R 614

Technical Report

STRUCTURAL BEHAVIOR OF REINFORCED CONCRETE BEAMS MADE WITH FAST-FIX 1 CEMENT

February 1969

MAR 1 3 1969

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STRUCTURAL BEHAVIOR OF REINFORCED CONCRETE BEAMS MADE WITH FAST-FIX 1 CEMENT

Technical Report R-614

YF 38.536.006.01.002

by

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ABSTRACT

The objective was to investigate the predictability and effectiveness of reinforced concrete beams made with Fast-Fix 1 cement. Fourteen simply supported beams were designed with the beam parameters systematically varied in order to observe a wide range of structural response when the beams were loaded to failure. The ultimate strength and mode of failure of test specimens were compared with predicted results based on the ultimate strength design assumptions of the ACI Code. In addition, "elastic" behavior in terms of beam stiffness was compared to calculated stiffnesses based on the working stress design assumptions of the ACI Code. The performance of two Fast-Fix beams was also compared with the test results of "duplicate" beams made with portland cement. Evaluation of the Fast-Fix beams indicated that their ultimate capacity was safely predicted by the ultimate strength theory and that their structural effectiveness was equivalent to beams made with portland cement.

CONTENTS

INTRODUCTION

Objective

The main objective of this study was to investigate the structural behavior of Fast-Fix 1 reinforced concrete beams, in particular their ultimate strength, in order to determine the effectiveness and predictability of such structural members. In addition, a study of "elastic" behavior in terms of beam stiffness was considered.

Background

Fast-Fix 1 cement was developed by the U.S. Air Force to provide a fast-setting, high early-strength concrete to enable rapid repair of bomb-damaged runways [1] Field experiments on slabs and crater repairs indicated that Fast-Fix performed exceedingly well; consequently, enthusiastic recommendations were made for further study into structural behavior of Fast-Fix concrete members.[2] These recommendations served as the stimulus for this investigation of the structural behavior of reinforced concrete beams made with Fast-Fix 1 cement.

Scope

To meet the objective of this investigation, 14 simply supported, reinforced Fast-Fix concrete beams were designed and loaded to failure. The ACI Building Code,[3] which was used as a design guide, was also used as a standard of comparison for determining the effectiveness and predictability of the Fast-Fix beams. Furthermore, the behavior of two Fast-Fix beams was compared with the results of tests on "duplicate"* beams made of portland cement concrete.[4]

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* That is, beams with geometry and reinforcement identical to that of the Fast-Fix beams but differing in the constituents of the concrete and therefore in such parameters as compressive strength.

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Specifically, ultimate strength response was investigated for a spectrum of beam failure modes. This spectrum included: (1) flexure failures—tension, balanced, and compression; and (2) shear failures—diagonal tension and shear compression. It is recognized that the ACI Code requirements imply designing for tension failure of a beam. The ductile characteristic of such a failure will generally provide adequate warning, in contrast to the suddenness of shear and compression failures. Since the validity of ACI Code assumptions for ultimate strength design was not known a priori, a knowledge of a failure spectrum was required to insure safe designing. In short, one must know when undesirable modes of failure are likely to occur if he is to avoid them. To this end, Fast-Fix beams were tested to compare the observed modes of failures and ultimate strengths with the ACI predictions.

The investigation of elastic behavior was similar in that the measured beam stiffnesses were compared with computed beam stiffnesses based on working stress design assumptions of the ACI Code. When applicable, a direct comparison was made with beam stiffness of duplicate portland cement concrete beams. Since the Fast-Fix beams were designed to display a spectrum of failure modes, the beam configurations inherently represented a wide range of beam stiffnesses, and thereby provided a good sample set to compare measured and calculated beam stiffness.

To achieve a failure spectrum, the following beam parameters were varied:

1. Compressive strength of concrete

2. Area and placement of longitudinal reinforcing steel

3. Shear span

EXPERIMENTAL WORK

Test Beams

Fourteen simply supported beams were statically loaded to failure. These 14 beams constituted seven pairs of companion specimens with the partners of each companion pair differing mainly in the compressive strength of the concrete. Of the seven pairs, three pairs were designed to fail in flexure and four pairs were designed to fail in shear. The three pairs intended for flexure failure were so designed that each pair would produce one of the three different types of flexural failure, namely, compression, balanced, and tension. Companion beams for shear failure were so loaded that they reflected the influence of the shear-span-to-depth ratio on shear resistance.

All beams tested were symmetrically loaded and had a constant 3.13 x 6.5-inch cross section, the same overall length (76 inches), and the same span (72 inches). To permit meaningful comparison, these dimensions were exactly the same as those used in a previous investigation[4] of the behavior of reinforced concrete beams made with portland cement.

In considering the values of compressive strength to be varied, there were two limitations; the first was the upper limit of compressive strength that could be achieved, and the second was the lower limit of compressive strength that a concrete must have to be of structural quality. The upper practical limit of compressive strength was found to be around 3,000 psi.[5] The value of 2,500 psi is implied by the ACI Code requirements as the lower limit of compressive strength for structural concrete. Compressive strengths of 2,500 and 3,000 psi could not have been produced without overlap, even with the best quality control. Accordingly, the two values of the nominal compressive strengths that were used in this investigation were 2,000 psi and 3,000 psi.

The two mixes used had the following properties:

	Mix 1	Mix 2
Nominal compressive strength (psi)	2,000	3,000
Proportions (by weight) cement:sand:crushed rock	1.0:1.8:1.2	1.0:0.67:0.44
Water-cement ratio (by weight)	0.4	0.3

Figure 1 gives a description of the test beam, and Table 1 lists the intended values of the beam parameters together with the anticipated failure modes for the seven pairs of companion beams.

Material Properties

Fast-Fix Concrete. Constituents of the concrete were Fast-Fix 1 cement, sand, crushed gravel (3/8-inch maximum size), and water. The characteristics and properties of these components are identical with those described in Reference 5. The two mix proportions used are given in Figure 2 along with typical compressive stress—strain curves of 6 x 12-inch plain concrete cylinders. Table 2 lists the time schedule and characteristics of the concrete for each test beam.
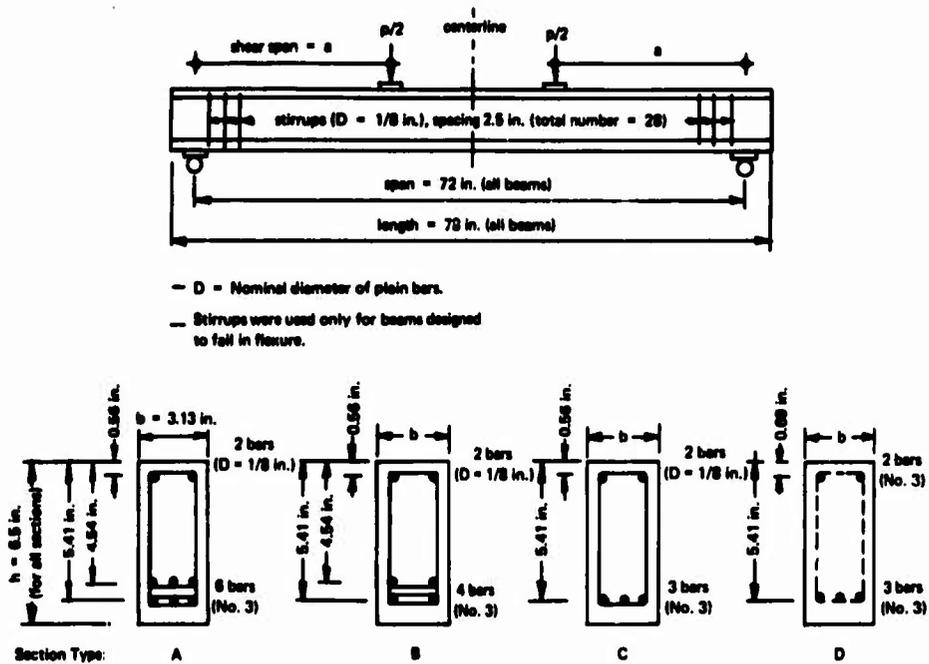
Figure 1. Details of test beams.
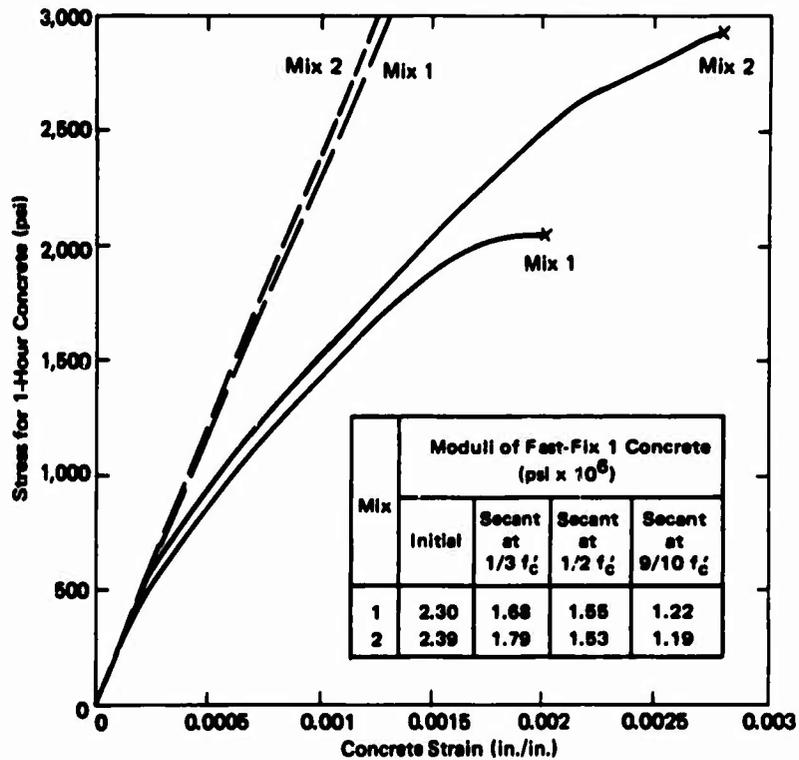
Figure 2. Typical stress—strain curves for Fast-Fix mix 1 and mix 2.

