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CONCRETE FOR TUNNEL LINERS:

EVALUATION OF FIBER REINFORCED QUICK SETTING CEMENT CONCRETE

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16. Abstract This study was undertaken to determine the behavior of steel fiber reinforced, quick setting cement concretes and provide information on this material needed for its application in tunnel liners.

Strength-time and creep behavior are evaluated for specific mix designs. The creep behavior does not appear to be substantially different than for plain concrete.

Interaction diagrams are presented for concretes made with Duracal cement and fiber contents of 0.9, 1.2 and 1.5 percent by volume. Various failure modes were obtained depending on the initial relative eccentricity of load.

Tensile stress-strain relationships are analytically obtained from flexural test data. These relationships make possible a post-crack analysis of a fiber reinforced concrete structure.

Fiber content, fiber orientation, and type of cement have little affect on Poisson's ratio but do influence the modulus of elasticity and the strength.

Durability of these concretes is examined through study of permeability, leaching, volume stability, disturbance of young concrete, effects of high temperature environments and sulfate resistance.

Pumping of fibrous concrete is investigated through laboratory testing. The presence of fibrous reinforcement significantly increases the volume of voids making the aggregate gradation and cement paste content critical parameters in designing pumpable mixes.

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PREFACE

This study was conducted in the Department of Civil Engineering of the University of Illinois at Urbana-Champaign. The study was supported by the Federal Railroad Administration, Department of Transportation through contract No. DOT FR 30022, under the technical direction of Mr. William N. Lucke. Project coordinator at the University of Illinois was Dr. Stanley L. Paul.

Dual-unit format is employed in the presentation of all data, figures, and tables included in this report. Data and tables are presented in the English system, with International System of Units (SI) values included in parentheses. Large tables are presented with SI values in a separate table. For example, the English values may be presented in Table 3 and the SI values in Table 3-SI. Figures are presented with auxiliary scales for SI units.

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b	width of beam section
c	depth to neutral axis in beam section
C	compression resultant force
d	depth of cross section
e	initial eccentricity measured from centroid of cross section
e/d	relative eccentricity
E_c	Young's modulus in compression
$F(k)$	creep rate constant
f_{avg}	average stress in the compression region
f_c'	ultimate compressive strength; 28-day cylinder strength
f_{cu}	compressive strength of test cylinder
f_r	modulus of rupture
h	depth of beam section
I	moment of inertia
K	permeability coefficient
P	applied load
P_{max}	applied load at failure

M	applied moment
M_{max}	applied moment at failure
t	time after loading, in days, for creep test
α	ratio of average stress in compression region to ultimate compressive strength
β	parameter which locates position of compressive resultant with respect to neutral axis
Δe	additional eccentricity from bending of test specimen
ε	strain
ε_c	strain in concrete
ε_{cu}	ultimate strain in concrete
ε_o	initial elastic strain
ε_t	total strain (elastic and creep) due to loading
ν	Poisson's ratio
ϕ	curvature at any point along a beam
ϕ_1	assumed uniform curvature along beam, except for crack-element
ϕ_2	assumed uniform curvature within crack-element

CHAPTER 1

INTRODUCTION AND SCOPE

Previous research (Parker, et al., 1973; Paul, et al., 1974) has focused on the applicability of steel fiber reinforced, regulated-set cement concrete as a material for tunnel linings. Further research has been undertaken with this material and similar research initiated for steel fiber reinforced, Duracal cement concrete.

Tunnel designers and constructors require information regarding the time-dependent behavior of concrete materials. Specifically the strength-time and creep behavior have been investigated for quick setting cement concretes. Through a series of laboratory tests and extensive analytical studies the behavior of these experimental concretes under combined axial and flexural loadings has been examined. Numerical techniques have been applied to data obtained from beam tests to generate the tensile stress-strain relationship for steel fiber reinforced, Duracal cement concrete. The effect of fiber orientation on elastic properties and strength was studied for cylinder and beam specimens. The durability of the experimental concretes was studied for a variety of special environments. The significance of efficiency of material placement was emphasized in an investigation of factors influencing the pumping of fiber reinforced concrete.

This report concerns research performed or initiated during the 1973-74 contract period. Studies of material durability require longer periods of observation. Results of short term tests accumulate and require analysis and detailed study. For these reasons this report should be regarded as a research summary. Comprehensive topical reports on the subjects considered in this report will be issued when the research of that particular subject is essentially complete. The program of research is at this time incomplete and the results reported here, while believed to be correct, are only tentative.

CHAPTER 2

EXPERIMENTAL CONCRETES AND LABORATORY PROCEDURE

2.1 INTRODUCTION

The purpose of this section is to present physical data regarding the raw materials, experimental concretes and laboratory procedures used throughout the research program. Throughout this chapter it will be assumed that these materials were used, unless specifically indicated otherwise.

2.2 MATERIALS

2.2.1 AGGREGATE

The coarse aggregate used was nominal 3/8-in. (10-mm) maximum size Wabash River pea gravel. Fine aggregate was a locally available torpedo sand.

Gradations, unit weights, bulk specific gravities and absorption capacities for the sand and gravel are shown in Tables 2.1 and 2.2.

2.2.2 CEMENT

Two types of quick setting cements were utilized in the experimental work reported. Regulated-set cement was supplied by the Huron Cement Division of National Gypsum Co. At different points in the research program the third and fourth burns of Huron regulated-set cement were used in the experimental concretes. Further reference to these cements will be as "reg-set 3" and "reg-set 4", respectively.

TABLE 2.1

SIEVE ANALYSIS OF SAND AND PEA GRAVEL

Sieve	Sand, percent		Pea gravel, percent	
	Retained	Passing	Retained	Passing
3/8-in. (10 mm)	0	100	1	99
No. 4	4	96	94	6
No. 8	12	88	99	1
No. 16	22	78	100	0
No. 30	40	60	100	0
No. 50	84	16	100	0
No. 100	99	1	100	0
No. 200	99	1	100	0
Fineness modulus	2.61		5.93	

TABLE 2.2

PROPERTIES OF SAND AND PEA GRAVEL

Property	Sand	Pea gravel
Unit weight, lb/cu ft (kg/m^3)	107.6 (1724)	100.6 (1612)
Bulk specific gravity, SSD	2.60	2.66
Absorption capacity, percent	2.2	2.2

A second quick setting cement used was Duracal, a product of U.S. Gypsum Co. Duracal cement is a blend of portland cement type 5 and a special gypsum. The Duracal cement during the work reported here was from the third, fourth, and fifth shipments received for research at the University of Illinois and will be referenced as Duracal 3, 4 and 5 as appropriate.

In some of the experimental work commercially available portland cement type 1 was used for purposes of comparison.

2.2.3 FIBER

Fibrous reinforcement used in all the experimental work reported herein was U.S. Steel Fibercon 0.010-in. by 0.022-in. by 1-in. (250-μm by 560-μm by 25-mm) steel fiber.

2.2.4 ADMIXTURES

Citric acid was used with regulated-set cement as a retarder. Other experimental admixtures were used and are described elsewhere as required.

2.3 MIX DESIGN

Mix designs for steel fiber reinforced, regulated-set cement concretes are shown in Table 2.3. To obtain necessary handling time water was either precooled to approximately 35 F (2 C) or added as a mixture of 40 percent liquid and 60 percent crushed ice. Mix designs for major portions of the testing of steel fiber reinforced Duracal cement concrete also appear

TABLE 2.3

MIXES FOR FIBER REINFORCED CONCRETE

Mix no.	Water-cement ratio	Mix materials, lb/cu yd						Fiber,[2] percent	Entr'd air percent	Unit weight, wet lb/cu ft
		Water	Cement	Sand[1]	Pea gravel[1]	Steel fiber	Citric acid			
RS0.9	0.50	325	650	1610	1130	120	2.0	0.9	4.1	142.8
RS1.2	0.45	326	725	1610	1080	160	1.75	1.2	3.7	143.8
RS1.5	0.45	360	800	1670	870	200	1.65	1.5	3.5	144.0
DC0.9	0.36	232	650	1640	1165	120	--	0.9	5.7	142.4
DC1.2	0.36	259	725	1610	1120	160	--	1.2	4.3	144.0
DC1.5	0.35	280	800	1580	1075	200	--	1.5	4.8	145.0

[1] Saturated, surface dry

[2] Percent by volume

TABLE 2.3-SI

MIXES FOR FIBER REINFORCED CONCRETE

Mix no.	Water-cement ratio	Mix materials, kg/m^3						Fiber,[2] percent	Entr'd air percent	Unit weight wet kg/m^3
		Water	Cement	Sand[1]	Pea gravel[1]	Steel fiber	Citric acid			
RS0.9	0.50	193	386	955	670	71	0.90	0.9	4.1	2314
RS1.2	0.45	194	430	955	640	95	0.80	1.2	3.7	2307
RS1.5	0.45	214	474	991	516	118	0.75	1.5	3.5	2278
DC0.9	0.36	137	386	973	691	71	--	0.9	5.7	2281
DC1.2	0.36	154	430	955	664	95	--	1.2	4.3	2307
DC1.5	0.36	166	474	938	638	118	--	1.5	4.8	2323

1 Saturated, surface dry

2 Percent by volume

in Table 2.3. Other mix designs will appear with the research for which they are applicable. No restrictions on water temperature were made for Duracal mixes.

2.4 TESTING PROCEDURE

Test specimens and testing procedures were standardized throughout the materials testing program. Specimens and procedure will be described in this section for reference throughout this report.

Compressive strengths were determined in accordance with ASTM C-39. Specimens were either 4-in. by 8-in. (100-mm by 200-mm) or 6-in. by 12-in. (150-mm by 300-mm) molded cylinders. Flexural strengths were determined in accordance with ASTM C-78, except specimens were not rotated with respect to position as molded unless otherwise noted. Specimen size was 3 in. by 3 in. by 15 in. (75 mm by 75 mm by 380 mm) with a span length of 14 in. (35.6 mm).

In the balance of this chapter compressive specimens will be referenced by their diameter; standard beam specimens will be referenced by the depth of cross section.

Expansion specimens were 3-in. by 3-in. by 10-in. (75-mm by 75-mm by 250-mm) prisms. Expansion specimens and length comparator were in accordance with ASTM C-490.

CHAPTER 3

TIME-DEPENDENT BEHAVIOR OF HARDENED FIBROUS CONCRETE

3.1 STRENGTH-TIME

3.1.1 INTRODUCTION

The increase of concrete strength with time is of significant interest to designers and constructors of any structure. Concretes with rapid setting cements, such as required for the extruded tunnel liner system (Parker, et al., 1971), require significant knowledge of early strength characteristics.

The strength-time development of steel fiber reinforced, regulated-set cement concretes has been reported for compression and flexure by previous investigators (Parker, et al., 1973; Paul, et al., 1974).

The scope of the current investigation in strength-time behavior is to determine the characteristics of steel fiber reinforced, Duracal cement concrete.

3.1.2 RESEARCH PROGRAM

The materials used in the strength-time investigation are described in Section 2.2. The cement was Duracal 5. The mix design appears in Table 2.3.

Seven separate casts were required to have a sufficient number of specimens for the complete investigation. The wet-mix properties of each cast is reported in Table 3.1.

TABLE 3.1

FRESH MIX PROPERTIES--STRENGTH-TIME TESTS OF STEEL FIBER REINFORCED, DURACAL CEMENT CONCRETE

Mix	Mix water temperature, F	(C)	Air temperature, F	(C)	Slump, in.	(mm)	Entrained air, percent	Unit weight wet, lb/cu ft	(kg/m^3)
1	59	(15.0)	70	(21.0)	7	(180)	4.6	146.0	(2338)
2	60	(15.5)	85	(29.5)	5-3/4	(145)	6.7	141.5	(2266)
3	58	(14.5)	85	(29.5)	5-3/4	(145)	6.7	142.0	(2274)
4	59	(15.0)	70	(21.0)	6-1/2	(165)	6.2	142.0	(2274)
5	59	(15.0)	70	(21.0)	7	(180)	6.1	142.0	(2274)
6	59	(15.0)	70	(21.0)	7	(180)	6.7	142.5	(2282)
7	59	(15.0)	70	(21.0)	7	(180)	6.7	142.0	(2274)

The test schedule was organized to allow testing of specimens at 2 hr, 8 hr, 1, 3, 7, 28, and 90 days after addition of water to the dry mix. For groups of specimens tested at seven days or less curing was at 72 F (22 C) 50 RH ("dry" cure) and 72 F (22 C) with full immersion in water ("wet" cure). At 28- and 90-day tests, specimens were cured in the dry and wet environments for the full curing period, and a further group cured 7 days in the wet environment and in the dry environment until testing ("wet-dry" cure).

For each condition of curing environment and testing age, 4-in. (100-mm) cylinders and 3-in. (75-mm) beams were tested as described in Section 2.4. The test program is summarized in Table 3.2.

TABLE 3.2

DATA FOR STRENGTH-TIME TESTS, STEEL FIBER REINFORCED, DURACAL CEMENT CONCRETE

		Cylinder failure load, kips (kN)			Beam failure load, lb (kN)		
Age	Mix	Wet	Dry	Wet-dry	Wet	Dry	Wet-dry
2 hr	1	11.75 (52.27)	11.15 (49.60)		460 (2.05)	430 (1.91)	
	2	10.40 (46.26)	10.40 (46.26)		330 (1.47)	400 (1.78)	
	3	9.50 (42.26)	10.35 (46.04)		280 (1.25)	320 (1.42)	
	4	10.60 (47.15)	9.75 (43.37)		360 (1.60)	330 (1.47)	
	5	10.15 (45.15)	9.05 (40.26)		450 (2.00)	400 (1.78)	
	Mean	10.48 (46.62)	10.14 (45.10)		376 (1.67)	376 (1.67)	
8 hr	1	11.50 (51.15)	12.75 (56.71)		390 (1.73)	510 (2.27)	
	2	10.20 (45.37)	13.40 (59.61)		440 (1.96)	430 (1.91)	
	3	10.50 (46.71)	13.20 (58.72)		380 (1.69)	390 (1.73)	
	4	12.50 (55.60)	12.35 (54.94)		400 (1.78)	450 (2.00)	
	5	11.90 (52.93)	12.00 (53.38)		410 (1.80)	430 (1.91)	
	Mean	11.32 (50.35)	12.74 (56.67)		404 (1.80)	442 (1.97)	
1 d	1	21.70 (96.53)	25.70 (114.3)		640 (2.85)	690 (3.07)	
	2	22.45 (99.86)	22.15 (98.53)		710 (3.16)	600 (2.67)	
	3	22.10 (98.31)	21.13 (93.99)		880 (3.91)	730 (3.25)	
	4	22.50 (100.1)	23.00 (102.3)		670 (2.98)	830 (3.69)	
	5	20.90 (92.97)	20.90 (93.00)		570 (2.54)	910 (4.05)	
	Mean	21.93 (97.55)	22.61 (100.6)		694 (3.09)	752 (3.35)	
3 d	1	26.00 (115.6)	35.40 (157.5)		910 (4.05)	980 (4.36)	
	2	27.10 (120.6)	32.60 (145.0)		850 (3.78)	1030 (4.58)	
	3	26.80 (119.6)	30.50 (135.7)		900 (4.00)	920 (4.09)	
	4	25.90 (115.2)	34.00 (151.2)		700 (3.11)	1200 (5.34)	
	5	27.65 (123.0)	31.50 (140.1)		800 (3.56)	930 (4.14)	
	Mean	26.69 (118.8)	32.80 (145.9)		832 (3.70)	1012 (4.50)	

(continued)

TABLE 3.2 (CONTINUED)

Age	Mix	Cylinder failure load, kips (kN)			Beam failure load, lb (kN)		
		Wet	Dry	Wet-dry	Wet	Dry	Wet-dry
7 d	1	28.10 (125.0)	39.20 (174.1)		840 (3.74)	1270 (5.65)	
	2	28.40 (126.3)	36.30 (161.5)		810 (3.60)	960 (4.27)	
	3	30.70 (136.6)	41.90 (186.4)		960 (4.27)	1290 (5.74)	
	4	30.90 (137.4)	40.10 (178.4)		870 (3.87)	1165 (5.18)	
	5	28.30 (125.9)	38.60 (171.7)		720 (3.20)	1130 (5.03)	
	Mean	29.28 (130.2)	39.22 (174.5)		840 (3.74)	1227 (5.46)	
28 d	1	36.00 (160.1)	52.60 (234.0)	53.10 (236.2)	1040 (4.63)		
	2	34.25 (152.4)	52.10 (231.8)	54.40 (242.0)			
	3	34.20 (152.1)	51.80 (230.4)	51.50 (229.1)	1190 (5.29)		
	4	36.40 (161.9)	42.30 (188.2)	51.60 (229.5)		1570 (6.98)	
	5	32.10 (142.8)	53.40 (237.5)	48.40 (215.3)	970 (4.31)		1130 (5.03)
	6				1010 (4.49)		1420 (6.32)
	6				970 (4.31)		1870 (8.32)
	6				1090 (4.85)		1780 (7.92)
	6						1460 (6.49)
	7					1610 (7.16)	
	7					1130 (5.03)	
	7					1180 (5.25)	
	7					1130 (5.03)	
	Mean	34.59 (153.9)	50.44 (224.4)	51.80 (230.4)	1045 (4.65)	1324 (5.89)	1532 (6.81)

(Continued)

TABLE 3.2 (CONTINUED)

Age	Mix	Cylinder failure load, kips (kN) Wet	Cylinder failure load, kips (kN) Dry	Cylinder failure load, kips (kN) Wet-dry	Beam failure load, lb (kN) Wet	Beam failure load, lb (kN) Dry	Beam failure load, lb (kN) Wet-dry
90 d	1		51.0 (226.9)	59.0 (262.4)	1400 (6.23)		
	2		49.0 (218.0)	58.7 (261.1)			
	3		53.3 (237.1)	58.4 (259.8)			
	4	40.6 (180.6)	53.5 (238.0)	61.25 (272.4)		2140 (9.52)	
	5	40.6 (180.6)	53.7 (238.8)	55.0 (244.7)	1130 (5.03)		1790 (7.96)
	6				1390 (6.18)		1740 (7.74)
	6				1400 (6.23)		2120 (9.43)
	6						1910 (8.50)
	6						1750 (7.78)
	7				1550 (6.89)	1670 (7.43)	
	7				1250 (5.56)	1880 (8.36)	
	7					2160 (9.61)	
	7					1900 (8.45)	
	Mean	40.6 (180.6)	52.1 (231.8)	58.5 (260.2)	1355 (6.03)	1950 (8.67)	1860 (8.27)

3.1.3 RESULTS

Early strengths are reported in Table 3.3. Results for the entire test series are reported in Figs. 3.1 and 3.2 for compression and flexure, respectively.

