SHEAR STRENGTH OF BENT CAPS BETWEEN COLUMNS

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Shear Strength of Bent Caps Between Columns

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ABSTRACT

Shear and bond stresses permissible in bent caps inside the supports, but close to them, were studied, for comparison with earlier studies on short overhanging ends. Behavior was not quite the same, bond splitting tending to lower the shear strength slightly below the very high limits permissible on the overhanging ends. It also appears necessary to calculate bond stresses, but the permissible stresses recommended both in shear and bond are in excess of those commonly specified. Although the strength added by vertical stirrups at a/d of 0.5 was negligible, it was found safe and efficient to combine the recommended shear value of concrete with the full shear value of stirrups as ordinarily computed; the stirrups delayed splitting in those lengths where splitting tended to lower shear strength and where stirrups were less effective the shear strength without stirrups was already higher.

SHEAR STRENGTH OF BENT CAPS BETWEEN COLUMNS

INTRODUCTION

In 1963-1964 the project "Design Criteria for Overhanging Ends of Bent Caps--Bond and Shear" was conducted as Project 3-5-63-52 under the Cooperative Highway Research Program and reported as of August, 1964. A paper under the same title was presented before the Highway Research Board Meeting in January, 1966, and has been accepted for publication by HRB.

That report contained several conclusions applicable to the design of the overhanging ends. These can be summarized as follows:

1. The ordinary simple flexural formulas used generally in beams were accurate insofar as maximum steel stress was concerned, but steel stresses did not decrease toward the cantilever load at anything like the same rate as the moment decreased.

2. Within shear spans of 0.5d to 1.2d the following permissible (increased) shear stresses were found to be acceptable:

   (USD) $v_c = V/bd = 320 + 140\ d/a$ for $f'_c = 4500$ psi
   $v_c = 302 + 132\ d/a$ for $f'_c = 4000$ psi

   where <u>a</u> is the distance of the load from the support and <u>d</u> the effective depth of the member; or

   (WSD) $v_c = 142 + 62.5\ d/a$ for $f'_c = 4500$ psi
   $v_c = 134 + 59\ d/a$ for $f'_c = 4000$ psi
   $v_c = 116 + 51\ d/a$ for $f'_c = 3000$ psi

   End anchorage of bars beyond the load was required, but adequate extension was commonly available.

3. Vertical stirrups in the short <u>a</u> distances did not show as contributing any substantial element of strength.

4. When adequate end anchorage was provided beyond the load, bond failures did not occur and omission of bond calculations was recommended.

Objectives

The present project was designed to determine the extent to which the above conclusions on bond and shear capacity apply within the span between piers, near the supporting column or pier. Since the typical bent cap span is short relative to its depth, a distance of 1.2d from each pier includes the most critical design sections for shear and bond stress.

Scope of Investigation

It was possible, by taking certain precautions, to make a separate test on each end of a complete pier cap. Thus a total of 18 specimens were tested but only 9 complete bent caps were used. These included sections with vertical stirrups, with horizontal bars acting as horizontal stirrups, and with no web reinforcing at all in the critical lengths.

SPECIMENS AND TESTING PROCEDURE

The test specimens could be considered as full-size prototypes, 36-in. deep overall, of typical designs except that their width (thickness) was reduced to a slice 12.5-in. thick, or roughly 40 percent of the typical bent cap width. The tension steel used was two #11 bars in each case, one-third that of a typical bent cap, but the steel was made A-432 grade (nominal f_y of 60 ksi) instead of the intermediate grade usually used in Texas. The area and yield point of the steel were such as to cause failure in shear rather than in flexure, and this occurred in all cases except specimen 1b-V. There was no reason to question or to investigate the flexural strength of these sections.

A typical pair of specimens is shown in Fig. 1a and Fig. 1b, with no stirrups inside the left column and vertical stirrups inside the right column. In three specimens horizontal bars on each face through the shear zone acted as horizontal stirrups, as shown in Fig. 1c. In two specimens small spirals were placed over the individual tension bars, as indicated in Fig. 1d and discussed later.

