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COMPOSITE SYSTEMS WITHOUT ROUGHNESS

CTA 35

SYNOPSIS

COMPOSITE SYSTEMS WITHOUT ROUGHNESS

Arguments are assembled in support of the concept of composite systems of tees and hollow-core slabs without intentional roughness as required by section 17.5.4 of the ACI Code.

Previously published research on composite systems of narrow beams with cast-on slabs indicates that the strength of concrete-to-concrete bond at the interface varies from 140 - 500 psi, depending on the degree of surface roughness.

An analysis of the final report of ACI-ASCE Committee 333 (attached here as Appendix B) indicates that the shear stresses presently allowed by the Code may be based on misinterpretation of earlier data.

CTA tests of 16 composite prestressed beams with 2 1/2" topping demonstrate that full ultimate moment can safely be developed at horizontal shear stresses of up to 150 and 350 psi by composite beams with shear span/effective depth ratios of 3.5 and 7.7 respectively. This was found to hold true even for intentionally smooth contact surfaces and cases where the surface was intentionally dirtied before casting the topping.

An investigation of horizontal shear stresses in composite double-tee and hollow-core members supports the idea of constructing such systems using a conventional float or vibrating screed finish. The minimum factors of safety against horizontal shear failure would be approximately 10 for double tees and 4 for hollow-core slabs.

INTRODUCTION.

Composite concrete flexural members are defined in Section 17.1.1 of the ACI Code as "concrete elements constructed in separate placements but so interconnected that the elements respond to loads as a unit." Precast beams and slabs often have topping cast on them to cover utility ducts, promote uniform camber, allow for heavy concentrated loads and tie members together for resisting seismic loads. To the extent that composite action is achieved, this topping is beneficial in reducing live-load deflection and increasing the ultimate flexural strength of the member. On the other hand, when composite action is lost, the topping is simply additional dead load.

The effectiveness of composite flexural members is determined primarily by their ability to transmit horizontal shear across the interface between the two elements. In design, this shear is treated in accordance with the requirements of Section 17.5.4 of the ACI Code, which specifies roughening of the contact surfaces, provision of steel ties, or both. This section of the code is based on investigations of composite tee beams formed by casting slabs on beams. Since the interface in such members is close to the neutral axis and the contact surface is narrow relative to the flange width, horizontal shear stresses across the interface are large and the method of connection is of critical importance. In hollow-core, single-tee and double-tee members with topping, the interface is a considerable distance above the neutral axis and the contact surface is wide, thus the horizontal shear stresses are rarely enough to necessitates ties. The intentional roughness required by the Code, however, can be costly in production and is subject to arbitrary rejection. It is, therefore, of practical interest to investigate the relationship between bond strength of untied joints and the roughness of the interface.

The purpose of this report is to examine the concept of composite systems without roughness, particular attention being directed toward the performance of such systems under service and cracking loads and the ability of composite beams to achieve the ultimate flexural capacity of a monolithic beam of identical properties.

PREVIOUS RESEARCH.

1. Revesz (1952).

Tests at the Imperial College of Science and Technology in London (ref. 1) involved 5 prestressed and reinforced composite tee beams without ties. Each of the beams exceeded its calculated ultimate flexural strength, including one beam which failed in horizontal shear at a stress of 134 psi ($\frac{VQ}{Ib}$ = 143 psi). This beam had a smooth contact surface joining a 21 1/2" flange to a 3" web. Load at the time of failure exceeded calculated ultimate by more than 12%. Four other specimens failed in flexure at an average of 228 psi horizontal shear. The author concluded that it is desirable to roughen the contact surfaces of the precast web and cast-in-place concrete of composite beams.

2. Ozell and Cochran (1956).

Tests of prestressed composite lintel beams at the University of Florida (ref. 2) included 9 specimens of varying depths, none of which employed ties across the interface between the bare lintel beam and the core. In the authors' words, "the surface of the bare lintel beam in contact with the core was extremely smooth." Yet none of the specimens failed due to horizontal shear or loss of composite action, shear flexure being the mode of failure in every case. The authors concluded that "natural bond between the bare beam and cast-in-place portion provides reliable resistance to horizontal shear."

3. Hanson (1960).

Tests of composite beams conducted by the Portland Cement Association (ref. 3) included one specimen with no ties crossing the interface. This beam failed at a calculated horizontal shear stress of 350 psi, under a mechanism described as shear compression preceded by rather substantial loss of composite action. The surface was described as intentionally roughened, the final finish having depressions and peaks approximately 3/8" below and above the average level.

Hanson concluded that the ultimate horizontal shear strength of a smooth bonded joint is about 300 psi and that of a rough bonded joint is 500 psi.

4. Saemann and Washa (1964).

Tests at the University of Wisconsin (ref. 4) included two composite T-beams without steel crossing the interface. The cast surface was retarded and mortar brushed out to obtain a surface with 1/8" deep depressions. Failure of the two specimens occurred by horizontal shearing at calculated stresses of 420 and 606 psi, and the authors concluded that the ultimate horizontal shear strength could be given by the equation

$$Y = \frac{2700}{X + 5} \text{ (psi)}$$

where X is the ratio of the shear span (or the distance to the first concentrated load) to the effective depth.

ACI Code Requirements for Horizontal Shear.

The evolution of present-day code requirements for horizontal shear in composite concrete members is outlined in Table A, below, which lists the allowable horizontal shear stresses for different types of surface and connectors. The first column contains the values recommended by ACI-ASCE Committee 333 in its 1960 report (ref. 5, attached here as Appendix B). The middle column shows the values which were adopted in Section 2505 of the 1963 ACI Code, while the values in the last column are the permissible horizontal shear stresses given in Section 17.5.4 of the 1971 ACI Code.

Table A: Permissible Horizontal Shear Stress (psi)

Surface	Ties		ACI-ASCE 333 (1960)	ACI 318-63 (1963)	ACI 318-71 (1971)
Rough	None	Working	--	40	--
		Ultimate	--	76	80
Smooth	Min.	Working	40	40	--
		Ultimate	80	76	80
Rough	Min.	Working	160	160	--
		Ultimate	320	304	350

Since the allowable stress of 80 psi first appears in the report of Committee 333, it is of interest to examine the data upon which this figure is based, most of which is taken from the test results presented in references 1, 2 and 3. Note that Table 2 on page 624 of the Committee's report is described as listing nine specimens which failed in horizontal shear; however, none of the specimens reported by Ozell and Cochran (Committee reference 9) failed in horizontal shear, and thus they appear to be listed erroneously in Table 2. The one specimen taken from Revesz (Committee reference 10) is listed in Table 2 as having failed at a horizontal shear stress of 122 psi, although Revesz records the same specimen at 134 psi and VQ/Ib is calculated to be 143 psi.

The four specimens credited to Hanson (Committee reference 8) have been reduced by an average of 30% from Hanson's values as taken from figure 15 of his report. An additional specimen tested by Hanson (BR-1) was not included by the Committee, even though it did fail in horizontal shear and provided valuable information about the performance of composite girders without ties.

The apparent discrepancies between the Committee 333 report and the test results given in the references are summarized in Table B, below.

Table B: Horizontal Shear Strength (psi)

Investigator	Specimen	Table 2 Committee	Original Reference	Comments
Revesz	J	122	134	Smooth surface with no ties, specimen exceeded calculated ultimate flexural load, VQ/Ib = 143
Ozell & Cochran	A2 C2 A3	78 100 122	- - -	Specimens named do not appear in referenced paper. No specimens failed in horizontal shear; investigators concluded that smooth contact surface was able to develop adequate bond.
Hanson	BS I BS II BRS I BRS II BR I	350 340 450 580 -	490 470 670 610 350	Results have been reduced an average of 30%. Specimen BR I without ties not considered by the Committee.

The 1971 ACI Code refers to tests by Saemann and Washa, Hanson, Mattock, and Kaar, and Grossfield and Brinstiel, as the basis for the requirements on horizontal shear. In all the above references, only three specimens without steel ties were tested to failure, and these achieved 350 psi (Hanson), 420 psi and 606 psi (Saemann and Washa) of horizontal shear, respectively. It must be concluded that the present-day allowance of 80 psi for an intentionally roughened contact surface without ties derives from the report of Committee 333 discussed above.

CTA TESTS.

A series of tests was conducted in the CTA Laboratory to study the performance of composite beams without ties and determine the relationship between the roughness of the interface and the degree of composite action. A full report of the tests is contained in Appendix A. The following parameters were considered.

1. Surface roughness -
   a) Smooth (hard-steel trowel finish).
   b) Intermediate (wood-float finish).
   c) Rough (serrations formed by dragging a sharp object across the wet surface).
2. Surface condition before casting slab -
   a) Clean (broom-swept before casting topping).
   b) Dirty (dust, oil and paper scattered before casting topping).
3. Topping concrete -
   a) Normal weight $f'_c = 2400 - 5000$ psi.
   b) Lightweight $f'_c = 3000 - 5000$ psi.
4. Shear span/effective depth ratio, X
   a) $X = 3.5$
   b) $X = 7.75$

Specimen	Span Length	Surface	Topping Weight (pcf)	Topping f'_c (psi)	P_u* Load Allowed by Code (kips) by V_{dh}	by V_{ci}	by M_u	P_u Test (kips)	Type of Failure	$\frac{V_u Q}{I b}$ (psi)	V_u/bd (psi)
S-1-I	4'	Intermediate-clean	150	3640	23	100	126	103.4	Shear	302	307
L-1-I	9'	Intermediate-clean	150	2400	23	48	52	50.8	Shear	139	151
S-2-I	4'	Intermediate-clean	155	3960	23	100	126	115.6	Shear	356	344
L-2-I	9'	Intermediate-clean	157	4760	23	48	52	56.4	Flexure	169	168
S-3-R	4'	Rough-dirty	153	4760	23	100	126	154.2	Shear flexure	477	459
L-3-R	9'	Rough-dirty	155	5080	23	48	52	55.5	Shear flexure	168	165
S-4-I	4'	Intermediate-clean	108	3220	23	100	126	150.4	Shear flexure	392	448
L-4-I	9'	Intermediate-clean	111	4340	23	48	52	57.3	Flexure	150	171
S-5-I	4'	Intermediate-dirty	113	4760	23	100	126	153.2	Flexure	421	456
L-5-I	9'	Intermediate-dirty	113	4620	23	48	52	58.3	Flexure	155	173
S-6-R	4'	Rough-dirty	114	4120	23	100	126	150.4	Shear flexure	411	448
L-6-R	9'	Rough-dirty	113	5450	23	48	52	58.3	Flexure	158	173
S-7-S	4'	Smooth-sandblast-clean	115	5500	23	100	126	152.3	Horizontal shear	429	453
L-7-S	9'	Smooth-clean	113	5450	23	48	52	60.2	Flexure	164	179
S-8-S	4'	Smooth-cement slurry-clean	110	4060	23	100	126	148.5	Horizontal shear	398	442
L-8-S	9'	Smooth-cement slurry-clean	109	4320	23	48	52	53.6	Shear flexure	139	159

* See sample calculations following Table 3a in Appendix A

Table C: CTA Test Results

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TEST RESULTS.

The performance of 16 test beams loaded to failure is recorded in Table C. No specimen failed at a load below the maximum allowed by the code for a monolithic member of the same dimensions. In the two cases where the failure was due to or accompanied by horizontal shearing, S-7-S and S-8-S, the test loads were 6.7 and 6.5 times greater than the maximum load which the code would have allowed for a composite beam with an intentionally roughened contact surface. Both of these specimens had smooth contact surfaces and were able to sustain loads in excess of their computed composite ultimate before failing.

Specimens S-5-I and L-5-I, which had a conventional wood float finish similar to that left by the vibrating screed often used in production, were deliberately dirtied with dust, bits of paper, and form oil before the topping was cast. Both of these beams failed in flexure at loads exceeding the calculated ultimate composite moment. Although the calculated horizontal shear stresses were 421 psi and 155 psi, respectively, there was no evidence of separation of the two elements of the composite beam.

If two specimens, S-1-I and S-2-I, are omitted from the averages because they failed prematurely due to vertical shear, the test beams are found able to achieve the computed ultimate flexural loads of the full composite section. In the range studied, no apparent correlation exists between the degree of roughness at the interface and the observed ultimate moment. Similarly, all test beams exhibited full composite action within the range of working loads, irrespective of the degree of surface roughness.

Comparison of the measured deflections of specimens with smooth and intermediate surfaces against those with rough surfaces indicates that, in the range of loads approaching ultimate, deflections increase with the smoothness of the interface. This phenomenon, indicative of greater relative slippage between elements, is observed only after cracking and does not affect the load carrying capacity of the member.

COMPOSITE SYSTEMS WITHOUT ROUGHNESS.

CTA test results, as well as the findings of other investigators, provide support for the concept of composite systems without the intentional roughness required by the ACI Code. Table D lists the maximum horizontal shearing stresses at ultimate load (1.4 D + 1.7 L) from amongst the most heavily loaded composite double-tee and hollow-core members in the CTC catalog. Even for an intentionally smooth interface, the factor of safety against failure due to horizontal shearing is at least 5 for all double tees and at least 2 for all hollow-core slabs. If the bond strength of composite systems finished by the conventional vibrating screed is taken conservatively as 255 psi (300 psi reduced by an understrength factor of .85), the respective factors of safety at full ultimate load are 9.1 and 3.3.

Table D: Maximum Ultimate Shear of Standard Composite Systems

Type of Member	Maximum $V_u Q/Ib$ (psi)	Maximum V_u/bd (psi)
16" D.T.	28	36
20" D.T.	23	33
24" D.T.	20	31
8" H.C.	77	96
12" H.C.	50	75

CONCLUSIONS.

1. The ACI Code requirements for horizontal shear may, in part, be based on incorrect interpretation of previously published research.

2. Previous research indicates that the shear strength of concrete-concrete bond ranges from 140-500 psi for very smooth to very rough contact surfaces.

3. Sixteen composite prestressed beams tested by CTA were able to exceed the maximum load capacity the ACI Code would have allowed for a similar monolithic section.

4. Within the range of variables considered, there was no apparent correlation between the interface condition (roughness and cleanliness) and the degree of composite action as measured by

   a) the effective moment of inertia of test beams under pre-cracking loads and

   b) the ability to achieve full ultimate moment of the composite section.

