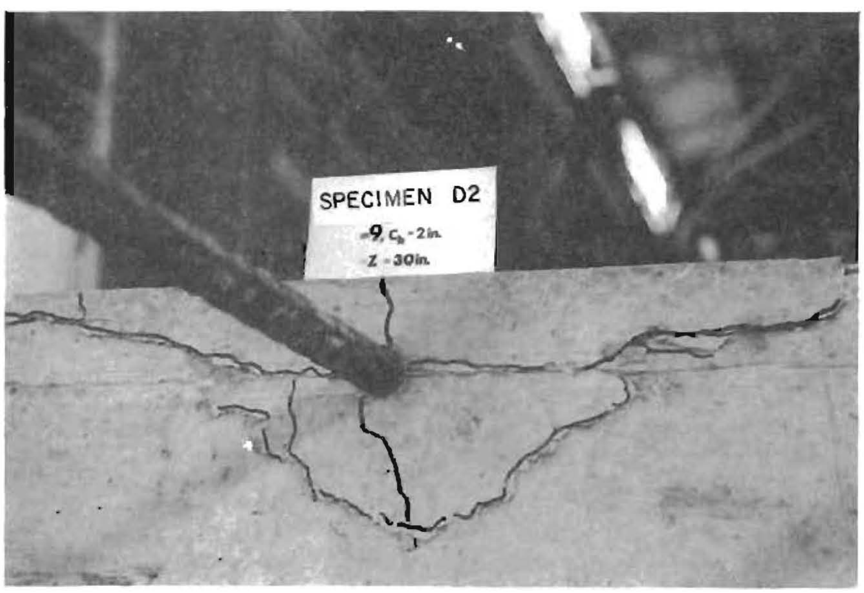


Fig. 2.3 V-notch splitting mode of failure of side bars of test Specimens D1 and D2

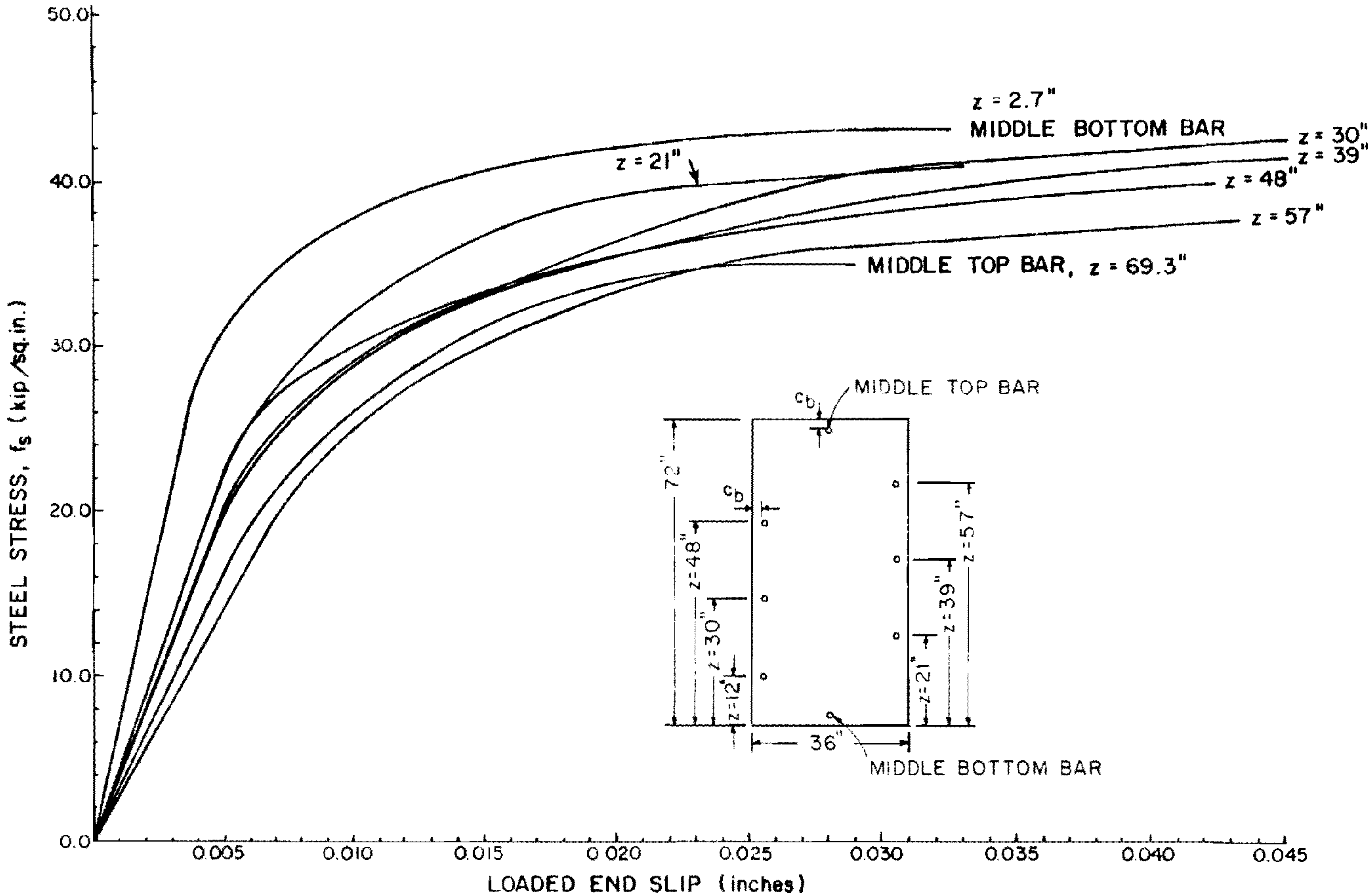


Fig. 2.4 Stress-slip relationship for 11 bars (Specimen D1, 3-in. slump)

TABLE 2.1 ULTIMATE BAR STRESS, SPECIMEN D1, #11 BARS

Casting position z in.	Side A				Side B				
	f'_c psi	f_{su} ksi	f'^{*}_{su} ksi	$\left(\frac{f'_{su}}{f'_{su\ bot.}}\right)$	f'_c psi	f_{su} ksi	f'^{*}_{su} ksi	$\left(\frac{f'_{su}}{f'_{su\ bot.}}\right)$	$\left(\frac{f'_{su}}{f'_{su\ bot.}}\right)$ average
Middle Top Bar	2715	33	35	0.81	2715	35	36	0.82	0.82
57	3150	39	48	0.87	3150	36	35	0.79	0.83
48	3150	40	39	0.90	3000	40	40	0.89	0.90
39	3150	42	41	0.96	3150	42	41	0.93	0.94
30	3150	44	43	0.99	3150	42	41	0.93	0.96
21	3150	42	41	0.96	3150	44	43	0.97	0.96
12	3150	--	--	--	2715	--	--	--	--
Middle Bot. Bar	2715	41	43	1.00	2715	42	44	1.00	1.00

$^{*}f'_{su} = f_{su} \times \left(\frac{3000}{f'_c}\right)^{\frac{1}{2}}$

TABLE 2.2 ULTIMATE BAR STRESS, SPECIMEN D2

Casting position z in.	#7 Side				#9 Side			
	f'_c psi	f_{su} ksi	f'^{*}_{su} ksi	$\left(\frac{f'_{su}}{f'_{su\ bot.}}\right)$	f'_c psi	f_{su} ksi	f'^{*}_{su} ksi	$\left(\frac{f'_{su}}{f'_{su\ bot.}}\right)$
Middle Top Bar	3125	41	40	0.85	3200	32	31	0.80
57	3400	44	41	0.89	3335	33	31	0.82
48	3400	45	42	0.90	3450	36	34	0.88
39	3400	45	42	0.90	3400	35	33	0.86
30	3400	45	42	0.90	3450	39	36	0.95
21	3450	48	45	0.96	3335	39	37	0.97
12	3400	48	45	0.97	3400	--	--	--
Middle Bot. Bar	3200	48	47	1.00	3125	39	38	1.00

$^{*}f'_{su} = f_{su} \times \left(\frac{3000}{f'_c}\right)^{\frac{1}{2}}$

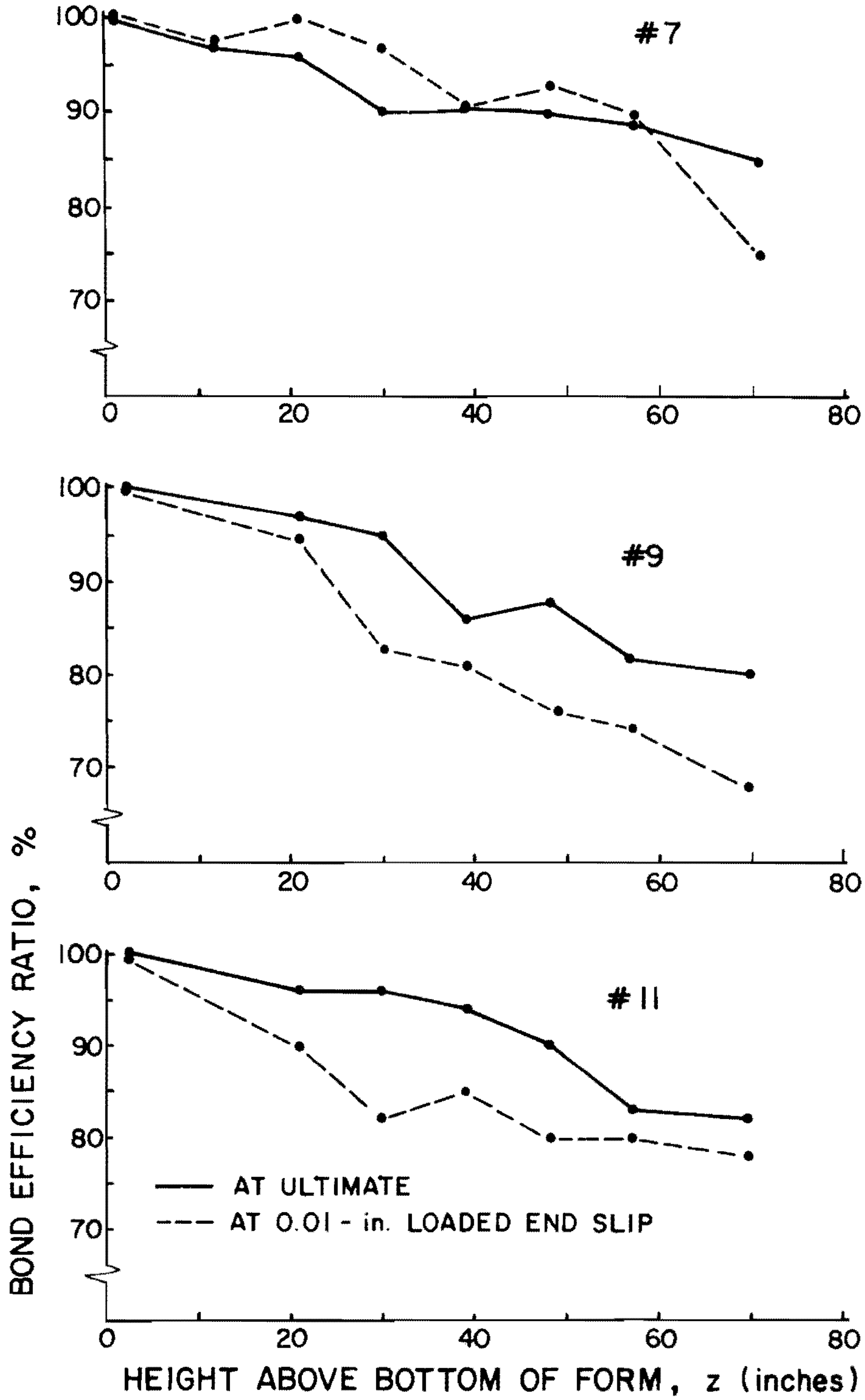


Fig. 2.5 Bond strength reduction--casting position relationship, normal slump (3") specimens

reduction in bond strength for a "top bar" is still greater than the reduction observed in Specimens D1 and D2 which had concrete cast with a 3 in. slump.