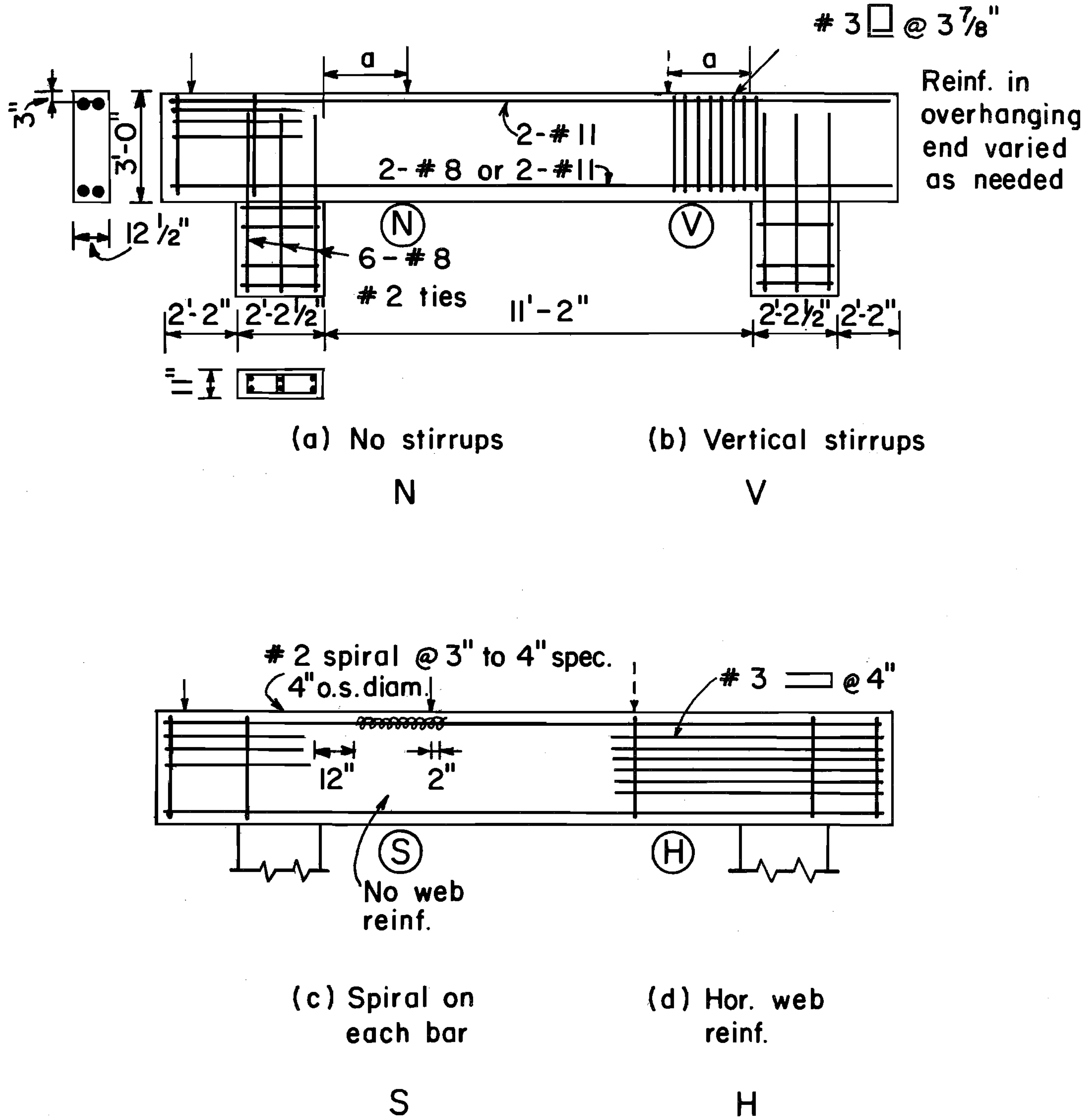

Fig. 1. Types of specimens.

The specimens were cast from transit mixed concrete made of high early-strength cement in the proportions indicated in the Appendix. They were cast in wooden forms, in the normal vertical position, cured under a plastic covering for approximately six days, and then taken out for preparation for testing the day before the test, usually on the seventh day. However, because of testing problems which developed, some specimens were tested as late as 11 days of age.

The specimen notation used in the earlier report can be simplified here because the beam section and A_s are constant for all specimens. The notation used is the following:

First - a serial number and letter, as 1a, 1b, 2a . . .

Second - the pattern of web reinforcing (Fig. 1).

Thus, 4b-V indicates the 4th casting, the second end tested, with vertical stirrups present.

Test Procedure

The specimens were tested on their side, resting on rollers made of pipe sections. They were loaded by the yoke assembly shown in Fig. 2. Since a balanced cantilever system did not quite simulate the actual loading, the inside load was made large enough to cause a positive reaction at the far end, as indicated in Fig. 3. This reaction was weighed by a load cell, thus definitely establishing the shear in the critical shear span.

For a second test the specimen was reversed end for end and, if the shear damage at the first tested support made it necessary, the reaction at the far end was moved into the undamaged beam section. The damage was always right at the column and only a short shift of the outer reaction was ever required. The cantilever overhang served only as a loading mechanism to create the negative moment desired over the support.

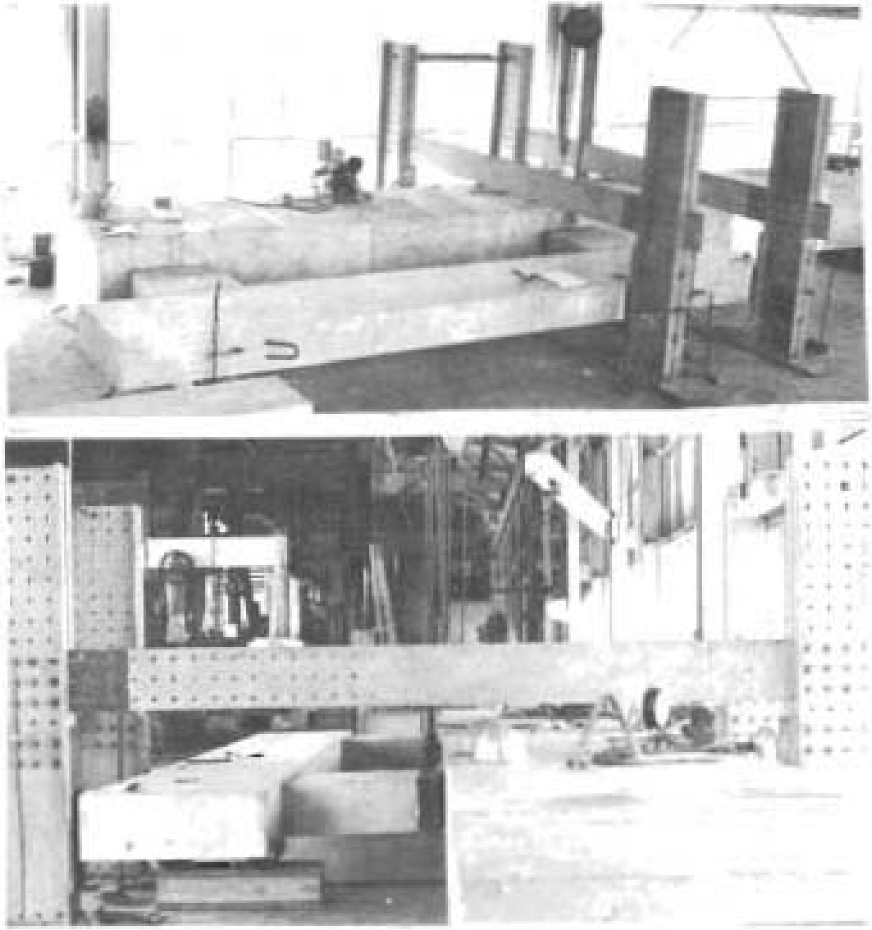

Fig. 2. Specimen in testing.