5. Under loads ranging from cracking to ultimate, specimens with smooth and intermediate surfaces exhibited greater deflections than those with rough surfaces, resulting in deflections at 70% of ultimate loads of:

   a) 10% greater for both smooth and intermediate specimens with a shear span/effective depth ratio of 8 (horizontal shear stress approximately 130 psi),

   b) 18% greater for intermediate specimens with a shear span/effective depth ratio of 4 (horizontal shear stress approximately 300 psi), and

   c) 29% greater for smooth specimens with a shear span/effective depth ratio of 4 (horizontal shear stress approximately 300 psi).

6. Comparing test results to the maximum horizontal shear stresses expected in composite hollow-core and double-tee elements, it is concluded that composite systems with nominal topping and wide contact surfaces, but without steel ties, can be expected to perform beyond load levels indicated by the ACI Code even without the intentional roughness prescribed by the Code.

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REFERENCES

1. Revesz, S., "Behavior of Composite T-Beams with Prestressed and Unprestressed Reinforcement," ACI Journal, V. 24, No. 6, Feb. 1953, pp. 585-592.

2. Ozell, A. M., and J. W. Cochran, "Behavior of Composite Lintel Beams in Bending," PCI Journal, V. 1, No. 1, May 1956, pp. 38-48.

3. Hanson, N. W., "Horizontal Shear Connections," Journal of the Research and Development Laboratories, Portland Cement Association, V. 2, No.2, May 1960, pp. 38-58.

4. Saemann, J. C., and G. W. Washa, "Horizontal Shear Connections between Precast Beams and Cast-in-Place Slabs," ACI Journal, V. 61, No. 11, November 1964, pp. 1383-1408.

5. "Tentative Recommendations for Design of Composite Beams and Girders for Buildings," Report of ACI-ASCE Committee 333, ACI Journal, V. 32, No. 6, December 1960, pp. 609-628.

APPENDIX A

TEST REPORT

PERFORMANCE OF COMPOSITE PRESTRESSED BEAMS WITHOUT STEEL TIES

OBJECTIVE.

A series of tests was conducted to determine the flexural performance of pretensioned beams composite with cast-on topping, with particular emphasis on deflections under service and cracking loads and ability to achieve full ultimate moment of the composite section.

SCOPE.

The investigation was limited to specimens designed so as to simulate the behavior of composite single and double tees, hollow core and other members with thin topping and a wide contact surface. Companion pairs of composite beams were fabricated to provide comparison between specimens with relatively high and low horizontal shear stresses at the interface under ultimate flexural loads. The following variables were considered:

1. Degree of contact surface roughness -- smooth, intermediate, and rough.
2. Strength and density of topping concrete -- 2500 - 5000 psi, 110 & 150 pcf.
3. Degree of contact surface cleanliness -- clean and dirty.

DESCRIPTION OF SPECIMENS.

The bottom beam used in the test program is shown in Figure 1 and Table 1. Figure 2 and Table 2 contain the material quantities and properties of the topping. Fabrication of the test beams is shown in Figures 3, 4, 5, and 6.

Three degrees of roughness were considered. Figure 7 shows the contact surface of a "smooth" specimen, achieved by finishing the surface of the bottom beam with a hard steel trowel. Figure 8 is an "intermediate" specimen, the finish having been applied with a wood float, and Figure 9 shows a "rough" surface, attained by dragging a bar across the top of the fresh concrete at regular intervals. Figure 10 is a "smooth" specimen which has been sand-blasted, while Figure 11 is the surface of a Basalt Rock Company double tee, formed by a vibrating screed.

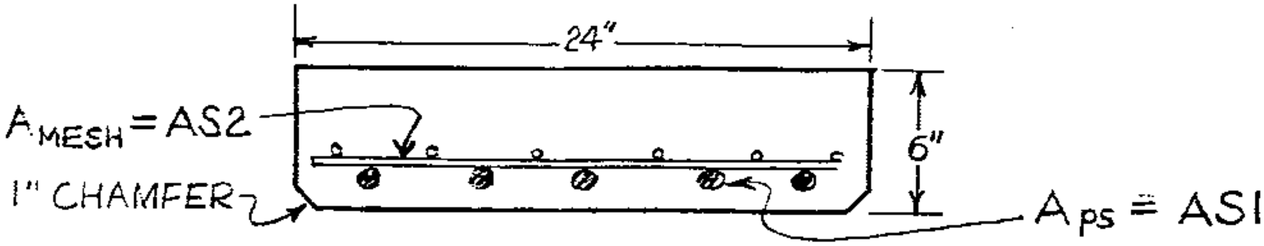

Figure 1: Bottom Beam

Specimens	AS1 (in^2)	Fps (kips)	Y_b-e (in)	AS2 (in^2)	f'_c (psi)	w_c (pcf)
S	.9186	141	1.5	.116	7062	155
L	.9186	141	1.5	.116	6933	155

Table 1: Bottom Material Quantities

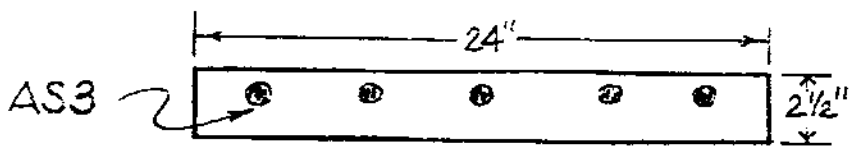

Figure 2: Top Slab

Specimen	AS3 (in^2)	f'_c (psi)	w_c (pcf)	Specimen	AS3 (in^2)	f'_c (psi)	w_c (pcf)
S-1-I	2.64	3640	150	L-1-I	1.76	2400	150
S-2-I	2.64	3960	154	L-2-I	1.76	4735	157
S-3-R	2.64	4760	153	L-3-R	1.76	5048	155
S-4-I	2.64	3220	108	L-4-I	1.76	4340	111
S-5-I	2.64	4760	113	L-5-I	1.76	4620	113
S-6-R	2.64	4120	114	L-6-R	1.76	5450	113
S-7-S	2.64	5500	115	L-7-S	1.76	5446	113
S-8-S	2.64	4058	110	L-8-S	1.76	4318	109

Table 2: Top Material Quantities

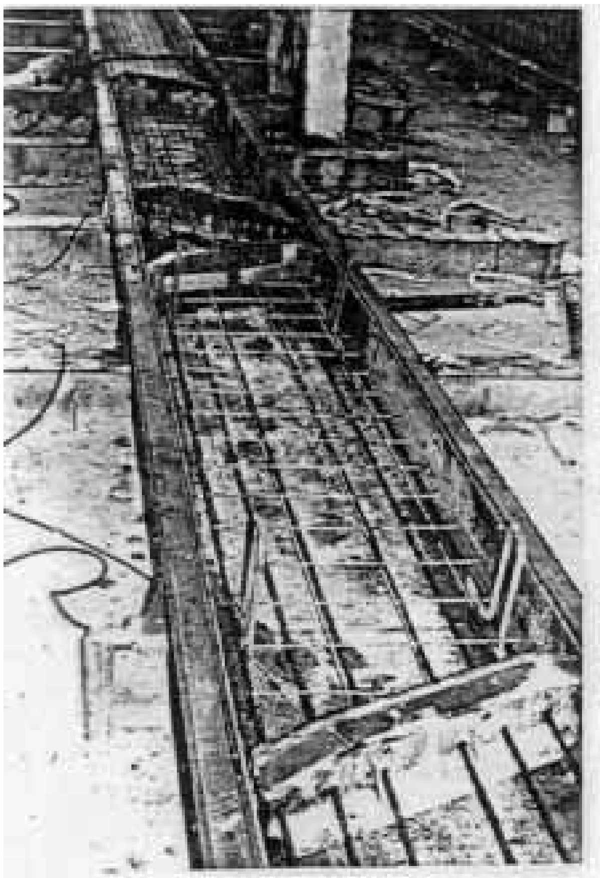

Figure 3

Figure 4

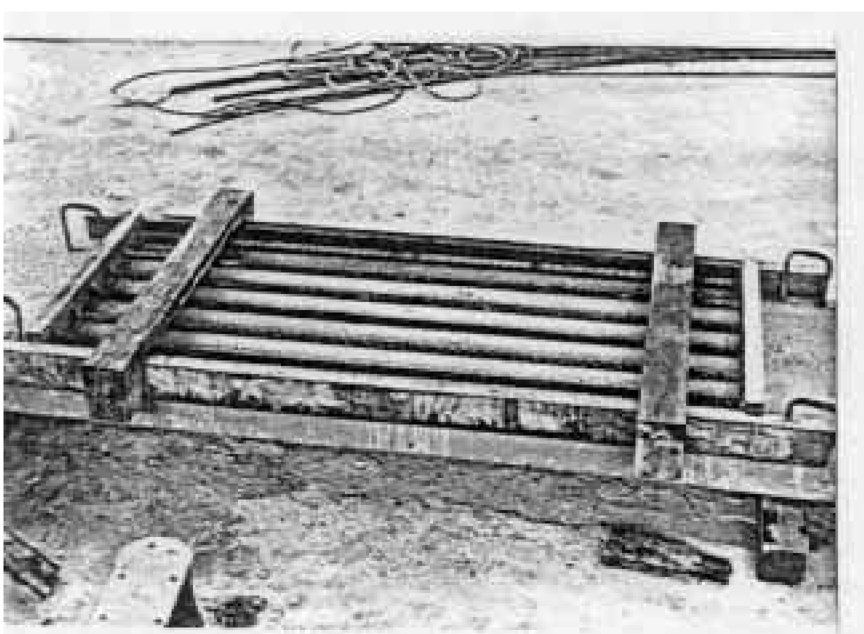

Figure 6

Figure 5

Figure 7 - "Smooth"

Figure 8 - "Intermediate"

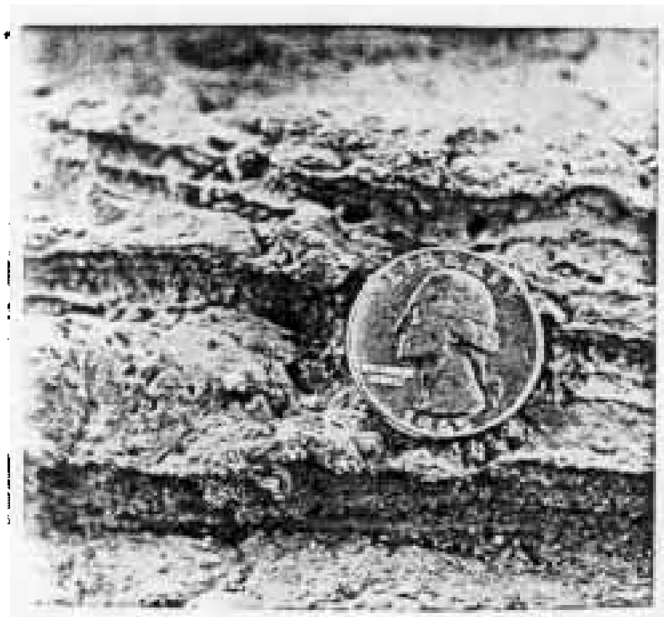

Figure 9 - "Rough"

Figure 10 - "Smooth-Sandblast"

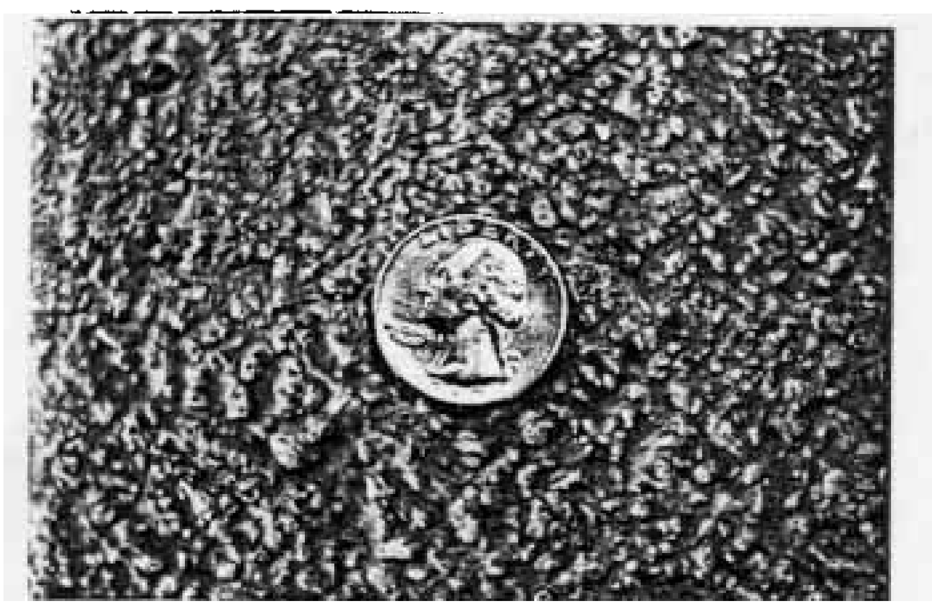

Figure 11 - Basalt Rock Company D. T.

Specimen	Age at Test (days) Bottom	Age at Test (days) Topping	Span Length	Interface Condition	P_u (kips)	Type of Failure	$V_u Q/Ib$ (psi)	V_u/bd (psi)
S-1-I	19	14	4'	Intermediate-clean	103.4	Shear	302	307
L-1-I	17	13	9	Intermediate-clean	50.8	Shear	139	151
S-2-I	19	13	4	Intermediate-clean	115.6	Shear	356	344
L-2-I	19	14	9	Intermediate-clean	56.4	Flexure	169	168
S-3-R	21	13	4	Rough-dirty	154.2	Shear-flexure	477	459
L-3-R	20	13	9	Rough-dirty	55.5	Shear-flexure	168	165
S-4-I	21	10	4	Intermediate-clean	150.4	Shear-flexure	392	448
L-4-I	20	10	9	Intermediate-clean	57.3	Flexure	150	171
S-5-I	22	10	4	Intermediate-dirty	153.2	Flexure	421	456
L-5-I	21	10	9	Intermediate-dirty	58.3	Flexure	155	173
S-6-R	22	8	4	Rough-dirty	150.4	Shear-flexure	411	448
L-6-R	21	8	9	Rough-dirty	58.3	Flexure	158	173
S-7-S	25	10	4	Smooth-sandblast-clean	152.3	Horizontal shear	429	453
L-7-S	21	7	9	Smooth-clean	60.2	Flexure	164	179
S-8-S	25	7	4	Smooth-clean-slurry	148.5	Horizontal shear	398	442
L-8-S	24	7	9	Smooth-clean-slurry	53.6	Shear-flexure	139	159

Table 3: Test Results

TEST PROCEDURE.