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Table 1. Summary of Planned Tests

Anticipated Failure Mode	No. of Test Beams	Shear Span, a* (in.)	Cross Section* Type for Nominal f_c' of—		Stirrups
			3,000 (psi)	2,000 (psi)	
Flexure:					
Compression	2	36**	A	A	1/8-in.-diam
Balanced	2	36**	B	C	bars uniformly
Tension	2	36**	D	D	spaced at 2.5 in.
Shear	2	27	D	D	none
	2	20	D	D	
	2	15	D	D	
	2	10	D	D	

* Shown in Figure 1.
** Midpoint loading.

Reinforcing Steel. The reinforcing steel was No. 3 deformed bars and 1/8-inch-diameter round bars. The average yield point for the No. 3 bars was 46 ksi, and the yield strength of 1/8-inch bars was 77 ksi (0.2% offset). Typical stress—strain curves are shown in Figure 3.

Forms

Beam forms were made of 1/4-inch-thick steel and were 78 inches long, 6.5 inches deep, and 3-1/3 inches wide (inside dimensions). The beam forms as well as the 6 x 12-inch control-cylinder molds were cleaned and oiled prior to each casting operation.

Loading Equipment

The beam was loaded by means of a hand-operated hydraulic pump driving a remote hydraulic ram. The ram was connected to a rigid steel frame below which the test beam was centered so that the ram's line of action was perpendicular to the beam's top surface and passed through the beam's center of gravity.

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Table 2. Time Schedule and Characteristics of Concrete

| Beam Designation | Time Table | | | | | Cement (sacks*/yd³) | Water-Cement Ratio (by wt) | Water (gal/sack*) | Slump (in.) | Concrete Strength (psi) | | | Density (lb/ft³) |
| | Date | Pour at | Set at | Test | | | | | | f'c | | f'sp | |
				Begin†	End‡					Begin†	End‡		
36FC3.0	7/11	0850	0902	1002	1045	15.4	0.30	3.6	10.75	2,985	3,030	264	138
36FC2.0	6/24	1301	1313	1413	1515	8.75	0.40	4.8	10.50	1,982	1,964	209	142
36FB2.9	7/24	0855	0909	1009	1145	15.4	0.30	3.6	10.75	2,760	3,078	275	136
36FB1.8	7/23	0849	0901	1001	1105	8.75	0.39	4.7	10.75	1,892	1,732	202	142
36FT2.8	7/10	1247	1259	1359	1505	15.0	0.31	3.7	11.50	2,760	2,766	251	135
36FT2.0	6/21	1300	1310	1410	1520	8.75	0.40	4.8	10	1,952	—	212	—
27FT2.7	7/18	0900	0916	1016	1153	15.4	0.30	3.6	11	2,709	2,771	264	137
27DT2.0	7/17	0900	0910	1010	1040	8.75	0.39	4.7	10	2,032	1,875	208	142
20DT3.0	7/15	0900	0913	1013	1110	15.4	0.30	3.6	11	2,778	3,198	257	137
20DT2.2	6/28	1314	1326	1426	1605	8.75	0.40	4.8	10.25	2,220	2,240	214	143
15DT3.1	7/22	0855	0909	1009	1145	15.4	0.30	3.6	11.25	3,066	3,088	271	137
15DT1.9	7/19	0900	0912	1012	1110	8.75	0.40	4.8	10.25	1,925	1,810	210	142
10SC2.9	7/16	0850	0903	1003	1150	15.4	0.30	3.6	11	2,945	2,920	244	136
10SC2.2	7/1	1249	1300	1400	1625	8.75	0.40	4.8	9.75	2,250	2,094	219	141

* One sack of Fast-Fix 1 cement weighs 100 pounds.

† Beginning of beam test.

‡ End of beam test.
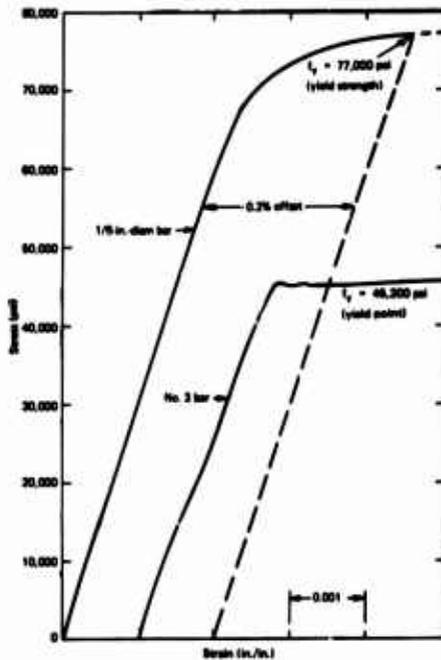
Figure 3. Typical tensile stress—strain curves for No. 3 deformed bars and 1/8-inch-diameter plain bars.

For midpoint loadings, the force provided by ram was transmitted to the beam through a 20-kip-capacity load cell and the assembly shown in Figure 4a. For two-point loadings, the same procedure was used with the addition of a stiffened I-beam to split the load symmetrically for the various shear spans as indicated in Figure 4b.

The applied load was measured with a Baldwin strain indicator in conjunction with the load cell. Centerline deflections were measured with an Ames dial gage (0.001 inch/division).

Control cylinders were loaded with a 400,000-pound-capacity universal testing machine. Compressive tests were conducted at the rate of 35 psi/sec (ASTM C 39-66), while split cylinder tests were performed at the rate of 110 psi/min (ASTM C 496-66).

Test Procedure and Techniques

Beam Fabrication. Reinforcing steel was held securely in place by light tie wire and 1/2-inch hydrostone cubes.

Mixing. The concrete constituents were mixed in a 6-ft^3-capacity mixer with a horizontal revolving drum. The mixing procedure was in accordance with the recommendations outlined in Reference 5.

Casting. The beam was cast while being continually vibrated with a form vibrator. Next, six 6 x 12-inch control cylinders were placed in two lifts with each lift vibrated by an internal vibrator. A typical casting operation is shown in Figure 5. Slump tests were conducted simultaneously with the placing operation.

Finishing. The beam and control cylinders were screeded and troweled to a smooth finish with a steel trowel. Set time was assumed when the trowel could no longer shape the concrete; these set times were recorded for each batch.

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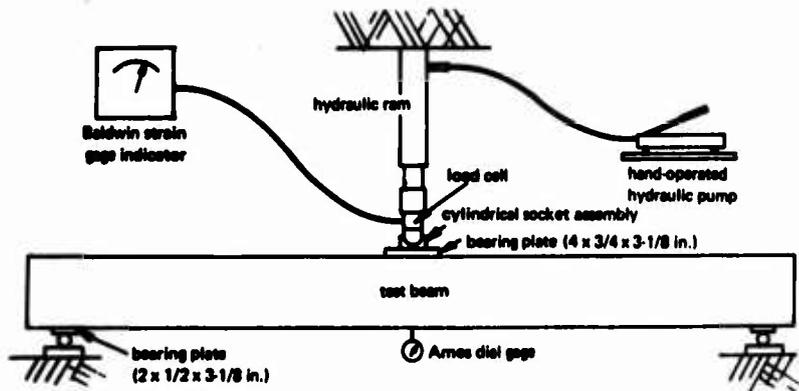
(a) Midpoint loading.
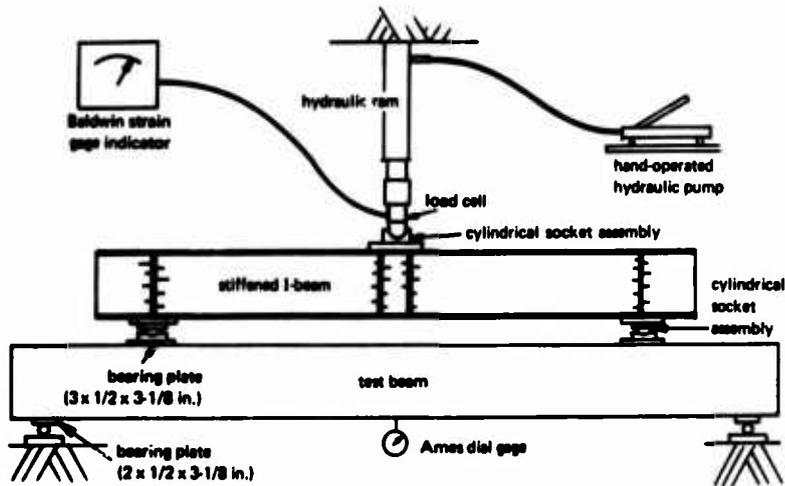
(b) Split-point loading.

Figure 4. Diagram of test setup.

Testing of Control Cylinders. Fifteen minutes beyond time of setting, the cylinders were removed from the molds and allowed to air dry until testing. Due to time limitations, standard caps were not made for the control cylinders; however, stiff cardboard disks were used in an effort to distribute the stress evenly. At the end of 1 hour from the time the concrete set, two cylinders were tested in compression, followed by two split cylinder tests. At the end of the beam test, the remaining two cylinders were tested in compression. The average compressive strength of the four compressive tests was used as the f_c' for beam calculations. In conjunction with compressive tests, load-deformation curves were obtained by means of a microformer-actuated compressometer connected to an x - y plotter.

Figure 5. Typical casting operation.

Testing of Beams. The reinforced concrete beams were removed from their forms 15 minutes after time of setting and were placed in the loading apparatus and made ready for testing. At 1 hour from time of concrete setting, the beam was loaded in increments of approximately 400 pounds to failure, or until the beam exhibited a ductile response, in which case, centerline deflections in increments of 0.05 inch were induced until failure. Following each deflection, total load and centerline deflection were recorded and the beam was inspected for cracks. Cracks were marked with the corresponding load increment number, thus giving a record of crack propagation. When failure occurred, the mode of failure and ultimate strength were recorded, after which the load was removed and permanent set was recorded. After photographs of the beam had been taken and developed, the concrete was chiseled away from both ends of each beam and the exact depths to the reinforcing steel were measured. All measured properties of the test beams are given in Table 3.