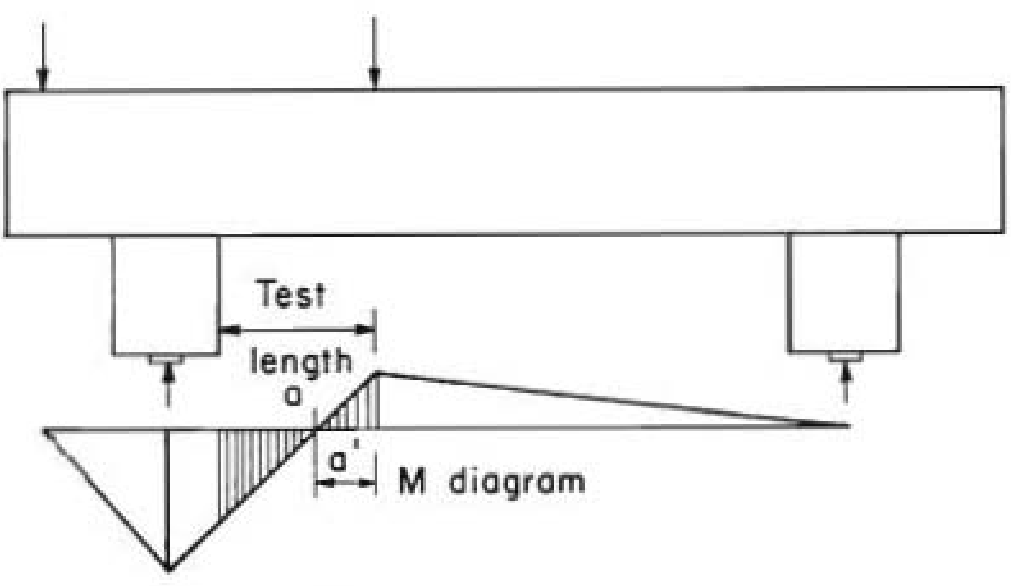

Fig. 3. Moment pattern used in test.

OBSERVATIONS AND ANALYSIS

Data

The ultimate loads, exact measured dimensions, properties, and calculated stresses are tabulated in Table 1. Nominal stresses were calculated from the basic equations for members of uniform section:

$$v = V/bd$$
$$u = V/(\Sigma o\ 0.9d)$$
$$f_s = M/(A_s\ 0.9d)$$

Concretes were kept somewhat lower in strength than for the previous series. The calculated shear and bond stresses recorded in Table 1 have been adjusted to values for f'_c = 4000 psi by multiplying actual calculated v and u values by $\sqrt{4000/f'_c}$, these adjusted values being noted as v_4 and u_4.*

It might be noted that only specimens 1b-V and 8b-V, beams with vertical stirrups and loaded at relatively large distances a, reached the nominal yield strength of the steel used (f_y = 60.7 ksi). Test 1b-V was stopped prior to shear failure, but after the steel had started to yield (crack width 0.04 in.), because questions arose as to the stability of the loading frame.

Type of Failure

Except for specimen 1b-V, every specimen failed by shear along a line from load to face of the supporting pier. The failure line (Fig. 4) was remarkably straight and in 12 specimens could be more accurately described as on the diagonal joining load and support rather than either slightly inside that line (2 cases) or slightly outside it (3 cases).

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*The earlier report data have been readjusted to the same base for all comparisons.

Table 1. Properties and Data

Spec.	Properties: f'_c psi	b in.	d in.	a in.	a/d	a' in.	a - a' in.	M/Vd	Test Results: V kips	v psi	v_4 psi	f_s ksi	u_4 psi
1a-N	4700	13.46	33.35	40.75	1.220	17.55	23.20	0.692	201	448	414	51.0	700
1b-V	4700	13.38	33.36	40.75	1.220	11.00	29.75	0.890	200*	(449)	(415)	64.4	(695)
2a-N	4220	13.35	32.85	16.75	0.510	5.10	11.65	0.355	336	765	745	42.2	1247
2b-H	4220	12.71	33.75	16.75	0.495	7.75	9.00	0.266	373	869	845	30.5	1348
3a-N	4460	13.01	33.49	16.75	0.500	5.10	11.65	0.355	320	750	710	38.7	1156
3b-V	4460	12.60	32.86	16.75	0.510	4.68	12.07	0.367	293	707	670	38.3	1060
4a-N	3050	12.57	32.82	34.00	1.030	11.90	22.10	0.663	127	303	347	30.3	546
4b-H	3080	12.53	33.36	34.00	1.018	13.60	20.40	0.605	146	345	393	32.6	618
5a-N	3470	12.44	33.75	28.00	0.830	7.98	20.02	0.595	148	355	381	32.1	593
5b-V	3470	12.42	33.56	28.00	0.835	8.12	19.88	0.593	242	581	624	52.2	975
6a-N	3830	12.52	32.92	40.75	1.230	17.20	23.55	0.713	170	412	421	45.6	660
6b-V	3830	12.67	32.48	40.75	1.250	17.20	23.55	0.725	224	545	555	61.6	880
7a-N	3640	12.57	33.49	22.75	0.680	7.06	15.69	0.468	213	509	534	36.1	834
7b-H	3570	12.49	33.37	22.75	0.683	6.73	16.02	0.480	243	555	587	40.0	920
8a-S	3600	12.54	32.33	34.00	1.050	11.90	22.10	0.684	179	441	465	42.6	760
8b-V	3600	12.55	32.65	34.00	1.044	5.93	28.07	0.860	231	564	594	70.0	933
9a-S	3510	12.56	33.19	28.00	0.845	7.97	20.03	0.602	181	436	465	39.2	730
9b-N	3510	12.54	33.87	28.00	0.828	1.77	26.23	0.775	182	429	457	49.5	717

*No failure in shear.