Selected stress-slip curves for bars in Specimen D3 (high slump) are shown in Fig. 2.6. The lower bars in the specimen (especially the bottom bars) showed a gradual increase in the rate of slip with increased load. However, the upper bars in the specimen showed very sharp increases in the rate of slip at relatively low loads. The effect of an increase in slump is quite apparent when the bond efficiency ratio ($f'_{su}/f'_{su\ bottom\ bar}$) is plotted against bar height in Fig. 2.7. The ratios for the #9 bars remain almost parallel up to a bar height of 39 in. but drop sharply thereafter. For the #7 bars, the ratio was substantially reduced at a height of only 12 in. The #7 bar was most likely affected to a much greater degree by the increase in slump than the #9 bar, because the #7 bar had only 1 in. cover while the #9 bar had 2 in. cover. One of the most serious side effects of an increase in slump (or water content) is an increase in shrinkage. With substantial shrinkage, the cracks between the bar and the surface are more likely to occur with small cover than with large cover.

Specimen D4 provided data for comparing influence of the orientation of bars and the direction of loading on the bond characteristics of the bars. Figure 2.8 shows the load-slip curves for the bars at z = 18 in. (For vertical bars this distance measures the mid point of bar embedment.) Horizontal bars generally reached higher stresses than did vertical bars. Vertical bars pulled in the direction of concrete settlement had the lowest strength. The effect of height of the bar during casting appears to be much clearer for the horizontal bars than for either type of vertical bar. Obviously, it is much easier for the water and weak concrete to build up under the horizontal bar than under the lugs of the vertical bars because of the greater area involved. The amount of interior concrete under the lugs of the vertical bars is likely to be highly variable and a function of method of compaction, bar congestion, and workability of concrete.

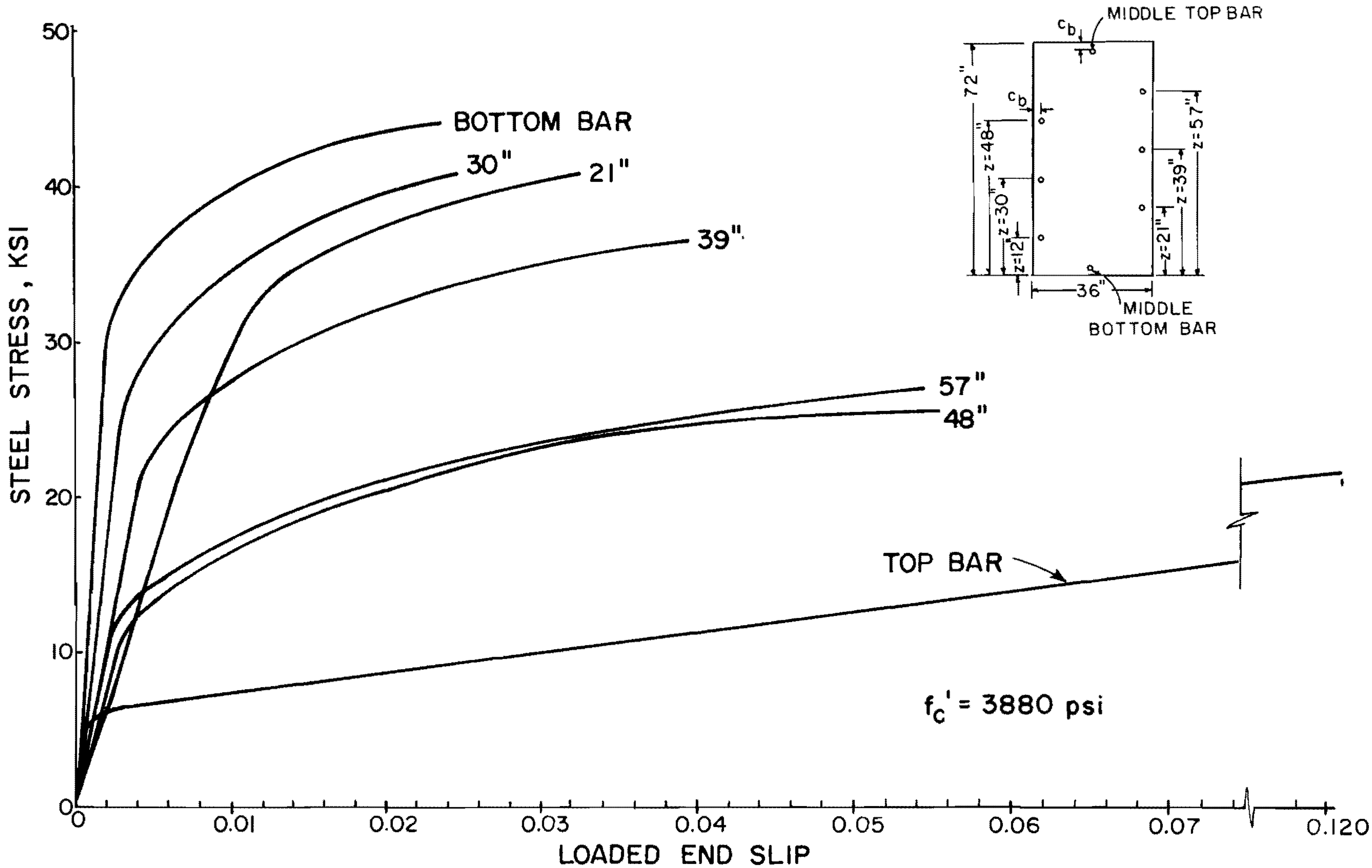


Fig. 2.6 Stress-slip relationship for 9 bars, Specimen D3, 8-1/2-in. slump

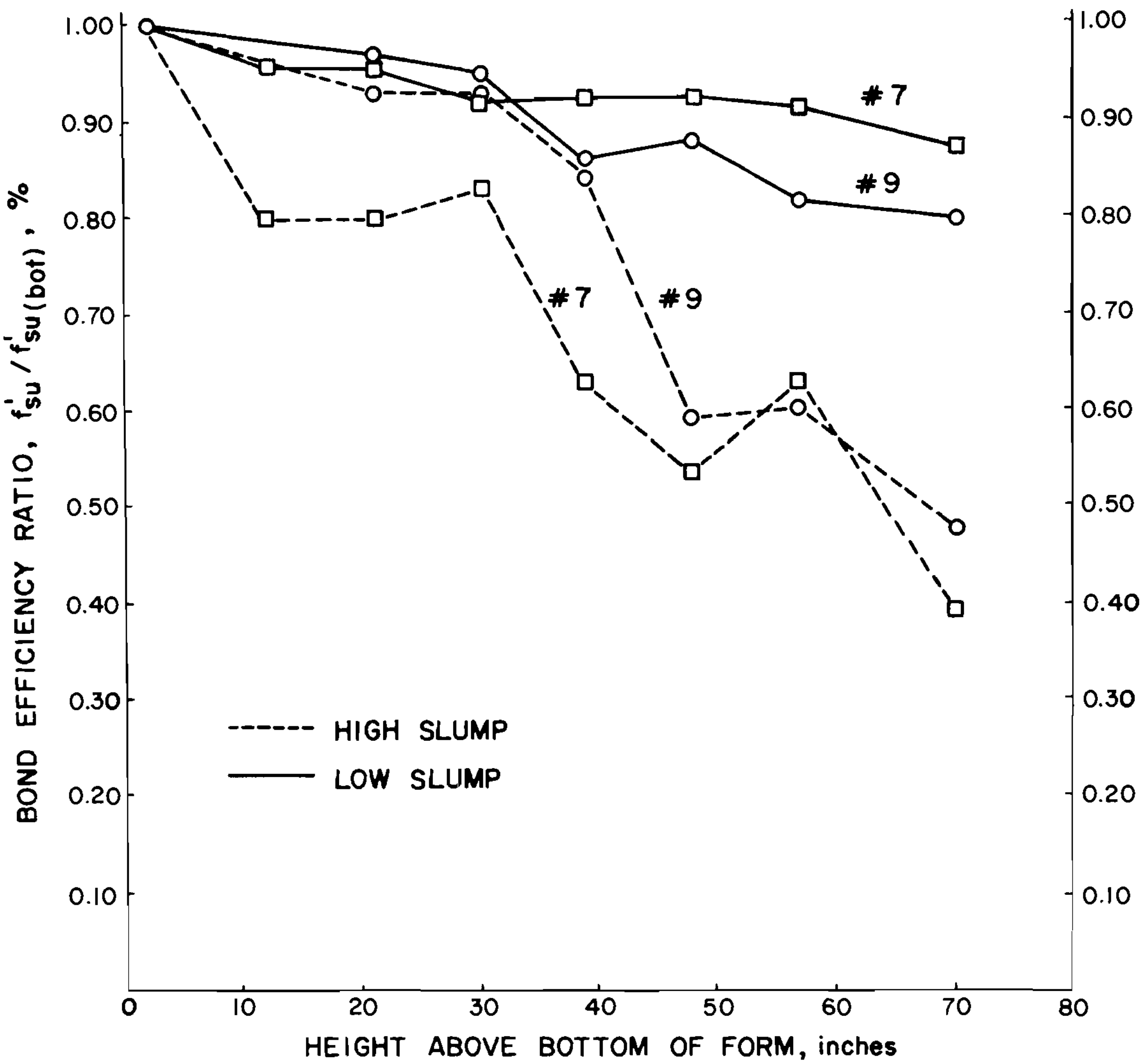


Fig. 2.7 Ultimate bond efficiency ratio vs. bar height, influence of slump

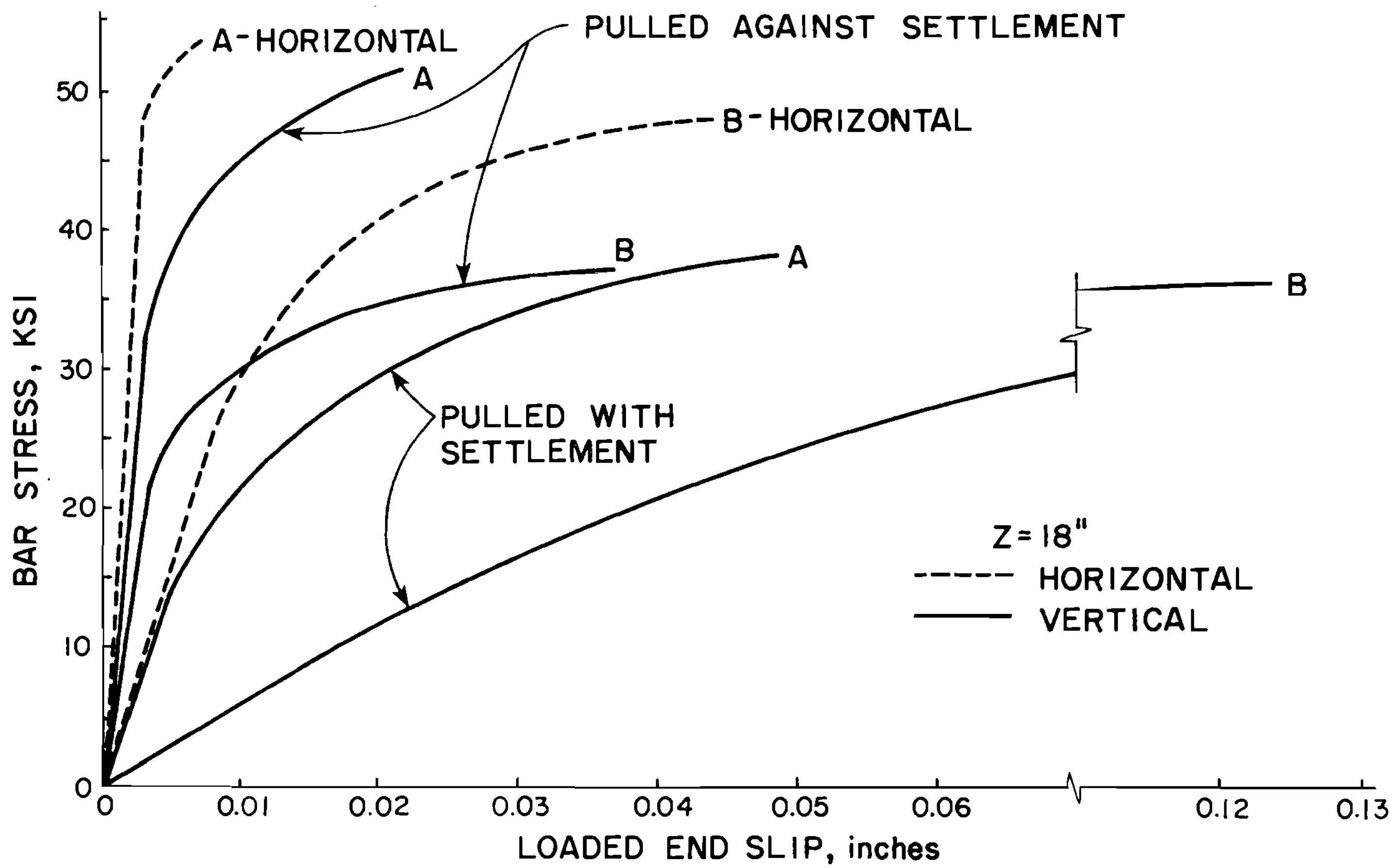


Fig. 2.8 Stress-slip curves for bars at z = 18 in., Specimen D4

Specimens S1 and S2 provided information regarding the influence of casting position on splices. Figure 2.9 shows the load-slip curves for face parallel and face perpendicular splices (Fig. 2.2) at z = 30 in. Both bars in the face-perpendicular orientation displayed a greater amount of initial and ultimate slip than either of the two bars in the face-parallel orientation. This behavior was typical of other splices and indicates that the side-by-side splice arrangement leads to an increase in the accumulation of weak concrete under the bars, which in turn leads to an increase in slip but little change in strength. The initial portion of the load-slip curves is the same for both bars in the face-parallel splice with the lower bar showing slightly more slip at higher loads. This was characteristic of most splices of this type because the weak concrete build-up was concentrated under the lower bar in the splice.