The beams were loaded with a single concentrated load applied at midspan by a hydraulic jack, as shown in Figure 12. After a few cycles to cracking load, the beams were loaded to failure in small increments. Deflections were measured at center span and near the support points by means of dial gages.

Table 3 contains the test results in summary form. Figures 13-20 and 21-28 show the failure modes of 4' and 9' specimens, respectively, in sequential order.

Table 3a presents, for comparison to actual performance, the maximum allowable loads on these specimens based on various criteria in the 1971 ACI Code. The derivation of these values is illustrated in the sample calculations following.

Table 3a: Code Allowances

Type of Specimen	Span (ft.)	P_u Allowed by Code (kips) V_{dh}	V_{ci}	M_u
S	4	23	100	126
L	9	23	48	51

Sample calculations (ℓ =9')

1. (Sections 17.5.3 & 17.5.4 a) $v_{dh} = 80 \text{ psi} = \dfrac{V_u}{(.85)(7)(24)}$ $\quad V_u = 11.4$ kips $\quad P_u = 23$ kips

2. (Sections 11.2.1 & 11.5.1) $v_u = v_c = 0.6\sqrt{f'_c} + 700\dfrac{V_u d}{M_u} = (0.6)\sqrt{7000}$

$+ (700)\dfrac{(7)}{54} = 141 \text{ psi}.\ 2\sqrt{f'_c} = 167.3 \text{ psi} \quad \text{use } 168 = \dfrac{V_u}{(.85)(7)(24)}$

$V_u = 24$ kips $P_u = 48$ kips.

3. (Sections 9.2.1.1) $\dfrac{P_u L}{4} = \phi M_u = (.9)(1525)$

$P_u = 50.8$ kips.

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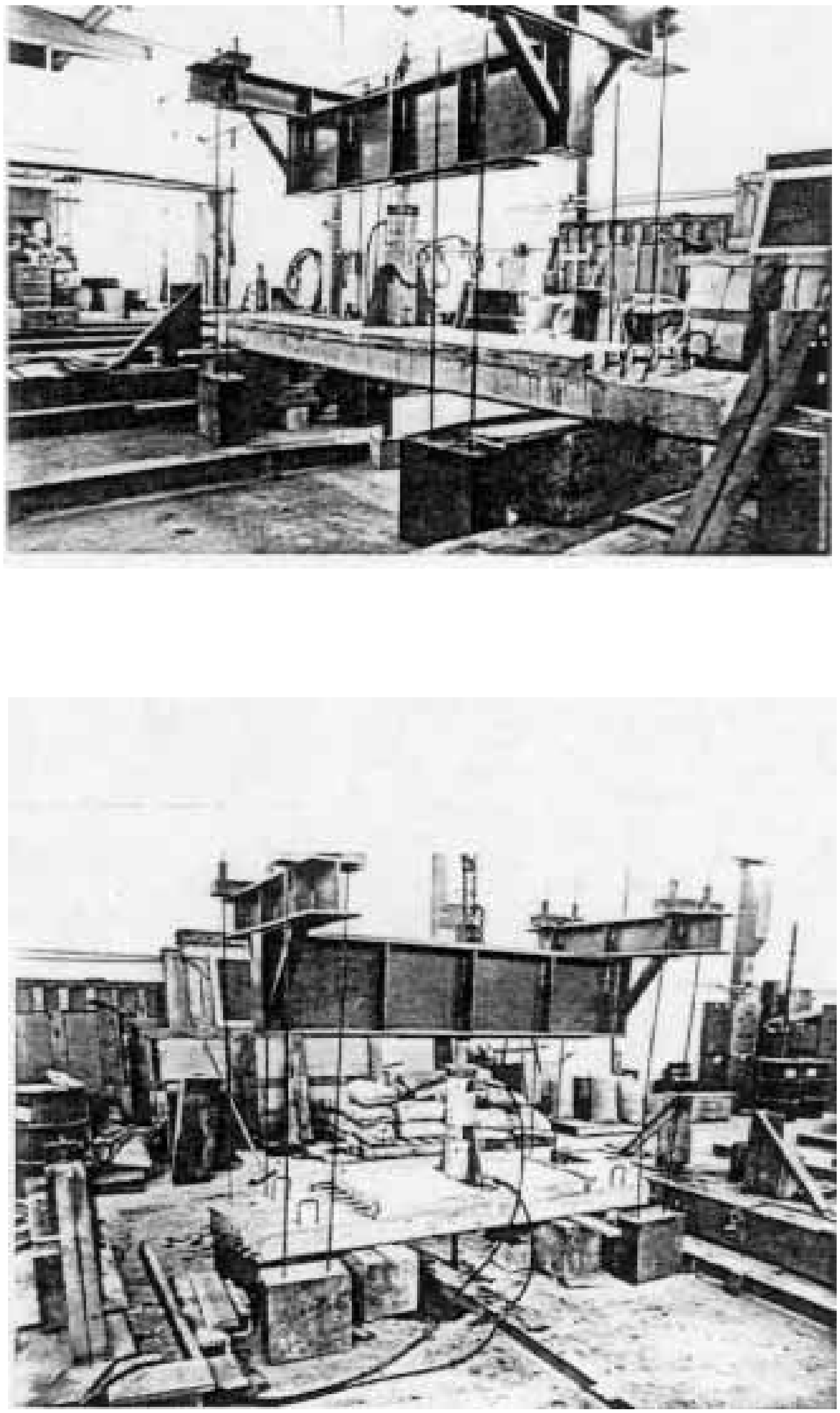

Figure 12 - Loading Setup

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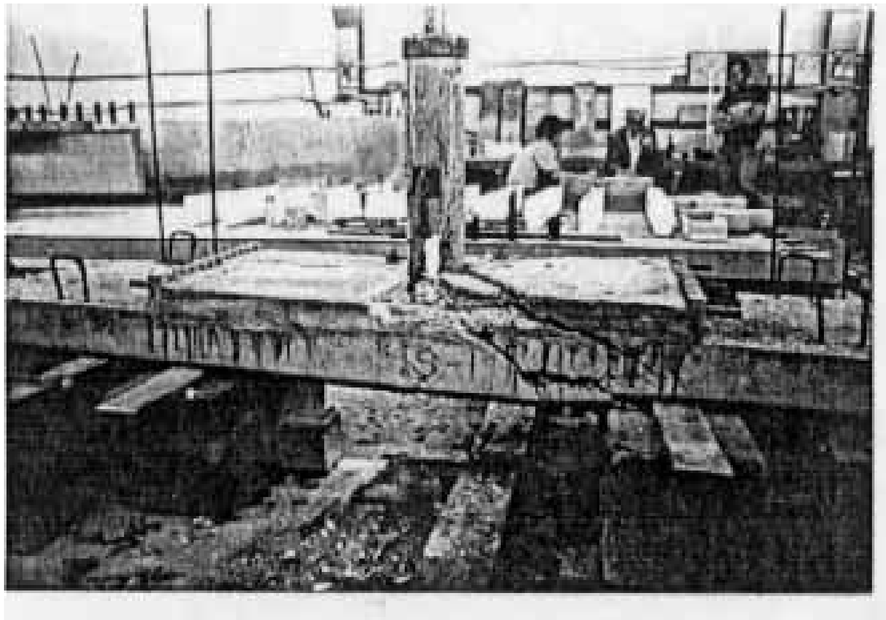