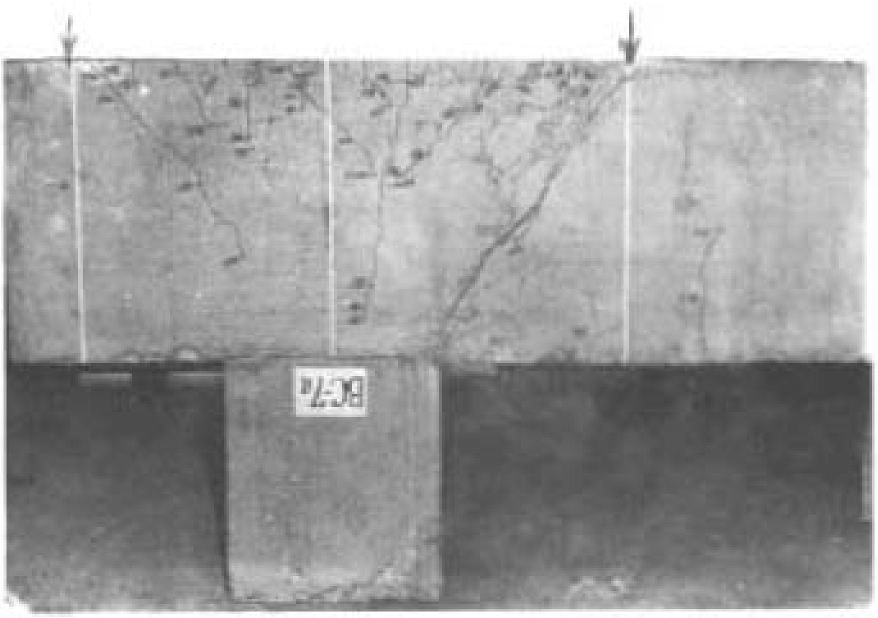

Fig. 4. Typical failure line between face of support and load.

In the cantilever ends, the failure pattern had been more diverse. Although the failure line from load to reaction was also common there where short anchorages led to bond failures, failure in many of the shear failure cases was on a line arcing considerably above such a diagonal and often tending to reach the top just at the inner face of the load bearing plate. With the longer end anchorages the failure tended to be associated with flexure cracks which were somewhat radial from the support. In the present interior tests, true flexural cracks from negative moment did not approach as close to the load as in the cantilever ends, because the moment reversed at the point of inflection between load and reaction.

The full significance of the straight line shear failure in the interior sections is not established. In members without stirrups, the failure tended to be sudden and essentially complete.

In four beams loaded at approximately 0.5d from the support there was no sign of significant bond complication. In all others except 1b-V (which was not carried to failure) and 4b-H, there was considerable evidence prior to shear failure of bond splitting along the top steel inside the load (Fig. 5). The splitting started from a moment crack and progressed across the point of inflection well towards the load before the shear failure occurred. In the case of specimens 4a-N and 5a-N the bond complication was considered the probable explanation of the lower shear strengths attained, as discussed later.

In the following beams positive moment flexural cracks developed closer to the support than the point of zero moment:

2b-H	6a-N
3a-N	7a-N
4b-H	7b-H
5a-N	8b-V
5b-V	9a-S

These cracks were of diminishing height as they approached the support. Since the bond splitting cracks in the top indicated tension in the bars nearly all the way to the load, it is obvious that in these beams tension existed in both top and bottom steel over some common length. The concrete

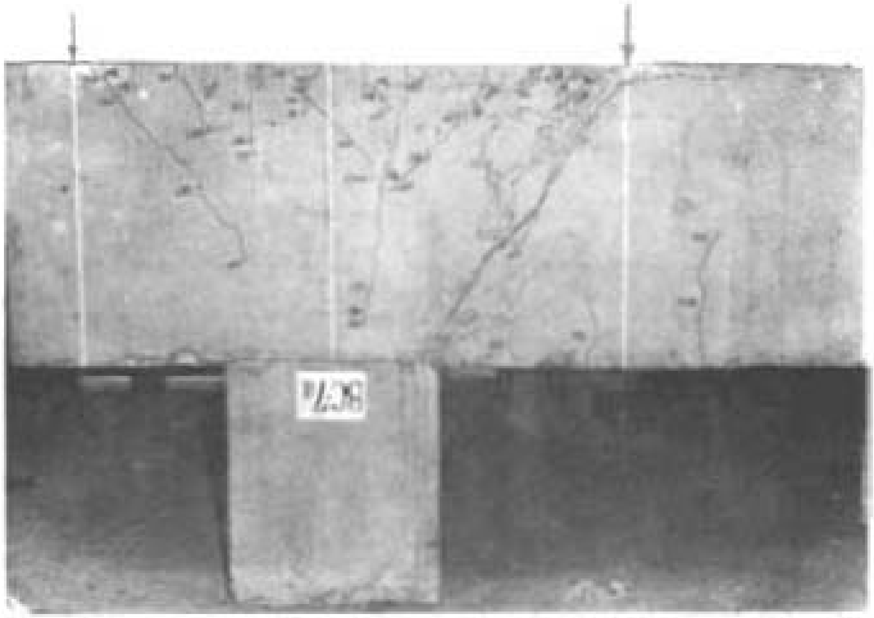

Fig. 5. Bond splitting between moment crack and load (which is beyond point of inflection).

between load and reaction point apparently acted as a diagonal compression strut with its horizontal component resisted by both top and bottom steel in proportions which varied from section to section. This emphasizes that the stresses in this short length between load and support differ in a major way from simple flexural stresses and are more like those existing in a simple cantilever, but with some similarities toward stresses in simple-span deep beams. The shear failure stress appeared almost as a tensile splitting stress along the axis of a heavily loaded diagonal compression strut.