In beams, the longitudinal reinforcement is generally placed along the top and bottom surfaces rather than along the side surfaces as the majority of the test splices were in this program. Therefore, a comparison of the splices at the tops and bottoms of the two test specimens should give a good indication of the differences in behavior between top and bottom splices in beams. Because the depth of the specimen was greater than that of most beams, the differences will be magnified. Figure 2.10 shows stress-slip curves for the top and bottom splices of Specimens S1 and S2. It can be seen that there is very little difference in the ultimate load capacity of the bottom splices. The initial straight line portions of the curves are also quite similar. Bar A of the top splice shows much greater slip near ultimate than either of the two bars of the face-parallel splice.

As can be seen in Fig. 2.11, the splice tests show considerably more variation than the anchored bars when bond efficiency ratios are compared. The splices at intermediate heights show greater relative strengths than the anchored bars at the same levels. Both top splices had lower relative strength than the top anchored bar. The variations

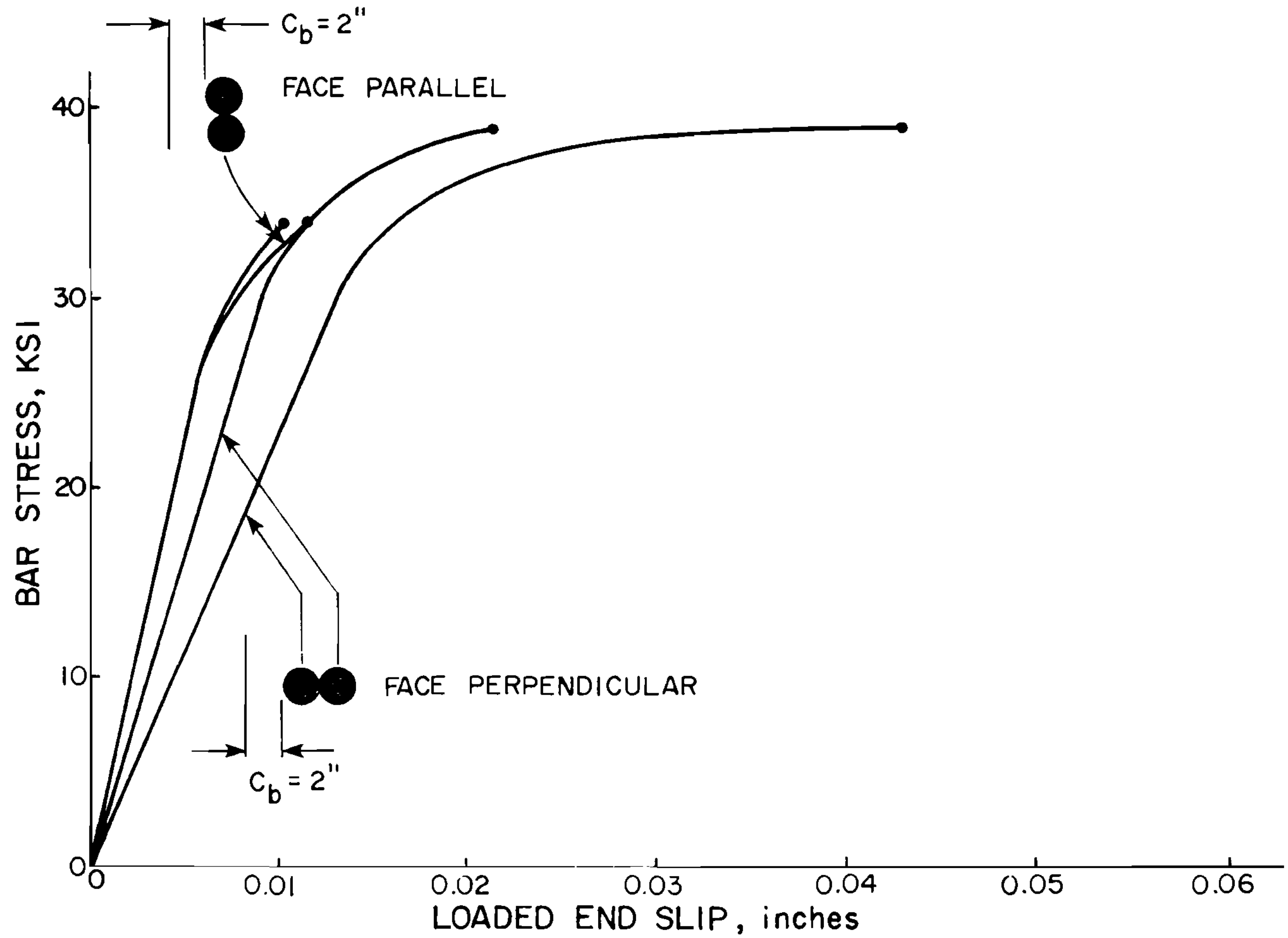


Fig. 2.9 Comparison of stress-slip curves for face-parallel and face-perpendicular splices at z = 30 in.