Figure 13

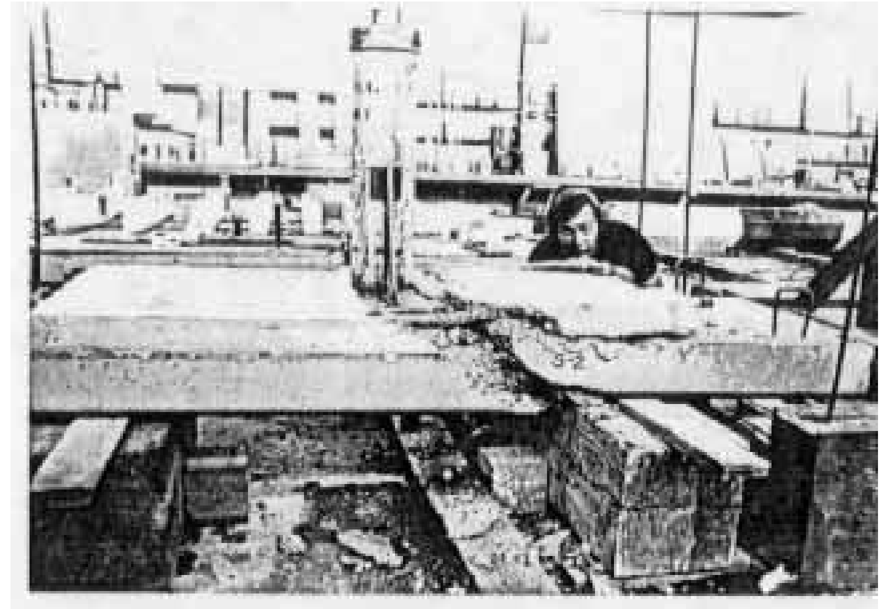

Figure 14

Figure 15

Figure 16

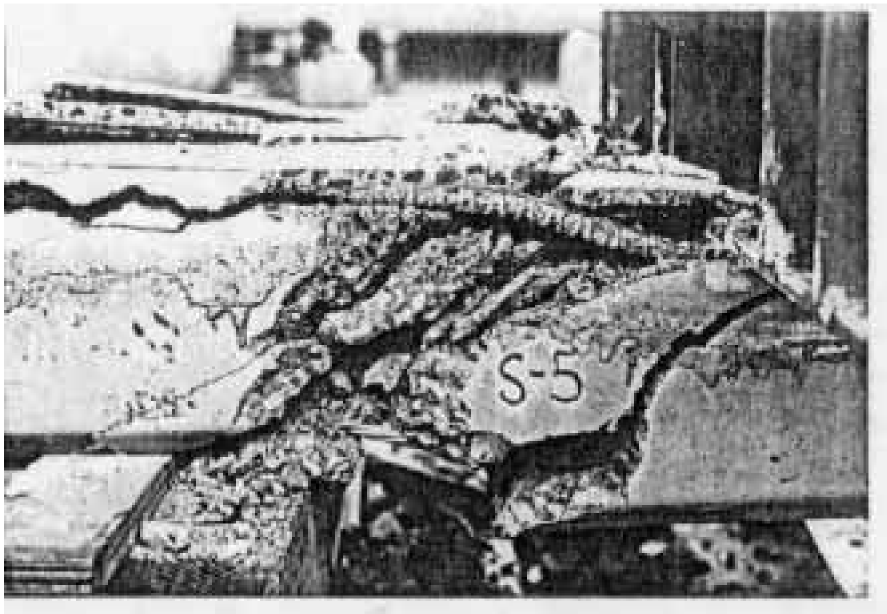

Figure 17

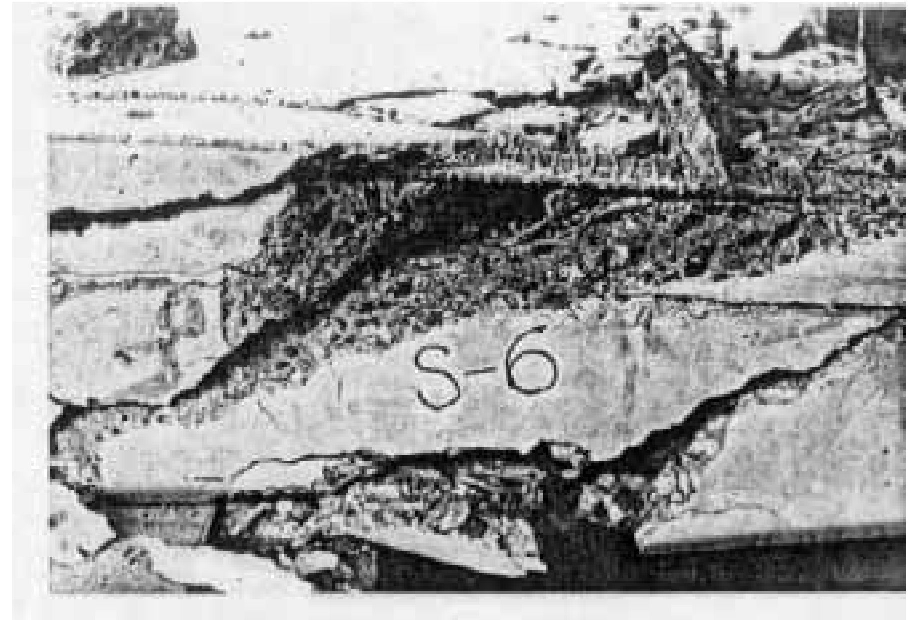

Figure 18

Figure 19

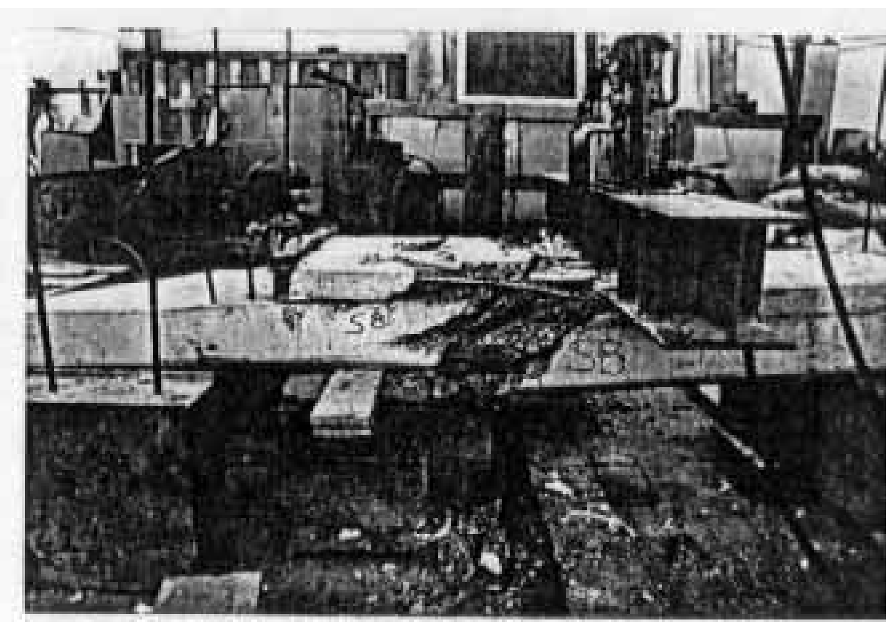

Figure 20

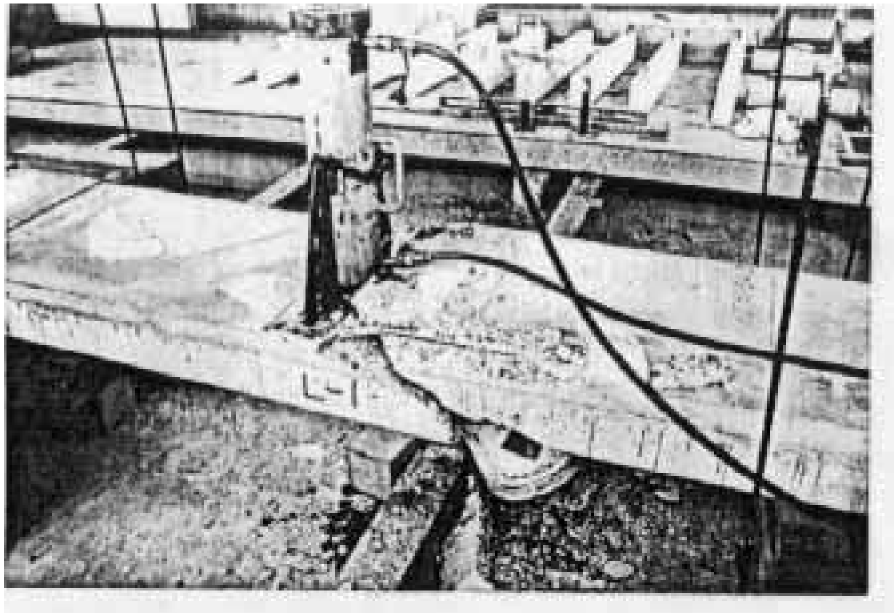

Figure 21

Figure 22

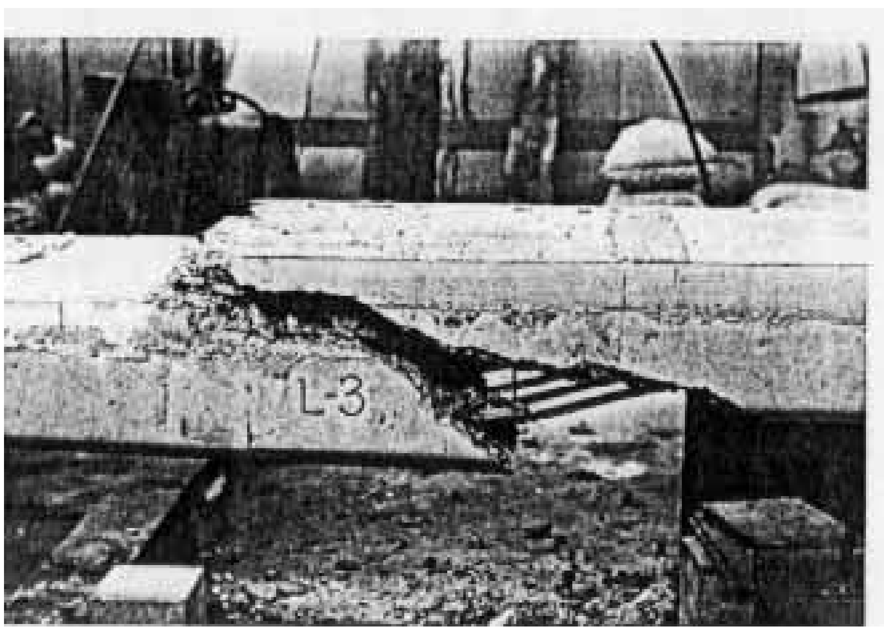

Figure 23

Figure 24

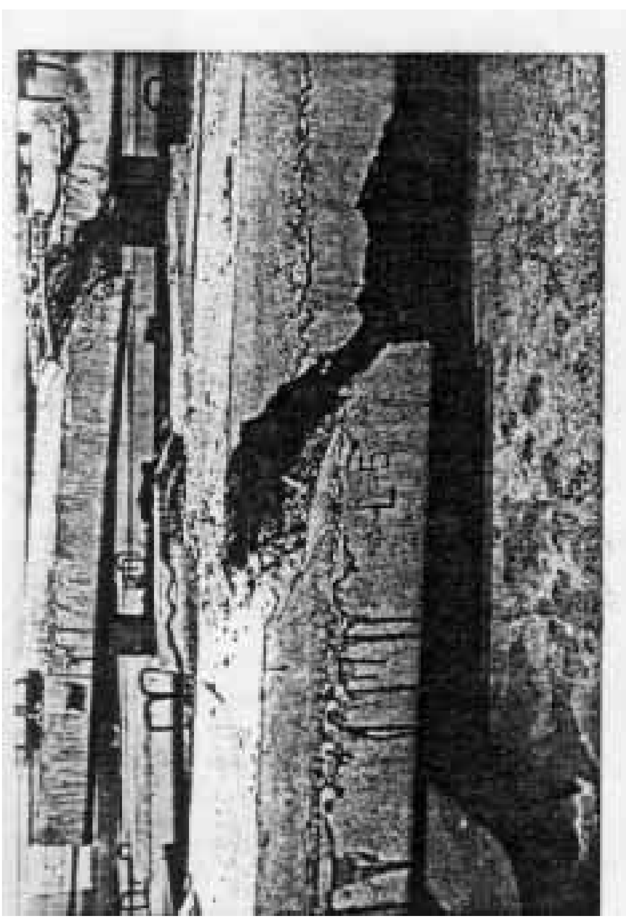

Figure 25

Figure 26

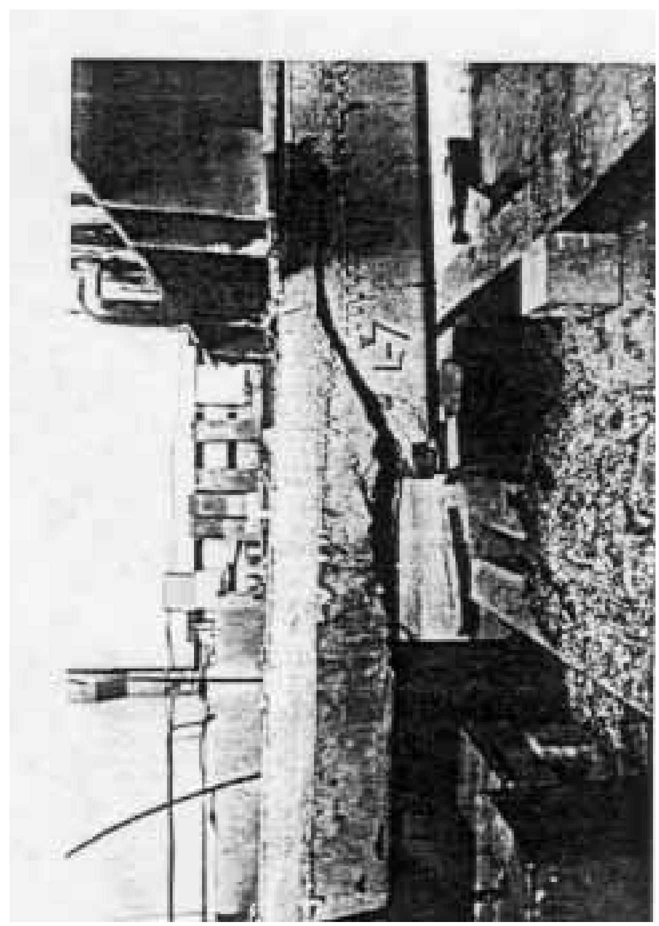

Figure 27

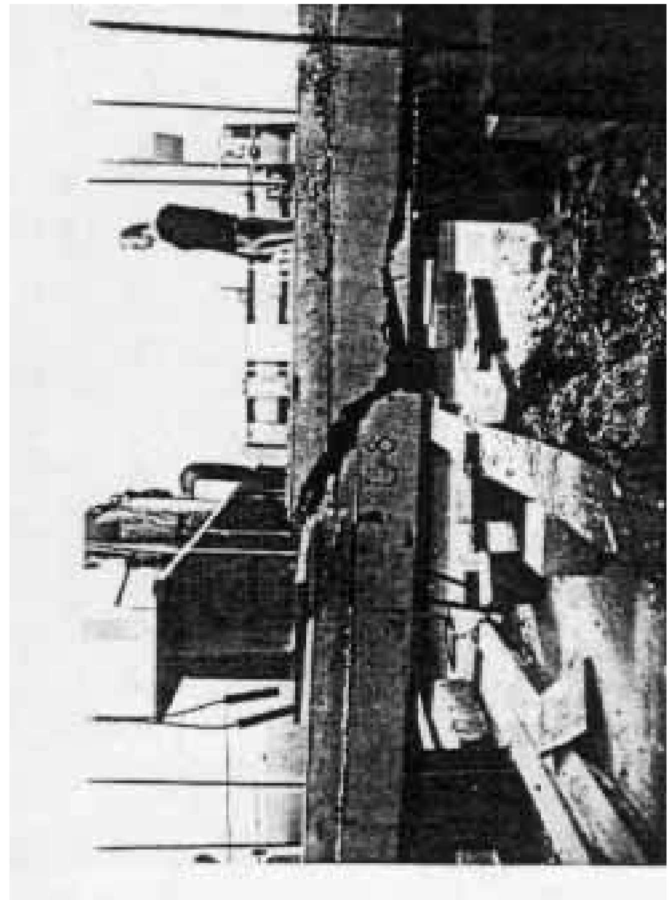

Figure 28

COMPARATIVE ULTIMATE FLEXURAL CAPACITY.

The principal matter of concern in the behavior of composite flexural members is their ability to achieve the ultimate moment of the full section without failing prematurely due to horizontal separation. Table 4 lists the theoretical and observed ultimate moments for all specimens, calculated in the following manner:

Sample calculation (specimen L-5-1)

$$P_u \text{ (test)} = 58.28^k$$

$$M_u \text{ (observed)} = \frac{P_u L}{4} = \frac{(58.28)(106)}{4} = 1544.4 \text{ k-in.}$$

$$M_u \text{ (theoretical)} = (AS3)(f_y)(7.5 - 1.5) + p'A_G\, f_{su} d \left(\frac{1 - 0.6\, P'\, f_{su}}{f'_c} \right)$$

$$\text{Where } p' = \frac{AS1 - (AS3 - AS2)\left(\frac{60}{270}\right)}{A_G}$$

$$= \frac{.9186 - (1.76 - .116)\ \frac{60}{270}}{(24)\ (8.5)} = .002712$$

$$\text{and } f_{su} = 270 \left[1 - 0.5\, p' \ \frac{270}{f'_c} \right] = 270 \left[1 - \frac{(0.5)\ (.002712)\ (270)}{4.62} \right] = 248.6$$

$$M_u = (1.76)(60)(6) + (.002712)(24)(8.5)(248.6)(7) \left[1 - \frac{(.6)(002712)(248.6)}{4.62} \right]$$

$$= 1512 \text{ k-in.}$$

Note that the span length is conservatively taken as 106" to allow for relocation of the point of support at high loads.

Table 4: Ultimate Moment

Specimen	M_u (k-in) Observed	M_u (k-in) Theoretical	Specimen	M_u (k-in) Observed	M_u (k-in) Theoretical
S-1-I	1189	1611	L-1-I	1345	1382
S-2-I	1330	1622	L-2-I	1494	1516
S-3-R	1772	1636	L-3-R	1470	1526
S-4-I	1729	1605	L-4-I	1520	1502
S-5-I	1763	1636	L-5-I	1544	1512
S-6-R	1729	1625	L-6-R	1544	1536
S-7-S	1751	1643	L-7-S	1594	1536
S-8-S	1708	1624	L-8-S	1420	1502

The importance of the interface roughness with respect to achieving ultimate flexural strength is illustrated in Table 5, below. It is apparent that, if two specimens which failed in vertical shear are dropped, the averages for varying degrees of roughness do not differ significantly.

Table 5: Comparative $\frac{M_u \text{ (observed)}}{M_u \text{ (theoretical)}}$

Smooth	Intermediate	Rough
1.038 (L-7-S)	* .738 (S-1-I)	1.083 (S-3-R)
.945 (L-8-S)	* .820 (S-2-I)	1.064 (S-6-R)
1.066 (S-7-S)	1.077 (S-4-I)	.963 (L-3-R)
1.052 (S-8-S)	1.078 (S-5-I)	1.005 (L-6-R)
	.973 (L-1-I)	
	.985 (L-2-I)	
	1.012 (L-4-I)	
	1.021 (L-5-I)	
1.025	1.024	1.029

* Omitted due to vertical shear failure.

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COMPARATIVE MOMENTS OF INERTIA.

One measure of the effectiveness of composite action is the ratio between the apparent moment of inertia as determined from measured deflections and the calculated moment of inertia of the full section. Tables 6 and 7 contain all deflection measurements and moments of inertia. The observed moments of inertia were calculated from data recorded in the range of working loads by the method demonstrated below for Specimen L-5-1. The deflection, $\bar{\delta}$, was obtained by subtracting the average of deflections measured near the support points from the deflection at center span. "a" and "b" are the ratios to the span length of the distances between these end dial gages and the centers of support.