Shear Strength

a. Without any web-reinforcement

The shear strengths, adjusted to $f'_c = 4000$ psi, are plotted in Fig. 6 against the ratio of a/d, where a is the distance from face of support to the load and d is the depth. Also plotted is the recommended ultimate shear established for the earlier cantilever end tests. In general the agreement is reasonably good, but there is some tendency for points to fall lower in the range of a/d from 0.8 to 1.1 and higher in the range 0.5 to 0.7, especially at 0.5. If all points are accepted as valid, it seems prudent to evaluate shear from a/d = 0.5 to 1.25 at

$$\text{(USD)} \qquad v_c = 200 + 150\ d/a \qquad \text{for } f'_c = 4000 \text{ psi}$$
$$= (3.16 + 2.37\ d/a)\sqrt{f'_c}$$

even though this penalizes the designer at a/d = 0.5 by some 25 percent. Actually specimens 4a-N and 5a-N experienced some bond weakness and could possibly be ignored as non-shear failures. However, on this limited number of tests, the more conservative approach is recommended, limiting both bond and shear in such cases unless stirrups are used.

b. With vertical stirrups

At a/d of 0.5 the vertical stirrups of specimen 3b-V seem to have done absolutely no good when compared to 3a-N without stirrups. However, at a/d of 0.83 it appears that stirrups contributed almost their full

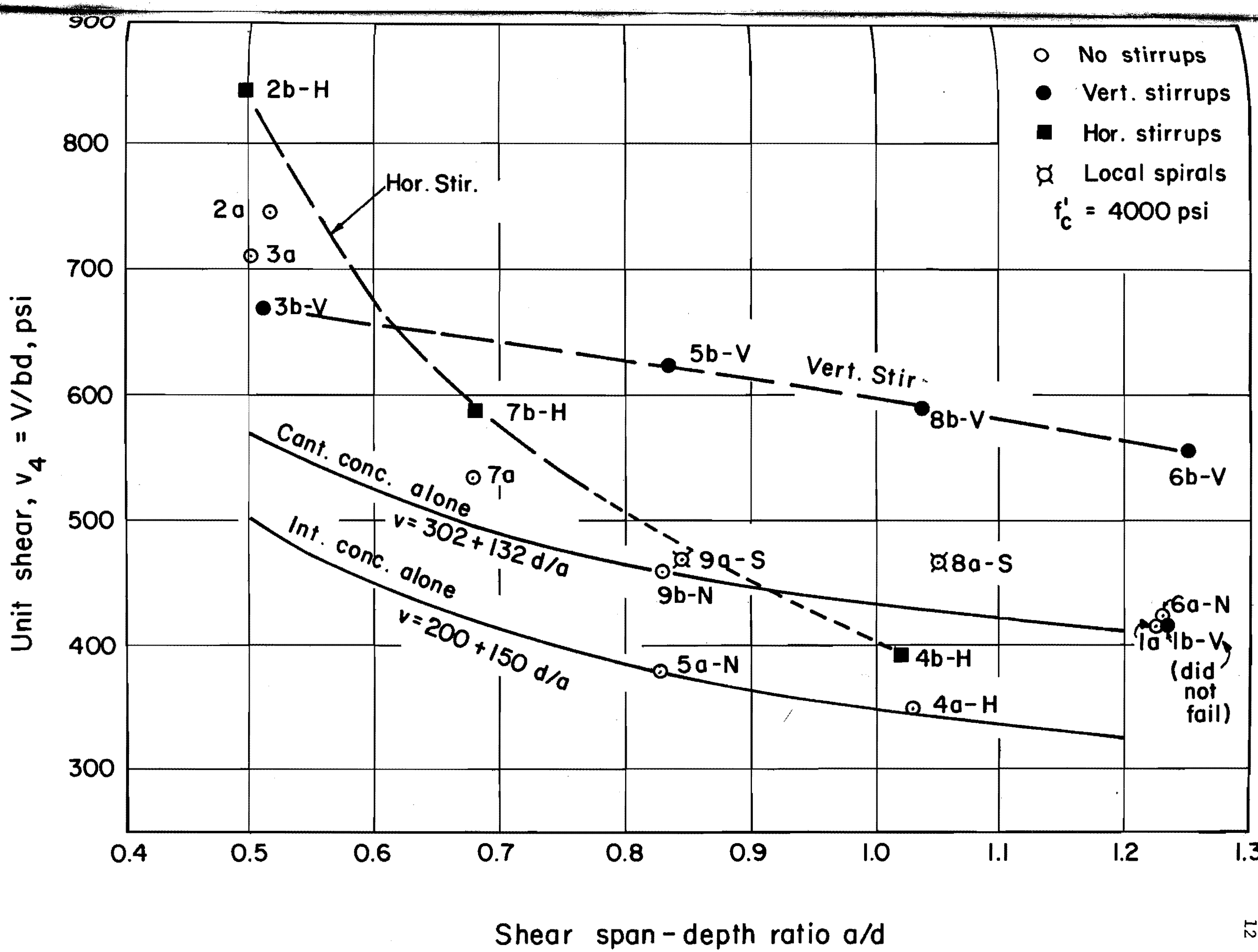

Fig. 6. Shear resistance in terms of a/d.

rf_y = 0.00458 x 40 = 183 psi based on the nominal f_y. Actually the adjusted strength of specimen 5b-V (from its f'_c of 3470 psi to 4000 psi) by the square root ratio method is not quite proper as plotted because the strength contributed by stirrups is not a function of f'_c. Without adjustment, the stirrups appear to have contributed 581 - 355 = 226 psi, but the value of specimen 5a-N used appears to be low because of bond complications. It might be better to compare 5b-V with 9b-N, which would show a stirrup contribution of 581 - 429 = 152 psi, slightly less than the nominal stirrup value.