Table 3. Measured Dimensions and Reinforcement of Test Beams

Beam Designation	Shear Span, a (in.)	Effective Depth, d (in.)	Top Steel Depth, d' (in.)	Width, b (in.)	Reinforcing Steel									Stirrups
					Tension				Compression					
					Bars		A_s (in.2)	ρ	f_y (ksi)	Bars		A'_s (in.2)	ρ'	f_y (ksi)	
					Quantity	Size No.				Quantity	Size				
36FC3.0	36.0	5.02	0.55	3.13	6	3	0.66	0.0420	46.6	2	1/8 in.	0.025	0.00159	77.0	1/8-in.-diam bars uniformly spaced at 2.5 in. (f_y = 77 psi)
36FC2.0	36.0	5.08	0.67	3.13	6	3	0.66	0.0415	45.4	2	1/8 in.	0.025	0.00157	77.0	
36FB2.9	36.0	5.16	0.80	3.14	4	3	0.44	0.0272	46.3	2	1/8 in.	0.025	0.00154	77.0	
36FB1.8	36.0	5.60	0.72	3.14	3	3	0.33	0.0188	47.0	2	1/8 in.	0.025	0.00142	77.0	
36FT2.8	36.0	5.46	0.75	3.13	3	3	0.33	0.0193	46.6	2	No. 3	0.22	0.0129	44.9	none
36FT2.0	36.0	5.57	0.75	3.13	3	3	0.33	0.0189	45.4	2	No. 3	0.22	0.0126	46.0	
27FT2.7	27.0	5.50	0.70	3.13	3	3	0.33	0.0192	45.6	2	No. 3	0.22	0.0128	47.0	
27DT2.0	27.0	5.58	0.80	3.13	3	3	0.33	0.0189	46.4	2	No. 3	0.22	0.0126	45.0	
20DT3.0	20.0	5.55	0.75	3.13	3	3	0.33	0.0190	46.4	2	No. 3	0.22	0.0127	47.1	
20DT2.2	20.0	5.58	0.83	3.13	3	3	0.33	0.0189	45.2	2	No. 3	0.22	0.0126	44.6	
15DT3.1	15.0	5.57	0.75	3.14	3	3	0.33	0.0189	45.3	2	No. 3	0.22	0.0126	46.5	
15DT1.9	15.0	5.50	0.72	3.13	3	3	0.33	0.0192	46.6	2	No. 3	0.22	0.0128	46.0	
10SC2.9	10.0	5.55	0.67	3.13	3	3	0.33	0.0190	45.9	2	No. 3	0.22	0.0127	45.9	
10SC2.2	10.0	5.47	0.69	3.13	3	3	0.33	0.0193	46.5	2	No. 3	0.22	0.0128	46.8	

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RESULTS AND DISCUSSION

General

Test results and predictions for 14 Fast-Fix beams are summarized in Tables 4 and 5 and plotted in Figure 6; methods and assumptions used in determining the predicted values are given in the Appendix.

The following beam identification system has been employed in this study. The first two numbers indicate the shear span in inches, and the next two letters indicate the observed failure mode as follows:

FC Flexural compression failure—crushing of concrete

FT Flexural tension failure—yielding of tension steel

FB Flexural balanced failure—simultaneous crushing of concrete and yielding of tension steel

DT Diagonal tension failure—diagonal cracking due to shear

SC Shear compression failure—shear cracking followed by crushing of concrete

The last two numbers give the ultimate strength of the concrete to the nearest 0.1 kip/in.^{2} Thus 20DT2.2 describes a beam having a shear span of 20 inches, an observed failure mode of diagonal tension, and an ultimate compressive strength of approximately 2.2 ksi.

Mode of Failure

The observed failure modes for all the test beams were the same as the ACI predictions except for beam 27FT2.7, which failed in tension rather than shear. The reason for this discrepancy is explained in some detail in a later section.

Fast-Fix companion beams 36FT2.8 and 36FT2.0 were observed to exhibit the same crack pattern and tension mode of failure as the previously tested beams of portland cement concrete. The portland cement concrete beams, which were identical in geometry, reinforcement, and shear span to these Fast-Fix beams, are described in Table 6.

Flexural Failure. Seven of the test beams failed in a flexural mode—two in compression, two in balanced, and three in tension.

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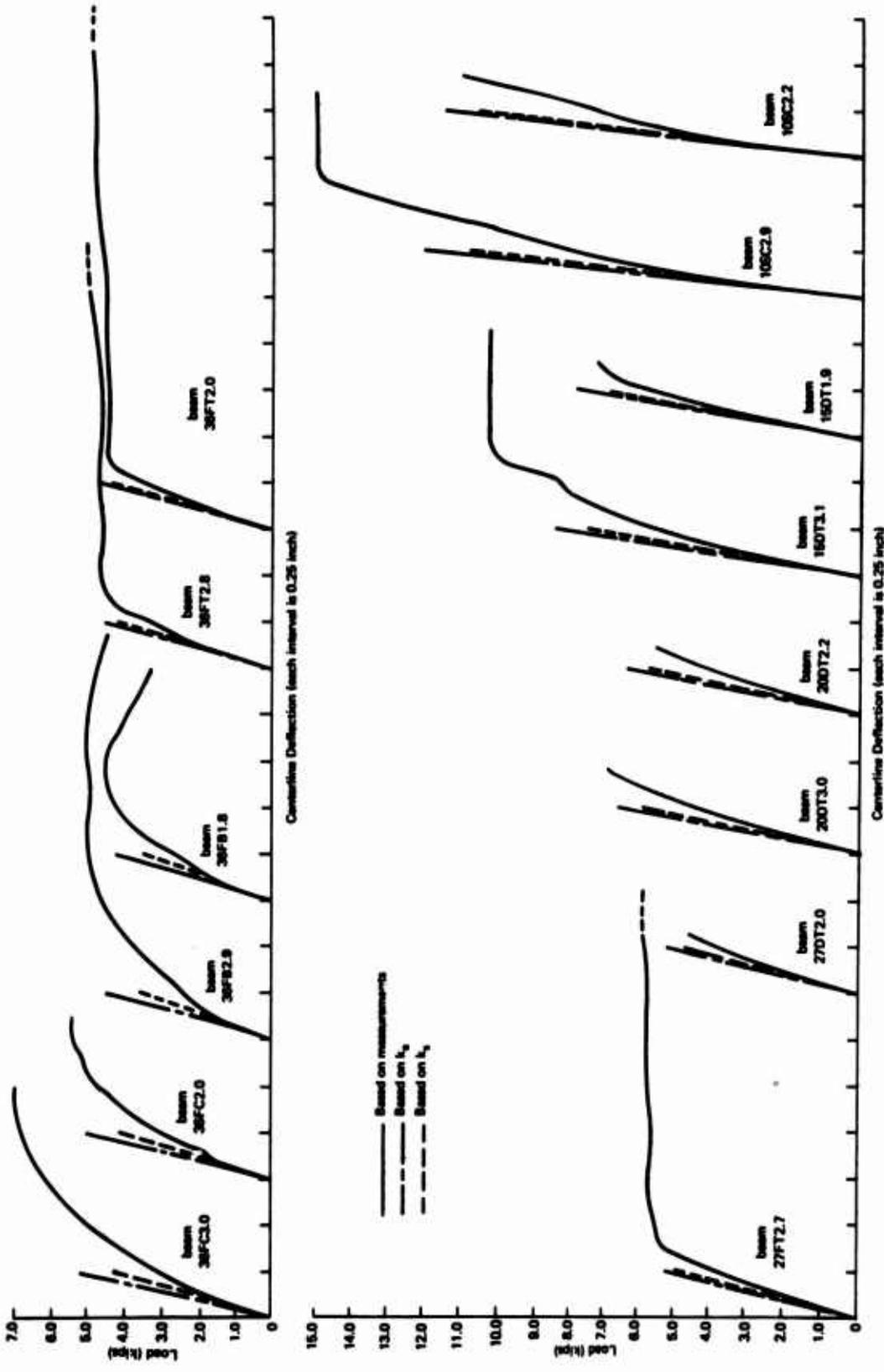
Figure 6. Load versus centerline deflection for all test beams.

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Table 4. Predicted and Actual Ultimate Strengths and Failure Modes of Fast-Fix Beams

Beam Designation	Beam Data				Ultimate Resistance and Failure Mode Based on—						
	a/d	f'_c (psi)	p/p_b	ACI Code			Experimental Results			
				Ultimate Load(s) (kips) based on—		Failure* Mode	Ultimate[†] Load (kips)	Maximum[†] Moment (in.-kips)	Maximum[†] Shear Stress (psi)	Failure* Mode
				Flexure	Shear					
36FC3.0	7.15	3,008	1.28	5.09	11.5	Flexure C	7.15	128	228	Flexure C
36FC2.0	7.10	1,976	1.80	3.71	11.0	Flexure C	5.55	98.7	175	Flexure C
36FB2.9	6.98	2,920	0.849	4.51	11.6	Flexure B[‡]	5.12	91.0	158	Flexure B
36FB1.8	6.42	1,812	0.928	3.66	11.7	Flexure B[‡]	4.55	80.7	129	Flexure B
36FT2.8	6.59	2,763	0.478	4.17	12.1	Flexure T	4.80	85.2	140	Flexure T
36FT2.0	6.46	1,952	0.570	4.07	11.9	Flexure T	4.65	82.5	133	Flexure T
27FT2.7	4.91	2,740	0.461	5.46	3.76	Shear	6.08	81.5	177	Flexure T
27DT2.0	4.84	1,954	0.589	5.50	3.27	Shear	4.72	63.2	135	Shear DT
20DT3.0	3.60	2,988	0.440	7.54	4.07	Shear	7.07	70.6	204	Shear DT
20DT2.2	3.58	2,226	0.528	7.25	3.59	Shear	5.81	58.0	166	Shear DT
15DT3.1	2.69	3,077	0.417	9.88	4.29	Shear	10.4	78.5	298	Shear DT
15DT1.9	2.73	1,868	0.609	9.88	3.43	Shear	7.50	56.5	218	Shear DT
10SC2.9	1.80	2,933	0.443	15.1	4.50	Shear	15.2	76.7	438	Shear SC
10SC2.2	1.83	2,198	0.547	14.8	3.96	Shear	11.2	56.7	327	Shear SC

* For flexure failure, the letters "C", "B", and "T" denote compression, balanced, and tension failures, respectively. For shear failure, the letters "DT" and "SC" denote diagonal tension and shear compression failures, respectively.

† Effect of dead loads is included.

‡ The expectation of balanced failure was due to the closeness of the p to p_b ratio to unity.