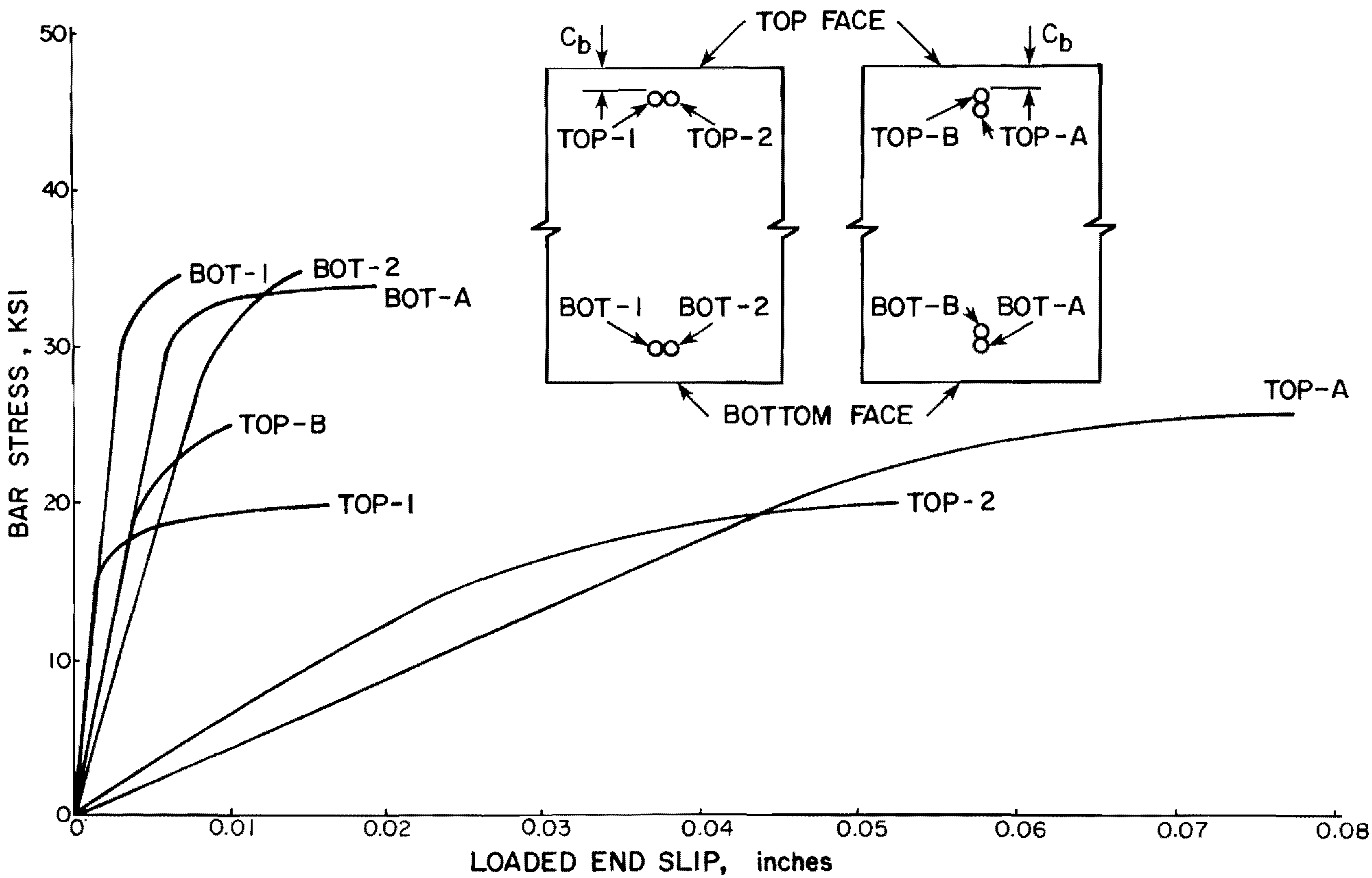


Fig. 2.10 Stress-slip curves for top and bottom splices in S1 and S2

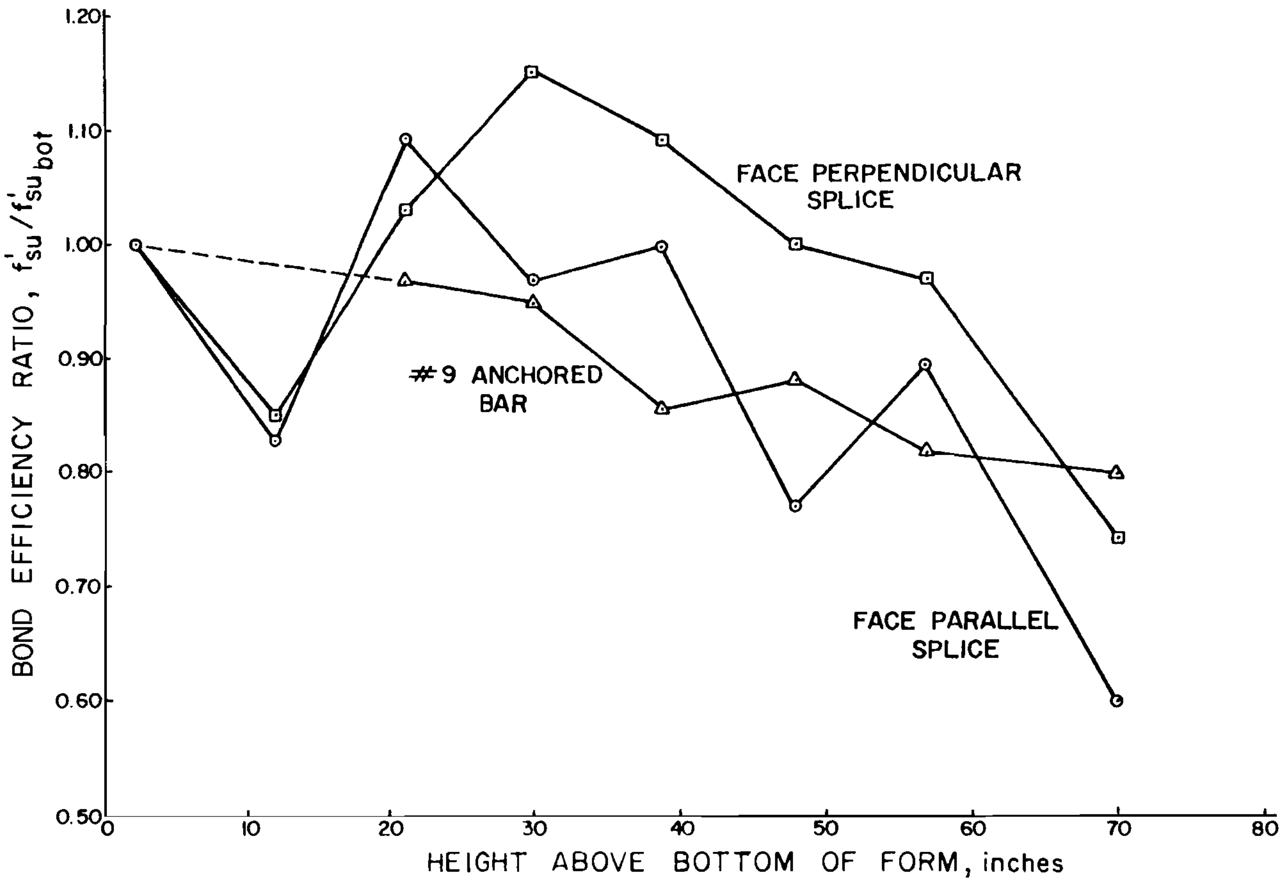


Fig. 2.11 Bond efficiency ratio vs. height for splice and development tests

are to be expected because of the many splice orientations relative to the direction of casting and casting position. The values of ultimate load capacity for both types of splices in the bottom position were very nearly equal. In the limiting case--that is, the top bar and splice values relative to the values of respective bottom tests--the splices appear to show a larger drop in capacity than the anchored bars. It is likely that the relative inferiority of the splices results from the fact that the splice test specimens were cast with a slump of 5.5 in. while the anchored bar specimen was cast with a slump of 3 in. The general trend in the reduction of strength with increase in height of concrete cast below the bars is nearly the same for both splices and development.

2.2 Tests to Evaluate Influence of Shear

2.2.1 Test Specimens--Splices. The test program consisted of twelve beams. Each beam was constructed with both bottom and top cast splice test zones. Each test zone is referred to as a specimen. The basic test specimen is shown in Fig. 2.12 and Table 2.3 summarizes details of the specimens. The clear cover to the longitudinal reinforcement was 2 in. The minimum side cover to the reinforcement was also 2 in. The clear spacing between the splices was 4 in. for #11 bars and 3 in. for #9 bars. All splice lengths were selected to ensure failure by concrete splitting and not steel yielding.

The main variables in this study were as follows:

(a) Level of Shear. Three different shear spans, 40 in., 53 in., and 80 in., were used to vary the level of shear. With an effective depth of 13.3 in., the a/d ratios were 3.0, 4.0, and 6.0, respectively.

(b) Transverse Reinforcement. Three levels of transverse reinforcement were used in the test specimens: (1) no transverse reinforcement, (2) the area of steel providing the ACI 318-77 and AASHTO minimum requirements (shear strength contributed by transverse reinforcement), and (3) the area providing twice the code minimum. Shear on some specimens exceeded the shear capacity of the concrete section. Therefore, the

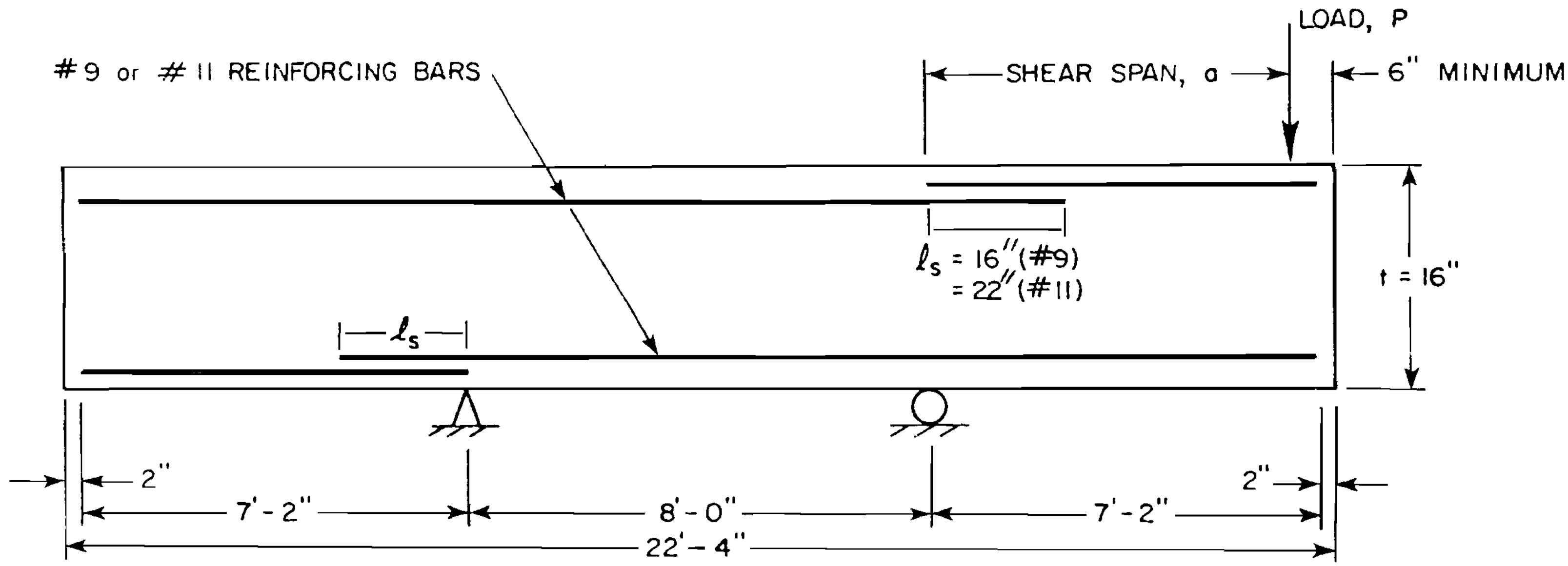


Fig. 2.12 Side view of test specimen (load shown for top cast splice test)

TABLE 2.3 DETAILS OF TEST SPECIMENS

Test No.	Bar Size	Shear Span a (inches)	Concrete: Age at Testing (days)	Concrete: f'_c (psi)	Slump (inches)	f_y (ksi) (Long. Reinf.)	f_{yt} (ksi) (Trans. Reinf.)	Stirrup Size	Stirrup Spacing s (inches)	Number of legs	Casting Position
1	#11	40	66	3700	4.5	60.1	60.3	#3	5	2	top
2	#11	40	75	3750	4.5	60.1	60.3	#3	5	2	bottom
3	#11	53	91	3775	4.5	60.1	60.3	#3	5	2	top
4	#11	53	94	3775	4.5	60.1	60.3	#3	5	2	bottom
5	#11	53	60	4125	7.0	60.1	74.5	6mm	4.5	2	bottom
6	#11	53	63	4150	7.0	60.1	74.5	6mm	4.5	2	top
7	#11	80	44	3825	5.5	60.1	----	---	--	-	top
8	#11	80	45	3825	5.5	60.1	----	---	--	-	bottom
9	#11	80	76	4200	7.0	60.1	74.5	6mm	4.5	2	bottom
10	#11	80	83	4200	7.0	60.1	74.5	6mm	4.5	2	top
11	#11	40	64	3850	5.5	60.1	74.5	6mm	5	4	top
12	#11	40	56	3850	5.5	60.1	74.5	6mm	5	4	bottom
13	#11	53	47	4025	3.5	60.1	70.0	6mm	4.5	2	top
14	#11	53	51	4025	3.5	60.1	70.0	6mm	4.5	2	bottom
15	#11	53	58	4125	3.5	60.1	70.0	6mm	4.5	2	bottom
16	#11	53	64	4125	3.5	60.1	70.0	6mm	4.5	2	top
17	#11	40	55	5425	10.5	60.1	70.0	6mm	4.5	2	top
18	#11	40	57	5425	10.5	60.1	70.0	6mm	4.5	2	bottom
19	#11	40	59	5050	6.5	60.1	70.0	6mm	4.5	2	bottom
20	#11	40	62	5050	6.5	60.1	70.0	6mm	4.5	2	top
21	#9	53	15	5650	7.0	62.8	----	---	--	-	top
22	#9	53	17	5650	7.0	62.8	----	---	--	-	bottom
23	#9	53	21	5700	7.0	62.8	70.0	6mm	4.5	2	bottom
24	#9	53	22	5700	7.0	62.8	70.0	6mm	4.5	2	top

transverse reinforcement was required to carry shear and to resist splitting along the splice.

(c) Configuration of Transverse Reinforcement. Approximately the same area of transverse reinforcement was provided by using two different configurations--two #3 legs @ 5 in. and four 6mm legs @ 5 in.

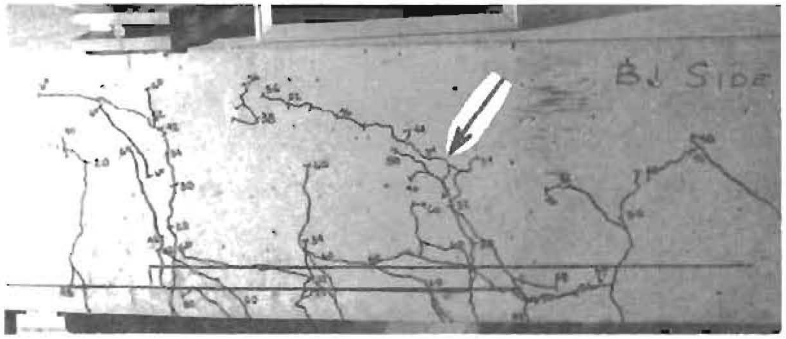
(d) Casting Position. Each beam specimen contained both a top cast and a bottom cast splice region. The top splices had 12.6 in. of concrete cast below the bar, thereby classifying the splices as top reinforcement by the ACI and AASHTO codes.

(e) Concrete Properties. Concrete strength and slump were varied. Concrete strength for beam specimens with #11 longitudinal bars ranged from 4025 psi to 5425 psi. Concrete strength for tests with #9 longitudinal bars was 5700 psi. The slump varied from a low of 3.5 in. to a high of 10.5 in. The 10.5 in. slump was obtained by adding a superplasticizer (HRWR) in powder form to the mix before casting.

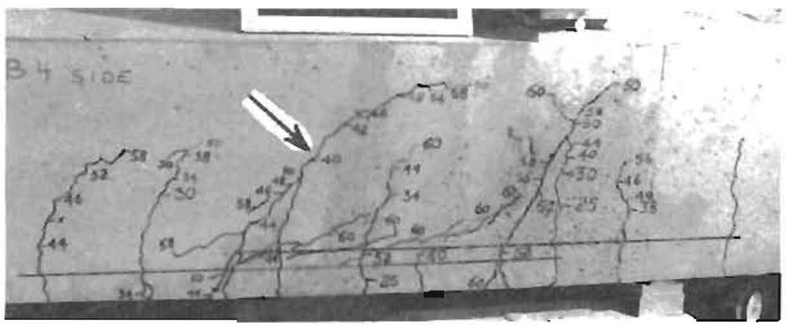
(f) Bar Size. Two different bar sizes were used. Twenty tests contained #11 longitudinal bars, and four contained #9 longitudinal bars. All were designed to develop about the same bond strength.

(g) Splice Location. In two tests the splice was shifted a distance d (13.3 in.) away from the support and the results compared with tests where one end of the splice started at the section where maximum moment was developed, i.e., right at the support of the overhanging part of the beam.