A comparison at a/d of 1.05 shows nearly the same result except that there is no second beam without stirrups to offset the low value of 4a-N, which also seemed to be caused by bond complications.

At a/d of 1.23 the comparison of 6b-V with 6a-N shows stirrups adding 545 - 412 = 133 psi compared to rf_y of 183 psi.

It seems that vertical stirrups do little good at a/d of 0.5 and probably are only some 75 percent effective in the a/d range from 0.8 to 1.23. As a design expedient, however, it is recommended that the use of the low recommended v_c equation be offset by counting the stirrups at full value throughout. The trend of strengths with vertical stirrups is quite steady, possibly because the stirrups delay the bond splitting.

c. With horizontal web reinforcement

At a/d of 0.5 the horizontal stirrups added 96 psi (at f'_c of 4220 psi). At a/d of 0.68 the increase is only about 46 psi (at f'_c of approximately 4600 psi), although this test and that without stirrups are both high compared to the recommended v_c. At a/d of 1.02 the horizontal stirrups appear to be of no value; in fact, the total is so low as to raise questions of how horizontal web steel could possibly lower strength. Without a satisfactory answer the value must be attributed to test scatter.

d. The use of a/d in preference to M/Vd

In a simple span or cantilever, under concentrated loads, M/Vd and a/d are the same numerically. For uniform load, M/Vd is usually considered a more meaningful designation, one more suitable to the varying shear. In these interior parts of the bent cap, M/V would be the distance from support to the point of inflection, that is, the distance a minus a', both as tabulated in Table 1.

In this report M/Vd has not been used for two reasons. One is that the failure did not appear related physically to the point of inflection, which varied considerably from the load point in most cases. The failure line was always from face of pier at the bottom to the load regardless of how the distance to the point of inflection from pier (a-a') varied. The second reason is that when the same data were replotted to M/Vd as abscissa, as in Fig. 7, the data appeared less organized. Values with vertical stirrups also appear more irregular in this plot. If M/Vd were simply substituted for a/d in the expression for shear in the concrete, as used for the overhanging ends, it would be manifestly unreasonable and even the value just recommended here would require serious readjustment for M/Vd values in the neighborhood of 0.6 or 0.7. For these data a/d seems to be a more suitable variable than M/Vd. Those accustomed to using M/Vd should note that M/Vd should not be used in the relations presented herein.

Bond Stress

Since no beam failed in bond stress, the bond stresses tabulated in Table 1 can be taken only as proof loads. It should also be noted that these are artificial values when horizontal web steel is used, since no allowance has been made either in the calculated f_s or calculated u for the assistance rendered by the horizontal stirrups. Since the ratio of Σo to b was constant, Fig. 8 is actually a reproduction of Fig. 6 to a slightly different scale.

It was noted earlier that many of the beams showed splitting related to bond stress. However, the highest bond stress, u_4 of 1060 to 1348 psi, occurred when the load was about d/2 from the support; and these beams