Sample Calculation (Specimen L-5-1):

Gage Pressure, y, (100 psi)	0	5	10	12	14
Deflection X (10^3 inches)	0	22.5	51	66.5	76

y = KX + c. Using least squares, K is best approximated by:

$$K = \frac{N(\Sigma x_i y_i) - \Sigma x_i \Sigma y_i}{N(\Sigma x_i^2) - (\Sigma x_i)^2} = \frac{(5)(2484.5) - (216)(41)}{(5)(13305.5) - (216)(216)} = .1795$$

$$P/\bar{\delta} = (.1795)(100)(1000)(18.8) = 337420 \quad (\text{lb/in})$$

$$\bar{\delta} = \frac{P\ell^3}{48EI}\left((1 - \frac{3(a+b) - 4(a^3+b^3)}{2}\right)$$

$$I = P/\bar{\delta}\left(\frac{\ell^3}{48E}\right)\left(1 - \frac{3(a+b) - 4(a^3+b^3)}{2}\right)$$

$$= P/\bar{\delta}\ \frac{(106)^3}{(48)(5300000)}\left(1 - \frac{3(.0967 + .1226) - 4((.0967)^3 + (.1226)^3)}{2}\right) =$$

$$= 3.167 \times 10^{-3}\, P/\bar{\delta} = 1068 \text{ in}^4$$

Table 6: Deflections of 4' Specimens

Specimen-Surface			$\bar{\delta}$ (Inches x 10^3) Gage Pressure (psi) (Piston Area = 18.8 in^2)										Observed	Theoretical
Specimen	a	b	500	1000	1500	2000	2500	3000	3500	4000	4500	5000	I	I
S-1-I	.136	.185	1	3	4	4	5.5	9.5					1176	1158
S-2-I	.152	.185	1.5	2	3	5	6	8.5	10	13.5	24	41	1265	1210
S-3-R	.163	.223	1	2	3.5	4	6.5	9.5	9	11.5	17.5	29	1148	1238
S-4-I	.147	.223	2	1.5	4	4.5	6.5	9.5	13	16.5	27	41	1018	963
S-5-I	.141	.217	0	1.5	2	3.5	4	5.5	7.5	11.5	18.5	33	1303	1053
S-6-R	.168	.245	1	1.5	4	4.5	5.5	7.5	8	12.5	19	28	1130	1032
S-7-S	.141	.190	3	4	6	8	10.5	11.5	13	19	29.5	42	1012	1090
S-8-S	.152	.185	.5	3	3.5	7	8.5	10	12.5	19.5	29.5	48	935	1012

Table 7: Deflections of 9' Specimens

Specimen - Surface	a	b	$\bar{\delta}$ (Inches x 10^3) Gage Pressure (psi) (piston area = 18.8 in.^{2})											Observed I	Theoretical I
			250	500	750	1000	1200	1400	1600	1800	2000	2200	2400		
L-1-I	.0613	.1415	15	30	45.5	61	77							978	1043
L-2-I	.066	.1203	12	24.5	39	53	66	87	132.5	193	280	388.5		1047	1172
L-3-R	.0684	.1462	10	20	33.5	47	58	75.5	108	155	230	320	428	1155	1232
L-4-I	.0566	.1368	13	29	46	64	81	105.5	152	213.5	292	383		948	986
L-5-I	.0967	.1226	9	22.5	35	51	65.5	76	105	148.5	207	273		1068	1008
L-6-R	.0896	.1226	9.5	23	36	50	64.5	82	110	151.5	217	288.5	376	1042	1040
L-7-S	.092	.1226	8	20	33.5	47.5	59.5	76	109.5	154				1109	1040
L-8-S	.087	.1273	8	18	32	46	58.5	80.5	115	171	241.5	333	427.5	1049	975

On the basis of geometry and material properties, the theoretical moment of inertia is calculated as follows:

$$y_c = \frac{(AS1)(n-1)(1.5) + (AS2)(n-1)(2.) + (AS3)(n-1)(7.25) + (24)(6)(3) + (24X)(2.5)(7.25)}{(AS1)(n-1) + (AS2)(n-1) + (AS3)(n-1) + (24)(6) + (24X)(2.5)}$$

$$= \frac{(.92)(4)(1.5) + (.12)(4)(2) + (1.76)(4)(7.25) + (24)(6)(3) + (24)(.508)(2.5)(7.25)}{(.92)(4) + (.12)(4) + (1.76)(4) + (24)(6) + (24)(.508)(2.5)}$$

$$= 3.836 \text{ in.}$$

$$I_c = \frac{24(6)^3}{12} + \frac{24 \times (2.5)^3}{12} + (AS1)(n-1)(y_c - 1.5)^2 + (AS2)(n-1)(y_c - 2)^2 + (24)(6)(y_c - 3)^2 + (24x)(2.5)(y_c - 7.25)^2 + (AS3)(n-1)(y_c - 7.25)^2$$

$$= 432 + 16 + 101 + 355 + 20 + 2 + 82 = 1008 \text{ in}^4$$

Table 8 shows the ratios of observed to theoretical moments of inertia for 16 specimens according to surface treatment. The average values demonstrate that there was no significant difference in effectiveness of composite action over the range of working loads.

Table 8: $\frac{I \text{ Observed}}{I \text{ Theoretical}}$ for different surface roughness

Smooth	Intermediate	Rough
1.066 (L-7-S)		
1.076 (L-8-S)	.938 (L-1-I)	.938 (L-3-R)
.928 (S-7-S)	.893 (L-2-I)	1.002 (L-6-R)
.924 (S-8-S)	.961 (L-4-I)	.927 (S-3-R)
	1.060 (L-5-I)	1.095 (S-6-R)
	1.016 (S-1-I)	
	1.045 (S-2-I)	
	1.057 (S-4-I)	
	1.237 (S-5-I)	
.999	1.026	.991

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COMPARATIVE DEFLECTIONS NEAR ULTIMATE LOAD.

Since even slight relative slippage along the interface may cause measurable increases in deflection, the average readings of specimens of different roughness were compared to determine the effect of the type of surface on the amount of deflection under loads approaching ultimate. The deflections in Tables 6 and 7 were "normalized" by subtracting the apparent residual deflection c, determined from a least-squares analysis, and dividing by a factor to correct for the location of the end dial gages. The resulting "deflections" were then compared using the average of specimens with a roughened contact surface as a basis. Figures 13 and 14 show the comparative deflections of smooth and intermediate specimens, the deflection of rough-surfaced specimens having been subtracted out. Both figures reveal a marked increase in relative average deflections around cracking load. The gap increased steadily towards ultimate load, reaching maximum recorded deflections in the 9' specimens of 10% greater for both smooth and intermediate contact surfaces than rough. In the 4' specimens, those with a smooth surface had maximum recorded deflections 29% greater than those with a rough surface, while the intermediate specimens had 18% greater maximum recorded deflections than the rough. Note that Figures 13 and 14 show the net difference in deflections, not the ratio. The significant difference in deflections at higher loads indicates that relatively more slippage took place in those specimens with smooth and intermediate contact surfaces after cracking. The ability of the composite beams to achieve full ultimate moment was not impaired by this slippage, however.

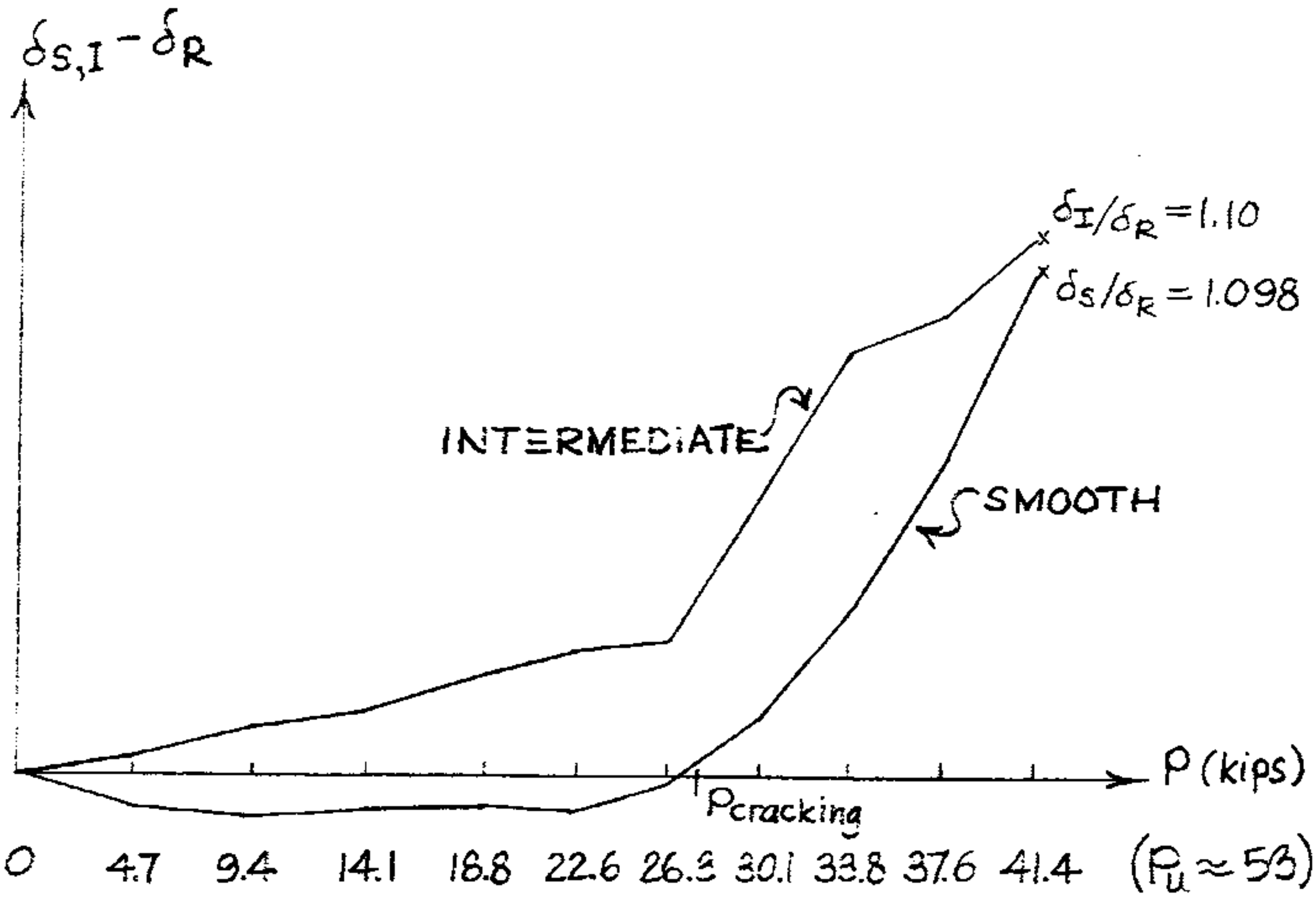

Figure 29
Excess Deflections of 9' Specimens

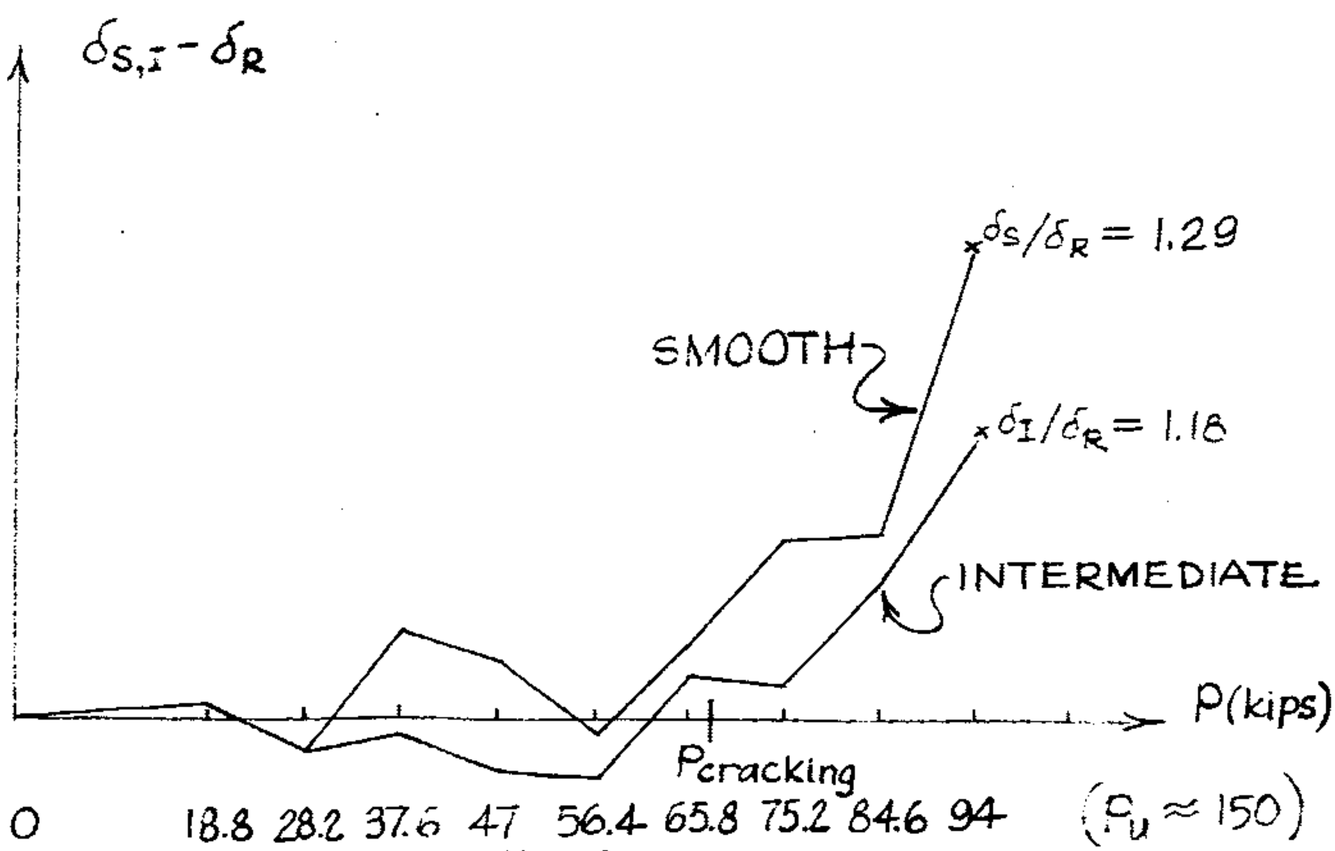

Figure 30
Excess Deflections of 4' Specimens

CONCLUSIONS.