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Table 5. Predicted and Measured Elastic Properties of Test Beams

| Beam Designation | Predicted Elastic Properties* | | | | | | Measured Beam Stiffness, k_m (kips/in.) |
| | $E_{ca} = 33w^{1.5}\sqrt{f'_c}$ | | | E_{cs} = Secant at 1/3 f'_c | | | |
	E_{ca} (psi x 10^6)	I_{ca} (in.^{4})	k_a (kips/in.)	E_{cs} (psi x 10^6)	I_{cs} (in.^{4})	k_s (kips/in.)	
36FC3.0	2.93	56.0	21.1	1.86	70.7	16.9	7.80
36FC2.0	2.48	62.9	20.1	1.69**	76.0	16.5	9.62
36FB2.9	2.89	48.1	17.9	1.68	65.6	14.2	6.97
36FB1.8	2.38	55.6	17.0	1.45	75.1	14.0	9.51
36FT2.8	2.72	52.9	18.5	1.68	77.1	16.7	11.10
36FT2.0	2.39	61.3	18.8	1.69**	80.4	17.5	12.70
27FT2.7	2.71	54.3	20.7	1.74	76.9	18.8	11.40
27DT2.0	2.47	59.7	20.7	1.64	82.1	18.9	12.60
20DT3.0	2.89	52.4	26.1	1.70	79.4	23.2	15.40
20DT2.2	2.66	56.1	25.7	1.69**	79.8	23.2	15.40
15DT3.1	2.94	52.2	33.5	1.69	80.5	29.7	18.50
15DT1.9	2.41	59.3	31.3	1.78	75.4	29.3	19.50
10SC2.9	2.83	53.7	48.2	1.68	81.2	43.2	22.90
10SC2.2	2.56	56.1	45.6	1.69**	77.9	41.7	26.10

* See List of Symbols for definitions.

** Values taken as average.

As expected for the cases of compression failure, the failures of beams 36FC3.0 and 36FC2.0 were sudden and the tension cracks did not penetrate deep into the compression zone (Figures 7 and 8). The crushing of concrete was accompanied by buckling of the compression reinforcement in the central region.

A condition of balanced flexural failure requires crushing of concrete and a concurrent yielding of tension reinforcement. Analytically, this condition may be satisfied easily; however, experimentally this condition becomes a matter of judgment. In view of this, beams 36FB2.9 and 36FB1.8 (Figures 9 and 10) were designated as balanced failures based on the following observations:

1. The load—deflection curves for these beams (Figure 6) showed there was limited ductility in comparison with those of a typical tension failure.

2. The tension cracks (Figures 9 and 10) penetrated into the "compression zone" higher than in the case of compression failures, but not as high as in the case of ductile tension failures.

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Table 6. Properties of Beams Made With Portland Cement*

Properties	Beam No.	
	C-1	C-2
Loading	midpoint	midpoint
Depth, d (in.)	5.40	5.40
Depth, d' (in.)	0.60	0.75
f'_c (psi)	4,860	4,850
Reinforcing steel		
Tension		
Bars	3 No. 3	3 No. 3
Area (in.^{2})	0.33	0.33
Compression		
Bars	2 No. 3	2 No. 3
Area (in.^{2})	0.22	0.22
Stirrups		
Diameter (in.)	1/8	1/8
Spacing (in.)	2.5	2.5
f_y (ksi)	52.0	52.0
f'_y (ksi)	52.0	50.0
Yield load (kips)	4.38	4.57
Yield displacement (in.)	0.27	0.26

* Data taken from Reference 4.
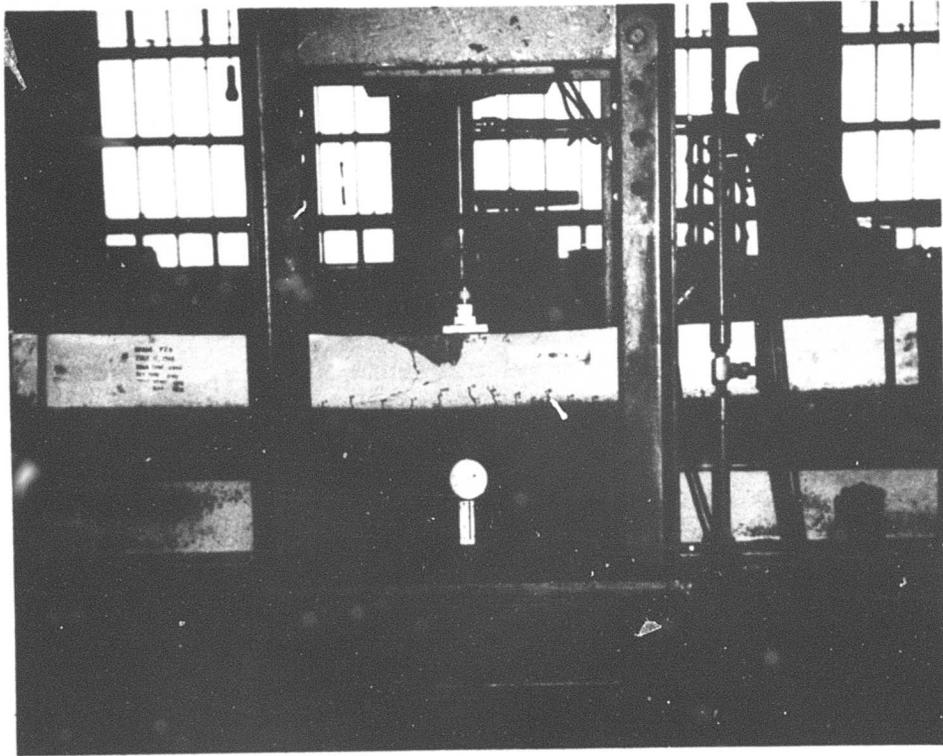
Figure 7. Flexural compression failure of beam 36FC3.0.
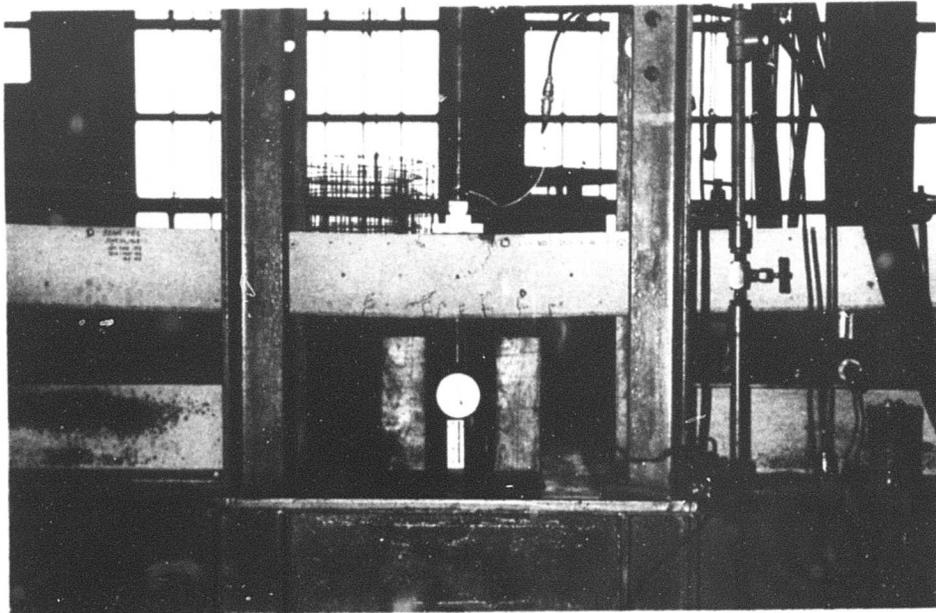
Figure 8. Flexural compression failure of beam 36FC2.0.
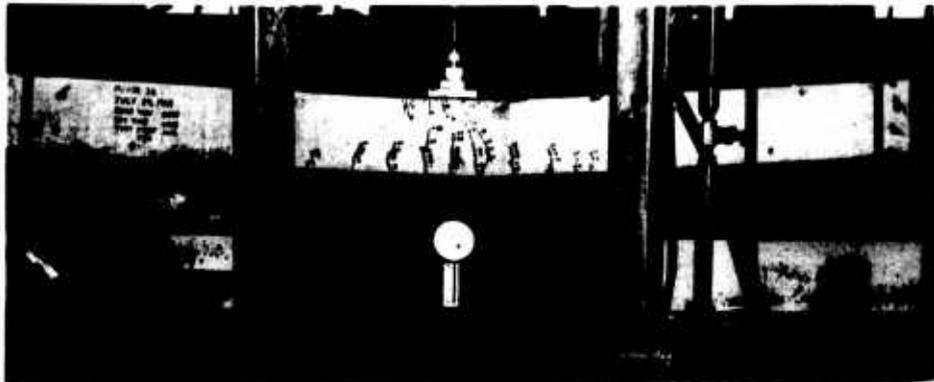
Figure 9. Flexural balanced failure of beam 36FB2.9.
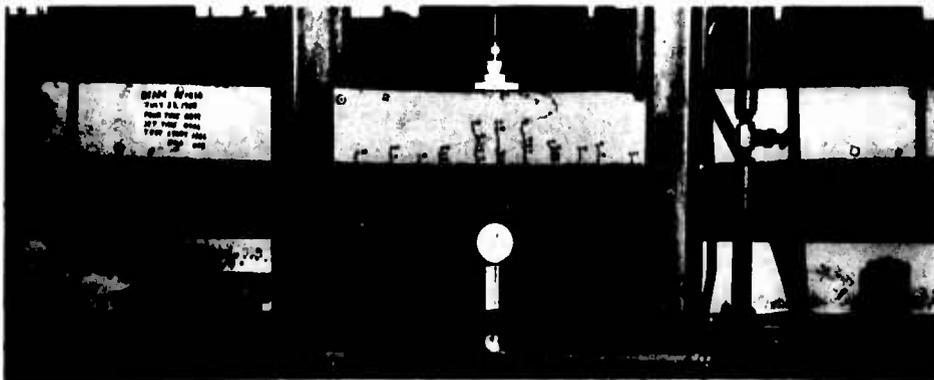
Figure 10. Flexural balanced failure of beam 36FB1.8.