TABLE 3.3

EARLY STRENGTH OF STEEL FIBER REINFORCED DURACAL CEMENT CONCRETE

Time after casting	Mean compressive strength, psi (MPa)		Mean flexural strength, psi (MPa)	
	Wet	Dry	Wet	Dry
2 hr	830 (57.2)	810 (55.8)	210 (14.5)	210 (14.5)
8 hr	900 (62.0)	1010 (69.6)	225 (15.5)	245 (16.9)
1 d	1750 (120.6)	1760 (121.3)	385 (26.5)	420 (29.0)
3 d	2120 (146.2)	2610 (180.0)	460 (31.7)	560 (38.6)
7 d	2330 (160.6)	3120 (215.1)	465 (32.1)	635 (46.9)

Compressive and flexural strengths of dry-cured specimens are greater than for companion wet-cured specimens at ages greater than 8 hr. The effect of the wet-dry cure can not be definitely stated.

At a given age the Duracal cement concretes exhibited lower strengths than do regulated-set cement mixes with a similar cement content reported by Paul, et al. (1974). This is a consequence of the Duracal composition which attains most early strength from the setting of the gypsum; hydration

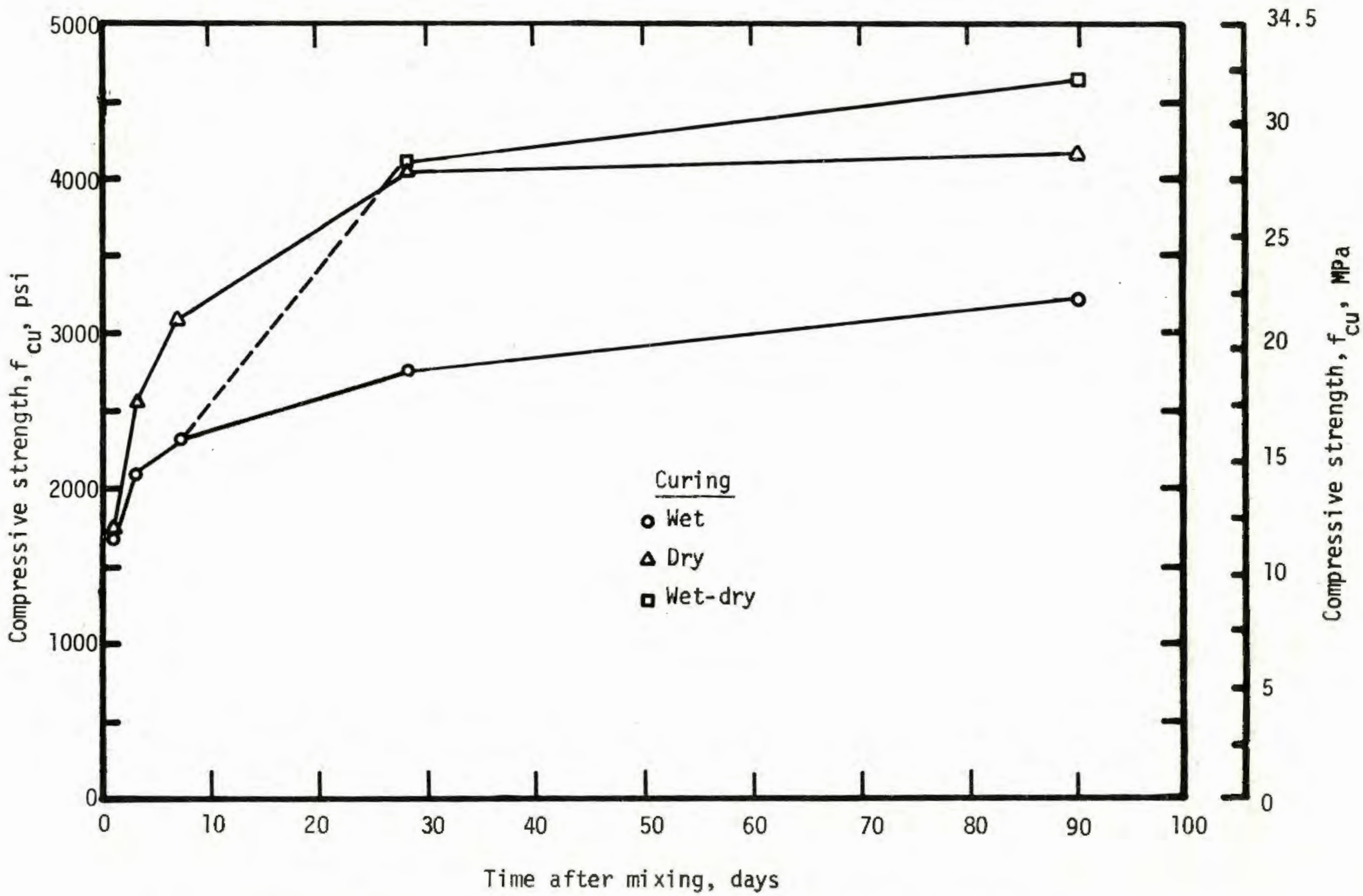


FIG. 3.1 COMPRESSIVE STRENGTH VS. TIME FOR DURACAL CEMENT CONCRETE WITH 1.2 PERCENT BY VOLUME STEEL FIBER REINFORCEMENT

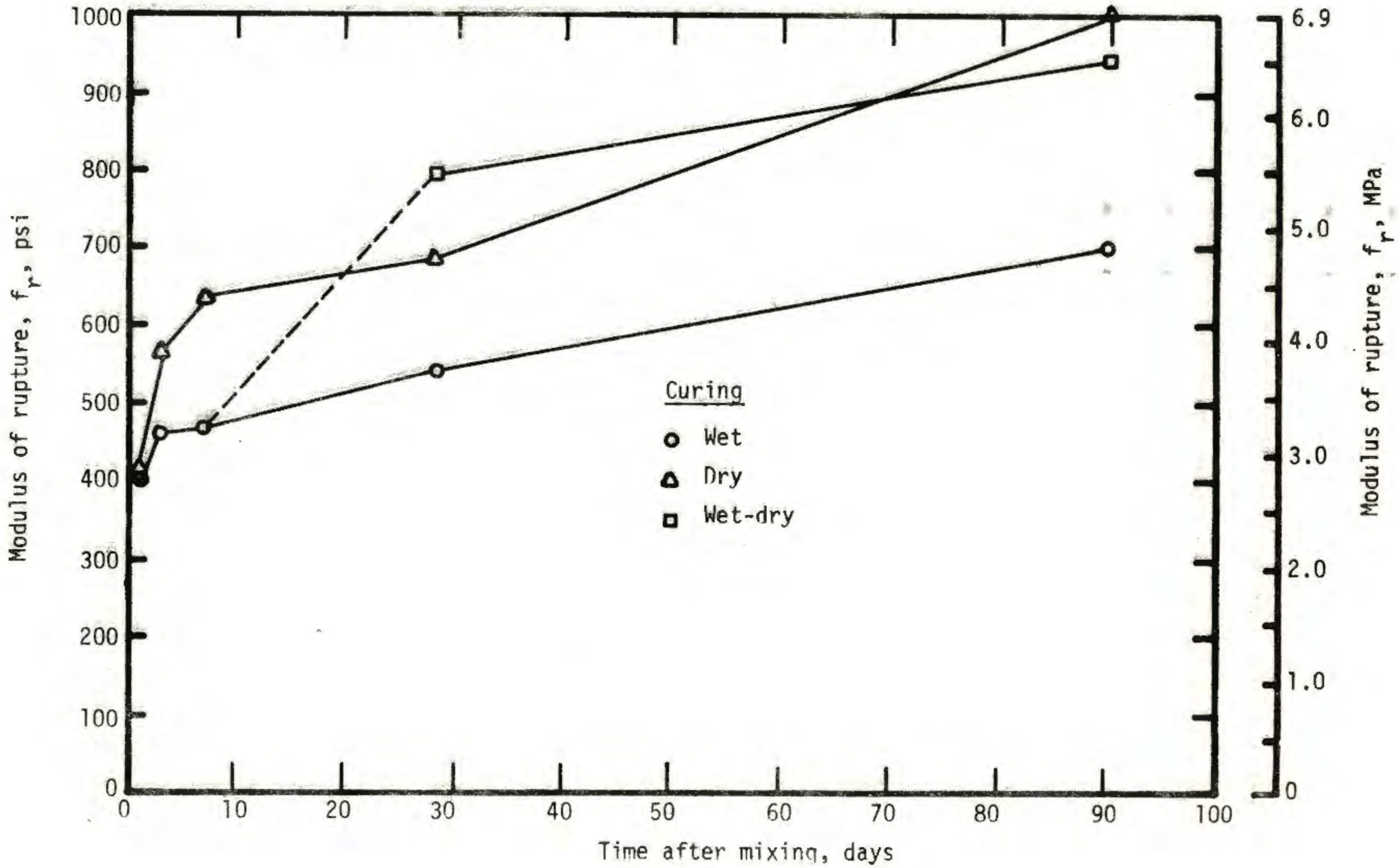


FIG. 3.2 FLEXURAL STRENGTH VS. TIME FOR DURACAL CEMENT CONCRETE WITH 1.2 PERCENT BY VOLUME STEEL FIBER REINFORCEMENT

of the portland cement results in longer term strength gain. The suitability of Duracal cement concretes to a particular application is thus primarily a function of requisite early strength and handling behavior.

3.2 CREEP

3.2.1 TESTING PROGRAM

Creep tests have been performed to develop additional information on the properties of hardened steel fiber reinforced, quick setting cement concretes. The test and curing procedures were in accordance with ASTM C-512. The test for steel fiber reinforced, regulated-set cement concrete used the 1.5 percent fiber mix of Table 2.3 for reg-set 3. The mix design for steel fiber reinforced Duracal cement concrete appears in Table 3.4. The mix design was based on Duracal 3 with 1.2 percent reinforcement. Fresh mix properties for reg-set 3 and Duracal 3 appear in Table 3.5.

Each group of specimens was loaded to 1000 psi (6.9 MPa), applied by a spring in a typical creep rack. For each test 6-in. (150-mm) vertically cast cylinders were used for the loaded and shrinkage control specimens. A cylinder was cut in half for use as a plug at the ends of each rack.

3.2.2 RESULTS

Total (elastic plus creep) strains are plotted against time after loading for the fibrous reg-set 3 and Duracal 3 cement concretes in Figs. 3.3 and 3.4, respectively. In each case a line of best fit is plotted through the data points. Shown in Table 3.6 are the strengths of control

TABLE 3.4

MIX DESIGN FOR CREEP TEST -
STEEL FIBER REINFORCED, DURACAL CEMENT CONCRETE

Material	Quantity	
	lb /cu yd	kg/m^3
Cement[1]	736	437
Water	280	166
Sand[2]	1665	988
Pea gravel[2]	1159	688
Steel fiber[3]	160	95

[1] Duracal 3

[2] Saturated, surface dry

[3] 1.2 percent by volume

TABLE 3.5

CREEP TEST FRESH MIX PROPERTIES

	Reg-set 3	Duracal 3
Slump in.(mm)	4-1/4 (105)	8-1/4 (210)
Air, percent	3.9	5.2
Unit weight wet, lb/cu ft (kg/m^3)	145.4 (2329)	148.5 (2379)

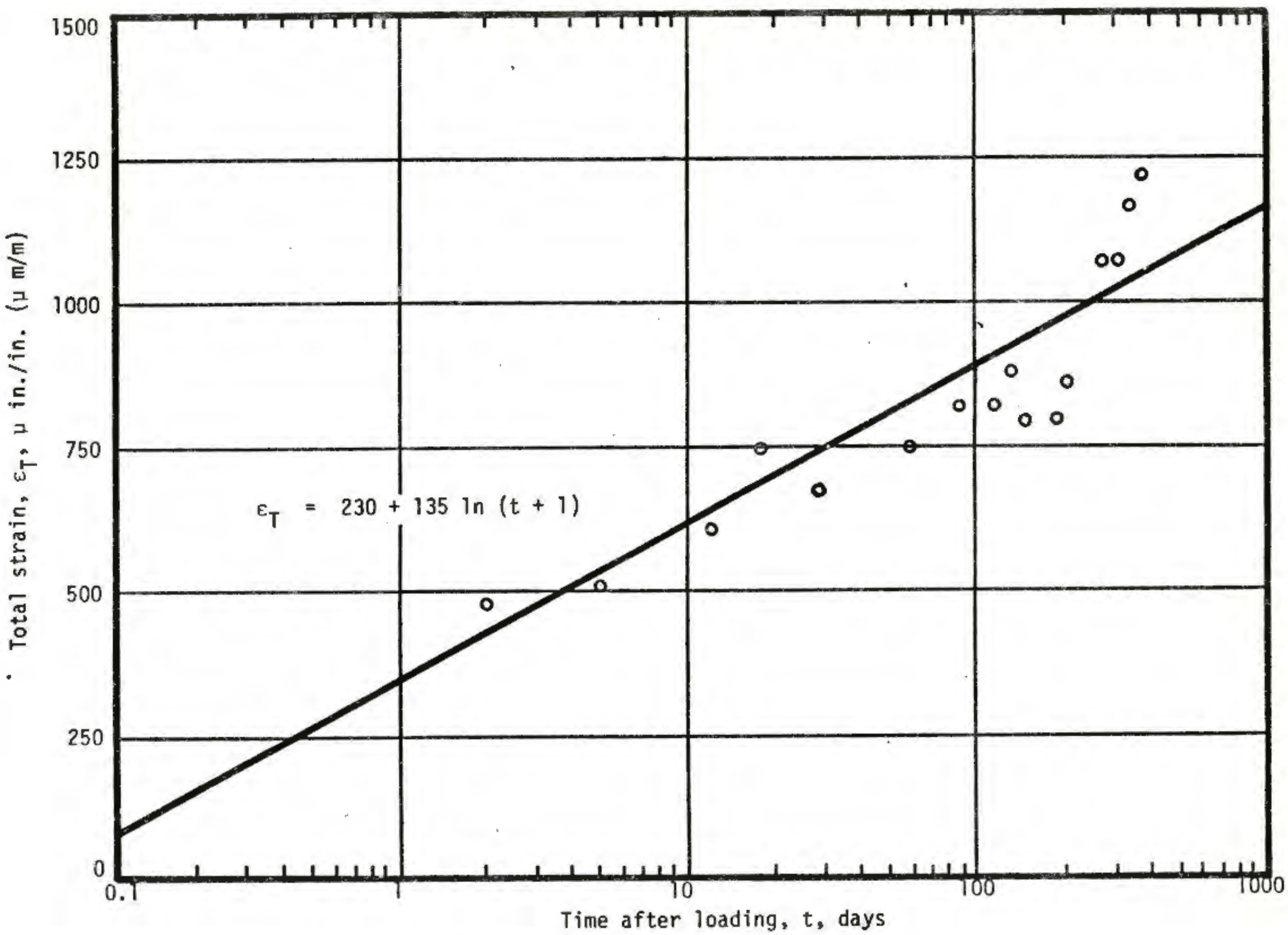


FIG. 3.3 CREEP TEST OF STEEL FIBER REINFORCED, REGULATED-SET CEMENT CONCRETE

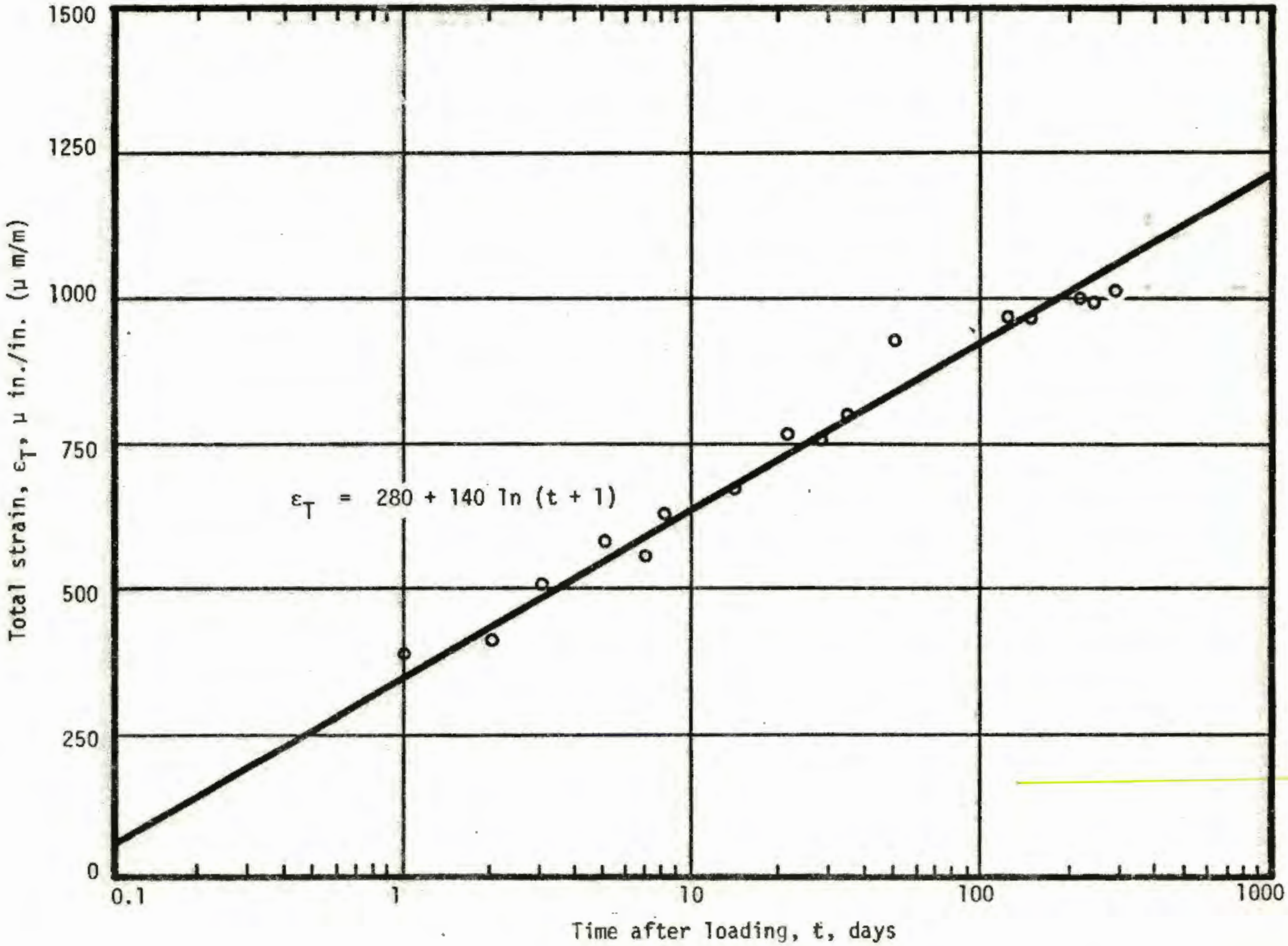


FIG. 3.4 CREEP TEST OF STEEL FIBER REINFORCED, DURACAL CEMENT CONCRETE

TABLE 3.6

CREEP TEST RESULTS

	Reg-set 3	Duracal 3
Compressive strength at loading, psi (MPa)	7720 (53.3)	3440 (23.7)
Instantaneous elastic strain, ε_o μ in./in. (μm/m)	230	280
1-day creep strain, μ in./in. (μm/m)	120	70
90-day creep strain, μ in./in. (μm/m)	650	660
1-yr creep strain μ in./in. (μm/m)	820	800
Creep rate, F(k) μ in./in. (μm/m)	135	140

cylinders, creep strains of interest and constants for the U. S. Bureau of Reclamation (1955) equation

$$\varepsilon_T = \varepsilon_o + F(K) \ln (t + 1) \qquad (3.1)$$

where

ε_T = total strain, μ in./in. (μm/m)

ε_o = instantaneous elastic strain, μ in./in. (μm/m)

F(K) = creep rate, μ in./in. (μm/m)

t = time after loading, days

The presentation of the results in this form is not meant to imply the independence of shrinkage and creep nor should a theoretical law be inferred (Neville, 1973; Philleo, 1966).