1. Sixteen composite prestressed beams have been tested and found to perform adequately with respect to the load allowed by the ACI Code for similar monolithic sections.

2. Two specimens exhibited failure modes due to or accompanied by horizontal shearing between the two elements, at calculated shear stresses of 398 and 429 psi.

3. Under loads in the range up to cracking, there was no apparent correlation between effective composite action as reflected in the relative moment of inertia of the section and:

a) the degree of surface roughness and cleanliness at the interface, and

b) the strength and weight of the topping.

4. There was similarly no apparent correlation between effective composite action as reflected in the ability of the member to achieve full ultimate flexural load and:

a) the degree of surface roughness and cleanlinesss at the interface, and

b) the strength and weight of the topping.

5. Increased deflections of smooth and intermediate specimens in the post-cracking range were attributed to slippage between the two elements. At a horizontal shear stress of approximately 130 psi and 75% of ultimate load, specimens with smooth and rough interfaces exhibited 10% greater deflections than those with a rough interface. At a horizontal shear stress of approximately 300 psi and 70% of ultimate load, specimens with intermediate and smooth interface exhibited 18% and 29%, respectively, greater deflection than specimens with a rough interface. This behavior was not observed in the range of working loads, nor was there any apparent detrimental effect on the ultimate load carried by such specimens.

6. The tests demonstrated conclusively that composite prestressed members with thin topping are able to perform as monolithic beams at least up to horizontal shear stresses of 150 psi and 400 psi for shear span/effective depth ratios of 8 and 4, respectively, even when the topping is cast on a smooth surface.

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APPENDIX B

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Tentative Recommendations for Design of Composite Beams and Girders for Buildings

Reported by ACI-ASCE COMMITTEE 333

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ACI-ASCE Committee 333 was organized in 1956 to prepare recommendations for the design and construction of structures composed of prefabricated beams combined with cast-in-place slabs. After a review of the existing information and practices, the committee has channeled one part of its activities toward preparation of recommendations for the design of composite beams and girders for buildings. The results of this work are reported herein. The committee expects to prepare further reports after the completion of research investigations now in progress.

The progress report is written in two parts. Tentative design recommendations are presented in the first part. The second part contains explanations of the provisions of the design recommendations.

CHAPTER 1 — GENERAL PROVISIONS

101—Definition

Composite beams and girders are comprised of prefabricated beams and of a cast-in-place reinforced concrete slab so interconnected that the component elements act together as a unit.

102—Scope

These recommendations apply to buildings subjected primarily to static loads. Structures containing prefabricated beams made of precast reinforced concrete, precast prestressed concrete, or either rolled or built-up steel-sections are included.*

The direction and coordination of the efforts leading to this progress report were assigned to Philip P. Page, Jr. The committee wishes to acknowledge his efforts, which were primarily responsible for the prompt completion of the report.

*Guides for the design of timber-concrete slabs and T-beams may be found in technical literature; see, for example, *Timber Design and Construction Handbook*, F. W. Dodge Corp., New York, 1956, pp. 214-229.

103 Design of slab

The slab may be designed as either a one-way or two-way slab in accordance with "Building Code Requirements for Reinforced Concrete (ACI 318-56)." In the design of the slab, the stresses caused by composite action may be neglected.

104—Design of prefabricated beams

104.1 Precast reinforced concrete beams — Reinforced concrete beams should be designed according to "Building Code Requirements for Reinforced Concrete (ACI 318-56)" except as otherwise stated in Chapter 2 of these recommendations.

104.2 Precast prestressed concrete beams — Prestressed concrete beams should be designed according to "Tentative Recommendations for Prestressed Concrete" (ACI-ASCE Committee 323, Jan. 1958)[1] except as otherwise stated in Chapter 3 of these recommendations.

104.3 Steel beams — Steel beams should be designed according to "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings" (AISC 1949) except as otherwise stated in Chapter 4 of these recommendations.*

105—Effective width of flange

105.1 Flange on both sides of beam — For composite T-beams having the slab on both sides of the prefabricated beam, the effective width of the concrete flange should not exceed one-fourth of the span length of the beam, and the overhanging width on either side of the prefabricated beam should not exceed eight times the thickness of the slab nor one-half the clear distance to the next beam.

105.2 Flange on one side of beam — For beams having the slab on one side only, the effective overhanging flange width should not exceed 1/12 of the span length of the beam, nor six times the slab thickness nor one-half the clear distance to the next beam.

106—Mixed construction

The use of noncomposite beams in systems using composite beams is permissible provided that it is consistent with the deformational characteristics of the structure.

107—Deflections

107.1 Live load deflections — Live load deflections should be computed on the basis of the moment of inertia of the transformed composite section using the full value of the modulus of elasticity E_c.

107.2 Dead load deflections

1. For beams shored during construction, the dead load deflections should be computed on the basis of the moment of inertia of the transformed composite section using one-half the value of the modulus of elasticity E_c.

2. For beams not shored during construction, the dead load deflections should be computed on the basis of the moment of inertia of the prefabricated beam alone except that deflections due to dead loads applied after the concrete slab has attained 75 percent of the specified 28-day strength should be computed according to Section 107.2.1.

108—Continuity

108.1 Determination of moments, shears, and thrusts — Moments, shears, and thrusts produced by external loads should be determined by elastic analysis. For the purposes of such analysis, the moment of inertia of the gross composite section may be used throughout the length of the beam.

108.2 Sections resisting negative moments

1. In negative moment regions of continuous or cantilever beams, the bending moment may be assigned either to the prefabricated section alone or to the composite section composed of the prefabricated section and the slab reinforcement. Such assignment shall be consistent with the design of shear connection (Section 108.3).

2. When the slab is continuous at the supports of beams, reinforcement should be provided sufficient to prevent excessive cracking of the slab.

108.3 Shear connection in regions of negative moments

1. When the negative moments are assigned to the prefabricated section alone, shear connection between the prefabricated beam and the slab need not be provided in the regions of negative moments.

2. When the negative moments are assigned to the composite section, shear connection must be provided throughout the full length of the beam.

109—Construction methods

109.1 Shoring — When shores are used, they should be kept in place until the cast-in-place concrete has attained 75 percent of the specified 28-day strength.

109.2 Camber — Necessary provisions should be taken in the design and construction to prevent excessive dishing of the slab in beams built with shores and excessive thickening of the slab in beams built without shores.

109.3 Treatment of beam surfaces — Surfaces of prefabricated beams in contact with the slab should be cleaned of any foreign or loose material before casting the slab. It is preferable to leave the contact surfaces of steel beams unpainted.

*The use of plastic design for composite construction will be the subject for future recommendations of ACI-ASCE Committee 333.

CHAPTER 2 — SLAB ON PRECAST REINFORCED CONCRETE BEAMS

201—Allowable stresses, moduli of elasticity, and load factors

201.1 Allowable stresses — Allowable stresses for reinforcing steel and concrete specified in "Building Code Requirements for Reinforced Concrete (ACI 318-56)" are recommended for the design of composite beams. In structures composed of elements with different concrete strength, the allowable stresses in each portion should be governed by the concrete strength of the portion under consideration.

201.2 Tension in concrete — The tensile resistance of concrete should be neglected.

201.3 Moduli of elasticity — The following values for the moduli of elasticity should be used:

1. Steel: $E_s = 30{,}000{,}000$ psi
2. Concrete: $E_c = 1000\ f_c'$ where f_c' is the 28-day compressive strength of the concrete under consideration.

201.4 Load factors — For designs based on Section 203, load factors given in the Appendix to "Building Code Requirements for Reinforced Concrete (ACI 318-56)" are recommended.

202—Determination of flexural stresses

202.1 Design method — The design of reinforced concrete members may be made with reference to allowable stresses, working loads, and the accepted straight line theory of flexure except in designs based on Section 203.

202.2 Loading conditions

1. For unshored construction the dead load of the precast beam and all other loads applied prior to the concrete slab attaining 75 percent of its specified 28-day strength should be assumed as carried by the precast beam alone. Live loads and dead loads applied after the concrete has attained 75 percent of its specified 28-day strength should be assumed as carried by the composite section.

2. For adequately shored construction all loads should be assumed as carried by the composite section.

202.3 Deformational stresses — Deformational stresses, including the effects of creep, shrinkage, and temperature, need not be considered except in unusual cases.

203—Determination of ultimate flexural strength

203.1 Design method — Ultimate strength of a composite section may be computed in the same manner as the ultimate strength of an integral member of the same shape following the procedure outlined in the Appendix to "Building Code Requirements for Reinforced Concrete (ACI 318-56)." In computing the ultimate strength of a section, no distinction should be made between shored and unshored beams.

203.2 Limitations — For beams designed on the basis of ultimate strength and built without shores, the effective depth of the composite section used in the computation of the ultimate moment should not exceed:

$$d_c = \left(1.15 + 0.25\,\frac{M_L}{M_D}\right) d_p$$

where d_c = effective depth of the tension reinforcement in the composite section
M_L = moment produced by live load and superimposed dead load
M_D = moment produced by dead load prior to the concrete attaining 75 percent of specified 28-day strength
d_p = effective depth of the tension reinforcement in the precast section

When the specified yield point of the tension reinforcement exceeds 40,000 psi, beams designed on the basis of ultimate strength should always be built with shores unless provisions are made to prevent excessive tensile cracking.

203.3 Construction loads — The precast beam alone should be investigated to assure that the loads applied before the concrete has attained 75 percent of its specified 28-day strength do not cause moment in excess of 1/1.8 times the ultimate moment capacity of the precast section.

204—Determination of shear

204.1 Reinforcement of the web — Web reinforcement should be designed in the same manner as for an integral T-beam of the same shape. All stirrups should be extended into the cast-in-place slabs.

204.2 Horizontal shear

1. The shear at any point along the contact surface may be computed as:

$$v = \frac{VQ}{I}$$

where v = horizontal shear per unit length of beam
V = total external shear at the section considered caused by both live and dead loads
Q = statical moment of the transformed area on one side of the contact surface about the neutral axis of the composite section
I = moment of inertia of the transformed composite section neglecting the tensile resistance of concrete

2. If the horizontal shear, v, exceeds the capacity of bond as recommended in Section 205.3, shear keys should be provided throughout the length of the member. Keys should be proportioned according to the concrete strength of each component of the composite member.

205—Shear connection

205.1 Shear along contact surface — Shear may be transferred along the contact surface by bond or shear keys. It should be assumed in the design that the entire shear is transferred either by bond or by shear keys.

205.2 Vertical ties — Mechanical anchorage in the form of vertical ties to prevent separation of the component elements in the direction normal to the contact surfaces should be provided. Spacing of such ties should not exceed four times the thickness of the slab nor 24 in. A minimum cross-sectional area of the ties in each foot of span of 0.15 percent of the contact area but not less than 0.20 sq in. is recommended. It is preferable to provide all ties in the form of extended stirrups.

205.3 Capacity of bond

1. The following values are recommended for the allowable bond stress at the contact surfaces:

(a) When minimum steel tie requirements of Section 205.2 are followed and the contact surface of the precast element is smooth (a smooth surface is one which has been cast against a form, troweled, or floated .... 40 psi

(b) When minimum steel tie requirements of Section 205.2 are followed and the contact surface on the precast element is rough .... 160 psi

(c) When additional vertical ties are used, the allowable bond stress on a rough surface may be increased at the rate of 75 psi for each additional area of steel ties equal to 1 percent of the contact area.

2. The capacity of bond at ultimate load may be taken as twice the values recommended in Section 205.3.1.

CHAPTER 3 — SLAB ON PRECAST PRESTRESSED CONCRETE BEAMS

301—General

Composite structures consisting of precast or cast-in-place slabs resting on prestressed concrete beams may be designed in accordance with "Tentative Recommendations for Prestressed Concrete" (ACI-ASCE Committee 323, Jan. 1958),[1] except that it is recommended to design the shear connection according to the provisions of Sections 204 and 205 of these recommendations.

CHAPTER 4 — SLAB ON STEEL BEAMS

401—Allowable stresses and moduli of elasticity

401.1 Allowable stresses

1. The allowable stresses for steel, except for reinforcing bars, specified in "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings" (AISC 1949) are recommended for the design of composite beams.

2. The allowable stresses for concrete and for reinforcing bars specified in "Building Code Requirements for Reinforced Concrete (ACI 318-56)" are recommended for the design of composite beams.

401.2 Tension in concrete — The tensile resistance of concrete should be neglected.

401.3 Moduli of elasticity — The following values of the moduli of elasticity should be used:

1. Steel: $E_s = 30{,}000{,}000$ psi
2. Concrete: $E_c = 1000\, f_c'$ where f_c' is the 28-day compressive strength of the concrete

402—Determination of flexural stresses

402.1 Design method

1. Flexural stresses should be determined at the working load level on the basis of the moment of inertia of the transformed composite section.

2. The transformed area of the composite section should be computed on the basis of the modular ratio $n = E_s/E_c$.

3. For beams built without temporary supports and designed according to Section 402.2, the section modulus of the composite section used in computations should not exceed the value:

$$S_c = \left(1.35 + 0.35\,\frac{M_L}{M_D}\right) S_s$$

where S_c = section modulus of the tension flange of the composite beam
M_L = moment produced by live load and superimposed dead load
M_D = moment produced by dead load prior to the concrete attaining 75 percent of specified 28-day strength
S_s = section modulus of the tension flange of the steel beam alone

4. The steel beam alone should be investigated to assure that the actual stresses in the steel do not exceed the allowable values for steel (Section 401.1.1) before the concrete has attained 75 percent of its specified 28-day strength.

402.2 Loading conditions — It may be assumed in the stress computations (except for provision of Section 402.1.4) that all dead loads and live loads are resisted by the composite section whether or not temporary supports are used.

402.3 Deformational stresses — Deformational stresses, including the effects of creep, shrinkage, and temperature, need not be considered except in unusual cases.

403—Shear

403.1 Web stresses — The web of the steel beam should be capable of carrying the entire external shear without exceeding the allowable shearing stress.

403.2 Horizontal shear between slab and beam

1. The horizontal shear at the junction of the steel beam and the concrete slab or haunch should be computed as follows:

$$v = \frac{VQ}{I}$$

where v = horizontal shear per unit length of the beam
V = total external shear at the section considered caused by dead and live loads
Q = statical moment of the transformed area on one side of the contact surface about the neutral axis of the composite section
I = moment of inertia of the transformed composite section

In computing the section properties, the modulus of elasticity of concrete E_c given by Section 401.3.2 should be used.