Failures of beams 36FT2.8 (Figure 11), 36FT2.0 (Figure 12), and 27FT2.7 (Figure 13) were due to yielding of tension reinforcement. The compression steel in these beams helped increase ductility to the extent that the test was terminated when the beam deflection at its center exceeded 2 inches; this condition was reached without a total collapse of the beam.

Examination of Figure 13 reveals that beam 27FT2.7, which was predicted to fail in shear, did show signs of weakness in shear by developing diagonal cracks prior to actual failure in tension. Once yielding of tension reinforcement commenced, the beam deflected as if the central portion were a plastic hinge allowing no further increase in the measured load (Figure 6). No increase in shear distress occurred after the yielding resistance had been reached.
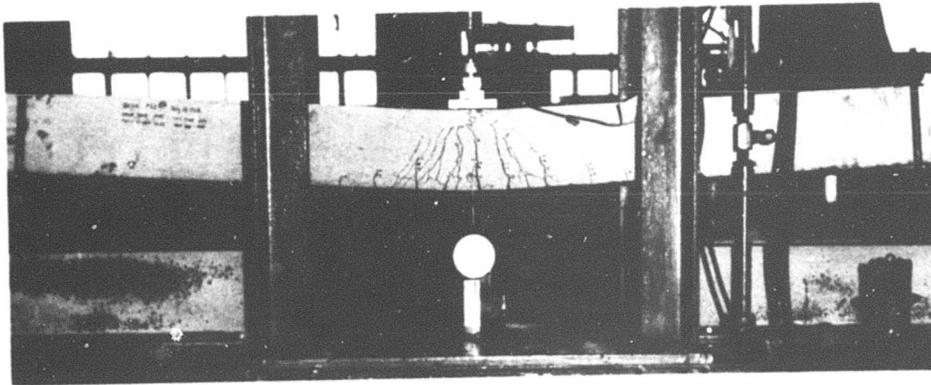
Figure 11. Flexural tension failure of beam 36FT2.8.
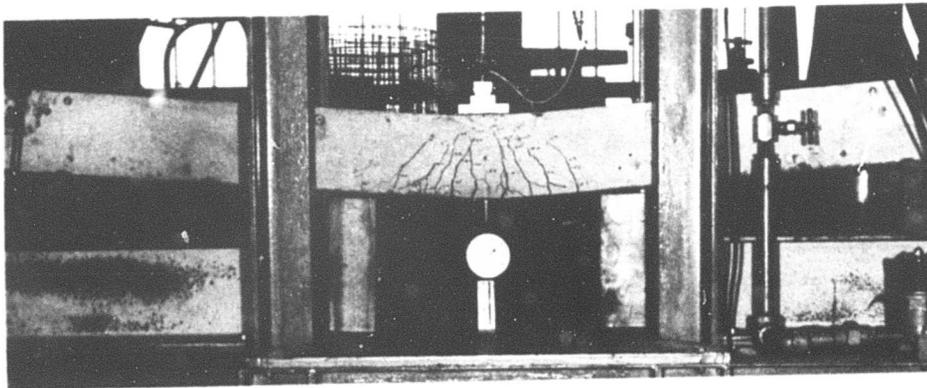
Figure 12. Flexural tension failure of beam 36FT2.0.
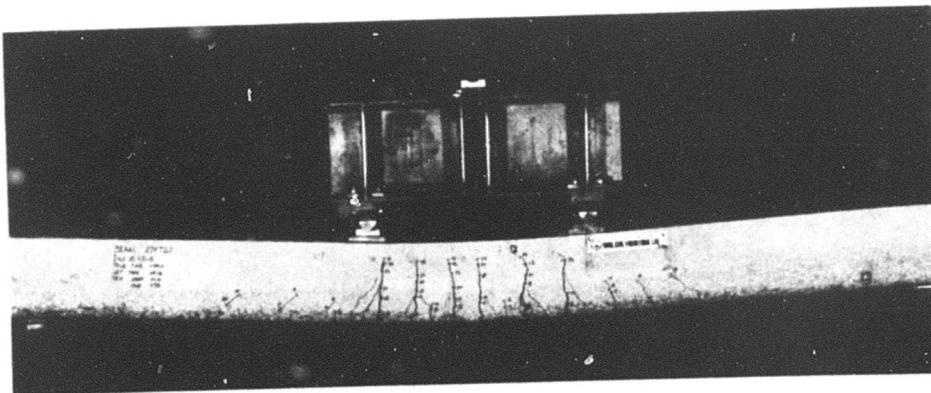
Figure 13. Flexural tension failure of beam 27FT2.7.

Shear Failure. Seven of the test beams failed in shear—five in diagonal tension and two in shear compression. The diagonal tension failure of beams 27DT2.0, 20DT3.0, and 20DT2.2 was followed by a secondary bond failure. This was evidenced by the longitudinal cracking at the tension steel level on both sides of each beam (Figures 14 through 16). Beams 15DT3.1 and 15DT1.9 (Figures 17 and 18) failed also in diagonal tension. However, some signs of bond distress appeared before the diagonal tension failure; this was possibly due to the shorter anchorage length of the tension steel.

Figures 19 and 20 show the final crack patterns for beams 10SC2.9 and 10SC2.2, respectively. The inclined shear cracks have appeared within the shear spans well before these beams reached their ultimate resistance. Beam failures occurred by crushing of concrete underneath one of the loading points. This type of failure is commonly called shear compression failure.
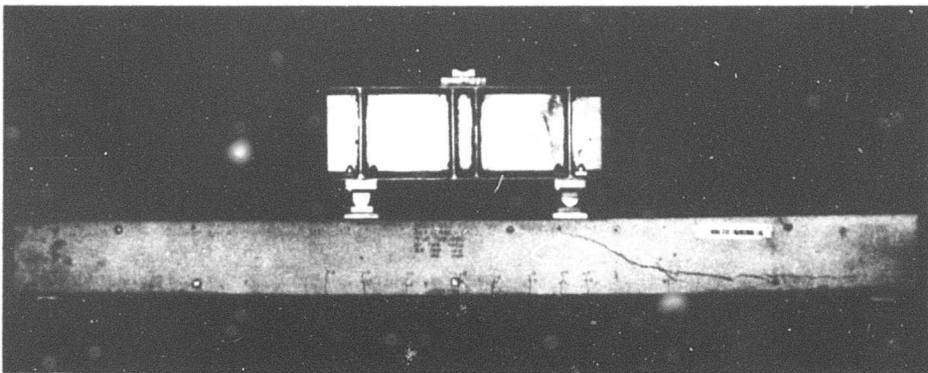
Figure 14. Diagonal tension failure of beam 27DT2.0.
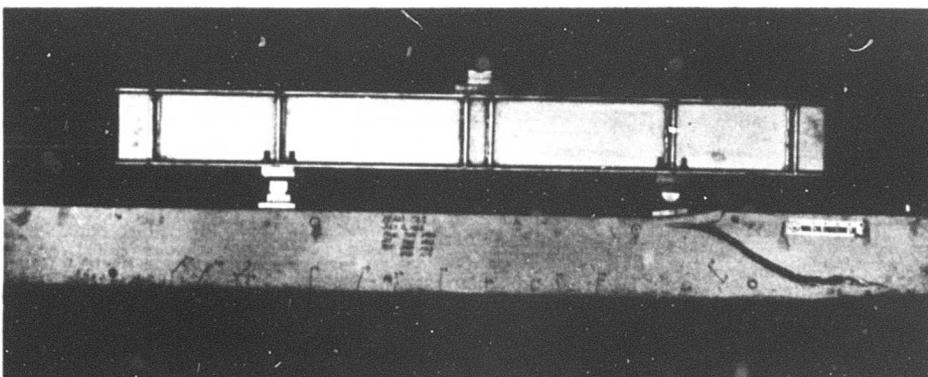
Figure 15. Diagonal tension failure of beam 20DT3.0.
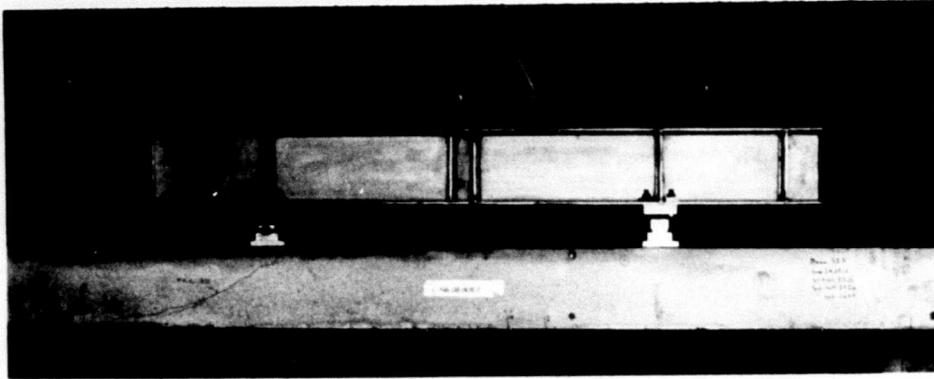
Figure 16. Diagonal tension failure of beam 20DT2.2.
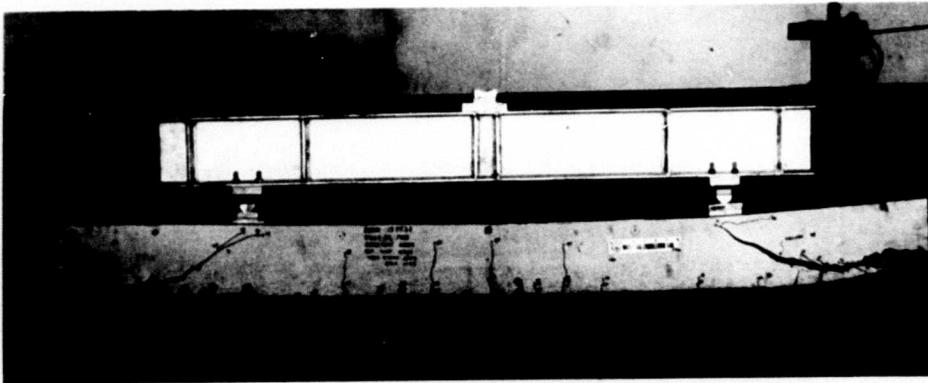
Figure 17. Diagonal tension failure of beam 15DT3.1.
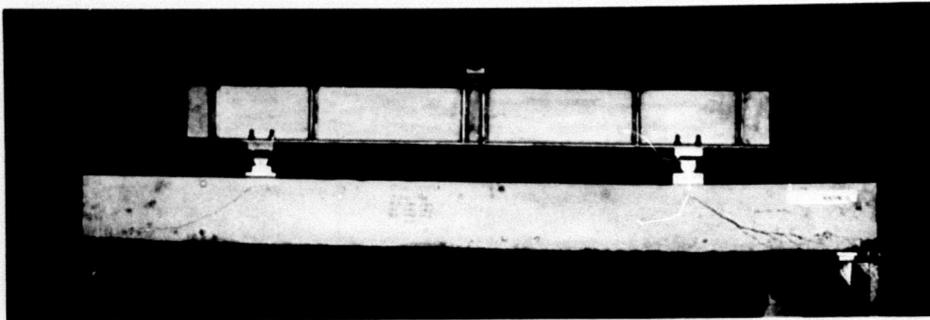
Figure 18. Diagonal tension failure of beam 15DT1.9.
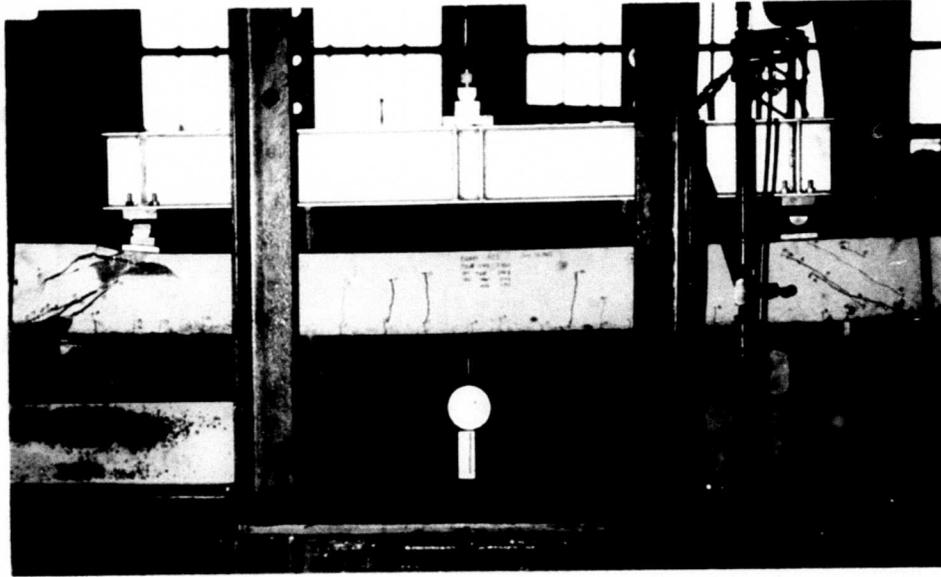
Figure 19. Shear compression failure of beam 10SC2.9.
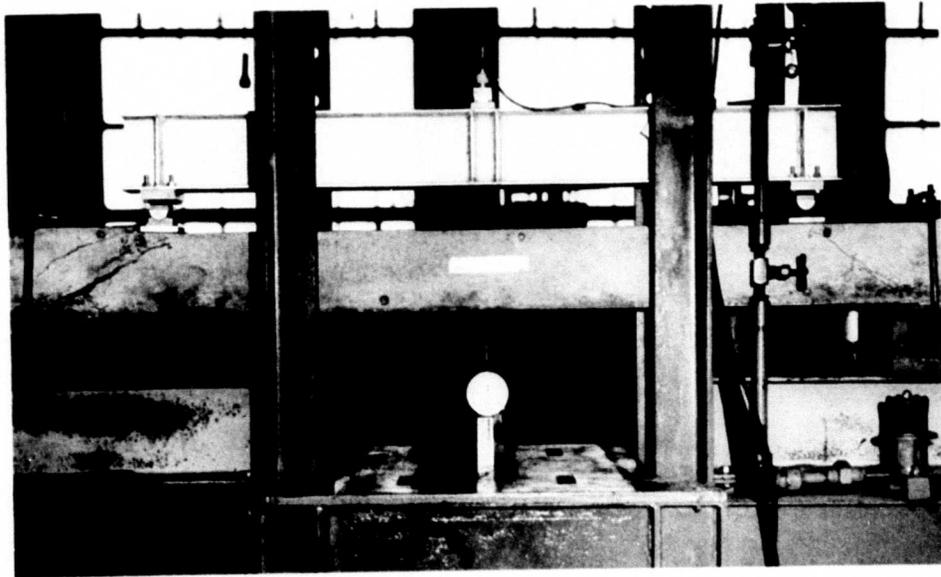
Figure 20. Shear compression failure of beam 10SC2.2.