Both concretes were loaded to levels where creep is assumed to vary linearly with stress. The creep rates obtained for the Bureau of Reclamation equation for the two concretes are nearly identical although the water-cement ratios and cement types are different. The observed creep strains are in the spectrum reported from references by Wallo and Kesler (1968) for plain concrete.

CHAPTER 4

BEHAVIOR OF FIBER REINFORCED CONCRETE UNDER AXIAL AND COMBINED LOADS

4.1 SCOPE

This research program was undertaken to study the behavior of fiber reinforced concrete under the action of combined compressive and flexural loadings, to determine its elastic properties, and to determine a tensile stress-strain relationship from flexural tests. The major variable in the study was the percentage of fiber present in a mix. All tests were performed with three fiber contents, 0.9, 1.2, and 1.5 volume percent. Tests were performed using Duracal and regulated-set cement concretes with the mix designs given in Table 3.3

Poisson's ratio, ν, and Young's modulus in compression, E_c, were determined for each fiber content for Duracal and regulated-set cement concretes from a series of cylinder tests. The effect of fiber orientation on these two quantites was also studied.

A series of beam-columns was cast from Duracal cement concrete to determine the effect of various fiber contents on the behavior of this fiber reinforced concrete under the action of combined compression and flexure. The failure envelope was determined for each fiber content.

The tensile stress-strain relationships for the three Duracal cement concrete mixes were determined from a series of flexural tests of 6-in. by 6-in. by 64-in. (150-mm by 150-mm by 1625-mm) beams. The tensile stress-strain relationships that were developed indicate the generalized tensile behavior of a beam element surrounding a crack. This information is essential in the post-crack analysis of a structure.

4.2 CURING PROCEDURE

For 21 days after casting all specimens were stored in a fog room at 75 to 80 F (23 to 27 C) 100 RH. For 7 additional days the specimens were stored in a nominal 72 F (22 C) 50 RH laboratory environment to facilitate the application of strain gages were required. All specimens were tested 28 days after casting.

4.3 YOUNG'S MODULUS IN COMPRESSION AND POISSON'S RATIO

4.3.1 SPECIMEN DESCRIPTION

A set of six 6-in. (150-mm) cylinders and three 3-in. (75-mm) beams were cast for each of the three fiber contents using both Duracal and regulated-set cement concretes.

To obtain the two extreme tendencies of fiber orientation three of each set of six cylinders were vibrated parallel to the longitudinal axis and three were vibrated perpendicular to the longitudinal axis. A long, thin body suspended in a liquid medium tends to become oriented in a direction perpendicular to that of wave motion. Thus, for specimens vibrated parallel to the longitudinal axis fiber orientation was perpendicular to the longitudinal axis of the cylinder while the fiber orientation was parallel to the longitudinal axis of cylinders vibrated perpendicular to the longitudinal axis.

Longitudinal strain was measured using a standard compressometer shown in Fig. 4.1. Lateral strains were measured using an unbonded gage shown in Fig. 4.2. Measurement of strain with an unbonded rather than a bonded gage was chosen to give an accurate measure of the average lateral strain, independent of minor local failures on the cylinder surface.

4.3.2 RESULTS

The resulting **values** of Poisson's ratio and Young's modulus corresponding to the various mixes and direction of vibration are shown in Table 4.1. These quantities were determined according to ASTM C-469. The values indicated in the table are the average of three separate tests.

The results indicate that Poisson's ratio, ν, was not influenced by any of the variables and that it can be taken as 0.14 ± 0.02.

The values of a secant Young's modulus in compression, E_c, shown in Table 4.1 indicate that E_c is higher for the regulated-set cement concrete than the Duracal cement concrete for corresponding fiber contents and orientations. The results also indicate a definite dependence of E_c on fiber orientation. The values of E_c within a batch were consistently higher in the cylinders for which the tendency of fiber orientation was parallel to the longitudinal axis than those in which the tendency of orientation was perpendicular to the longitudinal axis. However, the compressive strengths were higher in the cylinders with fiber orientation perpendicular to the longitudinal axis.

4.3.3 CONCLUSIONS

The results seem to indicate that Poisson's ratio is insensitive to difference in cement, fiber percentage, and direction of fiber orientation.

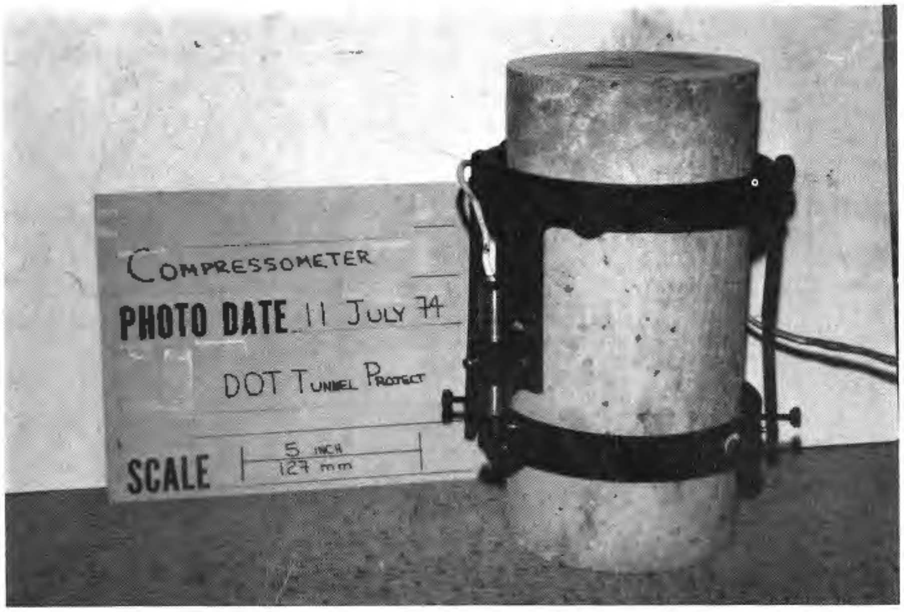


FIG. 4.1 COMPRESSOMETER

FIG. 4.2 LATERAL STRAIN BAND

TABLE 4.1

COMPRESSIVE YOUNG'S MODULUS AND POISSON'S RATIO FOR EXPERIMENTAL CONCRETES

Fiber content, percent	Cement	Direction of vibration	f'_c, psi (MPa)	f_r, psi (MPa)	E_c, 10^3 ksi (GPa)	ν
0.9	Duracal	Horizontal	3440 (23.4)	775 (5.36)	3.37 (23.2)	0.14
		Vertical	4040 (27.9)		3.04 (21.0)	0.14
1.2	Duracal	Horizontal	3840 (27.5)	1010 (6.95)	3.13 (21.6)	0.12
		Vertical	4190 (28.9)		2.93 (20.2)	0.12
1.5	Duracal	Horizontal	3870 (26.7)	950 (6.56)	3.23 (22.3)	0.16
		Vertical	4210 (29.0)		2.96 (20.4)	0.15
0.9	Reg-set	Horizontal	5610 (38.7)	875 (6.03)	3.53 (24.3)	0.12
		Vertical	6460 (44.5)		3.06 (21.1)	0.11
1.2	Reg-set	Horizontal	6570 (45.3)	1080 (7.45)	3.82 (26.3)	0.16
		Vertical	7250 (50.0)		3.47 (23.9)	0.15
1.5	Reg-set	Horizontal	6390 (44.1)	1110 (7.65)	3.68 (25.4)	0.16
		Vertical	6870 (47.4)		3.19 (22.0)	0.12

Further, it tends to be lower than for plain concrete due to the fibers inhibiting lateral expansion of the composite material. The modulus of elasticity shows a definite dependence on fiber orientation. The values given here for E_c and ν seem to be representative of those that can be used in design for the particular mixes studied. Speculation as to representative values of these quantities over a wide range of compressive strengths is impossible due to the narrow regions of compressive strength dealt with in this study.

4.4 BEHAVIOR OF FIBER REINFORCED CONCRETE UNDER COMBINED COMPRESSIVE AND FLEXURAL LOADING

4.4.1 SPECIMEN DESCRIPTION

Four beam-columns, the details of which are shown in Fig. 4.3, were cast for each of the fiber contents of 0.9, 1.2 and 1.5 percent by volume using the Duracal cement concrete. Three standard 3-in. (75-mm) flexural specimens and three standard 4-in. (100-mm) cylinders were cast with each of the beam-column specimens as control. The beam-columns were cast in a horizontal position and vibrated externally so as not to radically disturb fiber distribution in spots through internal vibration.

4.4.2 TESTING PROCEDURE

A beam-column under test is shown in Fig. 4.4. The strain distribution across the cross section at the center of the prismatic section was determined using a total of six strain gages placed on the specimen as indicated in Fig. 3.4. An average of each pair of gages on the compression face, tension face, and midway between the two faces was taken as the strain at that portion of the cross section. The strain gages had a 6-in. (152.4-mm) gage length which was considered adequate to give representative strain readings since the coarse aggregate used in the specimen was 3/8-in. (10-mm) pea gravel. The deflection of the prismatic section of the specimen was measured at five equally spaced points along this section by dial indicators to determine the actual eccentricity at failure. The load was applied to each specimen in 12 to 15 increments to failure and readings of all gages were taken at the end of each increment.

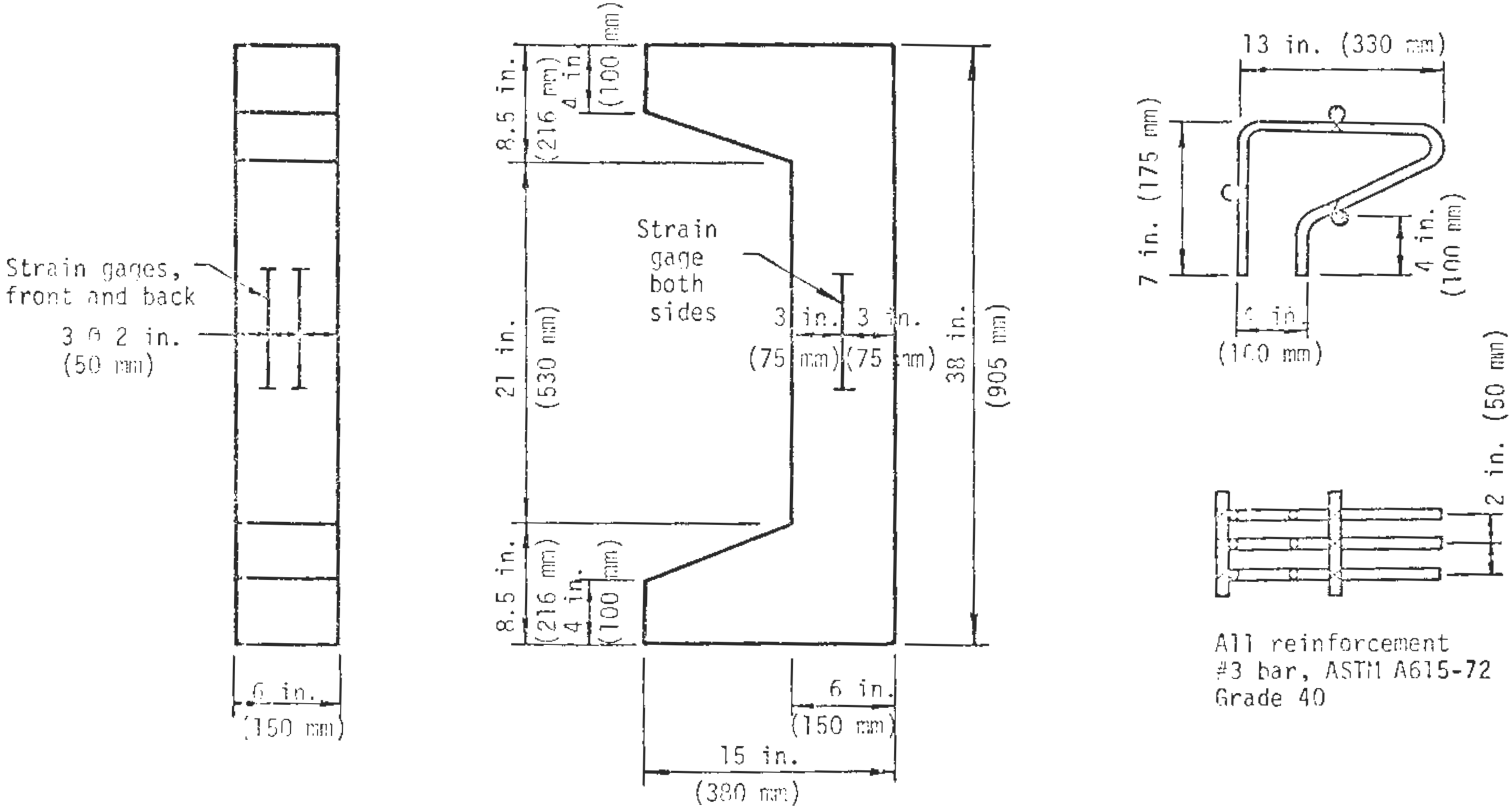


FIG. 4.3 DETAILS OF BEAM-COLUMN AND CAPITAL REINFORCEMENT

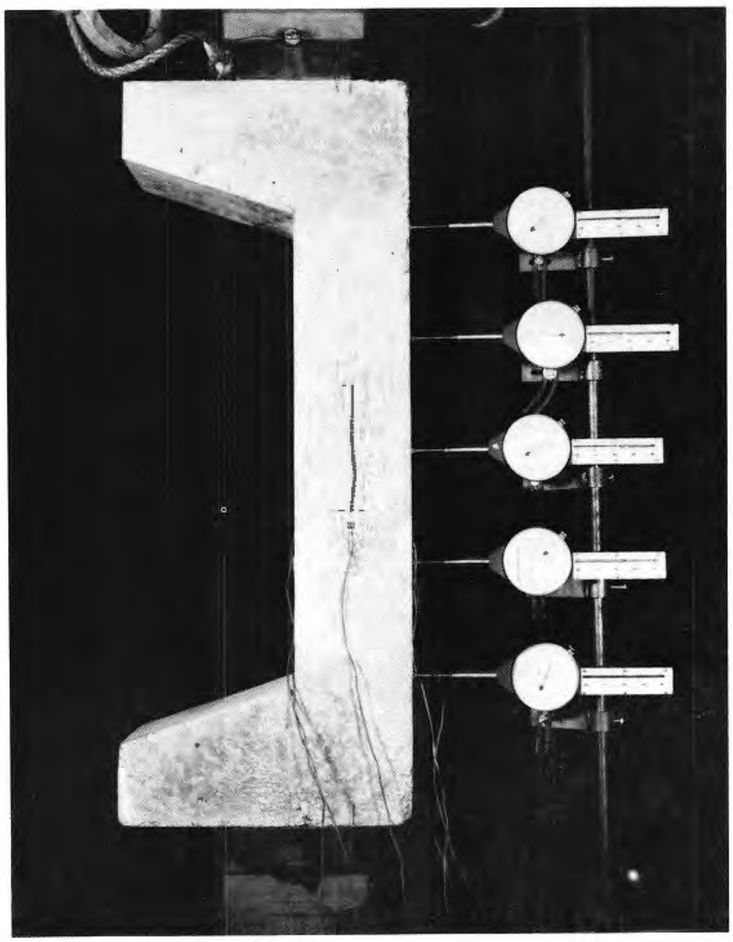


FIG. 4.4 BEAM-COLUMN TEST

Since reading of the strain gages and dial indicators at failure was impossible, the values of all gage readings corresponding to failure were obtained by plotting the readings against load and extrapolating to the failure load. Within each fiber group specimens were tested at eccentricities of 0.0, 0.5, 1.75 and 3.0 in. (0. 13, 44 and 76 mm). Previous research (Paul, et al., 1974) indicated that these points would be sufficient to define the ultimate interaction diagram for a given fiber percentage.