2.2.2 Behavior. The progression of cracking was correlated with measured steel strains. The crack pattern at failure shown in Fig. 2.13 is typical of many specimens. Tensile cracking produced by shear was manifested by diagonal cracks. Splitting cracks on the surface of the specimen produced by anchorage distress were characterized by closely spaced, short low angle diagonal cracks aligned with the axes of the splice. In every test, the first crack to appear on the specimen was a flexural crack at the end of the splice where the moment was greatest. The crack was followed immediately by a similar crack at the other end of the splice. Flexural cracking within the splice zone was fairly evenly



(a) Crack pattern B1 side



(b) Crack pattern B4 side

(c) Crack pattern on tension face, $P_u = 56.9k$

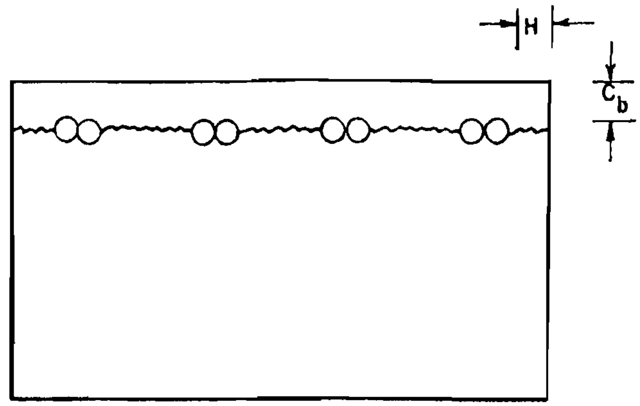
Fig. 2.13 Crack patterns, Test #5

spaced. As the load increased, evenly spaced flexural cracks also formed outside of the splice test zone, generally at the stirrup locations. Flexural cracks in the splice zone continued to extend and to bend diagonally toward the support, indicating the influence of shear.

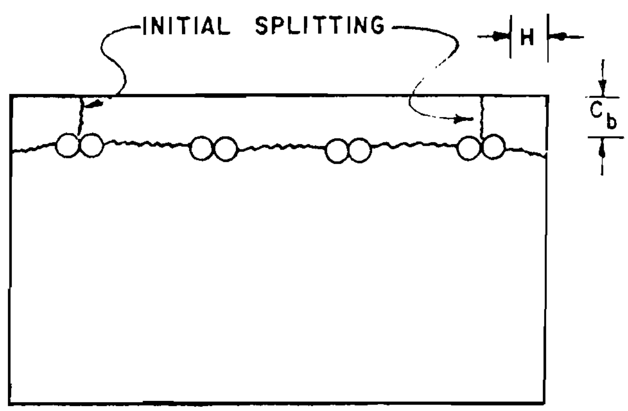
Failure of the specimen was imminent when splitting cracks formed in the splice zone. Splitting initiated on the tension face of the specimen at an edge bar. First splitting was usually observed at the end of the splice subjected to the higher stresses. As the load increased, splitting cracks formed on the side of the specimen. When the splitting cracks began to extend rapidly, bond was lost along the splice and the load dropped off substantially. Failures were quite sudden with little warning except for the growth of the splitting cracks.

Two modes of failure were observed: (1) side split mode (Fig. 2.14a), and (2) main face and side face split mode (Fig. 2.14b). In the side split mode the vertical clear cover is greater than the edge cover. The cover over the splice exhibits no splitting distress prior to failure. Splitting distress appears as side splitting cracks. Upon failure of the splice, the splitting proceeds horizontally until the cover concrete over the tension reinforcement is lifted with no longitudinal cracking in the cover. In the main face and side face split mode, initial splitting occurs in the vertical clear cover over the edge splices, generally both edges. As horizontal splitting cracks develop on the sides, the edge blocks of the concrete tend to break loose, destroying the bond along the outside edge splices. The remaining interior splices then fail by lifting of the clear cover over the reinforcement. In most of the tests conducted in this study, the failure was generally a main face and side face split mode with a slight variation.

Eight tests were conducted with 40 in. shear spans. These specimens were subjected to the highest shear forces. Very few flexural cracks formed except near the support. At loads less than 50 percent of ultimate, transverse cracking was limited to the tension face with cracks extending across the face. Very little transverse cracking occurred on the sides



(a) Side split mode ($H < C_b$)



(b) Main face and side face split mode
($C_b \leq H \leq 2C_b$)

Fig. 2.14 Modes of splitting failure

until the higher loads were reached, at which point the cracks began to extend at approximately 45° angles, reflecting the high level of shear.

Twelve tests were conducted using a 53 in. shear span, an intermediate level of shear. Diagonal shear cracks formed at approximately two-thirds of ultimate. Some of the inclined cracks propagated from the flexural cracks and others extended from the splitting cracks in the splice region. Splitting on the sides of the specimens was more pronounced than in the specimens tested on 40 in. shear spans.

Four tests were conducted using 80 in. shear spans. These specimens were subjected to the lowest levels of shear considered in the series. The amount of transverse reinforcement along the splice was varied. Flexural cracking was dominant with inclined cracks forming near failure. Cracking on the side faces remained nearly vertical as the load increased. The inclined shear cracking propagated from the flexural cracks and the splitting cracks in the splice zone. At failure the shear cracks became extensions of splitting cracks. Splitting was more pronounced in the specimens without transverse reinforcement than in those with minimum transverse steel required by ACI or AASHTO Specifications.

Steel strains across the splices correlated well with observed crack patterns. Typical strain distributions across the end of the splices at different load levels are shown in Fig. 2.15 for #9 bars (Test 24). The strain distributions at low levels were fairly uniform. At higher loads the edge bars picked up less strain than the interior bars. This was primarily due to the formation of splitting cracks in the region of the edge bars.

The distribution of steel strains along selected splices is shown in Fig. 2.16. The rate of change of bar stress (strain) along the splice was proportional to the bond stress developed along the splice length. At low levels of load, only a short length of lap near the ends of the splice was required to transfer the stress in the bar to the concrete. As the load increased, the length of lap required to transfer the stress increased.

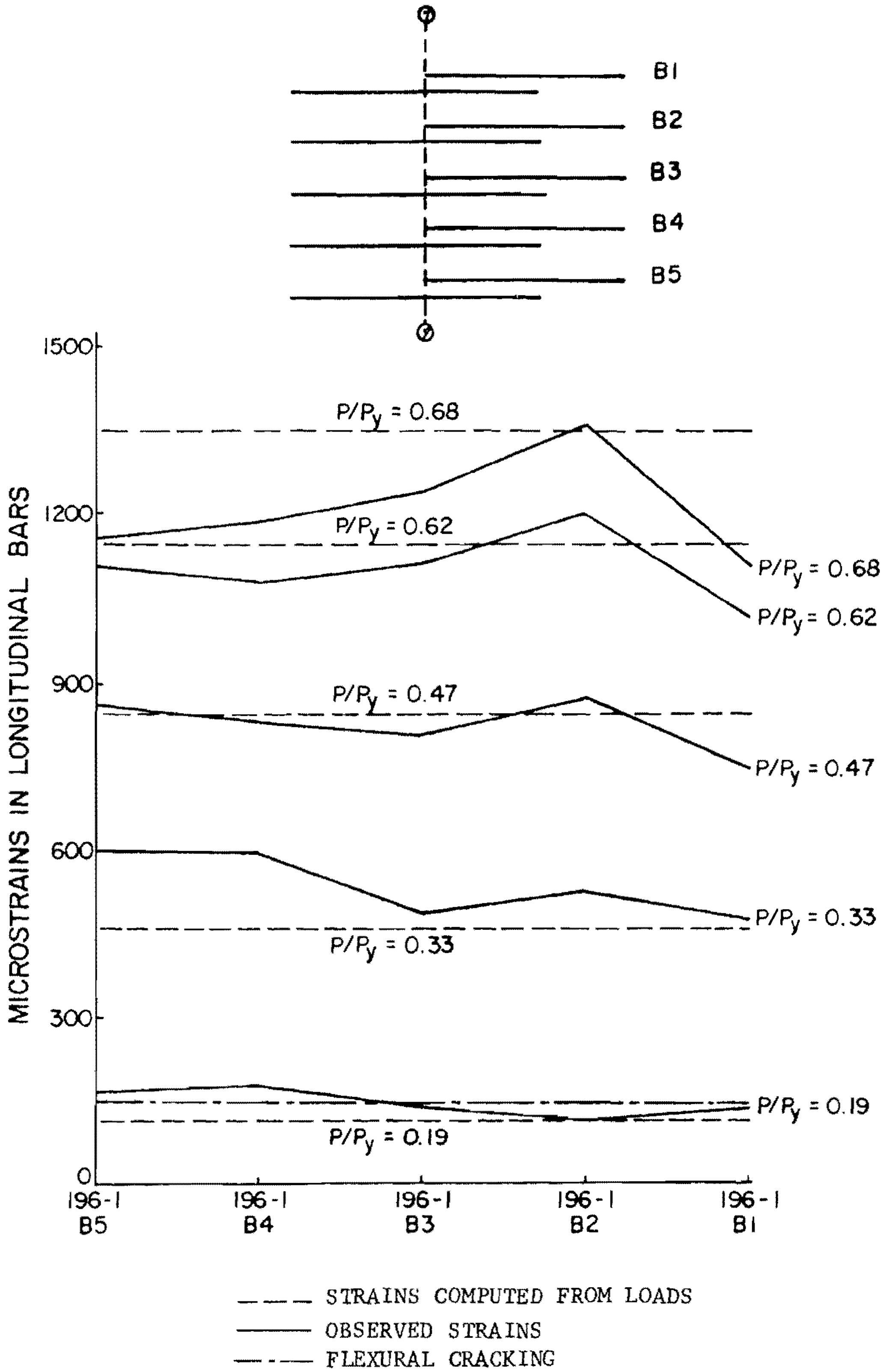
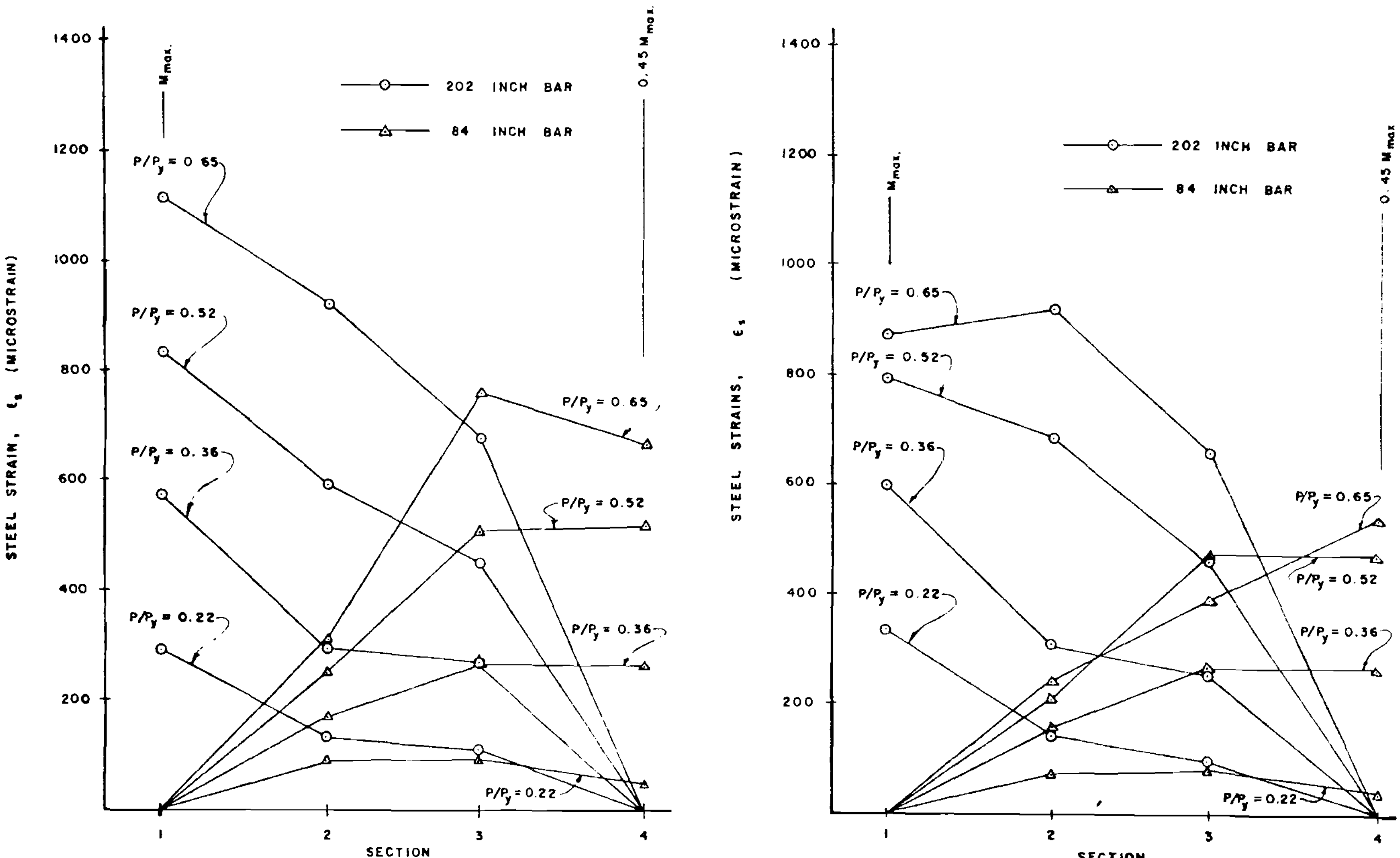