Ultimate Strength

Measured Versus Predicted Values. Measured and predicted values of ultimate moments of beams failing in flexure are presented in Figure 21. The ultimate moment of beam 36FC3.0 was some 30% greater than that of the companion beam 36FC2.0 due to the difference in the compressive strength of the concrete, a trend which is expected for compression failures. Also as expected is the fact that the ultimate moments were almost constant for beams failing in tension, namely, beams 36FT2.8, 36FT2.0, and 27FT2.7.

For beams failing in shear, both measured and predicted values of the ultimate shear stresses are shown in Figure 22. These ultimate shear stresses followed the same general trend as for beams made with portland cement concrete in that the shear strength increased with an increase of compressive strength or a decrease in the shear-span-to-depth ratio.
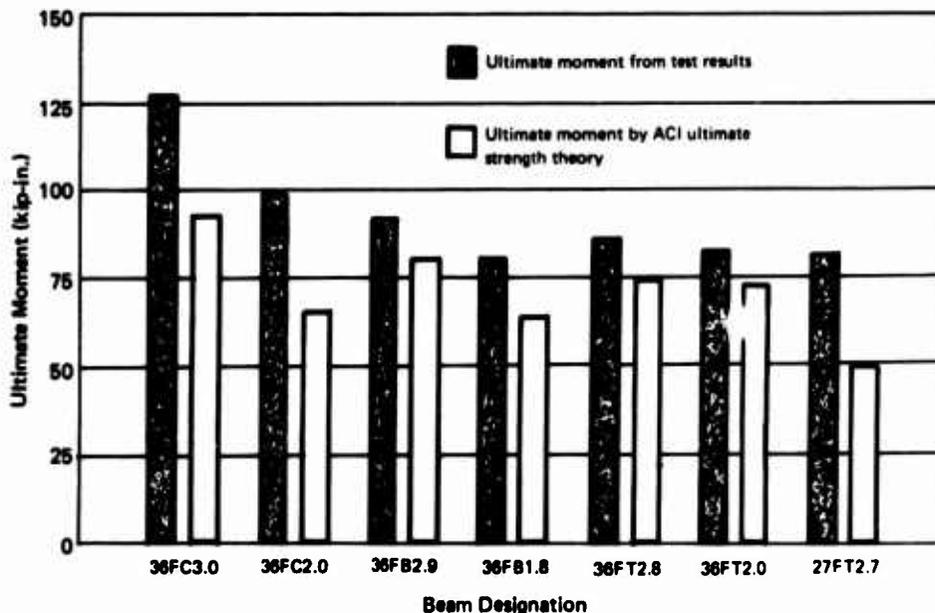
Figure 21. Beams failing in flexure: comparison of predicted and
measured ultimate shear stresses.

For all test beams, the measured ultimate resistances (Figures 21 and 22) are higher than the predicted values based on the ACI Code, which is intended for portland cement concrete. For reference, the ratios of the measured values to those of the predicted values of ultimate resistances for all test beams are shown in Figure 23. This ratio, which ranged from about 1.1 to 3.3, was higher for beams failing in shear than for those failing in tension. When any two companion beams are considered, Figure 23 also

shows that this ratio increased with the concrete strength for cases of shear failures but not for flexural failures. Moreover, the decrease of shear span for beams with shear failure resulted in an increase of this ratio of observed ultimate strength to predicted ultimate strength.
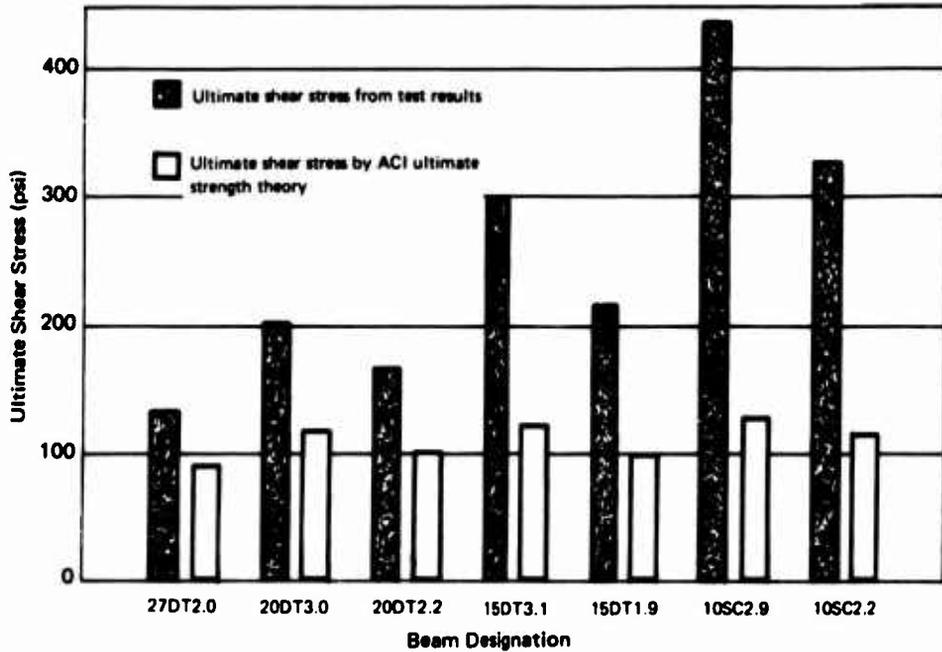
Figure 22. Beams failing in shear: comparison of predicted and measured ultimate shear stresses.
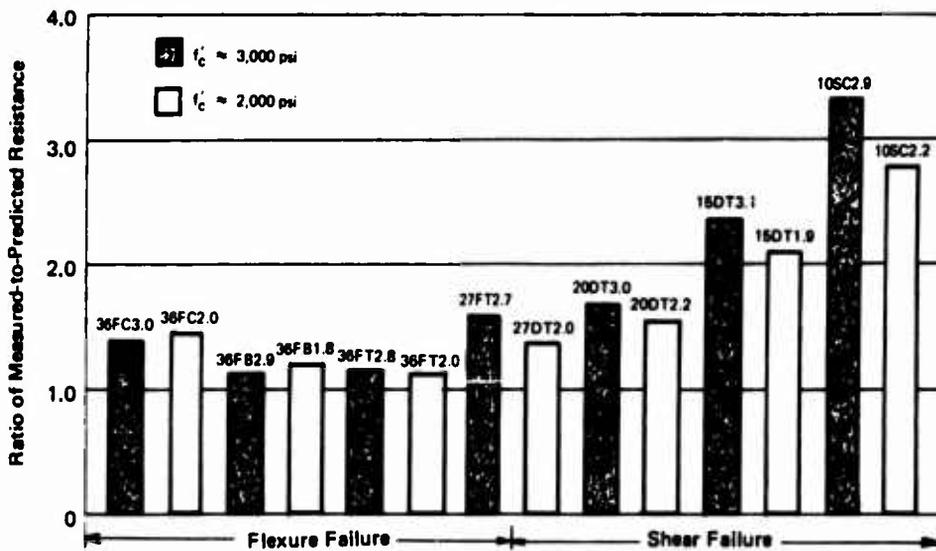
Figure 23. Ratios of measured-to-predicted resistances.