4.4.3 RESULTS

The ultimate interaction diagrams for Duracal cement concrete with fiber contents of 0.9, 1.2 and 1.5 percent are shown in Figs. 4.5, 4.6 and 4.7, respectively. It will be noted that there is an additional point on each of these graphs at the pure flexural loading point. This additional point was found from the results of the 6-in. by 6-in. by 64-in. (150-mm by 150-mm by 1625-mm) beams tests used for the determination of the tensile stress-strain relationships. A comprehensive summary of the results of the beam-column tests is given in Table 4.2. Typical strain distributions across the cross section at failure are shown in Fig. 4.8 for the four beam-columns containing 0.9 percent fiber.

Illustrations of the types of failures indicated for the specimens are shown in Figs. 4.9, 4.10 and 4.11. The ranges of relative eccentricity, e/d, over which these types of failure can be expected to occur have been previously determined (Paul, et al., 1974) to be 0.0 to 0.25 for compression failures, 0.25 to 0.33 for compression-tension failure, and greater than 0.33 for tension failures. The compression failures are characterized by

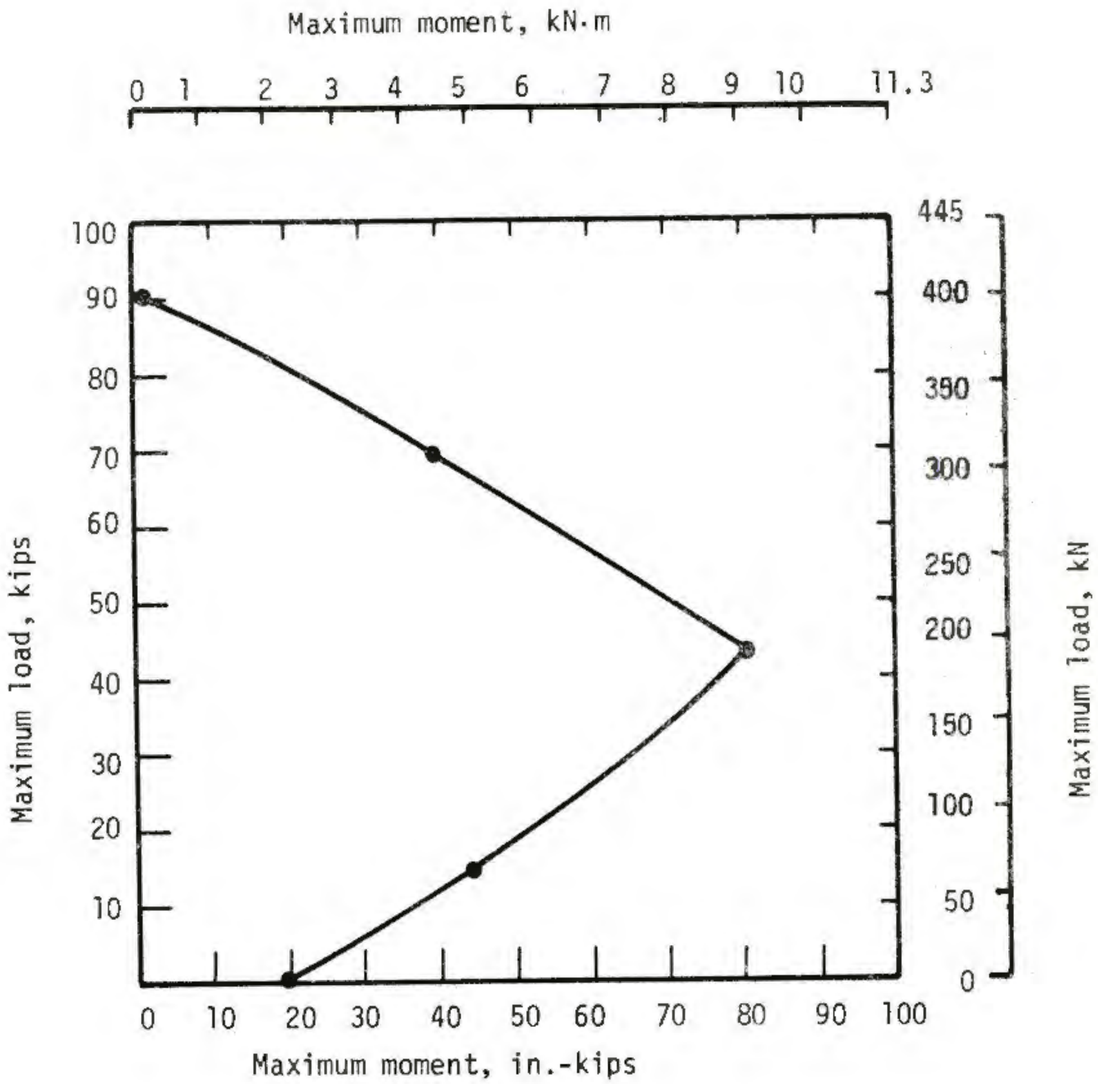


FIG. 4.5 ULTIMATE INTERACTION DIAGRAM FOR 0.9 PERCENT FIBER CONTENT - DURACAL CEMENT CONCRETE

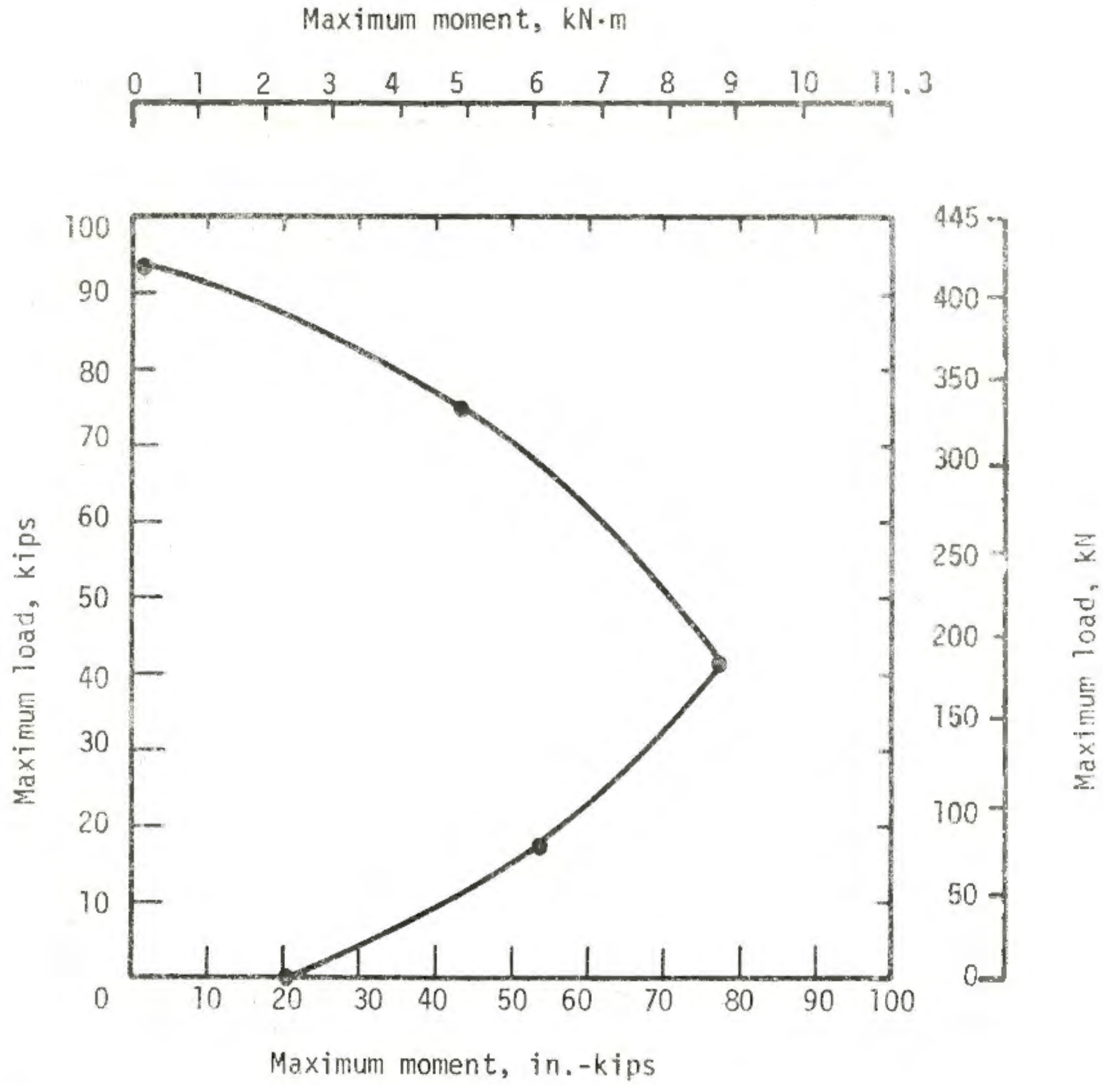


FIG. 4.6 ULTIMATE INTERACTION DIAGRAM FOR 1.2 PERCENT FIBER-DURACAL CEMENT CONCRETE

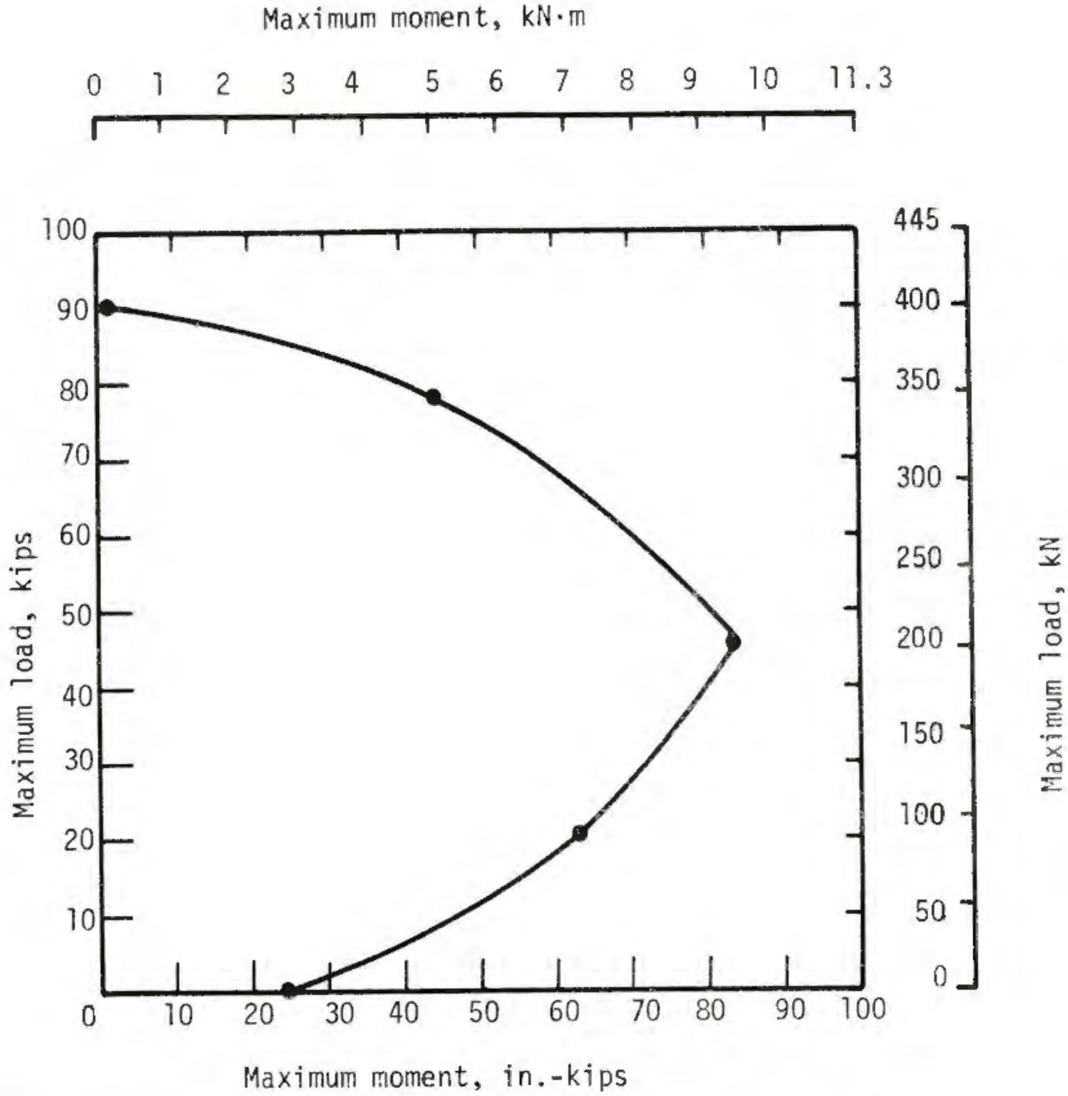


FIG. 4.7 ULTIMATE INTERACTION DIAGRAM FOR 1.5 PERCENT FIBER-DURACAL CEMENT CONCRETE

TABLE 4.2

RESULTS OF 8EAM-COLUMN TESTS

Specimen	Age at test, days	Fiber content, percent	e in.(mm)	p kips(kN)	M in.-kips (kN·m)	Δe in.(mm)	Compressive strength, psi (MPa)	Flexural strength psi(MPa)	ε_{cu}	Mode of failure*
MI-D-0.9A	28	0.9	0.0 (0.0)	90.48 (402.5)	1.27 (0.143)	0.014 (0.36)	3570 (24.6)	642 (4.43)	0.00209	C
MI-0-0.9B	28	0.9	9.50 (12.70)	69.40 (308.7)	39.38 (4.449)	0.068 (1.73)	3530 (24.3)	666 (4.59)	0.00266	C
MI-D-0.9C	28	0.9	1.75 (44.45)	43.50 (193.5)	80.54 (9.100)	0.102 (2.59)	3540 (24.4)	658 (4.54)	0.00205	CT
MI-D-0.90	28	0.9	3.00 (76.20)	14.31 (63.7)	44.15 (4.988)	0.085 (2.16)	3560 (24.6)	585 (4.03)	---	T
MI-0-1.2A	28	1.2	0.0 (0.0)	93.70 (416.8)	1.69 (0.191)	0.018 (0.46)	4010 (27.7)	723 (4.98)	0.00174	C
MI-D-1.2B	28	1.2	0.50 (12.70)	74.77 (332.6)	43.67 (4.934)	0.084 (2.13)	3680 (25.4)	772 (5.32)	0.00253	C
MI-0-1.2C	28	1.2	1.75 (44.45)	41.48 (184.5)	77.51 (8.757)	0.121 (3.07)	3830 (26.4)	823 (5.67)	0.00262	CT
MI-D-1.2D	28	1.2	3.00 (76.20)	17.48 (77.8)	53.91 (6.091)	0.086 (2.18)	3780 (26.1)	896 (6.18)	--	T
MI-D-1.5A	28	1.5	0.0 (0.0)	90.45 (402.3)	1.56 (0.176)	0.001 (0.03)	3970 (27.4)	984 (6.78)	0.00172	C
MI-0-1.5B	28	1.5	0.50 (12.70)	77.89 (346.5)	44.09 (5.981)	0.066 (1.68)	3940 (27.2)	921 (6.35)	0.00241	C
MI-0-1.5C	28	1.5	1.75 (44.45)	44.49 (197.9)	83.82 (9.470)	0.134 (3.40)	3880 (26.8)	885 (6.10)	0.00272	CT
MI-D-1.5D	28	1.5	3.00 (76.20)	20.39 (90.7)	63.27 (7.149)	0.103 (2.62)	3840 (26.5)	942 (6.49)	--	T

* C denotes a compression failure; CT denotes a combined tension and compression failure and T denotes a tension failure.

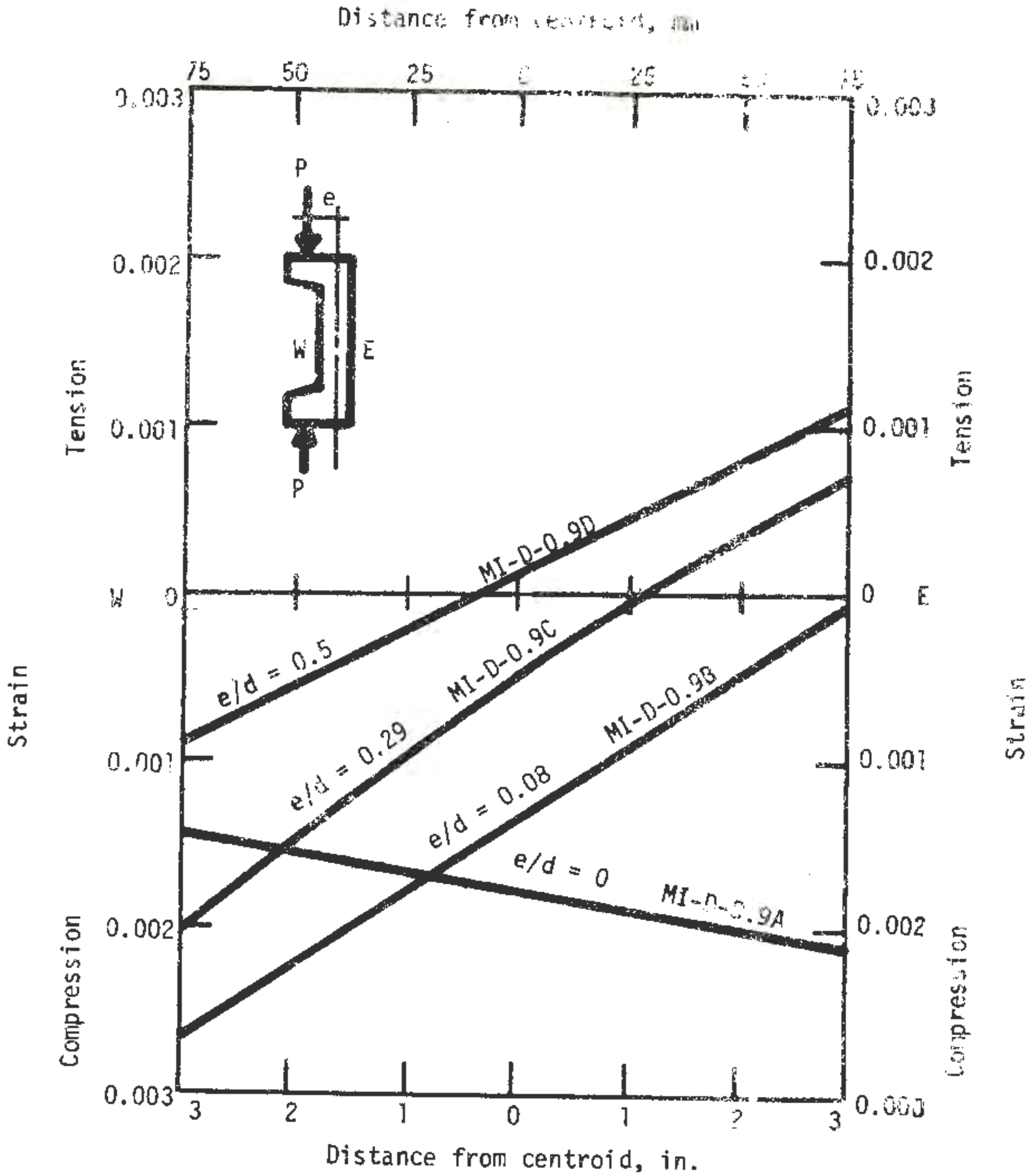


FIG. 4.8 ULTIMATE STRAIN DISTRIBUTIONS

FIG. 4.9 TENSION FAILURE

FIG. 4.10 COMPRESSION-TENSION FAILURE

FIG. 4.11 COMPRESSION FAILURE

severe spalling on all surfaces at failure. The tension failures are characterized by tensile cracks forming through the specimen with no spalling of concrete on the compression face at failure. The combined compression and tension failures are characterized by almost simultaneous spalling on the compression face and tensile cracking on the tension face.