Fig. 2.15 Steel strain distribution across the end of the splice, Test 24



(a) Splice B3--Interior

(b) Splice B1--Exterior

Fig. 2.16 Steel strains along the splice, Test 11, 40-in. shear span

To understand the behavior of the transverse reinforcement in the splice region, plots of load versus stirrup strain were studied. Figure 2.17 shows the load-strain plots for gages S1 and S4 on three instrumented stirrups along a #11 bar splice. These gages give an indication of the effects of side splitting. Examination of the crack patterns indicates that splitting cracks appeared on the surface of the concrete at loads very close to failure. At loads above 55 kips, the stirrup strains increased rapidly.

2.2.3 Test Results. A summary of the test results is shown in Table 2.4. To evaluate the main variables, average bond stresses are compared. The average bond stresses are divided by the square root of the concrete compressive strength. Table 2.4 shows the measured and calculated bond stresses normalized for concrete strength.

Based on the results from the specimens tested in this study, the following observations and conclusions were made:

(1) Level of Shear. The level of shear had an inconsequential effect on the strength of lapped splices. With substantial increases in the level of shear, only negligible changes in the bond strength were observed.

(2) Transverse Reinforcement. Transverse reinforcement was found to be effective in resisting splitting produced by anchorage distress in addition to its traditional role as primary reinforcement for the diagonal tension produced by shear stresses on the section. Inclusion of transverse reinforcement was found to substantially improve the performance. With transverse reinforcement, the splitting distress was less severe and greater deflections prior to failure were observed. The increases in calculated bond strength attributed to the transverse reinforcement were small even though the transverse reinforcement substantially increased the calculated shear strength of the section.

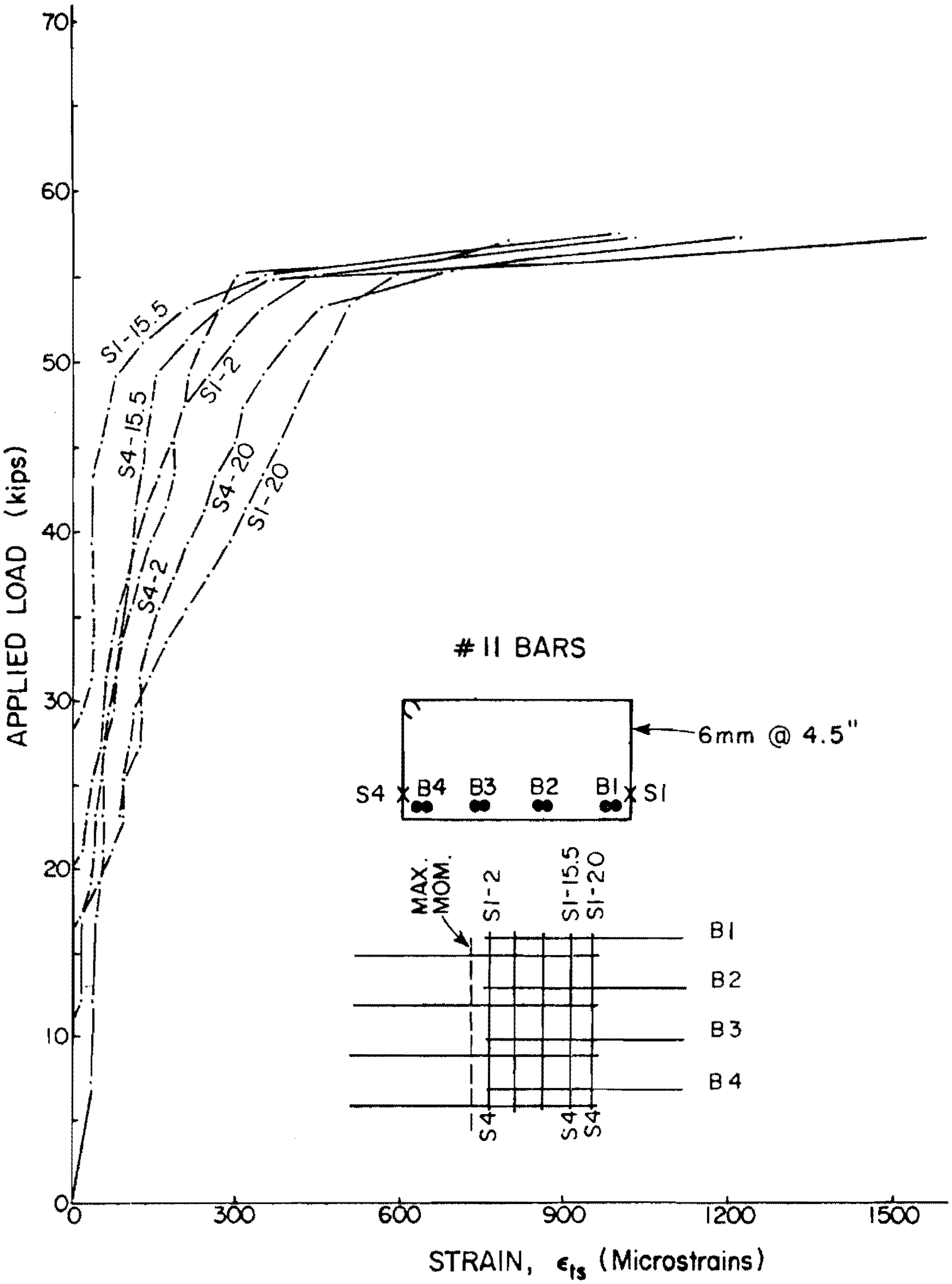


Fig. 2.17 Load vs. strain in stirrups, Test 14, bottom cast, 53-in. shear span

TABLE 2.4 SUMMARY OF TEST RESULTS

Test	Casting Position	Bar Size	Shear Span a (in.)	Slump (in.)	f_c' (psi)	Measured f_s (ksi)	K_{tr} (Eq. 2.7)	$\frac{u_t}{\sqrt{f_c'}}$ (1) (Eq. 2.5)	$\frac{u_{cal}}{\sqrt{f_c'}}$ (2) (Eq. 2.9)	$\frac{u_t}{u_{cal}}$	$\frac{u_{top}}{u_{bottom}}$ (Observed)
1	T	#11	40	4.5	3700	31.8	0.94	8.37	7.38	1.13	0.89
2	B	#11	40	4.5	3700	35.7	0.94	9.41	9.60	0.98	
3	T	#11	53	4.5	3775	33.9	0.94	8.83	7.38	1.20	0.95
4	B	#11	53	4.5	3775	35.8	0.94	9.33	9.60	0.97	
5	B	#11	53	7.0	4150	39.7	0.43	9.62	9.09	1.06	0.92
6	T	#11	53	7.0	4150	35.6	0.43	8.84	6.99	1.26	
7	T	#11	80	5.5	3825	34.7	0	8.98	6.66	1.35	1.00
8	B	#11	80	5.5	3825	34.8	0	9.01	8.66	1.04	
9	B	#11	80	7.0	4200	39.7	0.43	9.82	9.09	1.08	0.93
10	T	#11	80	7.0	4200	37.0	0.43	9.15	6.99	1.31	
11	T	#11	40	5.5	3825	37.1	0.75	9.61	7.24	1.33	0.94
12	B	#11	40	5.5	3825	39.4	0.75	10.21	9.41	1.09	
13	T	#11	53	3.5	4025	37.0	0.41	9.37	6.98	1.34	0.96
14	B	#11	53	3.5	4025	38.6	0.41	9.77	9.07	1.08	
15	B	#11	53	3.5	4125	41.7	0.41	10.43	9.07	1.15	0.92
16	T	#11	53	3.5	4125	38.6	0.41	9.64	6.98	1.38	
17	T	#11	40	10.5	5425	35.5	0.41	7.73	6.98	1.11	0.82
18	B	#11	40	10.5	5425	43.1	0.41	9.38	9.07	1.03	
19	B	#11	40	6.5	5050	43.7	0.41	9.86	9.07	1.09	0.85
20	T	#11	40	6.5	5050	37.[illegible]	0.41	8.42	6.98	1.21	
21	T	#9	53	7.0	5650	39.7	0	9.32	6.70	1.39	0.91
22	B	#9	53	7.0	5650	43.6	0	10.23	8.71	1.17	
23	B	#9	53	7.0	5700	54.2	0.41	12.67	9.12	1.39	0.70
24	T	#9	53	7.0	5700	37.7	0.41	8.81	7.02	1.25	
									Average	1.19	0.90
									Standard Deviation	0.14	0.08

Notes: (1) Includes no transverse reinforcement term. Calculated directly from measured steel stress

(3) K_{tr} term included.

$u_{cal} = u_c + u_{tr}$, casting position factor = 1.3

Therefore, the transverse reinforcement is fully effective in carrying shear and in resisting splitting along the splice. The entire area of transverse reinforcement can be considered in calculating shear capacity and splice lengths.

(3) Configuration of Transverse Reinforcement. The use of intermediate tie legs at each splice to provide the required area of transverse reinforcement improved the splice strength as compared to using only two legs as in a single perimeter hoop.

(4) Casting Position. The test results showed a decrease in splice strength for top splices with Z = 13.3 in. Top splices had average strengths of 90 percent (with a standard deviation of about 8 percent) of the bottom splice strength.

(5) Concrete Slump. Top splices performed more efficiently in concrete with lower slumps than in high slump concrete. Further research is urgently needed to evaluate the influence of high slump concrete produced with the use of HRWR additives on the bond strengths of top reinforcement.

(6) Splice Location. Shifting the splice a distance d away from the section of maximum moment did not improve the capacity of the splice. The load sustained was about the same as if the splice had been located at the critical section (maximum moment).

CHAPTER 3

DESIGN RECOMMENDATIONS

3.1 Casting Position

3.1.1 Current ACI and AASHTO Provisions. In the previous discussion of the test results, the effect of casting position was analyzed in terms of a reduction in bar stress or bond capacity and corresponds to code limitations on allowable bond stress. As mentioned in Chapter 1, current ACI and AASHTO codes define the anchorage and splice requirements in terms of development and splice lengths rather than bond stresses. The relationship between bond stresses and bar lengths is reciprocal. For "top bars" as defined in codes, a 30 percent reduction in allowable bond stresses ($u_{top} = 0.7u_{bottom}$) means a 40 percent increase in required development length ($\ell_d \approx 1/u \therefore 1/0.7 = 1.4$). To facilitate the incorporation of the present test results into design specifications, all test results will be discussed in terms of a casting position factor, $\ell_d/\ell_{d\ bottom\ bar}$, which is defined as a factor for multiplying the development or splice length of a "bottom bar" to obtain the anchorage length of a bar located at any height in the fresh concrete. According to this definition, the current codes [4,5] specify that the basic development length shall be multiplied by a casting position factor of 1.4 when more than 12 in. of concrete is cast below the bar.