Shear and Tension Resistance Envelope. An interesting way for investigating the interplay between shear and tension failures is by considering the envelope of shear and tension resistance curves. Resistance curves are graphs of ultimate resistance as a function of some selected parameter(s). For the problem in hand, the shear-span-to-depth ratio and the concrete strength are the main parameters; therefore, the ultimate load that can be carried by the beam is expressed in terms of these parameters. This is done for the test beam as follows:

Tension Resistance. The maximum moment M is given by

$$M = \left(\frac{P}{2}\right) a \tag{1}$$

where a is the shear span and P is the total load on the beam. The ultimate load, P_u, is reached when the maximum moment is equal to the ultimate moment resistance, M_u, based on yielding of the tension steel. Thus

$$P_u = \frac{2 M_u}{a} = \frac{2 M_u/d}{a/d} \tag{2}$$

The effective depth, d, was introduced in order to have P expressed in terms of a/d; that is, the shear-span-to-depth ratio. The effect of concrete strength on the ultimate load appears when calculating the ultimate moment. In all cases of tension failure, this effect is very small (about 2%) and is usually neglected.

Shear Resistance. The maximum shear force, V, is given by

$$V = \frac{P}{2} \tag{3}$$

The ultimate load is obtained when the maximum shear force reaches the ultimate shear resistance. For beams without web steel, the ultimate shear resistance is given by bdv_c where b and d are the width and depth of beam cross section, and v_c is the ultimate shear stress carried by concrete alone. This is given by the ACI Code[3] Section 1701(d) as

$$v_c = \phi \left(1.9 \sqrt{f'_c} + 2{,}500 \frac{pVd}{M}\right) \tag{4}$$

where ϕ is a capacity reduction factor taken here as unity, and p is the reinforcement ratio (A_s/bd). For the test beams, the moment-to-shear ratio (M/V) is equal to the shear span a, and therefore the above expression can be written in the form

$$v_c = 1.9 \sqrt{f'_c} + 2{,}500 \frac{p}{a/d} \tag{5}$$

Accordingly, the ultimate load in pounds is given by

$$P_u = 2bd\left(1.9\sqrt{f_c'} + 2,500\frac{p}{a/d}\right)$$ (6)

For the beam cross section designated by "D" in Figure 1, the solid curves in Figure 24 represent tension resistance as well as the shear resistance for concrete strengths of 2,000 psi and 3,000 psi. The fact that both curves of shear resistance fall below the tension resistance curve implies that a beam should fail in shear when the applied load reaches the value obtained from the shear resistance curve for the appropriate f_c' and a/d ratio. This conclusion is true only if the ACI Code predictions are valid.

To check the validity of the ACI Code expressions for ultimate resistances in shear and in tension, data points representing all the beams which failed in shear (seven beams) or in tension (three beams) are also presented in Figure 24. The effect of stirrups for beam 36FT2.8 and beam 36FT2.0 was accounted for by a step increase in the shear resistance curves at the corresponding value of a/d; this was indicated by the arrows in Figure 24. These step increases raised the shear resistances of the two beams above the tension resistance, thereby ensuring a tension failure.
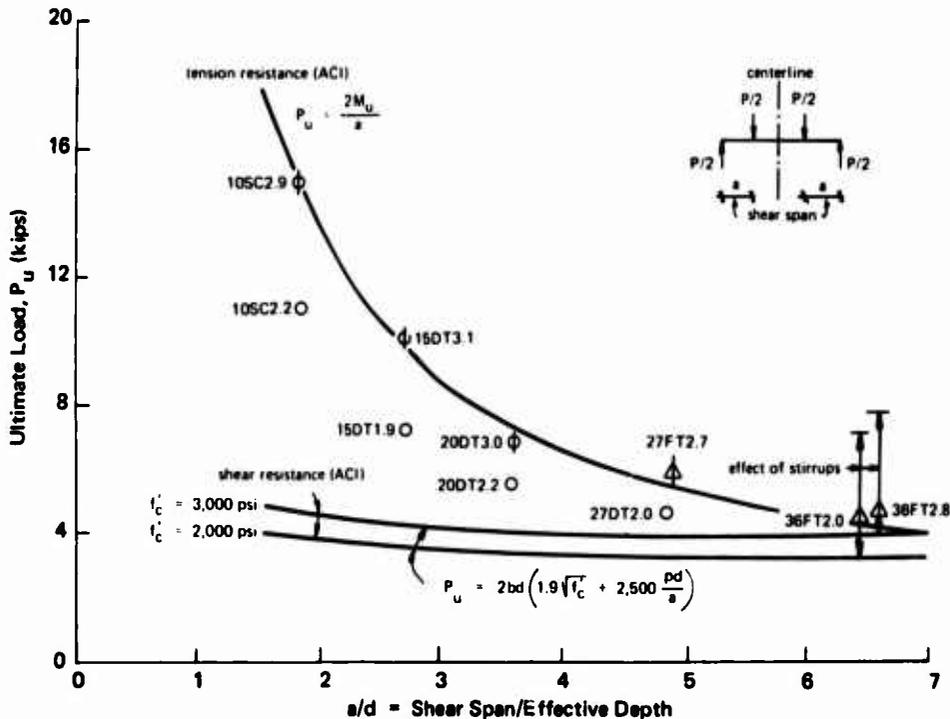
Figure 24. Correlation of ultimate load with shear-span-to-effective depth ratio for beams failing in tension and in shear.

While all the data points for beams failing in shear lay well above the curves representing the predicted ultimate shear resistance, the data points for beams failing in flexure lay right above the tension resistance curve. This implies the previously stated conclusion that the ratios of observed-to-predicted ultimate resistances of beams failing in shear are generally much higher than for those failing in tension.

Data points for beams with concrete strength of 3,000 psi that failed in shear lay so high above the shear resistance curve that they are in the neighborhood of the tension resistance curve. In particular, data points of beams 10SC2.9 and 15DT3.1 lay on or above the tension resistance curve. If the tension resistance curve is accurate, these beams should exhibit some degree of ductile response. Figure 6 verifies that this is true for both beams. Therefore, the actual failures of these beams were close to being borderline cases between shear and tension even though the resistance envelope based on the ACI Code indicates definite shear failures.

A special case of interest was beam 27FT2.7, which failed in tension instead of in shear as predicted. The prediction of shear failure was based on the fact that the calculated shear resistance curve in Figure 24 was lower than the calculated tension resistance curve. But the same figure shows that the calculated shear resistance is not realistic when compared to data points representing actual shear failures. Obviously, realistic shear resistance curves would be those representing the actual test results. Based on such curves, the tension resistance of beam 27FT2.7 would be less than the shear resistance, and therefore tension failure would be expected.

With this in mind, the ratio of measured ultimate strength to predicted ultimate strength for beam 27FT2.7 was corrected as shown by the white line in Figure 23. This corrected figure represents the ratio of measured tension resistance to predicted tension resistance, rather than the unrealistic ratio of measured tension resistance to predicted shear resistance.

Comparison With Beams of Portland Cement. Table 7 presents a direct comparison of ultimate strength of companion beams 36FT2.8 and 36FT2.0 with results of similar beams made of portland cement (Table 6). From this table it is seen that the capacity of the Fast-Fix beams exceeded its predicted value by a higher margin than that of beams made with portland cement.

Table 7. Comparison of Ultimate Strength of Fast-Fix and
Portland-Cement Concrete Beams

Type of Cement	Beam Identification	Calculated Ultimate Moment (in.-kips)	Measured Ultimate Moment (in.-kips)	Ratio of Measured-to-Calculated Ultimate Moment
Fast-Fix	36FT2.8	74.1	84.0	1.13
	36FT2.0	73.2	81.1	1.11
Portland	C-1	84.4	78.9	0.935
	C-2	83.6	82.3	0.985

Elastic Stiffness

Elastic behavior was investigated in terms of beam stiffness, k, which is defined as the load(s) divided by the centerline deflection. Measured values of beam stiffness, k_m, given in Table 5 were obtained from load–deflection curves of Figure 6 for that "straight" portion of the curve above tensile-cracking load. Calculated beam stiffnesses k_a and k_s, shown in Table 5, are based on a cracked transformed section for two moduli of concrete: (1) the modulus recommended by the ACI Code[3] Section 1102(a), which is given by $E_{ca} = 33 \, w^{1.5} \sqrt{f_c'}$ where w is the unit weight of concrete in pounds per cubic foot; and (2) secant modulus at $1/3 \, f_c'$, measured from compressive stress–strain curves of control cylinders and denoted as E_{cs}. A graphical representation of these calculated beam stiffnesses is shown in Figure 6.