The ultimate compressive stress distribution can be described using the nomenclature of Winter and Nilson (1972) where $\alpha = f_{avg}/f_c'$ and β defines the location of the compression resultant as a fraction of the depth to the neutral axis from the compression face. Specimens MI-D-0.9B, MI-D-1.2B, and

MI-D-1.5B were used to determine α and β as reported in Table 4.3. These specimens were chosen since the compressive strains on the tension faces were very small at failure. By considering equilibrium of the failed cross section as shown in Fig. 4.12, α and β are determined as follows:

$$\alpha = \frac{P_{max} + \frac{E_c \varepsilon_c}{2} (c - h)\, b}{f_c' \, bc} \tag{4.1}$$

$$\beta = \frac{h}{2c} - \left(\frac{M_{max} + \frac{E_c \varepsilon_c}{2} (c - h)\left(\frac{c}{3} + \frac{h}{6} \right) b}{\alpha \, f_c' \, b \, c^2} \right) \tag{4.2}$$

TABLE 4.3

STRESS BLOCK CONSTANTS

Specimen	Fiber content percent	α	β
MI-D-0.9B	0.9	0.53	0.40
MI-D-1.2B	1.2	0.56	0.40
MI-D-1.5B	1.5	0.54	0.40
Average		0.54	0.40

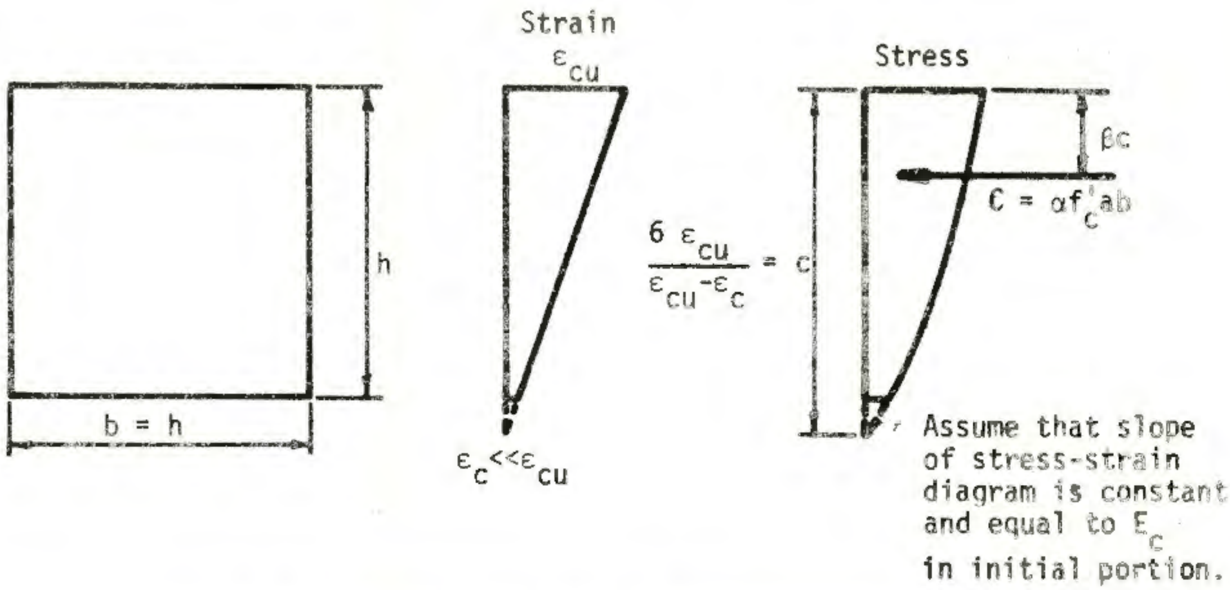


FIG.4.12 ULTIMATE STRAIN AND STRESS DISTRIBUTIONS FOR BEAM-COLUMNS

An average value of E_c taken from the cylinder tests was used although inclusion of that term in the above equations is academic since its effect is negligible due to the small magnitude of ε_c.

4.4.4 CONCLUSIONS

The ultimate interaction diagrams presented here are for the Duracal mixes shown in Table 2.3. When used in design, an appropriate factor of safety should be determined by the designer. In an ultimate strength design, like that of ACI 318-71, the results of Table 4.2 indicate that the compressive strain in the outer fibers at failure in compression, ε_{cu}, may be taken as 0.0023 and that linearity of strain is a valid as sumption. When using the convention of a fully developed compressive stress block, the values of α and β associated with these mixes can be con sidered to be 0.54 and 0.40, respectively.

4.5 TENSILE STRESS-STRAIN BEHAVIOR IN FLEXURE FOR STEEL FIBER REINFORCED DURACAL CEMENT CONCRETE

4.5.1 SPECIMEN DESCRIPTION

A series of 6-in. by 6-in. by 64-in. (150-mm by 150-mm by 1625-mm) beams was tested to determine the generalized tensile stress-strain behavior, the behavior of an element of the beam around a crack. With each set of three large beams three 4-in. (100-mm) cylinders and three 3-in. (75-mm) standard flexural specimens were cast as control. The large beams were cast in a horizontal position and vibrated externally so as not to disturb fiber orientation at points by internal vibration.

4.5.2 TESTING PROCEDURE

A 6-in. by 6-in. by 64-in. (150-mm by 150-mm by 1625-mm) flexural specimen under test is shown in Fig. 4.13. The distance between the outer simple supports was 60 in. (1525 mm) and the length of the section under pure flexure was 40 in. (1015 mm). The deflections of the central portion of the beam were measured at the center and 18 in. (457 mm) each side of the center by cantilevered deflection gages. In this way, only the center deflection of a 36-in. (915 mm) segment under pure flexural loading was used in the analysis. This minimized the inaccuracies that are induced as a result of the disturbances in stress distribution around the loading points. The

FIG. 4.13 TEST OF LARGE BEAM SPECIMEN

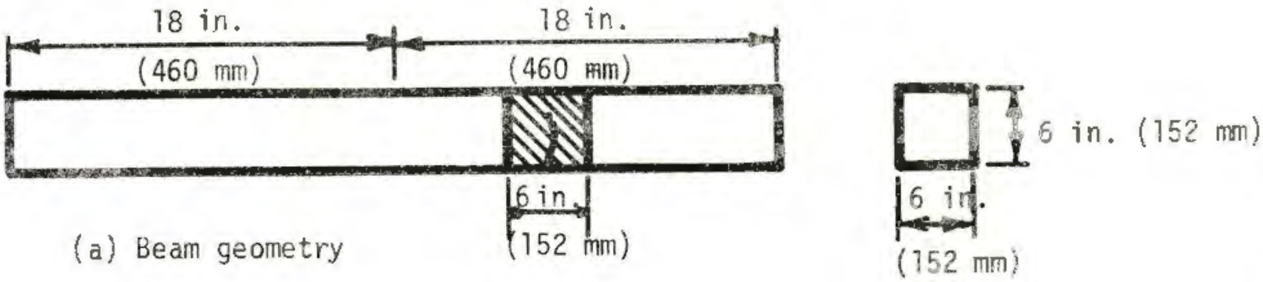
difference between the deflection measured at the center and the average of the deflections at the two outer points were recorded continuously while load was applied.

4.5.3 METHOD OF ANALYSIS

The derived tensile stress-strain relationships are the generalized tensile stress-strain behavior of an element in a flexural specimen surrounding a crack as indicated by the shaded area of Fig. 4.14a. The length of the element over which the effects of cracking are distributed is assumed to be the depth of the beam. The point at which cracking begins in a specimen is at the discontinuity of slope in the load-deflection curve for the specimen as indicated in Fig. 4.15a. This point of first cracking has also been noted by Snyder and Lankard (1972). The shape of the tensile stress-strain relationship should be similar to that of a load-deflection curve obtained from a uniaxial tension test as shown in Fig. 4.16.

In order to proceed from the load-deflection curve to the moment-curvature relation for the crack-element, the following assumptions are made:

1. The proportional limit is the point of discontinuity in the load-deflection curve.
2. The curvature in the beam section up to the proportional limit is constant along the length and determined from $\phi = \frac{M}{EI}$.
3. After cracking initiates the material outside the crack-element continues to behave in a linearly elastic fashion; within the crack-element there is a uniform curvature of



(a) Beam geometry

(b) End moments and deflected shape

(c) Moment diagram

(d) Curvature diagram

FIG. 4.14 BEAM SECTION USED FOR ANALYSIS

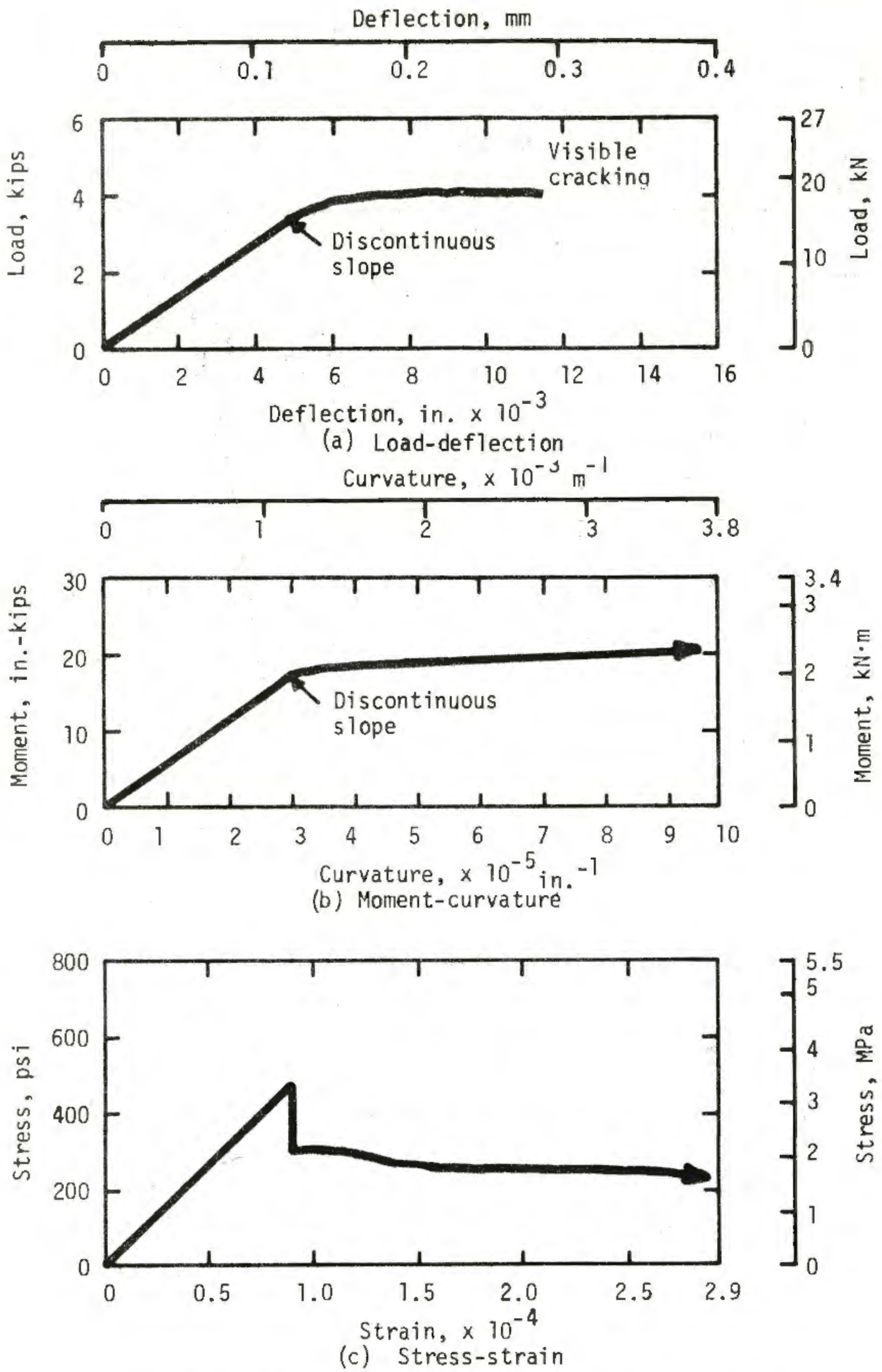


FIG. 4.15 DEVELOPMENT OF STRESS-STRAIN RELATIONSHIPS

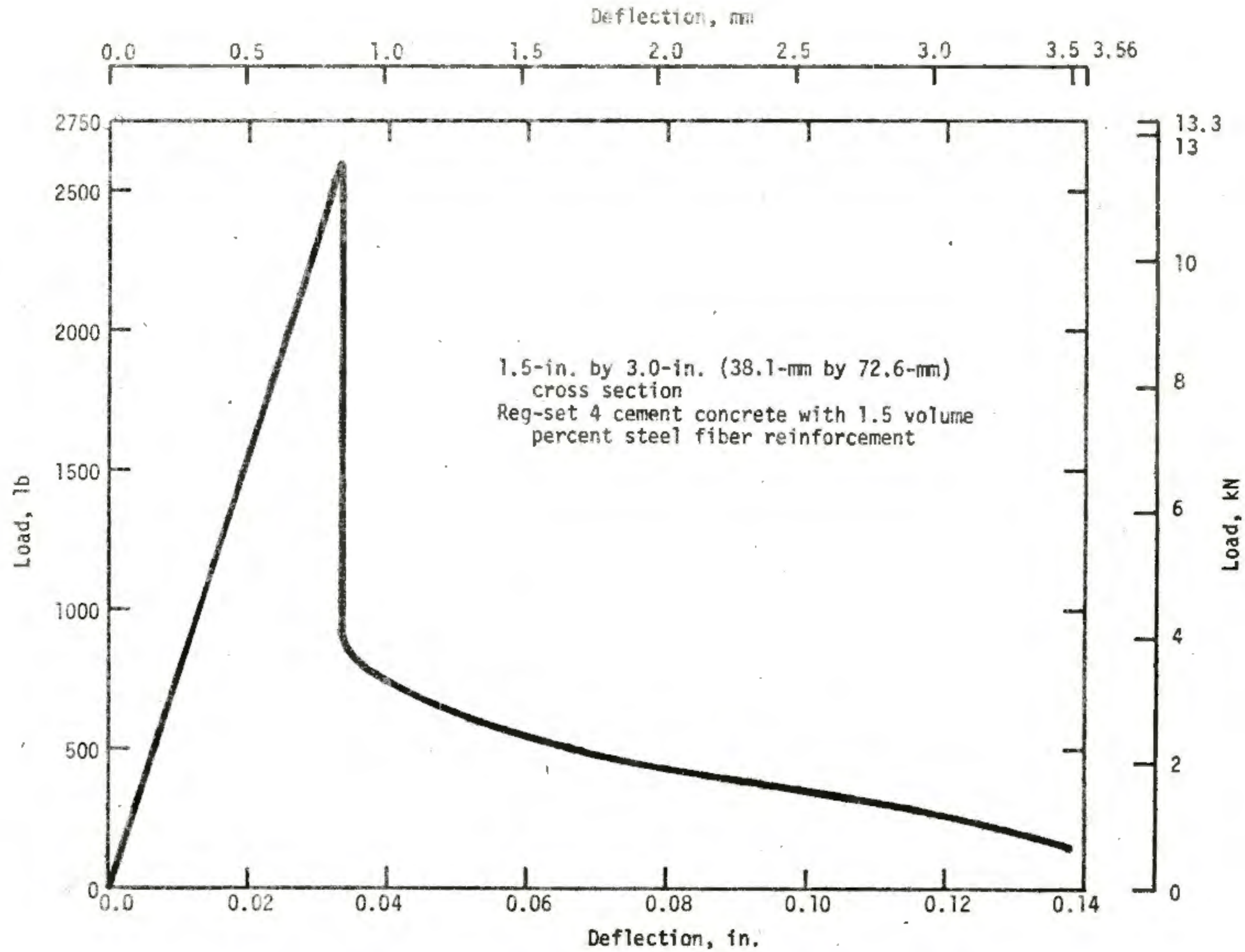


FIG. 4.16 LOAD-DEFLECTION CURVE FOR UNIAXIAL TENSION TEST

magnitude greater than the curvature in the uncracked region of the beam.