Figure 3.1 shows the casting position factor as a function of bar or splice height for all the tests of the present investigation in which casting position was varied (excluding beam tests). The heavy dark line shows that the current ACI and AASHTO specifications are very conservative. It is clear that a single cut-off point at a height of 12 in. is irrational. Regardless of slump, there is a definite trend toward an increase in casting position factor with increasing depths of fresh concrete.

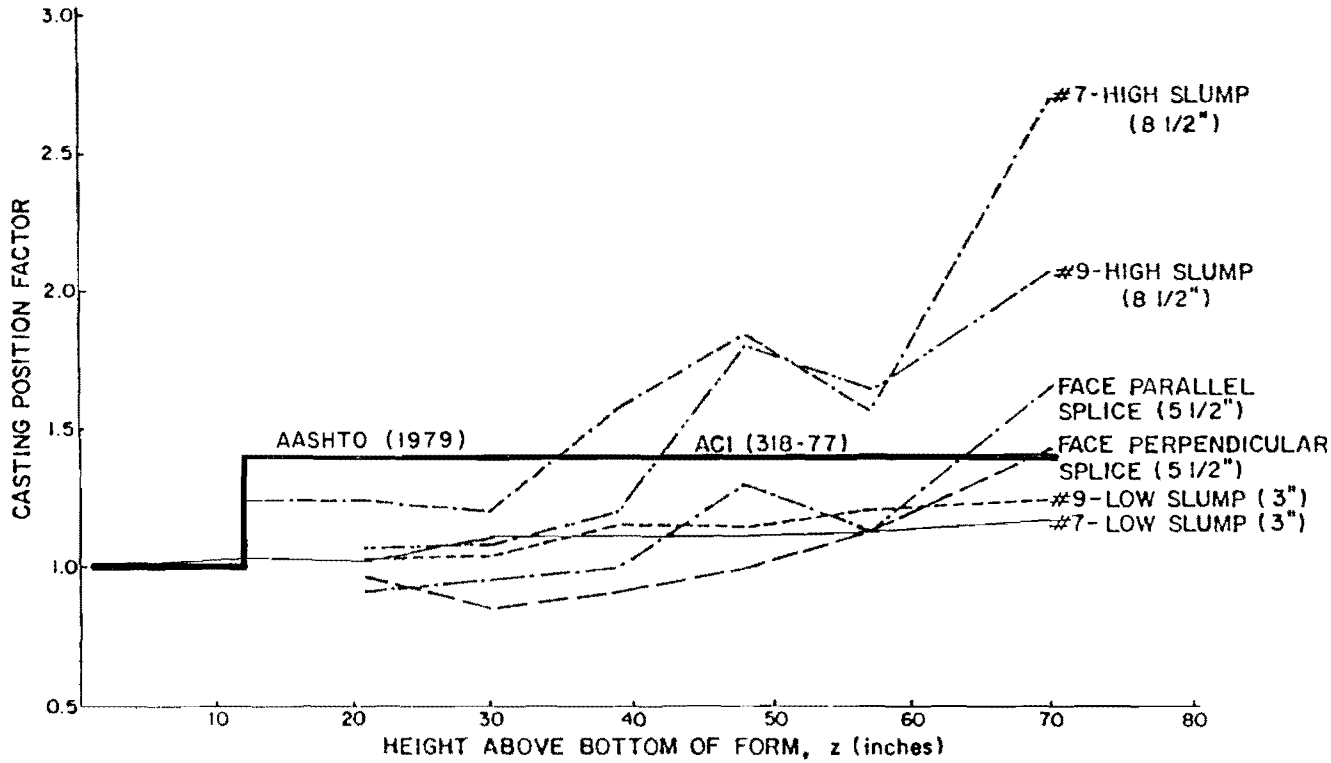


Fig. 3.1 Casting position factor vs bar height for all tests

3.1.2 Proposed Recommendations. The curves clearly show that concrete slump is a very important variable in determining the effects of casting position. This is especially true when very large depths of concrete are cast below the bars or splices. The recommendations presented here will be presented in terms of the several ranges of concrete slump investigated, < 4 in., 4 to 6 in., > 6 in. For design purposes, two approaches are possible. First, a linear function in terms of z permits the designer to calculate a casting position factor for all values of z. However, it may be preferable to have casting position factors in a tabular form. For this approach, the linear function can be approximated by a series of steps.

Figure 3.2 shows the test results from the low slump series of tests together with recommended values of casting position factor for slumps of less than 4 in. Also shown are the test results from a series of splice tests run by Ferguson [10] with a slump of 3 in. The recommended values of casting position factor for concrete slumps of less than 4 in. are:

$1.0 + 0.005z$ for $z \geq 12$ in.

or in steps

1.0	for $z < 12$ in.
1.1	for 12 in. $\geq z \leq 24$ in.
1.2	for 24 in. $\geq z \leq 48$ in.
1.3	for $z > 48$ in.

where z is defined as the depth of concrete cast below a horizontal bar.

Figure 3.3 shows the test results from the splice tests. The recommendations for casting position factor for concrete having a slump of between 4 in. and 6 in. are also shown. Test results for test splices having casting position factors less than 1.0 are omitted. The recommended values of casting position factor for this range of slump values are:

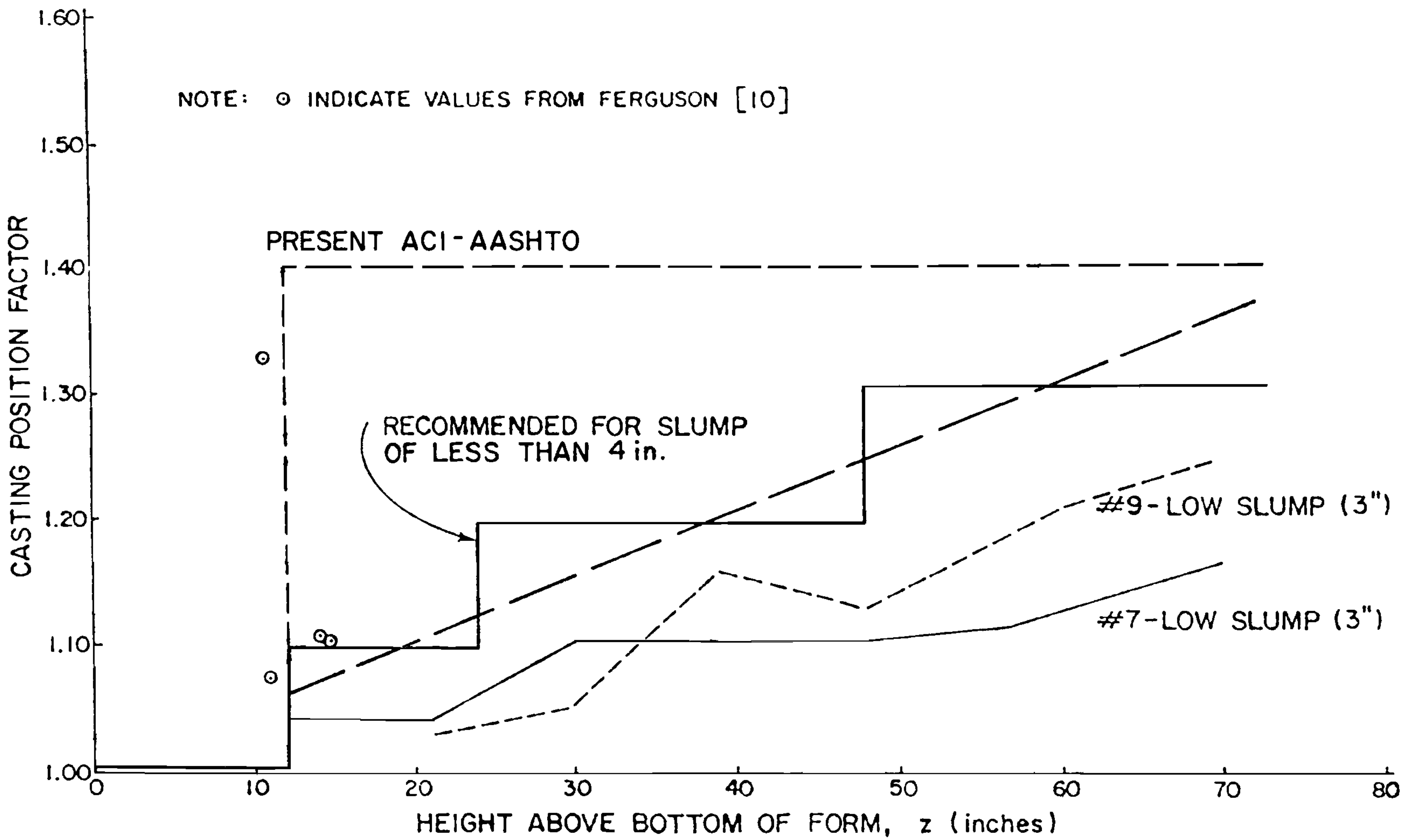


Fig. 3.2 Recommended casting position factor for concrete having a slump of less than 4 in.

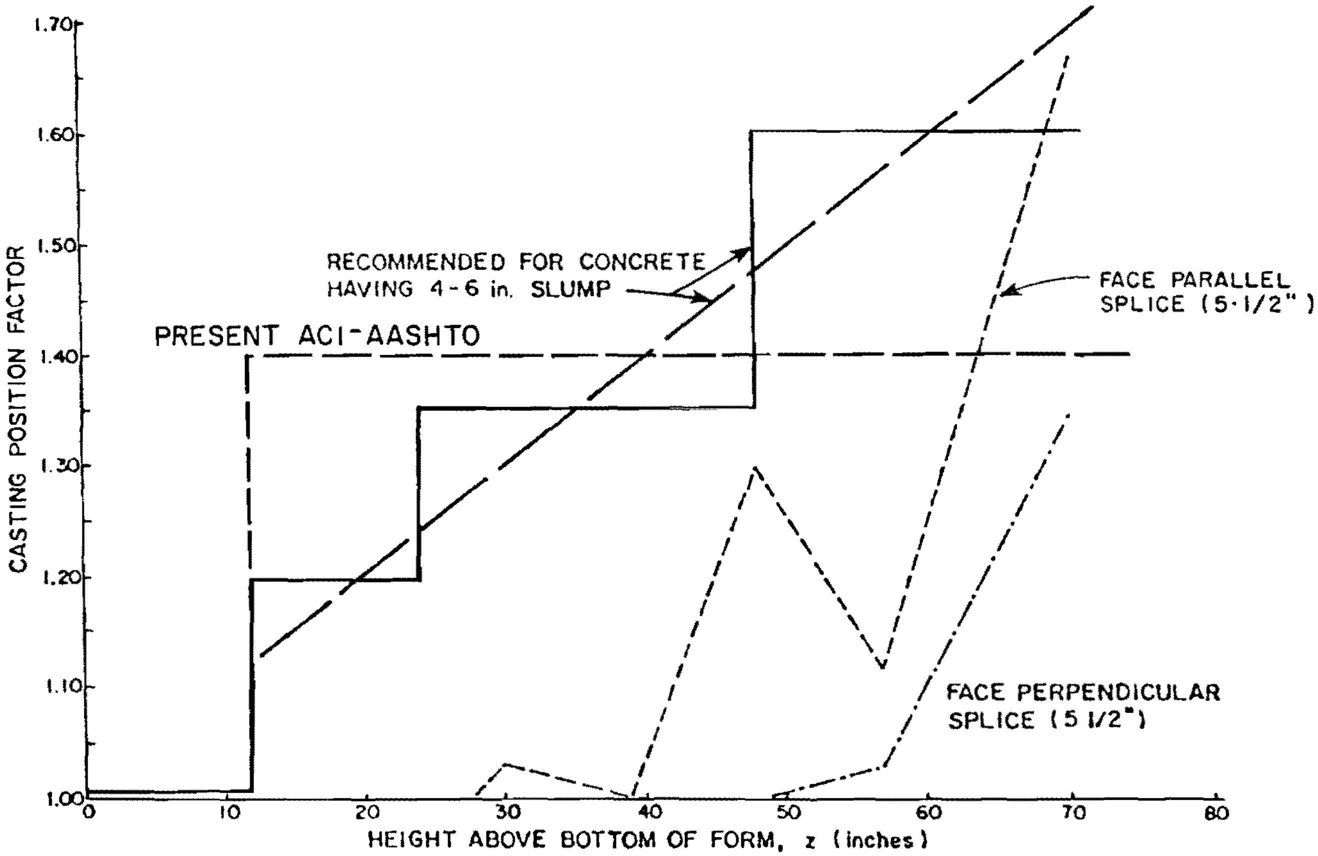


Fig. 3.3 Recommended casting position factor for concrete having a 4-6 in. slump

$1.0 + (0.01)z$ for $z > 12$ in.

or in steps

1.0	for $z < 12$ in.
1.1	for 12 in. $\geq z \leq 24$ in.
1.35	for 24 in. $\geq z \leq 48$ in.
1.6	for $z > 48$ in.