2. The horizontal shear should be assumed to be transferred entirely by shear connectors except as noted in Section 403.2.3.

3. In beams fully encased in concrete, the entire horizontal shear may be assumed to be transferred by bond and friction provided that such beams are designed in accordance with Section 13 of "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings" (AISC 1949). No shear connectors or ties are required in such beams.

403.3 Concrete haunch — The horizontal shearing stresses in the concrete haunch between the steel beam and the concrete slab should not exceed the allowable shear stress for concrete. If the allowable shear stress is exceeded, the haunch should be provided with web reinforcement. The shear connectors may be considered as web reinforcement.

404—Shear connectors and ties

404.1 Allowable loads for connectors — The shear connectors should be designed on the basis of an allowable load given as follows:

1. Stud shear connectors:

$$q = 105\, d^2 \sqrt{f_c'} \text{ for } h/d \text{ equal or larger than } 4.2$$

$$q = 40\, hd \sqrt{f_c'} \text{ for } h/d \text{ smaller than } 4.2$$

where q = allowable load per one stud, lb
h = height of stud, in.
d = diameter of stud, in.
f_c' = 28-day compressive strength of concrete, psi

2. Spiral shear connectors:

$$q = 1900\, d \sqrt[4]{f_c'}$$

where q = allowable load per one pitch of spiral, lb
d = diameter of bar, in.

3. Channel shear connectors:

$$q = 90\, (h + 0.5\, t)\, w \sqrt{f_c'}$$

where q = allowable load per one channel, lb
w = length of channel, in.
t = thickness of web, in.
h = maximum thickness of flange, in.

4. For connectors other than the above, the allowable load should be developed from test data.

404.2 Spacing and cover of connectors

1. Shear connectors should be spaced along the beam in accordance with the formula

$$s = \frac{q}{v}$$

where s = spacing of shear connectors
q = allowable load per shear connector or on a group of connectors placed at the same section
v = horizontal shear per unit length of beam defined in Section 403.2.1

2. Shear connectors should have at least 1 in. concrete cover in all directions.

404.3 Ties

1. Mechanical ties should be provided between the slab and the beams to prevent separation. Such ties may be a part of the shear connectors.

2. The maximum spacing of ties should not exceed 24 in.

Explanations of Tentative Recommendations

The ACI-ASCE committee on composite construction has prepared tentative recommendations for the design of composite beams and girders in buildings. Since no code is available for the design of composite buildings, the release of these *tentative* recommendations is considered desirable even though important research projects on composite beams are now in progress. As further data are developed from the research projects, the committee expects to make appropriate modifications in its future reports.

In producing these recommendations the committee was faced with the problem of being consistent with existing codes for the materials involved. It felt that it was not the province of this committee to rewrite accepted practice but rather to correlate and draw upon it producing new concepts only where no accepted practice previously existed.

Lastly, the committee felt that any recommendations produced because of the need for a usable guide should be written in terms familiar to most designers. Thus, the recommendations have been limited generally to working stress design, even though several provisions are based on, or refer directly to, the conditions at ultimate strength.

The recommendations are limited to structures with concrete made from conventional hard rock aggregates. No data were available to the committee on the behavior of composite beams made with lightweight aggregate concretes.

Composite action

To explain the reasoning behind the committee's recommendations, it will be necessary first to review the action of a simply supported composite beam. After the prefabricated member has been erected, its lower flange is subjected to a tension which can be computed by general flexural theories (Fig. 1). Immediately after casting, the slab which is still plastic adds no strength but merely dead load increasing the lower flange stresses. After the slab has hardened, it becomes the top flange of the composite section; additional loads causing further tension in the lower flange are resisted by the entire composite section. However, this tension is less per *unit* load because the composite section is stronger. Since a *unit* load applied before commencement of composite action creates greater flange stresses than one applied after, attempts have been made to place as much of the load as possible on the composite section rather than on the prefabricated beam alone. The classic method was to shore the beam.

The discussion so far has dealt with stresses in the elastic range; that is, with stresses below the yield point rather than at the ultimate. The recommendations of ACI-ASCE Committee 323,[1] dealing with precast prestressed stems and cast-in-place slabs, state that the ultimate strength of the composite member is the same as that of a monolithic T-beam of equal dimensions. The fact that the composite beam may be built with or without shores does not influence the ultimate strength.

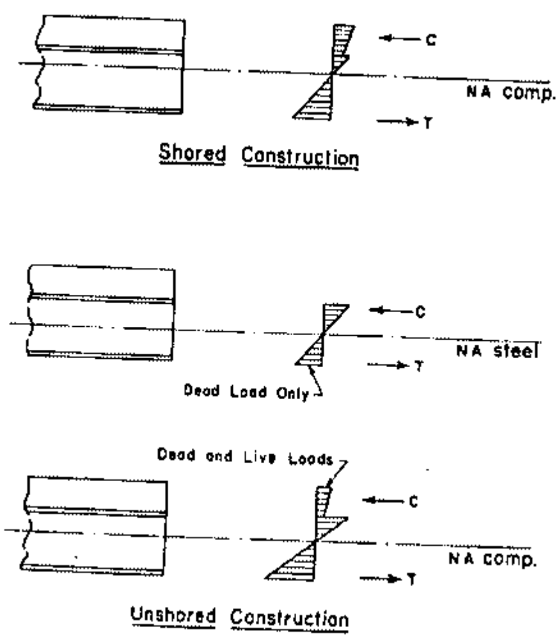

Fig. 1 — Stress distribution at working load

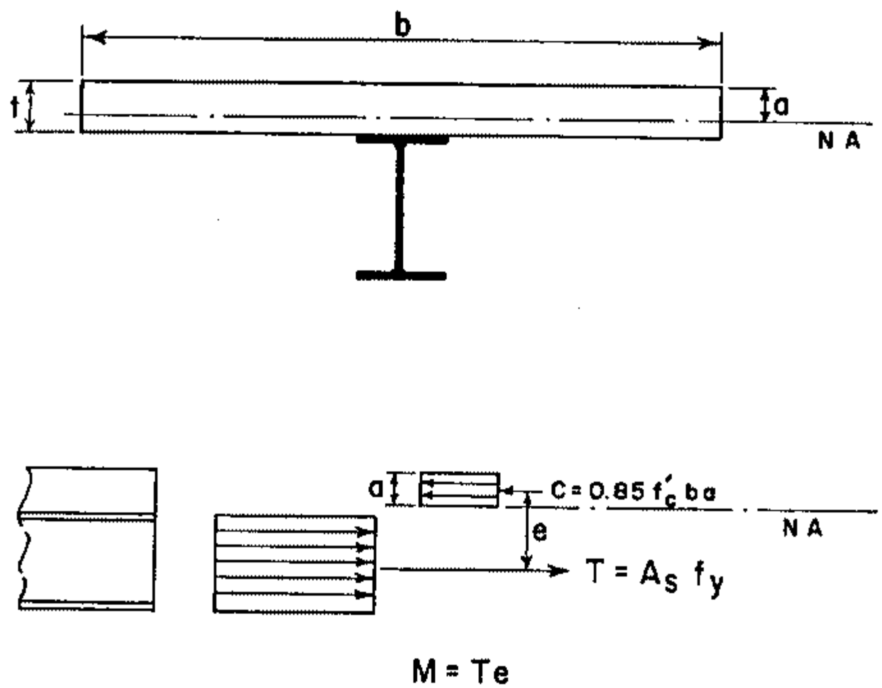

Fig. 2—Stress distribution at ultimate bending moment

A similar situation exists in a steel beam with a cast-in-place slab. Assuming that *at ultimate load* the neutral axis lies in the concrete slab, the stress distribution corresponding to the ultimate bending moment may be approximated as shown in Fig. 2.* The entire tensile force is carried by the steel beam stressed uniformly to its yield point. The entire compressive force is carried by the concrete above the neutral axis uniformly stressed to 85 percent of its cylinder strength. The location of the neutral axis is such that the total force in the concrete equals the total force in the steel and the ultimate bending moment is this total force times the distance between the centers of gravity of the two stress blocks. This fully plastic stress distribution is independent of the manner in which the stresses are induced into the beam.

Table 1 contains comparisons between computed and actual ultimate loads. Data for 15 beams with depths of steel section varying between 3 and 24 in. have been tabulated. The actual ultimate loads are those observed in the tests. The computed ultimate loads have been calculated from the reported properties of materials used in the tests. It may be noted that for all beams that failed in flexure the correlation between

*A similar theory can be developed for beams with the neutral axis below the top surface of the steel beam.

TABLE 1 — COMPARISON OF COMPUTED ULTIMATE LOADS WITH TEST DATA

Specimen	Reference	Depth of steel beam,* in.	Average† yield point, ksi	Size of slab,* in. x in.	Strength of concrete, ksi	Type of shear connector	Ultimate load: Computed, kips	Ultimate load: Test, kips	Ultimate load: Test/Computed	Mode of failure
Beams without shoring§										
Double T	2	18	37.9	131x6	3.93	stud	156.3	172.8	1.11	flexure
C	3	8	37.6	24x4.9	3.60	bar + loop	54.5	57.2	1.05	flexure
B24W	4	24	36.8	72x6.3	5.10	channel	116.9	111	0.95	did not fail
B21W	4	21	37.0	72x6.1	5.75	channel	96.0	79	0.82	horizontal shear
T1	5	3	37.4	18x1.8	5.10	channel	13.0	13.0	1.00	flexure
T2	5	3	37.4	18x1.7	1.82	channel	9.7**	10.2	1.05	flexure
T3	5	3	37.4	18x1.8	4.25	channel	12.2	8.7	0.71	horizontal shear
1	6	8	43.8	30x4.5	7.04	spiral	75.7	77.0	1.02	flexure
2	6	8	39.0	30x4.5	7.38	spiral	65.0	70.5	1.08	flexure
Beams with shoring or preloaded††										
D	3	8	37.9	24x4.9	3.30	bar + loop	45.8	44.1	0.96	flexure
E	3	8	38.0	24x4.8	3.61	bar + loop	54.5	54.9	1.01	flexure
B24S	4	24	36.1	72x6.2	5.67	channel	115.1	115	1.00	did not fail
B21S	4	21	37.6	72x6.3	6.53	channel	99.2	102	1.03	flexure
3	6	8	39.8	30x4.5	7.04	spiral	66.0	67.5	1.02	flexure
4	6	8	42.7	30x4.5	7.38	spiral	74.5	71.8	0.96	flexure

*Rounded-off values.
†Average weighted according to areas and corresponding yield points.
‡Dead load not included.
§Average test/computed for flexural failures:1.052.
**Neutral axis in the steel section; for all other specimens neutral axis in the slab.
††Average test/computed for flexural failures: 0.996.

the computed and observed ultimate loads is good: the difference between the two loads varies between —4 and +11 percent. The observed load is substantially smaller than the computed one only for Specimens B21W and T3 designed with extremely weak shear connections.

The specimens listed in Table 1 are divided into two groups: those built without temporary supports and those either shored or loaded with additional weights during construction. Using the same method of computation, the ultimate moment capacity was predicted with essentially equal accuracy for both shored and unshored beams. The ultimate flexural strength of a composite steel-concrete beam is independent of the construction method.

Based on a design stress of 20,000 psi and a yield stress of 33,000 psi, the factor of safety at first yielding of a noncomposite steel beam is 1.65. At ultimate strength the stress distribution is plastic and thus the factor of safety against failure is higher. For a symmetrical rolled section, the ultimate load is approximately 1.85 times the design load. On the other hand, for a composite beam designed on the assumption that the composite section takes all loads, the ultimate load is approximately 2.2 to 2.5 times the design load. The addition of a bottom cover plate does not change these values appreciably. If the composite beam were designed on the assumption that the steel beam alone resisted the dead loads, the ratio of ultimate to design load would be still higher.

As a result of these observations the committee makes the following recommendation:

> The design of composite beams made up of a prefabricated steel beam and a cast-in-place concrete slab may be based upon working stresses calculated by assigning all loads to the composite section regardless of construction method. That is, all loads may be considered in the design as resisted by the composite section even though shores are not employed during construction.

This recommendation is limited to composite sections with steel beams (Section 402). It is not recommended for the design of composite sections with either precast reinforced concrete beams or precast prestressed concrete beams.

For sections composed of a slab and a precast reinforced concrete beam, the committee recommends two alternate methods of design: a working stress design (Section 202) and an ultimate strength design (Section 203). The working stress procedure takes into account the construction method, while the ultimate strength procedure is independent of the method of construction. Both procedures follow principles well established in the design of reinforced concrete.

The ultimate strength of a composite concrete T-beam is independent of the method of construction. Thus, the load factors used in the ultimate strength design provide the necessary overload capacity regardless of the type of construction used. The recommendation of the ultimate strength design as an alternate method for reinforced concrete sections is warranted both by the availability of information and by the familiarity of the designers with such design.

The need for a working stress procedure is self-evident. However, in contrast to composite sections with steel beams, the reserve strength of a composite concrete beam beyond first yielding of the reinforcement can be small. Therefore, it is recommended that the construction method in the working stress design of concrete beams be considered.

Finally, the committee considered satisfactory the currently available method for the design of composite sections made with prestressed concrete beams (Chapter 3).

Construction considerations

Although the procedure recommended for the design of sections with steel beams provides adequate safety against failure of the composite beam, it does not limit either the erection stresses or the actual stresses at working load. To guard against possible damage to the steel section, Sections 402.1.3 and 402.1.4 place limits on the total stress and on the stresses in the steel beam prior to hardening of the slab.

In beams built with temporary supports the actual stresses exceed the computed values only as a result of volume changes of concrete. The increases in the governing tension in the steel section resulting from volume changes are usually small and may be safely neglected. On the other hand, in beams built without temporary supports, the actual stress may be substantially in excess of the computed value because the dead load is in reality resisted by the steel section alone, while the recommended procedure assigns it to the composite section. The limit on the section modulus of the composite beam (Section 402.1.3) places a maximum of 27,000 psi on the total stress caused by dead and live loads in an unshored beam. This provides safety against yielding comparable to that of a noncomposite beam.