Under these assumptions, using the measured load and deflection and applying the moment-area theorems, the moment-curvature relationship in the crack-element is determined.

At cracking there must be a sharp drop in the tensile stress-strain relationship corresponding to the slope discontinuities of the load-deflection and moment-curvature diagrams indicated in Fig. 4.15 a and b. The tensile stress-strain relationships are obtained from the moment-curvature diagram by making the further assumptions:

1. The tensile stress-strain relation is linear up to the proportional limit.
2. The elastic moduli in tension and compression are equal.
3. Compressive behavior is linear to failure of the specimen.
4. Strain varies linearly through the depth of the cross section.
5. All points on the cross section have identical stress-strain behavior.

Under the assumption of linearly elastic behavior up to the proportional limit, the peak of the tensile stress-strain curve is defined. The magnitude of the stress drop at this point is determined by consideration of the magnitude of the slope change in the moment-curvature curve at the point of discontinuity. The remainder of the tensile stress-strain relationship is then obtained by gradually incrementing the moment and curvature of the beam element and considering equilibrium of the cross section after each step to obtain the corresponding increment in the stress-strain relationship.

The above procedure was followed for each of the specimens within a fiber group; the resulting individual stress-strain relationships were averaged to yield a generalized tensile stress-strain relationship for a given fiber percentage.

4.5.4 RESULTS

The resulting generalized tensile stress-strain relationships for the three fiber contents are shown in Fig. 4.17. In this figure, it will be noted that the strain scale only extends to 0.0007. This is due to limits that were placed on the deflection scale of the load-deflection curve in order to get the resolution necessary for the accurate determination of the slope change at the proportional limit. Shown in Fig. 4.15 is the sequence in the derivation of the tensile stress-strain relationship from the load-deflection curve for one of the specimens with a 1.2 percent fiber content. The complete measured load-deflection diagram is shown in Fig. 4.15a, but only portions of the resulting moment-curvature diagram and stress-strain relation are shown in Fig. 4.15b and c, respectively. It was also noted that in every large flexural specimen, the maximum load was reached before the initiation of visible cracking, as indicated in Fig. 4.15a.

Shown in Table 4.4 is a comparison of the modulus of rupture computed for the 6-in. by 6-in. (150-mm by 150-mm) cross section beams and the 3-in. by 3-in. (75-mm by 75-mm) cross section beams as calculated by assuming elastic behavior up to the ultimate load carrying capacity. Both specimen types were cast in the same manner, yet the results indicate that the modulus of rupture is significantly lower for the larger cross section.

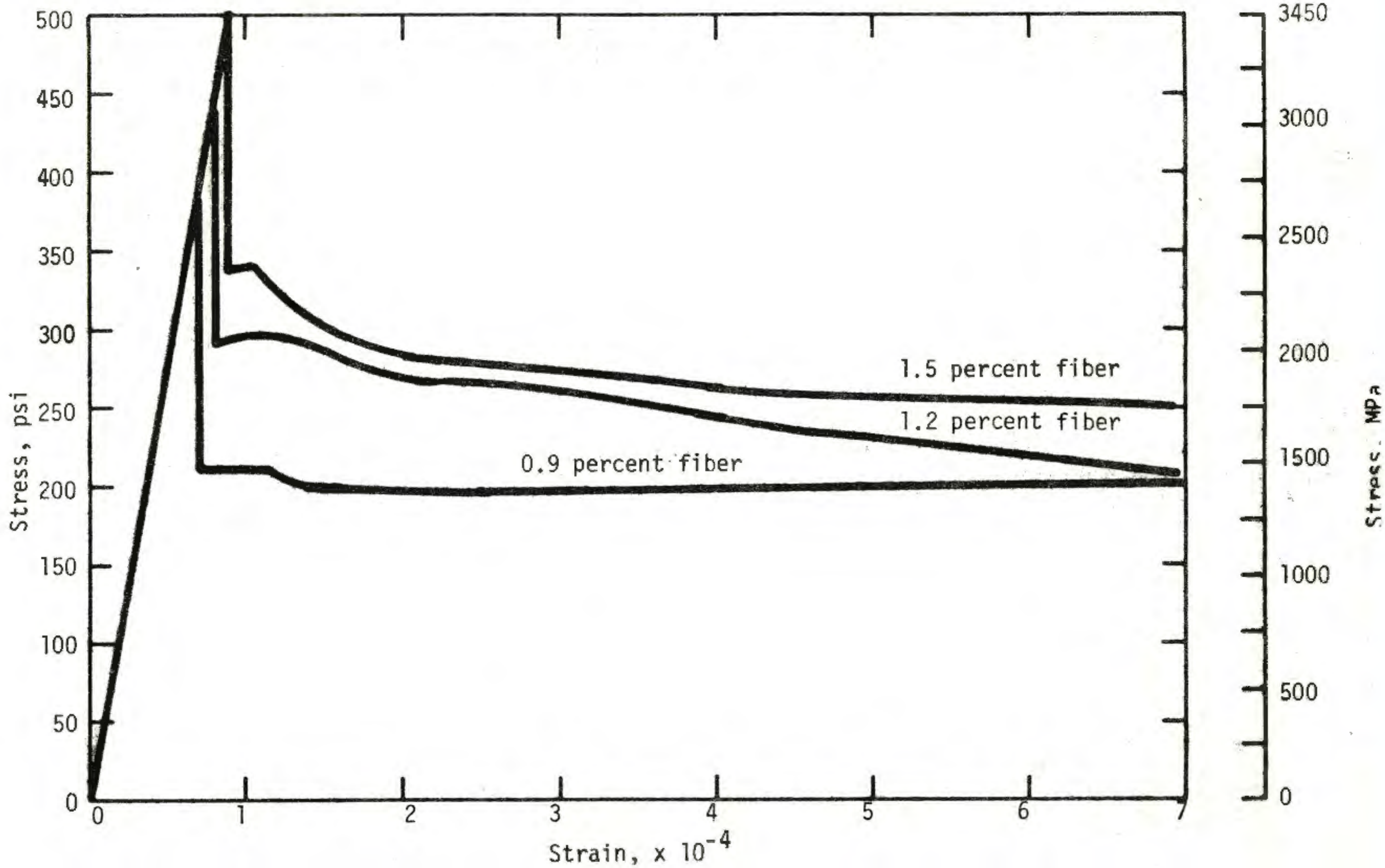


FIG. 4.17 GENERALIZED TENSILE STRESS-STRAIN RELATIONSHIPS FOR A BEAM ELEMENT--FIBER REINFORCED DURACAL CEMENT CONCRETE

TABLE 4.4

EFFECT OF SPECIMEN SIZE ON APPARENT MODULUS OF RUPTURE

Fiber content, percent	f_r, psi (MPa) 3 in. by 3 in. (75 mm by 75 mm)	6 in. by 6 in. (150 mm by 150 mm)	Difference in apparent strength, percent	f'_c, psi (MPa)
0.9	700 (4.82)	545 (3.76)	28.3	4410 (30.4)
1.2	765 (5.27)	580 (4.01)	31.6	3950 (27.2)
1.5	840 (5.79)	680 (4.70)	23.0	4180 (28.8)

4.5.5 CONCLUSIONS

The generalized tensile stress-strain relationships presented here may be subject to some error arising from the assumption that the width of the crack-element was equal to the depth of the beam. This seemed a reasonable assumption, but one which should be investigated further. Also, the effects of changing the size and shape of the cross section on the apparent strength and behavior should be studied further.

Formation of visible cracks did not take place until after the specimens reached ultimate capacity and the load carrying capacity was decreasing. This result indicates that care must be exercised in using this material at a critical section in which the moment is large and the axial force small since there would be no visible indication of imminent failure of the section.

CHAPTER 5

EXPOSURE TO SPECIAL ENVIRONMENTS

5.1 PERMEABILITY, LEACHING AND VOLUME STABILITY

Permeability, leaching and volume stability are significant properties in the determination of material durability. In this section information is presented regarding these properties of the experimental concretes considered throughout this report.

The materials used in the laboratory studies have been described in Section 2.2. Mix designs were selected to be of practical interest. For each cement a mix was proportioned with 1.5 percent fiber to give a slump of approximately 5 in. (125 mm). A second mix was based on the first with fibrous reinforcement replaced with an equal weight of pea gravel. The third mix was the same as the first with the fibrous reinforcement simply omitted. Mix designs and fresh mix properties are shown in Table 5.1. Compressive and flexural strength specimens were cast from each mix for purposes of control and comparison. Results of strength tests for different curing environments are presented in Table 5.2.

5.1.1 PERMEABILITY

Test results show a wide range of permeability coefficients but for each cement the results are consistent within a range. Variability of the permeability test increased with the more permeable concretes. The results of the permeability tests are presented in Table 5.3. Each concrete will be discussed in the following paragraphs.

TABLE 5.1

FRESH MIX PROPERTIES FOR PERMEABILITY STUDY

Mix no.	Cement type	Water-cement ratio	Mix materials, lb/cu yd (kg/m^3): Water	Cement	Sand[1]	Pea gravel[1]	Steel fiber	Fibers, percent	Citric acid, g	Slump,[2] in. (mm)	Entrained air, percent	Unit weight, wet lb/cu ft (kg/m^3)
7	Duracal 4	0.30	240 (142)	800 (475)	1580 (938)	1075 (638)	200 (119)	1.5	--	4-1/2 (115)	3.6	152.5 (2445)
8	Duracal 4	0.26	208 (123)	800 (475)	1580 (938)	1275 (756)	--	--	--	8 (205)	5.2	149.5 (2395)
9	Duracal 4	0.28	224 (133)	800 (475)	1580 (938)	1075 (638)	--	--	--	9-1/2 (240)	4.8	147.0 (2355)
10	Reg-set 4	0.47	376 (223)	800 (475)	1580 (938)	1075 (638)	200 (119)	1.5	1310	5 (130)	3.5	148.5 (2380)
11	Reg-set 4	0.40	320 (190)	800 (475)	1580 (938)	1275 (756)	--	--	1300	5-1/4 (135)	4.1	145.0 (2320)
12	Reg-set 4	0.48	384 (228)	800 (475)	1580 (938)	1075 (638)	--	--	1310	9 (230)	2.0	144.0 (2305)
13	Type 1	0.45	360 (214)	800 (475)	1580 (938)	1075 (638)	200 (199)	1.5	--	4-1/2 (115)	2.8	151.5 (2425)
14	Type 1	0.42	336 (137)	800 (475)	1580 (938)	1275 (756)	--	--	--	7 (180)	2.7	148.0 (2370)
15	Type 1	0.44	352 (142)	800 (475)	1580 (938)	1075 (638)	--	--	--	8-1/2 (215)	2.4	146.5 (2345)

[1] Saturated surface dry
[2] Taken 5 min after water added to dry mix

TABLE 5.2

STRENGTHS OF CONCRETE FOR PERMEABILITY TESTS

Mix no.	Cure[1]	Compression, psi (MPa) — Time after mixing, days: 3		7		14		Flexure, psi (MPa) — Time after mixing, days: 3		7		14	
7	A	4300	(29.65)	4550	(33.44)	5250	(36.20)	895	(6.17)	965	(6.66)	1005	(6.91)
	F	3850	(26.54)	4525	(31.20)	4275	(29.48)	905	(6.23)	850	(5.86)	1545	(10.63)
	W	--		4125	(28.44)	4150	(28.61)	--	--	--	--	--	--
8	A	4250	(29.30)	4825	(33.27)	4525	(31.20)	550	(3.80)	645	(4.46)	690	(4.75)
	F	3875	(26.72)	4400	(30.34)	4250	(29.30)	560	(3.87)	730	(5.03)	670	(4.62)
	W	--		4050	(27.92)	3750	(25.86)	--		--		--	
9	A	3200	(22.06)	4625	(31.89)	5250	(36.20)	520	(3.59)	545	(3.76)	680	(4.67)
	F	3525	(24.30)	4000	(27.58)	4300	(29.65)	655	(4.53)	605	(4.17)	610	(4.22)
	W	--		3375	(23.27)	3750	(25.86)	--		--		--	
10	A	3375	(23.27)	3850	(26.54)	4050	(27.92)	650	(4.49)	745	(5.13)	1055	(7.27)
	F	3975	(27.41)	4125	(28.44)	4750	(32.75)	885	(6.11)	775	(5.34)	995	(6.86)
	W	--		6000	(41.37)	6600	(45.51)	--		--		--	
11	A	4025	(27.75)	4525	(31.20)	4750	(32.75)	655	(4.52)	620	(4.27)	702	(4.84)
	F	3750	(25.86)	4550	(31.37)	5050	(34.82)	715	(4.93)	920	(6.34)	850	(5.87)
	W	--		6500	(44.82)	6775	(46.71)	--		--		--	

(continued)

[1] Curing conditions:

A = 72 F (22 C) 50 RH
F = 72 F (22 C) 100 RH
W = 72 F (22 C) immersed in water

TABLE 5.2 (CONTINUED)

Mix no.	Cure[1]	Compression, psi (MPa) — Time after mixing, days: 3	7	14	Flexure, psi (MPa) — Time after mixing, days: 3	7	14
12	A	3200 (22.06)	3600 (24.82)	4125 (28.44)	425 (2.93)	425 (2.93)	485 (3.34)
	F	3750 (25.66)	4150 (28.61)	4950 (34.13)	510 (3.52)	605 (4.16)	515 (3.54)
	W	--	5050 (34.82)	6000 (41.37)	--	--	--
13	A	3325 (22.93)	3850 (26.54)	4750 (32.75)	620 (4.28)	890 (6.13)	725 (5.01)
	F	3400 (23.44)	4600 (31.72)	5175 (35.68)	640 (4.42)	730 (5.03)	1010 (6.97)
	W	--	4450 (30.68)	6925 (47.75)	--	--	--
14	A	3025 (20.86)	4125 (28.44)	4125 (28.44)	515 (3.54)	565 (3.90)	590 (4.06)
	F	3400 (23.44)	4550 (31.37)	5050 (34.82)	545 (3.77)	570 (3.92)	640 (4.42)
	W	--	6225 (42.92)	6700 (46.19)	--	--	--
15	A	3025 (20.86)	4000 (27.58)	4250 (29.30)	410 (2.82)	502 (3.46)	--
	F	3200 (22.06)	4300 (29.65)	4925 (33.96)	515 (3.55)	695 (4.79)	--
	W	--	5750 (39.64)	5525 (38.09)	--	--	--

[1] Curing conditions:

A = 72 F (22 C) 50 RH
F = 72 F (22 C) 100 RH
W = 72 F (22 C) immersed in water

TABLE 5.3

PERMEABILITY COEFFICIENTS

Mix no.	Cure	Coefficient of permeability, nm/sec		
		Time after casting, days		
		3	7	14
7	Air	4.00	7.60	4.30
	Fog	1.00	1.35	3.70
8	Air	0.95	1.10	3.60
	Fog	1.60	1.65	1.70
9	Air	3.10	2.30	2.80
	Fog	6.30	0.85	1.70
10	Air	0.43	0.27	0.14
	Fog	0.25	0.15	0.15
11	Air	0.13	0.16	0.20
	Fog	0.15	0.35	0.11
12	Air	0.31	0.29	0.14
	Fog	0.25	0.14	0.16
13	Air	1.50	1.70	5.20
	Fog	3.10	2.70	3.30
14	Air	1.10	0.72	0.88
	Fog	0.63	0.59	1.00
15	Air	0.77	0.95	0.90
	Fog	0.94	1.00	0.75

Duracal cement concretes were the most permeable. Early tests with Duracal 3 indicated that permeability decreased with curing time and that water-cured specimens were less permeable than those cured in air or fog. In later tests of Duracal 4, permeability remained constant or increased with curing time. Fog-cured specimens were less permeable than air-cured. No difference was detected in permeability of specimens containing fibrous reinforcement, but any such difference could be obscured by the variability of the experimental results. For Duracal 4 most permeabilities were in the 1.0 to 3.0 nm/sec range.

Concretes made with reg-set 4 cement were least permeable. Results for all specimens were more uniform for each curing period and environment. As with Duracal cement concretes, there was no detectable difference in permeability between fibrous and plain concrete specimens. Calculated coefficients of permeability, K, ranged from 0.1 to 0.3 nm/sec.

Concretes made with type 1 cement exhibited permeabilities bounded by the two experimental concretes. Permeability decreased with curing time and was less for fog-cured specimens than for air-cured. In contrast to the experimental concretes, a definite increase in permeability was noted for specimens containing fibrous reinforcement. For specimens with fibrous reinforcement, K ranged from 1.0 to 3.0 nm/sec while for plain concrete specimens K ranged from 0.6 to 1.0 nm/sec.