Figure 25 gives the ratios of measured stiffness to calculated stiffness (k_m to k_a and k_m to k_s) for each beam. It is evident that all of these ratios are less than unity, indicating that the measured stiffness is less than either of the calculated stiffnesses for each beam. The ratio k_m to k_a has an average value of 0.55 while the ratio k_m to k_s averages 0.62; this reflects the fact that the secant modulus, E_{cs}, is less than the ACI modulus, E_{ca}, as can be noted in Table 5.

Stiffnesses of beams 36FT2.8 and 36FT2.0 are compared in Table 8 with those of the duplicate beams made with portland cement. The ratios of measured-to-calculated stiffnesses (k_m to k_a) of the Fast-Fix beams are not quite as high as those of the portland cement concrete beams.

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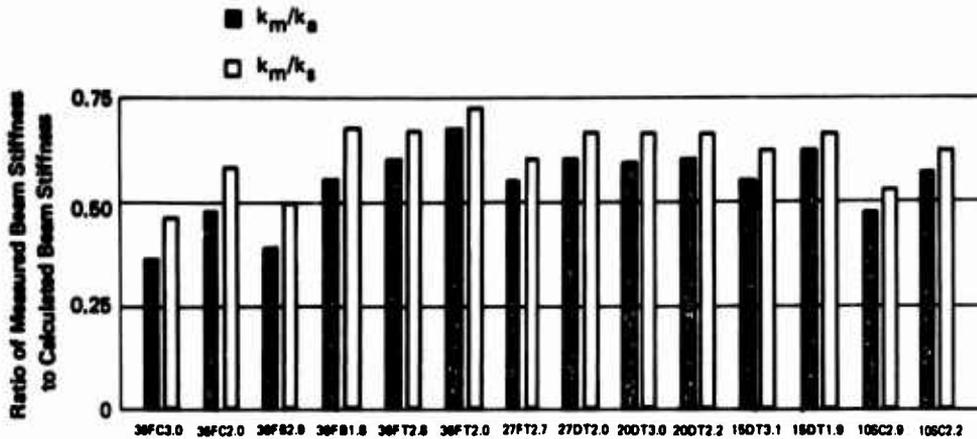
Figure 25. Ratios of measured-to-predicted beam stiffnesses.

Table 8. Comparison of Stiffness of Fast-Fix and
Portland Cement Concrete Beams

Type of Cement	Beam Identification	Calculated Stiffness, k_s (kips/in.)	Measured Stiffness, k_m (kips/in.)	Ratio of Measured-to-Calculated Stiffness
Fast-Fix	36FT2.8	18.5	11.1	0.600
	36FT2.0	18.8	12.7	0.675
Portland	C-1	19.9	13.9	0.698
	C-2	19.6	14.5	0.739

FINDINGS

1. The ultimate strength of Fast-Fix beams follows the same general trend
as that for beams made with portland cement concrete in that an increase
of f_c' results in an increase of the ultimate resistance for compression and
shear failures, but not for tension failures. Also, an increase of the shear-
span-to-depth ratio results in a decrease of ultimate shear resistance.

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2. The actual ultimate resistance of the test beams ranged from 1.1 to 3.3 times the computed values based on the ACI Code (ultimate strength design). The ultimate resistance in tension was closest to the predicted resistance, whereas the ultimate shear resistance exceeded its predicted value by the widest margin.

3. Throughout the test program, brittle shear and compression failure modes occurred only as predicted. In addition, beams which were predicted as borderline cases between brittle and ductile failures failed in a ductile tension mode. This trend is desirable as it insures the ductility required for safe designing.

4. The ratio of measured beam stiffness to calculated values (based on the ACI Code working stress design assumptions) averaged only 0.55. This low average is not unusual for beams made of portland cement.

5. Comparing the performance of two Fast-Fix beams with "duplicate" beams made of portland cement concrete revealed that the ultimate strength of the Fast-Fix beams was at least as high as that of the portland cement concrete beams. On the other hand, the stiffnesses of beams made with portland cement concrete were slightly greater than those of the Fast-Fix beams.

RECOMMENDATION

Reinforced concrete beams made with Fast-Fix cement can be used as structural elements. The Ultimate Strength Design Section of the ACI Building Code Requirements[3] is recommended for safe prediction of the ultimate resistance of the Fast-Fix beams.

Appendix

ANALYTICAL ASSUMPTIONS AND METHODS

ULTIMATE STRENGTH ANALYSIS

In accordance with ACI Code[3] the following assumptions were made for calculating ultimate strength.

1. Strain in the concrete was assumed directly proportional to the distance from neutral axis, and strain in reinforcing steel assumed equal to the strain in concrete at the same position.

2. The stress—strain relation at ultimate strength was idealized by a rectangular stress block.

3. The maximum strain at the extreme compression fiber at ultimate flexural strength was taken as 0.003 in./in.

4. The ultimate shear stress capacity at a given section o′ unreinforced web was taken as, $v_c = 1.9 \sqrt{f'_c} + 2,500 \, (p \, V \, d/M)$.

With the first three assumptions above and the requirement of equilibrium, an iterative method was used to obtain ultimate strengths for flexural failures. Mode of failure is designated as compression, balanced, or tension depending on whether the above requirements are satisfied by having tension steel (1) not yet at yield, (2) at yield, or (3) already yielded.

Ultimate strengths of shear failures were computed directly from assumption 4 with the addition of web reinforcement resistance when applicable.

The ultimate flexural resistance was compared with the ultimate shear resistance, and the lower of the two values taken as controlling ultimate strength.

Calculated values of beam stiffness, defined as the load(s) divided by the centerline deflection, were based on the assumptions of a cracked transformed section and that stresses are proportional to strains. With these assumptions and the requirement of equilibrium, beam stiffness for the loadings considered is given by $k = 48 \, E \, I/a(3 \, L^2 - 4 \, a^2)$.

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LIST OF SYMBOLS

A_s — Area of tension reinforcement (in.^{2})

A_s' — Area of compression reinforcement (in.^{2})

a — Shear span: for the test beams, "a" is the distance between the support and the closest load point (in.)

b — Beam width (in.)

d — Distance from extreme compression fiber to centroid of tension reinforcement (in.)

d' — Distance from extreme compression fiber to centroid of compression reinforcement (in.)

E — Modulus of elasticity of concrete (psi)

E_{ca} — Modulus of concrete based on ACI Code Section 1102(a) (psi)

E_{cs} — Secant modulus of concrete at 1/3 f_c' measured from the actual stress—strain curves of control cylinders (psi)

f_c' — Compressive strength of standard concrete cylinder (psi)

f_{sp} — Splitting tensile strength of concrete (psi)

f_y — Yield strength of reinforcement (ksi)

f_y' — Yield strength of compressive reinforcement (ksi)

I — Moment of inertia (in.^{4})

I_{ca} — Moment of inertia of cracked transformed section based on E_{ca} (in.^{4})

I_{cs} — Moment of inertia of cracked transformed section based on E_{cs} (in.^{4})

k — Beam stiffness defined as the load(s) divided by the centerline deflection (lb/in.)

k_a — Calculated beam stiffness defined as the load(s) divided by the centerline deflection and based on E_{ca} and I_{ca} (kips/in.)

k_m — Measured beam stiffness (kips/in.)

k_s — Calculated beam stiffness defined as the load(s) divided by the centerline deflection based on E_{cs} and I_{cs} (kips/in.)

L — Span (in.)

M — Bending moment (in.-kip)

M_u — Calculated ultimate resisting moment (in.-kips)

P — Total load(s) (kips)

P_u — Total load(s) at ultimate (kips or pounds)

p — Ratio of tension reinforcement (= A_s/bd)

p' — Ratio of compression reinforcement (= A_s'/bd)

p_b — Ratio of tension reinforcement for flexural balanced failure with the influence of A_s' taken into account

V — Shearing force (kips)

v_c — Ultimate shear stress carried by the concrete alone (psi)

w — Unit weight of concrete (lb/ft^3)

ϕ — Capacity reduction factor

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DOCUMENT CONTROL DATA - R & D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

1. ORIGINATING ACTIVITY *(Corporate author)*	2a. REPORT SECURITY CLASSIFICATION
Naval Civil Engineering Laboratory Port Hueneme, Calif. 93041	Unclassified
	2b. GROUP

3. REPORT TITLE

STRUCTURAL BEHAVIOR OF REINFORCED CONCRETE BEAMS MADE WITH FAST-FIX 1 CEMENT

4. DESCRIPTIVE NOTES *(Type of report and inclusive dates)*

Final; May 1968–September 1968

5. AUTHOR(S) *(First name, middle initial, last name)*

S. B. Nosseir and M. G. Katona

6. REPORT DATE	7a. TOTAL NO. OF PAGES	7b. NO. OF REFS
February 1969	35	5
8a. CONTRACT OR GRANT NO	9a. ORIGINATOR'S REPORT NUMBER(S)	
b. PROJECT NO. YF 38.536.006.01.002	TR-614	
c.	9b. OTHER REPORT NO(S) *(Any other numbers that may be assigned this report)*	
d.		

10. DISTRIBUTION STATEMENT

This document has been approved for public release and sale; its distribution is unlimited.

11. SUPPLEMENTARY NOTES	12. SPONSORING MILITARY ACTIVITY
	Naval Facilities Engineering Command Washington, D. C.

13. ABSTRACT

The objective was to investigate the predictability and effectiveness of reinforced concrete beams made with Fast-Fix 1 cement. Fourteen simply supported beams were designed with the beam parameters systematically varied in order to observe a wide range of structural response when the beams were loaded to failure. The ultimate strength and mode of failure of test specimens were compared with predicted results based on the ultimate strength design assumptions of the ACI Code. In addition, "elastic" behavior in terms of beam stiffness was compared to calculated stiffnesses based on the working stress design assumptions of the ACI Code. The performance of two Fast-Fix beams was also compared with the test results of "duplicate" beams made with portland cement. Evaluation of the Fast-Fix beams indicated that their ultimate capacity was safely predicted by the ultimate strength theory and that their structural effectiveness was equivalent to beams made with portland cement.

DD FORM 1473 (PAGE 1)
1 NOV 65

S/N 0101-807-6801

14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Concrete beams						
Fast-Fix cement						
Structural behavior						
Flexural compression failure						
Flexural balanced failure						
Flexural tension failure						
Shear compression failure						
Diagonal tension failure						

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