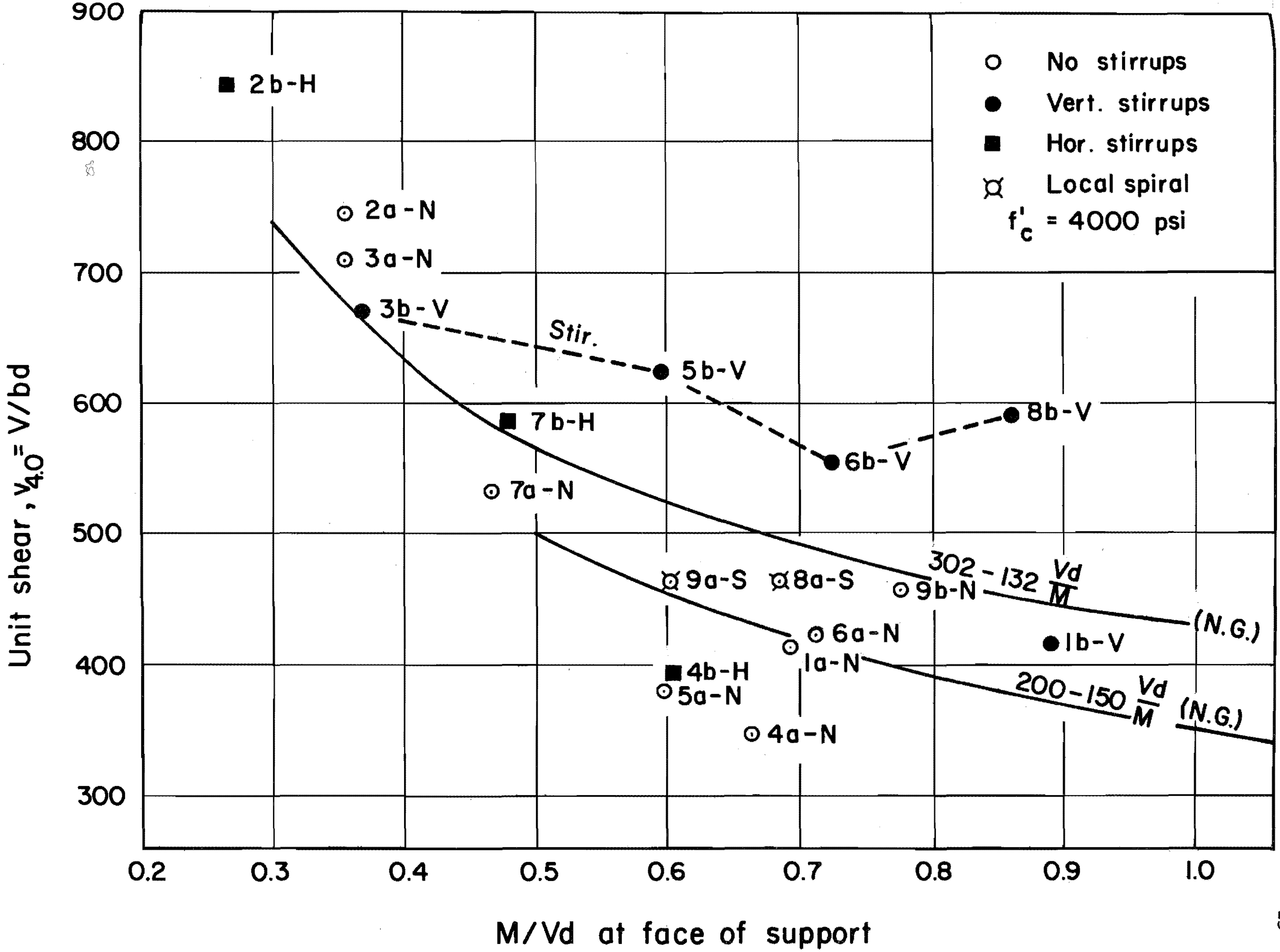

Fig. 7. Shear resistance in terms of M/Vd is not as usable as Fig. 6.

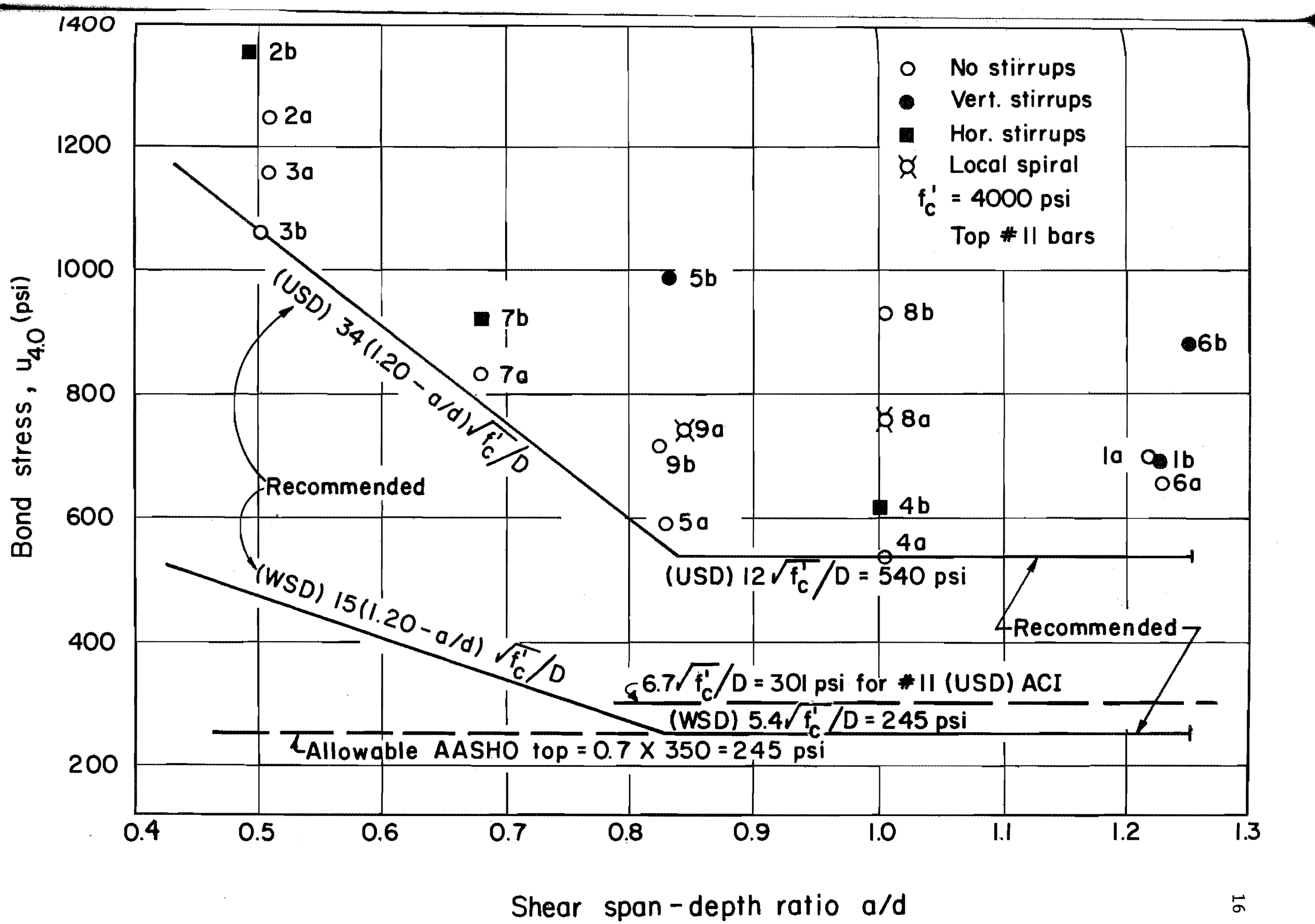

Fig. 8. Bond stress in terms of a/d.

showed almost no bond distress. The bond splitting seems to lower the shear strength of specimens 4a-N and 5a-N by 25 and 17 percent, respectively, below values found for the cantilevers. The corresponding bond stresses were u_4 of 546 and 593 psi, the lowest of all values observed. These were not bond failures but shear failures, with the end of the bar remaining tightly embedded beyond the load, although bond distress seems to have played a part in the shear failure. The recommended shear stresses already reflect this weakness.