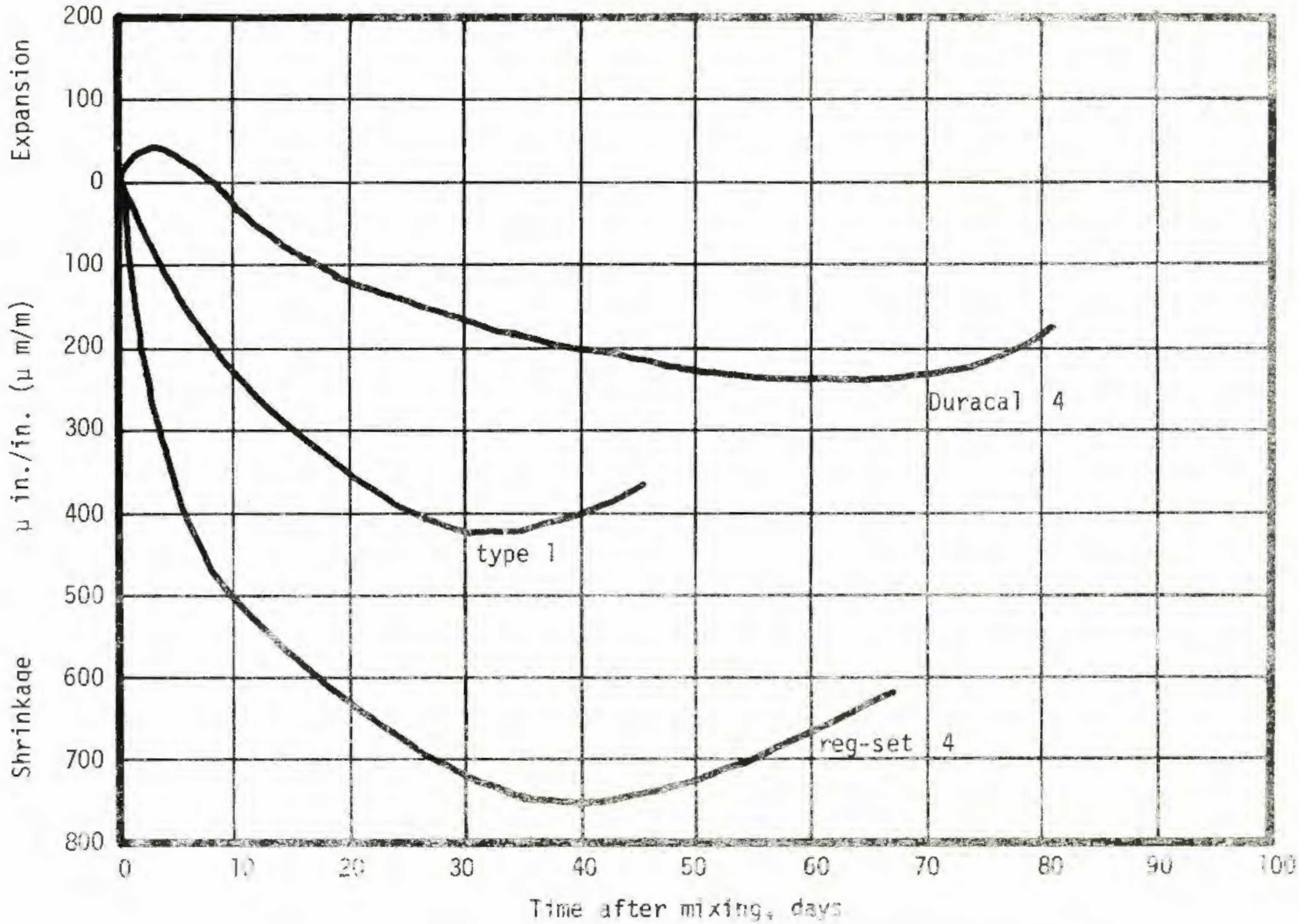
Regulated-set cement specimens possess permeability coefficients an order of magnitude less than Duracal cement specimens, and less than type 1 specimens by a factor of from two to ten. Comparison of the results for all of the tests indicate that permeability decreases with water-cement ratio, decreases with strength increase, and that permeability can be greatly affected by cement type. On the basis of these tests, there can be no generalization about the presence of fibrous reinforcement on concrete permeability.

5.1.2 LEACHING

As evidenced by the strength data of Table 5.2, leaching was a problem only with Duracal cement concretes. Duracal cement concrete mixes 7, 8, and 9 exhibited lower compressive strengths for water-cured specimens than for air- or fog-cured specimens in both 7- and 14-day tests. Regulated-set cement concrete mixes 10, 11, and 12, and type 1 cement concrete mixes 13, 14, and 15 exhibited compressive strength increases of about 30 percent for water-cured specimens.

5.1.3 VOLUME STABILITY

Volume stability measurements continue but trends are available from observations to date. Average volume stabilities for air-cured and fog-cured specimens are illustrated in Figs. 5.1 and 5.2. Duracal cement concrete is the most stable and least susceptible to dimensional change with changes in temperature of humidity, or as the result of hydration. Air-cured Duracal cement concrete specimens shrink the least of the air-cured specimens. Fog-cured Duracal cement concrete specimens expand and then shrink but the total dimensional change is small. Type 1 cement concrete shrinks for both air- and fog-cured specimens; dimensional changes are greater than for Duracal. Regulated-set cement shrinks very fast during the first few days of hydration and then continues to shrink at a much reduced rate.



[illegible] AVERAGE VOLUME STABILITY FOR AIR-CURED SPECIMENS

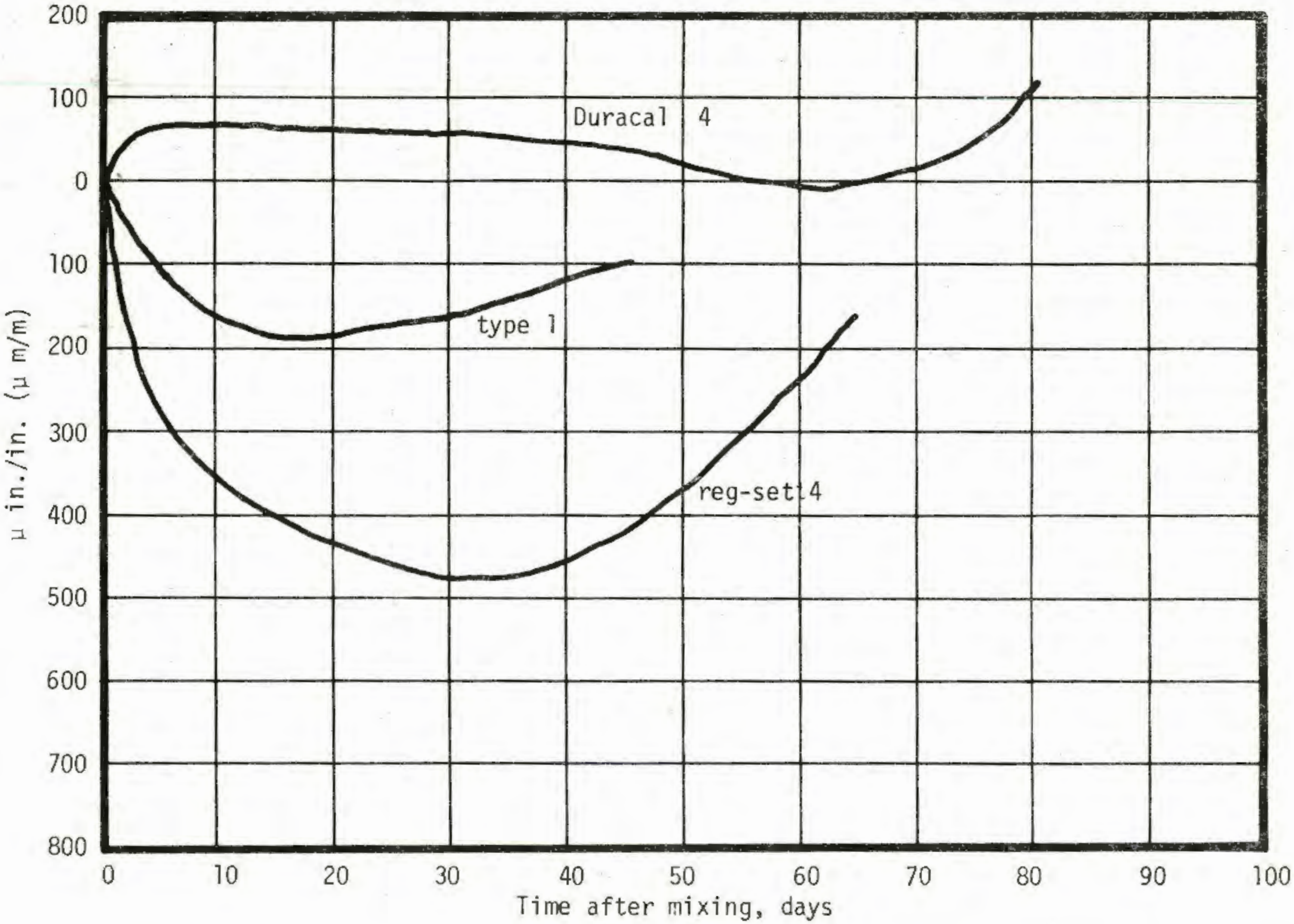


FIG. 5.2 AVERAGE VOLUME STABILITY FOR FOG-CURED SPECIMENS

The temperature and humidity conditions of both air-cured and fog-cured specimens were not uniform, although conditions of fog curing were intended to be uniform. It was observed that during significant changes, especially a combination of increased temperature and humidity, all specimens of the three cement types changed nearly equally, and when the conditions returned to normal the dimensions were restored to those before the change. For small changes in humidity or temperature, regulated-set cement concrete specimens exhibited the greatest change, while Duracal cement concrete specimens were not affected. All of the concretes shrink at a decreasing rate except fog-cured Duracal cement concrete, which expanded before the onset of shrinkage. Changes in curing conditions result in dimensional changes. Such changes are most evident in regulated-set cement concretes and least evident in Duracal cement concretes. Shrinkage was always greatest for air-cured specimens.

5.2 DISTURBANCE OF YOUNG CONCRETE

The placement of an extruded tunnel lining system and its subsequent vibration could very well be accompanied by unwanted disturbance of concrete which had been previously placed.

It is necessary to determine the effects of periodical revibration of placed steel fiber reinforced Duracal cement concrete in terms of fiber distribution, and flexural and compressive strength. The effects of revibration on strength of young steel fiber reinforced regulated-set cement concrete are reported by Paul, et al. (1974).

Two batches, designated batches 2 and 3 were cast from the DC1.2

mix design appearing in Table 2.3. From each batch, ten 4-in. (100-mm) cylinders and six 3-in. (75-mm) beam specimens were made. For each batch, the cylinder and beam specimens were divided between control and test groups, half going to each.

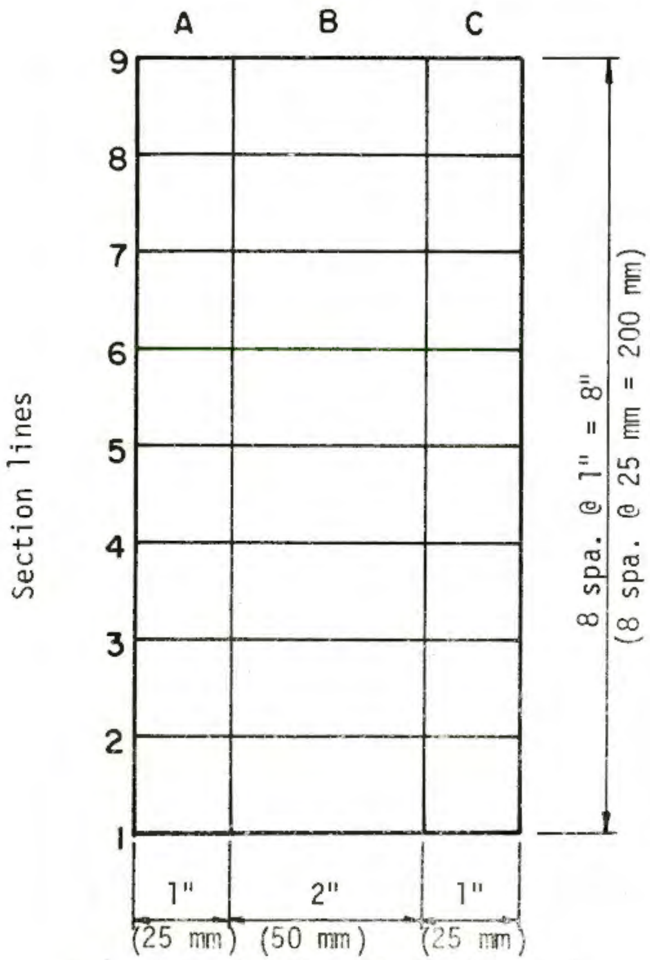
After placement of the fibrous concrete in molds, control and test specimens were given an initial 10 sec external compactive vibration. The test group was subsequently vibrated externally at 15 min intervals for 15 sec up to and including 60 min from completion of the initial compactive vibration effort. The test and control groups were then tested for strength at 2 hr ± 15 min from completion of casting.

Upon completion of the strength tests, all specimens were cut and fiber counts made. Cylinders were cut longitudinally about their center line and the beams were cut laterally 3 in. (75 mm) from an end. Fiber counts were made using transparent grids, constructed as shown in Fig. 5.3.

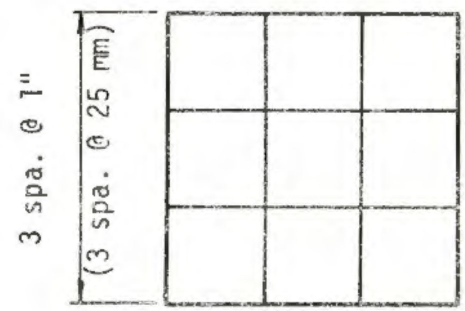
Compressive and flexural strengths are reported in Table 5.4. The results of batch 2 show a 13 percent strength decline for revibrated specimens, while results for batch 3 show only a 3 percent reduction. Flexural strengths also exhibited a strength decline for revibrated specimens; 2 percent for batch 2 to 5 percent for batch 3.

A strength decline is evident for both compressive and flexural strength of steel fiber reinforced, Duracal cement concrete, although the magnitude of this decline is not a significant portion of the overall strength.

For both batches, fiber distribution did not vary to a significant degree with revibration. Figures 5.4 and 5.5 indicate the average fiber



(a) Grid for longitudinal cut in cylinders.



(b) Grid for sawed beams.

FIG. 5.3 GRID CONSTRUCTIONS USED FOR CYLINDER AND BEAM FIBER DISTRIBUTION COUNTS

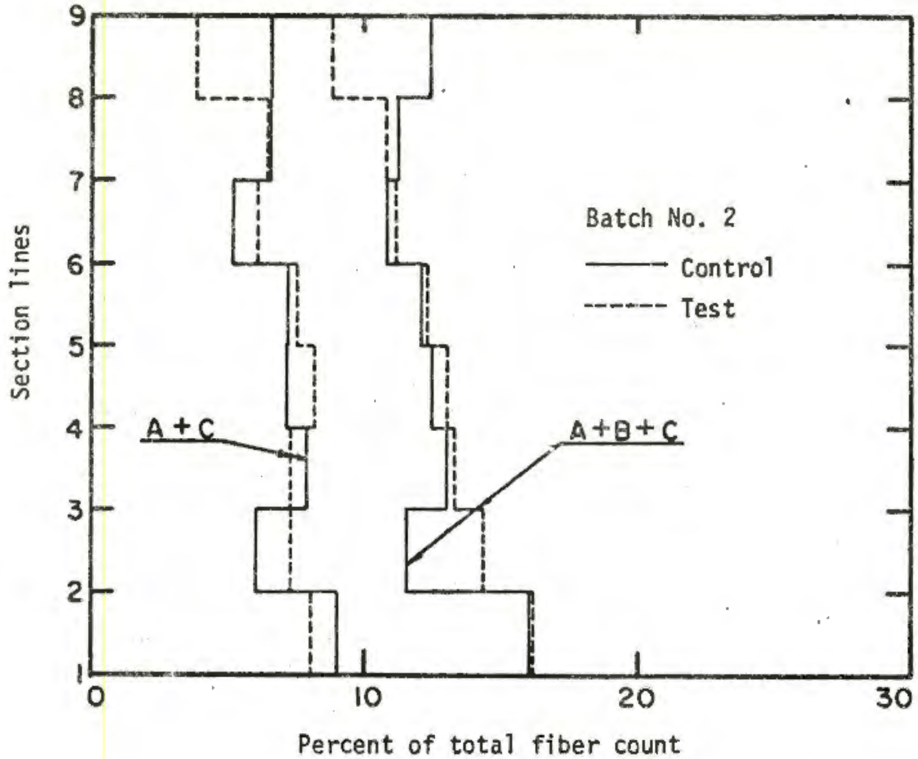


FIG. 5.4 FIBER DISTRIBUTION, BATCH NO. 2

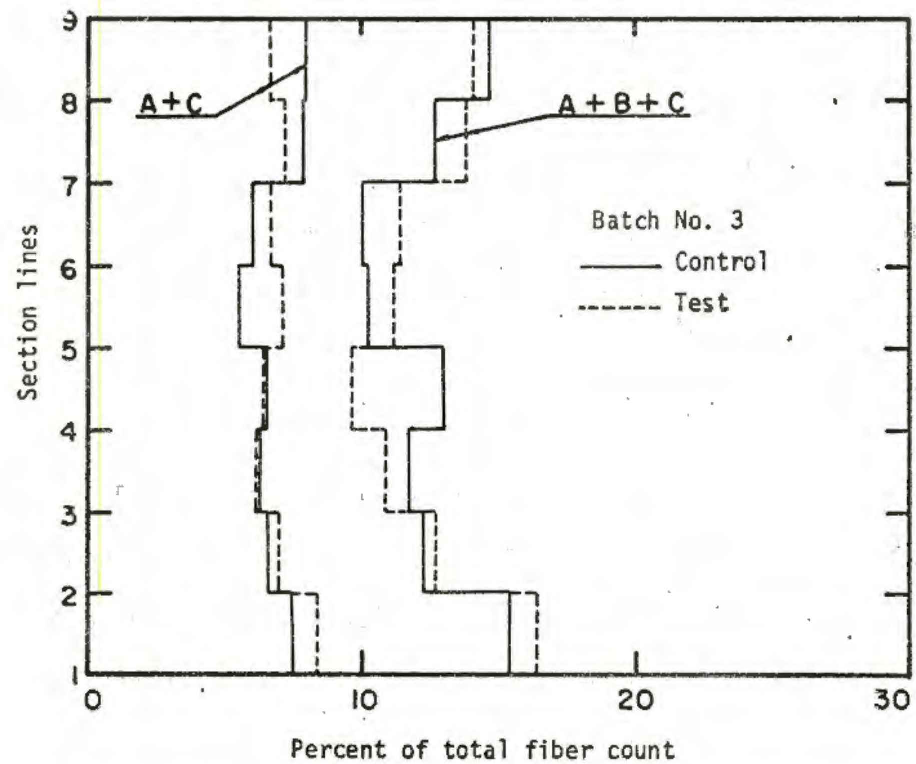


FIG. 5.5 FIBER DISTRIBUTION, BATCH NO. 3

TABLE 5.4

EFFECT OF REVIBRATION ON EARLY STRENGTH STEEL FIBER REINFORCED DURACAL CEMENT CONCRETE

Batch	Compressive strength, psi (MPa)[1]		Flexural strength, psi (MPa)[2]	
	Control	Revibrated	Control	Revibrated
2	910 (6.25)	780 (5.42)	530 (3.65)	525 (3.62)
3	1080 (7.45)	1050 (7.24)	565 (3.40)	590 (4.08)

[1] Average of 3 cylinders for batch 2, 5 cylinders for batch 3
[2] Average of 3 beams

distribution over cylinder cross section for the experimental batches. In the fiber distributions, the A + C line indicates the fiber count in the boundary regions of the specimen; the A + B + C line indicates the total amount of fibers present between section lines. Test and control groups for both batches did not deviate from each other by more than approximately 4 percent at any particular section line considered.