To avoid damage to the steel section during construction, it is recommended in Section 402.1.4 to limit the stresses in the bare steel beam before hardening of the slab to the conventional allowable values for steel. This provision is particularly needed for unsymmetrical steel sections with light compression flanges.

The ultimate strength procedure for the design of sections with precast reinforced concrete beams provides adequate safety against failure of the composite beam but additional limitations are needed to guard against damage to the precast beam during erection and against excessive widening of the tension cracks at working loads. The provisions of Sections 203.2 and 203.3 serve this purpose.

Except for small changes caused by differential shrinkage, the stresses in a composite beam fully shored during construction are the same

as in a cast-in-place monolithic beam. In beams built without temporary supports, the weight of the precast beam and of the concrete slab is resisted by the precast section alone. However, the ultimate strength of the composite section is the same regardless of the construction procedure. Thus actual stresses caused by dead and live loads are higher if shores are not used. The limitation on the effective depth of the composite beam (Section 203.2) limits the total stress caused by dead and live loads to 75 percent of the specified yield point f_y of the tension reinforcement.

The limit of 0.75 f_y not only keeps the stresses well below the yield point but also prevents excessive tensile cracking of the beam provided that the yield point is only moderately high. The committee recommends that, unless provisions are made to prevent excessive widening of tensile cracks, composite beams designed on the basis of ultimate strength with the yield point of the steel in excess of 40,000 psi should not be constructed without the use of shores. When the yield point does not exceed 40,000 psi, the actual stresses in the tension reinforcement in a composite beam built without temporary supports will be no higher than the maximum values attainable in monolithic beams designed by the ultimate strength design procedure recommended in "Building Code Requirements for Reinforced Concrete (ACI 318-56)."

The provision of Section 203.3 guards against damage to the precast section during construction.

Shear connection

It has been assumed tacitly in the preceding discussion that the shear connection between the prefabricated beam and the cast-in-place slab is capable of developing the ultimate moment of the composite section. It may be observed in Fig. 2 that at ultimate load the shear connection must develop the fully plastic force C (or T) through horizontal shear in the length between the sections of zero and maximum bending moments. If the connection is able to yield and still develop the fully plastic or ultimate strength of the beam, the exact distribution of horizontal shear is not critical. On the other hand, if the connection is brittle or if its yielding causes a significant loss of composite action, the distribution of this horizontal shearing force has to be considered. Experimental studies are now in progress which are expected to supply the information needed for formulating an ultimate strength procedure for the design of the shear connection.

Another approach to the design of the shear connection makes use of the elastic horizontal shear formula. Such approach, based on experimental observations, was developed for the design of composite bridges[7] and was adapted for this report. It provides a connection capable of developing the fully plastic moment capacity of the composite beam.

TABLE 2—HORIZONTAL SHEAR FAILURES OF COMPOSITE CONCRETE-CONCRETE BEAMS

Specimen	Reference	Type of joint surface	Ties	a/d*	Horizontal shear strength, psi
BS-I	8	Smooth	#3— 6 in. on centers	3.0	350
BS-II	8	Smooth	#3—16 in. on centers	4.2	340
A2	9	Smooth	none	6.5	78
C2	9	Smooth	#4— 6 in. on centers	6.5	100
A3	9	Smooth	none	3.3	119
J	10	Smooth	none	8.0	122
BRS-I	8	Rough	#3— 6 in. on centers	3.0	450
BRS-II	8	Rough	#3—16 in. on centers	4.2	580
III-0.6-1.66	11	Rough	#3— 6 in. on centers	3.8	418

*Ratio of length of shear span to effective depth of tension reinforcement.

The AASHO procedure,[7] derived for steel-concrete beams, makes use of the useful capacity of mechanical connectors and of a variable factor of safety. The factor of safety is intended to furnish a connection capable of developing the ultimate flexural strength of the composite beam; it accounts for moving loads, for the different proportions of moments and shears resisted by the noncomposite and the composite sections, for the properties of the cross section and for the level of design stresses. The connection and the beam have to resist the same ultimate load. At working load, however, the beam is always resisting both the dead load and the live load while the connection may resist only the live load. In such case the factor of safety needed in the design of the connection is higher than that of the beam. If, however, the live and dead loads are distributed in the same manner along the beam, as is usually the case in building design, and if the assumption is made that all loads are resisted by the composite section, then the required factor of safety for shear connectors is equal to the factor of safety of the beam. The allowable values for various types of connectors may then be determined as the ratio of their strength to this constant factor of safety.

In the interest of simplicity, the committee recommends that the horizontal shear be computed from the elastic formula, using the vertical shear caused by *all* loads acting on the *beam* regardless whether shoring is used or omitted during construction (Section 204.2.1 and 403.2.1). It is important to note that the use of the *total* load in computing the horizontal shear is one of the basic assumptions in the derivation of the recommended design procedure.

Composite sections with steel beams designed as recommended by this committee usually have a factor of safety between 2.2 and 2.5. The useful strength of mechanical connectors, used on steel beams, may be determined from empirical formulas given in the AASHO specifications.[7] In using the AASHO formulas, the committee recognized their conservative nature in relation to static loading and, therefore, selected the safety factor of 2.0 as adequate when arriving at the recommended formulas for allowable loads (Section 404.1).*

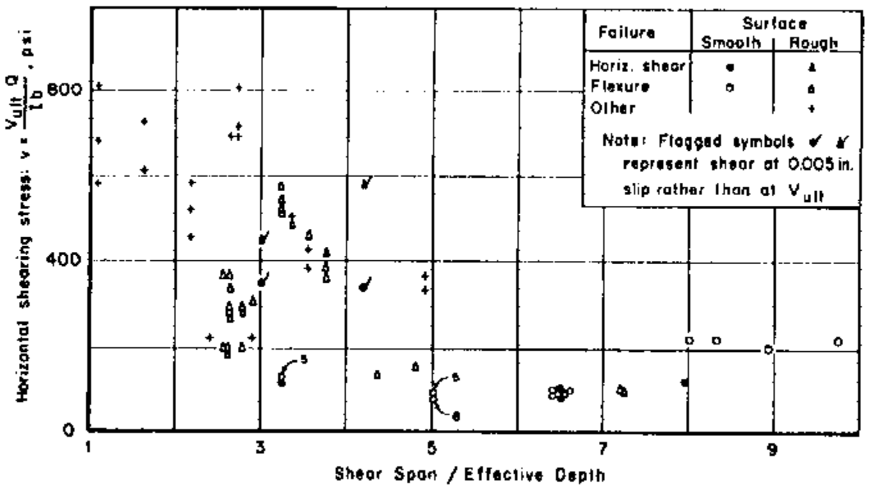

Fig. 3—Shear in composite concrete-concrete beams

The question of bond between the *steel* section and the *concrete* slab has always been a perplexing one. Tests have shown that it usually exists but no one can guarantee its duration. For this reason, the tentative recommendations give no credit to bond. Fully embedded beams constitute the only exception to this rule (Section 403.2.3).

For the transfer of shear between a *concrete* beam and a *concrete* slab the use of bond or of shear keys is recommended. Several experimental studies and experience have shown that bond in conjunction with steel ties provides a very effective and reliable connection. However, the tests have shown also that there is a practical limit to the capacity of the bond connection.

In the 78 tests of composite beams studied by the committee, nine beams failed in horizontal shear. The horizontal shear at which failure occurred (computed from the elastic formula), the type of surface, and a description of steel ties are given in Table 2 for all nine beams. The six specimens with a smooth surface failed at horizontal shearing stresses ranging from 78 to 350 psi, while the three specimens with a rough surface failed at horizontal shearing stresses between 418 and 580 psi. The remaining 69 specimens, which did not fail in horizontal shear, included both rough and smooth surfaces. The maximum horizontal shears obtained in the tests of all 78 beams are shown in Fig. 3.

The data for specimens with smooth contact surface cover well the range of moderate to large slendernesses (ratios of shear span *a* to ef-

*Preliminary data from an investigation now in progress at Lehigh University have indicated that ultimate strength of a composite beam might be developed with a substantially smaller number of connectors than that obtained by the procedure recommended at present.

fective depth of tension reinforcement d ranging from 3 to 9.75). It may be noted in Fig. 3 and in Table 2 that the data representing horizontal shear failures suggest a possible decrease of strength with increasing slenderness. The data for specimens with rough contact surface are limited to the range of small and moderate slendernesses (a/d ranging from 1 to 5); only two specimens with rough surfaces were slender ($a/d = 7.25$) and these did not fail. Furthermore, all three specimens with rough surface that failed in horizontal shear had a/d ratios less than 4.25. It is hoped that an investigation now in progress will provide information on slender beams with rough surfaces.

In view of the evidence discussed briefly in the preceding paragraph the committee considered it advisable at this time to base the recommended bond values for smooth surfaces on the strength of 80 psi, and for rough surfaces on the strength of 400 psi. The factor of safety for beams designed according to these recommendations ranges from 2.0 up. In view of the uncertainty concerning the bond strength in the range of larger a/d values, a factor of safety of 2.5 was used to obtain the allowable bond stresses (Section 205.3.1) for the rough surfaces. For smooth surfaces, better covered by the available test information, a factor of safety of 2.0 was selected.

The committee strongly recommends the use of steel ties crossing the contact area in all composite concrete T-beams. For light concrete joists and for slabs with precast beams completely embedded on three sides, this recommendation may be too severe. The committee considered such construction outside the scope of the tentative recommendations.

The minimum amount of ties recommended by the committee is based on the value suggested by Committee 323.[1] A small increase is suggested in bond values for beams with ties in excess of the minimum. This latter allowance is based on the results of recent tests of push-off specimens.[8]

In view of a lack of experimental data, the committee is not prepared to make any detailed recommendations concerning the design of shear keys.

Deflection

As a general rule, composite beams which are shallower and employed on longer spans than conventional beams are more deflection sensitive. The use of shores reduces the total deflection but creates construction problems. If the shore is properly placed, the deflection is a function of the load, span, and stiffness of the composite section. If the shore is driven too tight, the energy stored in the prefabricated beam will produce an additional effective load which will increase the deflection of the slab. Furthermore, if the floor is screeded level and the beam is shored high, the slab will be made too thin at midspan with resulting loss of strength.

In the case of an unshored beam, the deflection must be computed separately for the dead load—resisted by the prefabricated beam alone—and for the live load—resisted by the composite section. The total deflection is greater than for a beam built with shores but the floor may be screeded level at its final elevation. The beam itself must be checked for deflection under its weight plus that of the wet concrete and formwork to assure that excessive thickening of the slab does not occur at midspan. The tentative recommendations carry warnings about these potential problems of deflection.

Other factors

The question of deformational stresses was investigated in some detail. Such stresses are always present in composite members at working load stress levels. However, they are wholly internally balanced and hence have no effect upon the ultimate capacity. The committee felt that in ordinary buildings deformational stresses may be safely ignored except in unusual cases.

Creep of concrete affects the deflections of a composite beam. In cases such as beams carrying heavy masonry partitions, the long-time effect could cause serious cracking of the partitions even though the strength of the beam would be unaffected. The simplest manner of correcting the deflection computation for this long-time effect is to reduce the value of the modulus of elasticity of the concrete. It is recommended to use one-half the short time value of the concrete modulus to account for creep deflections.

The tentative recommendations contain a few guide lines for continuous design. The recommendations concerning the elastic properties of composite beams for the purpose of frame analysis follow the current practice for reinforced concrete T-beams. On the other hand, the recommendations for the design of the negative moment sections are in accord with the current bridge practice.

Finally, the recommendations concerning the effective slab width are based on the current practices both for composite beams and for reinforced concrete T-beams.

The recommendations have been prepared as a result of a need for a guide. Composite construction is being employed more and more as its advantages are being demonstrated.

The task of the committee is by no means finished. Its further work and direction of its investigations will be influenced by the reaction of the engineering profession to these tentative recommendations.

REFERENCES

1. ACI-ASCE Committee 323, "Tentative Recommendation for Prestressed Concrete," ACI Journal, V. 29, No. 7, Jan. 1958 (*Proceedings* V. 54), pp. 545-578.

2. Thürlimann, B., "Composite Beams with Stud Shear Connectors," *Bulletin* No. 174, Highway Research Board, 1958, pp. 18-38.

3. Graf, O., and Brenner, E., "Versuche an Verbundträgern," *Berichte* No. 19, Deutschen Ausschusses für Stahlbau, 1956, 82 pp.

4. Viest, I. M.; Siess, C. P.; Appleton, J. H.; and Newmark, N. M., "Full Scale Tests of Channel Shear Connectors and Composite T-Beams," *Bulletin* No. 405, University of Illinois Engineering Experiment Station, 1952, 155 pp.

5. Siess, C. P.; Viest, I. M.; and Newmark, N. M., "Small-Scale Tests of Shear Connectors and Composite T-Beams," *Bulletin* No. 396, University of Illinois Engineering Experiment Station, 1952, 133 pp.

6. Mains, R. M., "Report of Tests of Composite Steel-Concrete Beams," Unpublished report of the Fritz Engineering Laboratory, Lehigh University, 1943, 13 pp.

7. *Standard Specifications for Highway Bridges*, American Association of State Highway Officials, 7th Edition, Division I, Section 9, 1957, pp. 105-109.

8. Hanson, N. W., "Horizontal Shear Connections," *Journal of the Research and Development Laboratories*, Portland Cement Association, V. 2, No. 2, May 1960, pp. 38-58.

9. Ozell, A. M., and Cochran, J. W., "Behavior of Composite Lintel Beams in Bending," *Journal*, Prestressed Concrete Institute, V. 1, No. 1, May 1956, pp. 38-48.

10. Revesz, S., "Behavior of Composite T-Beams with Prestressed and Unprestressed Reinforcement," ACI JOURNAL, V. 24, No. 6, Feb. 1953 (*Proceedings* V.49), pp. 585-592.

11. Kaar, P. H.; Kriz, L. B.; and Hognestad, E., "Pilot Tests of Continuous Girders," *Journal of the Research and Development Laboratories*, Portland Cement Association, V. 2, No. 2, May 1960, pp. 21-37.

This report was submitted to letter ballot of the committee which consists of 18 members; 17 members returned their ballots, of whom 16 have voted affirmatively and 1 negatively.

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