The significance of this progressive bond splitting needs consideration. It appears that when the load is at the point of zero moment, as in a cantilever end, the load itself helps to stop the splitting by the vertical compression it creates. In the present case the splitting could continue past the point of inflection, and it did so. Although it was always slowed as it approached the load, apparently its damage was already done in specimens 4a-N and 5a-N and diagonal tension failure appeared somewhat prematurely. To establish that bond splitting was the basic problem, companion beams 8a-S and 9a-S were made, different only in that a small spiral* was placed around the individual bars (Fig. 1c). Splitting also developed in these beams, but more slowly, and shear strength was improved and not sub-normal.

It should be noted that bars in specimens 4a-N and 5a-N (just discussed) furnished much more than a computed development length since the bars were continuous to the far end of the cap. Still the local splitting led to shear weakness even though the extended bar remained firmly embedded. If the bent had been designed with #14S or #18S, one suspects that splitting would have developed earlier and shear strength would have been further decreased.

In a previous series of bond tests** end anchorage beyond a point of inflection had shown itself some 10 percent less effective than a corresponding length within the point of inflection. This and the lowered strengths just mentioned indicate that, in spite of the high bond stresses recommended below, the total neglect of bond stress calculations (as already recommended***

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* #2 at about 3" spacing, 4" outside diameter.

** Ferguson, Phil M., and Thompson, J. Neils, "Development Length of High Strength Reinforcing Bars," ACI Journal, Jan. 1965, pp. 72-91.

***Ferguson, Phil M., "Design Criteria for Overhanging Ends of Bent Caps--Bond and Shear," Report to Center for Highway Research, August, 1964.

for cantilever ends) would not be appropriate here. Instead, the 546 psi value of u_4 for specimen 4a-N will be taken as an upper stress limit.

Since all the main tensile steel was #11, the minimum bond stress of 546 psi is equivalent to $12.2\sqrt{f'_c}/D$, which is 182 percent of the ACI (USD) permissible stress of $6.7\sqrt{f'_c}/D$ for top bars and 222 percent of the (WSD) AASHO specification value of 0.7 of 350 psi. (Since these beams had 32" of concrete cast below the bars, with concrete of 2" to 3" slump, the bars clearly classify as top bars.) With a safety factor of 2.25 for WSD, these tests justify the following recommended bond stresses for top bars, where a is the distance to the primary load:

USD	$u = 34(1.20 - a/d)\sqrt{f'_c}/D$	for $a/d < 0.84$
	$u = 12\sqrt{f'_c}/D$	for $0.84 < a/d < 1.25$
WSD	$u = 15(1.20 - a/d)\sqrt{f'_c}/D$	for $a/d < 0.84$
	$u = 5.4\sqrt{f'_c}/D$	for $0.84 < a/d < 1.25$

The last value is identical with the present AASHO specification of $0.7 \times 350 = 245$ psi for $f'_c = 4000$ psi (only).

These USD values are considerably higher than the ACI allowable. Nevertheless, they are certainly justified by these proof values and further investigation might permit still higher limits.

These WSD values are also much higher than the AASHO specification for the smaller a/d values, but the last value, for $a/d > 0.84$, does not represent any increase at $f'_c = 4000$ psi and represents a decrease at lower f'_c strengths. This anomaly exists because the AASHO specification has not been correlated with recent bond research information and generally gives a reduced factor of safety with large bars. The bond stresses recommended for a/d less than 1.25 are actually some 60 percent above those which can be recommended for general use, that is, for larger a/d values.

For primary loads farther than 1.2d from support no new data are available and no changes from present practice can be recommended. If the data gaps were filled, it is probable a smoother transition and more favorable bond values could be established to an a/d of 2.0 or more.

CONCLUSIONS

When loads are applied to the top of the bent cap and the reaction is below, as in these specimens, the behavior in bond and shear resistance near the support is somewhat similar to that in the cantilever end, but not identical. The conclusions apply to gravel concrete and have not been validated for lightweight aggregate concrete.

1. The shear failure was more uniformly along a direct line from load to face of support.

2. Bond splitting was no problem when a/d was 0.5, but at larger a/d values the splitting progressed beyond the point of inflection to the load and in two cases (without stirrups) seemed to reduce the shear strength.

3. The shear strength without stirrups can be safely taken for $f'_c = 4000$ psi as

$$v_c = 200 + 150\ d/a \qquad \text{for } 0.5 < a/d < 1.25.$$

   For other concrete v_c can be varied as the square root of f'_c:

$$v_c = (3.16 + 2.37\ d/a)\sqrt{f'_c} \qquad \text{for } 0.5 < a/d < 1.25.$$

4. Stirrups are less effective on most specimens than normally computed, and at a/d = 0.5 are of little value. However, since shear capacity at a/d = 0.5 is quite high and since elsewhere bond splitting does less damage when stirrups are present, it is safe to use the shear in item 3 increased by the full (nominal) value of vertical stirrups.

5. Horizontal stirrups were more efficient at a/d = 0.5, but their value dropped rapidly at larger a/d values. They arenot recommended because of their limited range and the scarcity of detailed test information on them.

6. It appears that a/d is a more meaningful variable than M/Vd in these tests.

7. Bond stresses must be calculated but increased bond stresses on top bars are recommended as shown in Fig. 8 for a/d less than 1.25, the range investigated. No recommendations can be made for a/d more than 1.25 because the larger values have not been included in this investigation. (It is noted that the AASHO allowable bond stresses are not in line with recent research and the recommendations are actually lower than some of the AASHO allowables.)

APPENDIX

CONCRETE MIX

<u>Quantities per cubic yard</u>

High-early strength cement	5 sacks
Puzzolith	5 quarts
Darex (air entraining)	2 oz
Coarse aggregate, gravel	2130 lbs
Fine aggregate	1150 lbs
Water	21.4 gal