Test results for development length tests with a slump of greater than 6 in. are shown in Fig. 3.4. The recommended values of casting position factor for concretes with slumps in this range are:

$1.0 + (0.02)z$ for $z > 12$ in.

or in steps

1.0	for $z < 12$ in.
1.3	for 12 in. $\geq z \leq 24$ in.
1.8	for 24 in. $\geq z \leq 48$ in.
2.2	for $z > 48$ in.

As indicated in Ref. 1, the reduction in bond capacity of vertical bars is about half that of horizontal bars. However, the basic bond capacity of the vertical bars seems to be only about 75 percent of the horizontal bar capacity. Rather than trying to determine a value of z for a vertical bar and defining casting position factors, a single casting position factor of 1.3 is recommended for all vertical bars where the center of the splice or development length has more than 24 in. concrete cast below. The data are so limited that it was not felt prudent to make more specific recommendations. Obviously, some transition is necessary for bars oriented at angles other than horizontal or vertical. However, no data are available for evaluating bars in other orientations relative to the direction of concrete placement. It is also strongly recommended that more research be done in the area of the bond capacity of vertical or other inclined bars relative to that of horizontal bars.

The values of casting position factor versus bar or splice height for all ranges of slump together with the current ACI and AASHTO specifications are shown in Fig. 3.5. It is recommended that values for slumps of less than 4 in. be used in design only when the designer is

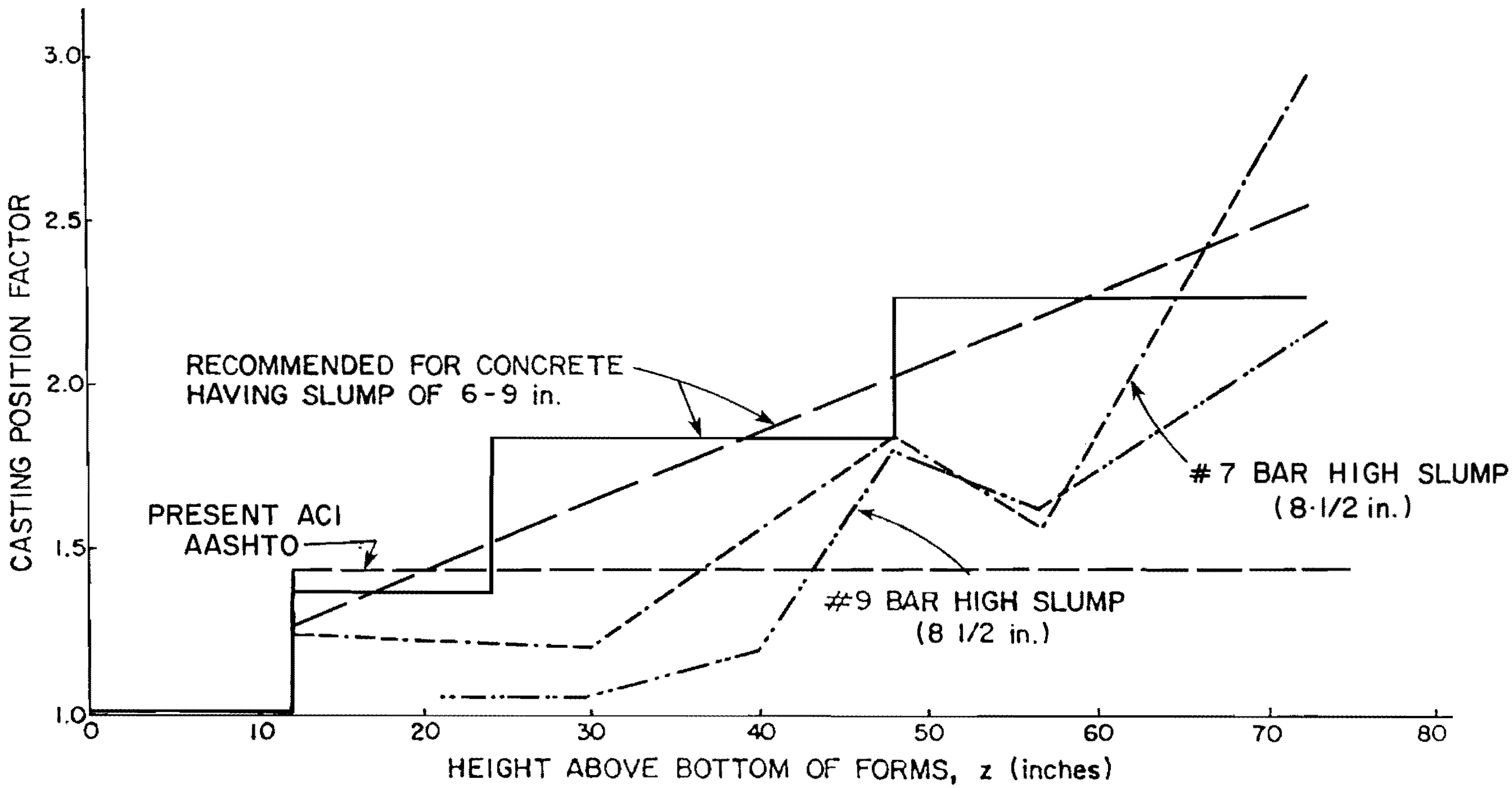


Fig. 3.4 Recommended casting position factor for concrete having a 6-9 in. slump

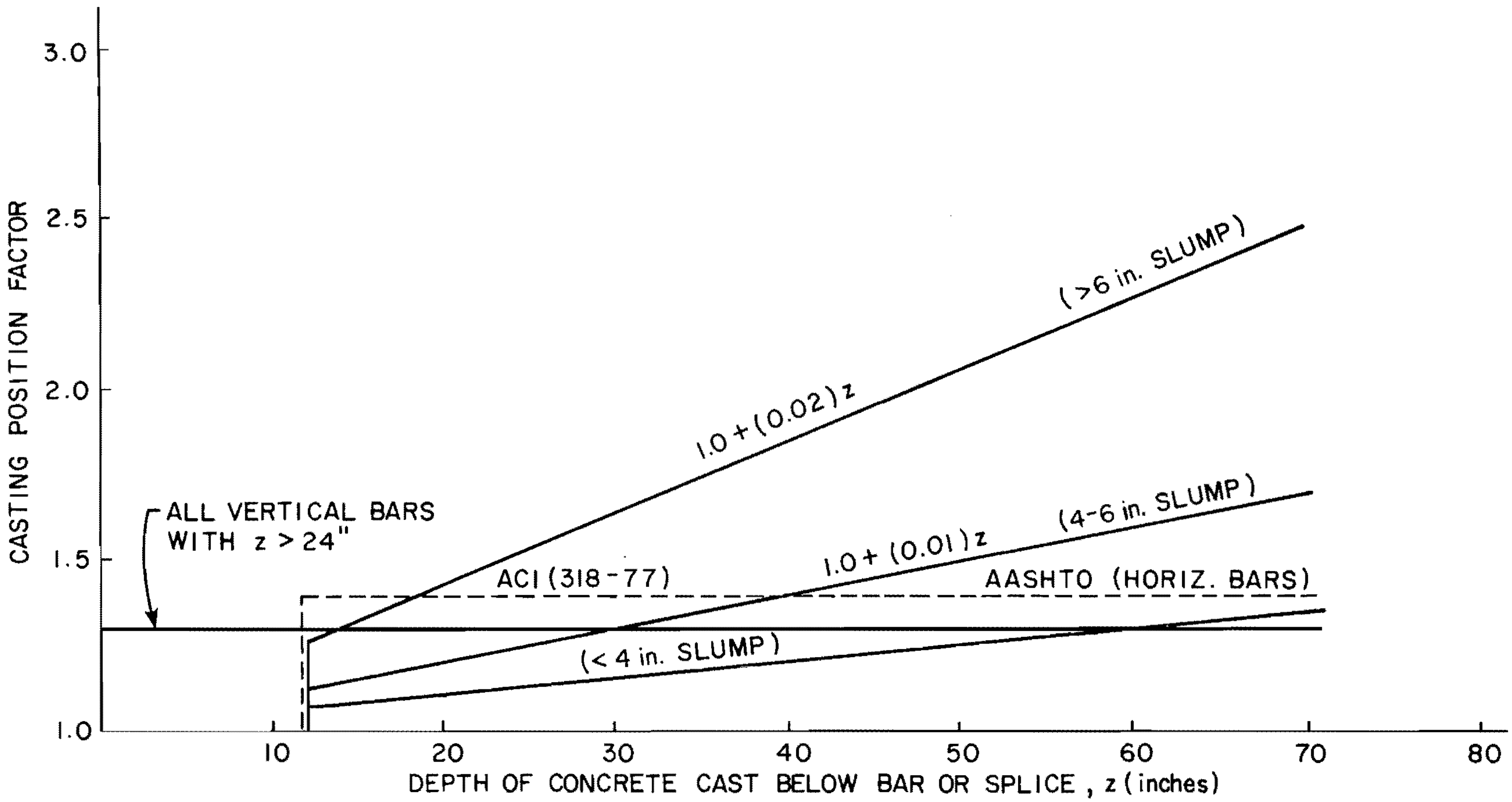


Fig. 3.5 Recommended casting position factors for all ranges of slump investigated

confident that the control over the concrete consistency in the field is sufficient to warrant its use. In cases where concrete slump is likely to be high, the upper curve for the range of slumps from 6 in. to 9 in. should be used. These recommendations apply to both anchored and spliced deformed bars.

3.1.3 Design Code Format. The following formats are proposed for inclusion in design specifications.

A. The basic development or splice length shall be multiplied by the following factors for

(1) Top horizontal reinforcement placed so that more than 12 in. of fresh concrete is cast in the member below the reinforcement.

Concrete with slump < 4 in.	$1 + 0.005\ z$
Concrete with slump 4 to 6 in.	$1 + 0.01\ z$
Concrete with slump > 6 in.	$1 + 0.02\ z$

where z is the depth of concrete cast below the bar.

(2) All vertical bars with more than 24 in. of fresh concrete cast below the center of the splice or development length 1.3.

B. The basic development or splice length shall be multiplied by the following factors:

	Z, in.*	Slump < 4 in.**	Slump 4 - 6 in.	Slump > 6 in.
Horizontal Bars	< 12 in.	1.0	1.0	1.0
	12-24 in.	1.1	1.2	1.3
	24-48 in.	1.2	1.35	1.8
	> 48 in.	1.3	1.6	2.2
Vertical Bars	< 24 in.	1.0	1.0	1.0
	> 24 in.	1.3	1.3	1.3

*Depth of fresh concrete (prior to initial set) cast below horizontal bars or the center of the splice or development length of vertical bar.

**Requires effective field control of concrete consistency.

3.2 Shear and Bond Interaction

The interaction between bond stresses and shear appears to be negligible. The performance of splices located in regions of shear was not influenced as shear was varied. Based on the tests performed in this program, there does not appear to be any reason to reduce bond stresses (or to increase splice or development length) in the presence of shear. It should be noted that as the shear increases, the moment gradient increases and the stresses along the bar change rapidly. Therefore, the stresses are at critical levels along a relatively short length of the anchored bar.

3.3 Transverse Reinforcement

The geometry of the transverse reinforcement did not significantly influence the strength of the spliced bars tested in this program. It appeared, however, that bars in wide sections perform somewhat better if multiple leg ties and stirrups are used. With multiple legs, more bars are contained by corners of the hoops and stirrups and can withstand larger member deformations prior to failure. The results indicated that the stirrups were effective in carrying shear and also resisting splitting of concrete around the anchored bars. This supports the use of the area of shear reinforcement in computing the confinement provided by transverse reinforcement for evaluation of development length as recommended in Ref. 9.

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