5.3 EFFECT OF HIGH TEMPERATURE ENVIRONMENT

The performance of tunnel liner materials in high temperature environments is a further significant aspect of material durability. Series of strength tests were made on concrete which had been subjected to 350 F and 600 F (175 C and 315 C), respectively. Standard 6-in. (150-mm) cylinders and 3-in. (75-mm) beams were cast for each temperature using the DC1.2 mix of Table 2.3 with Duracal 4 and Duracal 5.

All specimens were air-cured at 73 F (23 C) in the laboratory environment for two days prior to high temperature exposure. This precaution was taken as a result of experience with specimens of steel fiber reinforced regulated-set cement concrete which exploded from vaporization of entrapped water (Paul, et al., 1974).

Following the initial curing period test specimens were subjected to the high temperature environment for 1 or 7 days; control specimens remained in the laboratory environment until time of test.

Fresh mix properties for the different casts are reported in Table 5.5. Results of strength tests are reported in Table 5.6.

One- and seven-day exposure strengths are not significantly influenced by the 350 F (175 C) environment. For the 600 F (315 C) environment the one-day exposure strength exhibited a 45 percent strength decline over control strengths while the strength decline at 7 days was 35 percent. Further investigation is necessary to determine behavior for intermediate temperature environments and curing conditions.

TABLE 5.5

FRESH MIX PROPERTIES FOR HIGH-TEMPERATURE TESTS

Mix	Water temperature, F	(C)	Slump, in. (mm)	Air, percent	Unit weight wet, lb/cu ft	(kg/m^3)
T1	56	(13.3)	7-1/2 (190)	4.8	147.0	(2355)
T2	62	(16.7)	7-1/2 (190)	4.4	149.0	(2387)
T3	70	(21.1)	7-1/2 (190)	4.4	149.0	(2387)
T9	70	(21.1)	7 (180)	6.7	146.0	(2339)

TABLE 5.6

TYPICAL COMPRESSIVE AND FLEXURAL STRENGTHS FOR HIGH TEMPERATURE EXPOSURE

MIX	1-Day exposure strength, psi (MPa)						7-Day exposure strength, psi (MPa)					
	72 F (22 C)		350 F (175 C)		600 F (315 C)		72 F (22 C)		350 F (175 C)		600 F (315 C)	
	comp	flex	comp	flex	comp	flex	comp	flex	comp	flex	comp	flex
T1			3130 (21.59)	815 (5.62)					3300 (22.78)	825 (5.68)		
T2	3140 (21.65)	590 (4.07)					3820 (26.32)	620 (4.27)				
T3									3160 (21.78)	700 (4.83)		
T9					1700 (11.73)	320 (2.20)	2990 (20.60)	605 (4.18)			1930 (13.31)	325 (2.24)

5.4 SULFATE RESISTANCE

5.4.1 INTRODUCTION

The susceptibility of steel fiber reinforced, regulated-set and Duracal cement concretes to sulfate attack is being investigated. Data are being obtained on the linear expansion, weight change, and flexural strength of specimens subjected to sulfate and control environments.

The investigation of durability in the sulfate environment is necessarily a long term test. An economical and relatively fast procedure which provides indicative data on sulfate resistance has been developed by Eustache and Magnan (1972). Appreciable sulfate attack can be detected after 60 days immersion of 20-mm by 20-mm by 100-mm (0.8-in. by 0.8-in. by 4.0-in.) specimens in a 5 percent solution of anhydrous magnesium sulfate ($MgSO_4 \cdot 7H_2O$). This test utilizes a sand-cement mortar having a water-cement ratio of about 0.50. For the purpose of relevance to the development of new materials for tunnel liners, the Eustache-Magnan test was abandoned in favor of larger specimens cast from potential field mix designs.

Thorvaldson, Vigfusson and Larmour (1927) reported sodium sulfate to be slower than other sulfates, such as magnesium sulfate, in terms of reaction with concrete. However, sodium sulfate (white alkalai) deposits are the most common sulfates found in soils. A sodium sulfate environment would thus provide a representative indication of the behavior of fibrous concrete in situ. The use of more reactive sulfates or other concentrations, is however, a valid area for further research.

5.4.2 PROGRAM OF RESEARCH

The test environment for sulfate exposure is immersion in a 10 percent by weight solution of sodium sulfate. The control environments are plain and lime-saturated tap water. Solutions are changed at 14-day intervals to maintain uniform concentration of solute.

Expansion specimens were as described in Section 2.4. Flexural specimens were 3-in. by 3-in. by 10-in. (75-mm by 75-mm by 250-mm) beams tested in third-point loading. The mix designs of Table 2.3 were used; fresh mix properties appear in Table 5.7.

Initial determinations of length and weight were made at 2 hr following casting, then specimens were immersed in the appropriate environment. The current schedule calls for repeated measurements at 1, 2, 3, 7, 14, 21, 28 and 60 days. Flexural strengths are determined at 14, 21, 28 and 60 days as the average of five 3-in. (75-mm) beam specimens.

5.4.3 RESULTS

Flexural strengths obtained to date are reported in Table 5.8. Weight gain with time is reported in Figs. 5.6 to 5.10. Linear expansion with time is reported in Figs. 5.11 to 5.15.

At the writing of this report only the data from specimens cast from regulated-set cement and 0.9 percent steel fiber are available for the 60-day period. Specimens stored in the sulfate environment exhibited expansions approximately five times greater than the companion lime-saturated water specimens. Higginson and Glantz (1953) propose a length increase of 0.20 percent as a failure criterion for expansion. The 60-day expansion

TABLE 5.7

FRESH MIX PROPERTIES FOR SULFATE ENVIRONMENT TEST[1]

Mix[2] designation	Slump, in.	(mm)	Entrained air, percent	Unit weight wet, lb/cu ft	(kg/m^3)	Mix water temperature, F	(C)
DC-L-0.9	5-1/4	(135)	4.1	142.1	(2276)		
DC-L-1.2	5-1/4	(135)	4.1	145.0	(2323)		
DC-L-1.5	7-1/2	(190)	4.0	142.6	(2284)		
RS-L-0.9	6	(150)	3.5	140.7	(2254)	44	6.4
RS-L-1.2							
RS-L-1.5							
DC-S-0.9							
DC-S-1.2	8	(205)	5.5	140.7	(2254)		
DC-S-1.5							
RS-S-0.9	5	(125)	2.7	139.7	(2238)	40	4.2
RS-S-1.2							
RS-S-1.5							
DC-W-0.9	5-1/4	(135)	2.8				
DC-W-1.2	8-1/2	(215)	3.3	141.6	(2268)		
DC-W-1.5							
RS-W-0.9							
RS-W-1.2	3-1/2	(90)	3.3	140.7	(2254)		
RS-W-1.5							

1 Including anticipated mixes

2 Key to mix designation: DC=Duracal 4 cement; RS=reg-set 4 cement; L=Lime saturated water; S=10 percent by weight Na_2SO_4; W=tap water; 0.9=0.9 volume percent fiber; etc.

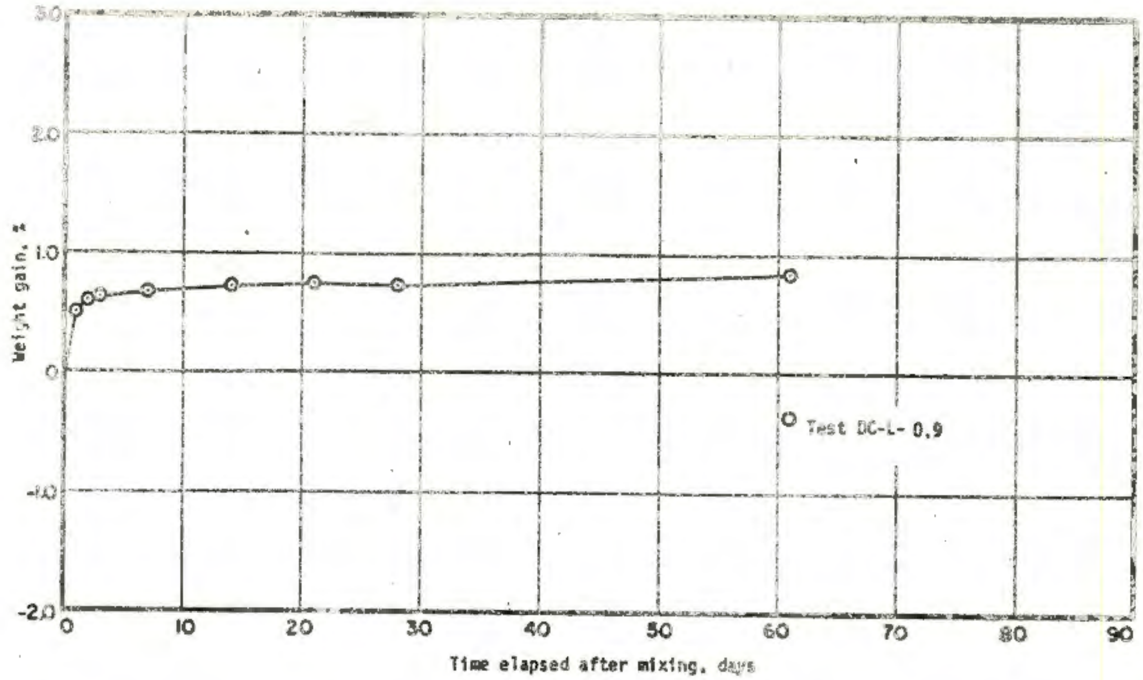


FIG. 5.6 WEIGHT GAIN FOR DURACAL CEMENT CONCRETE WITH 0.9 PERCENT FIBER

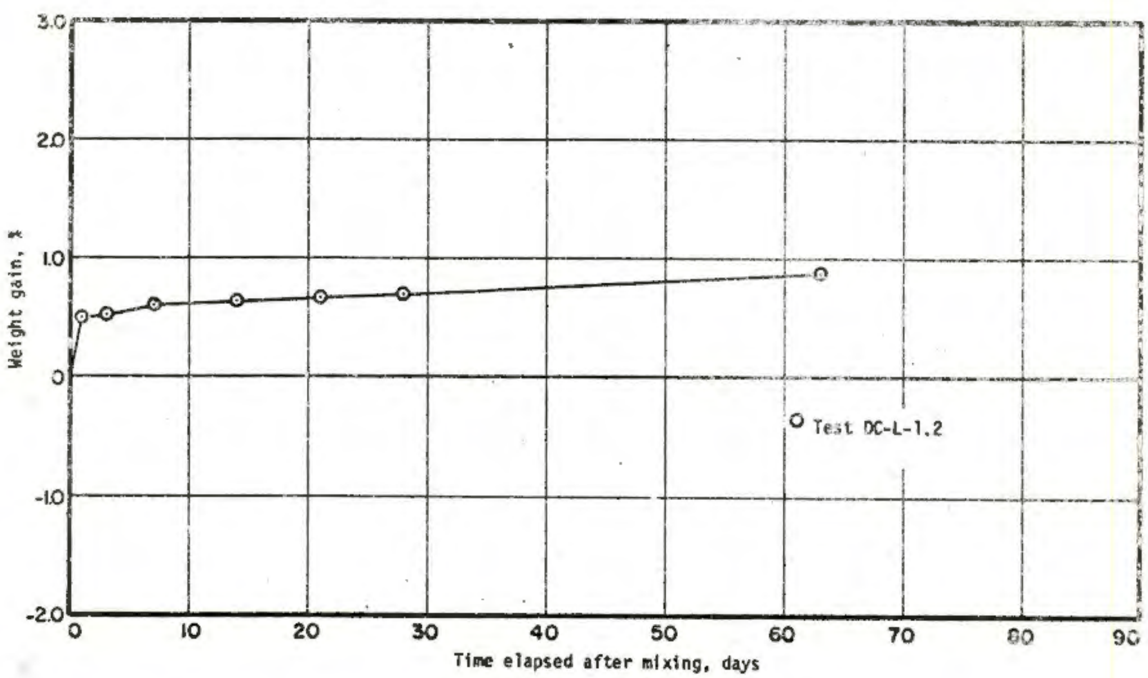


FIG. 5.7 WEIGHT GAIN FOR DURACAL CEMENT CONCRETE WITH 1.2 PERCENT FIBER

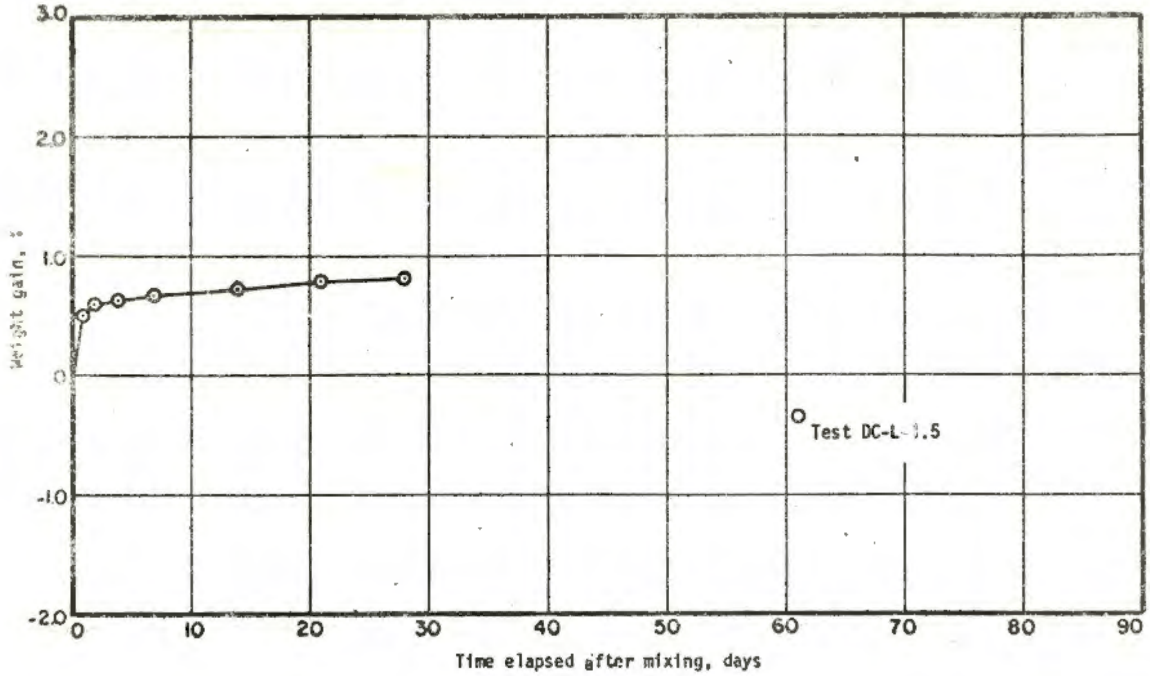


FIG. 5.8 WEIGHT GAIN FOR DURACAL CEMENT CONCRETE WITH 1.5 PERCENT FIBER

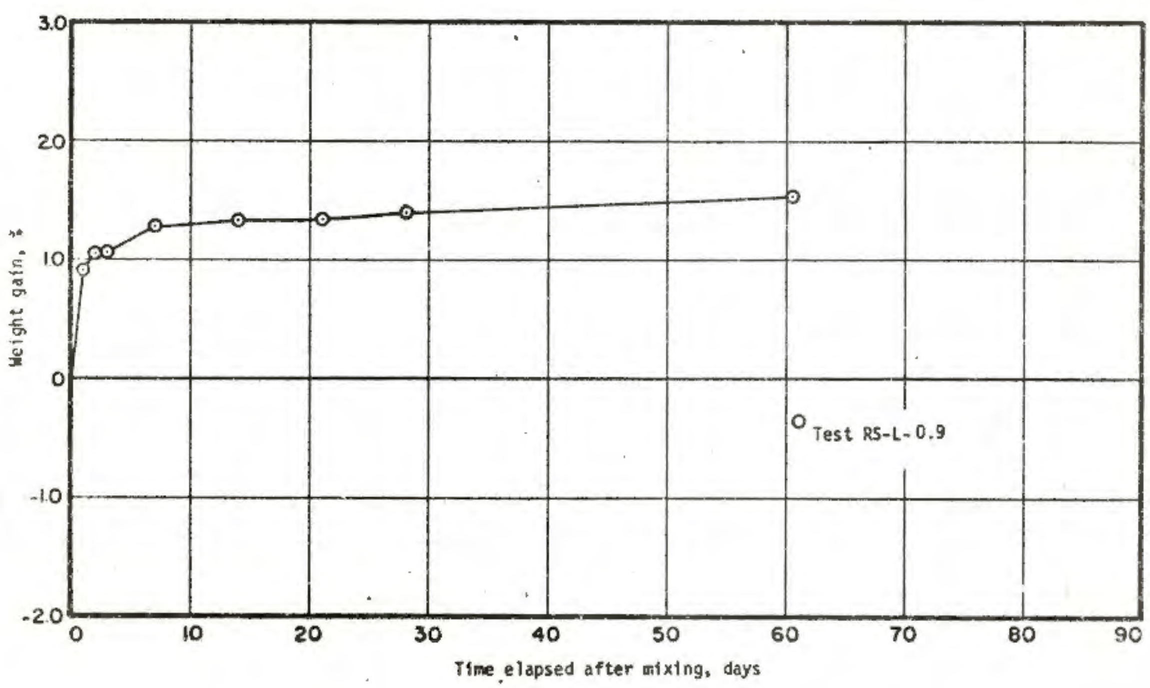


FIG. 5.9 WEIGHT GAIN FOR REGULATED-SET CEMENT CONCRETE WITH 0.9 PERCENT FIBER

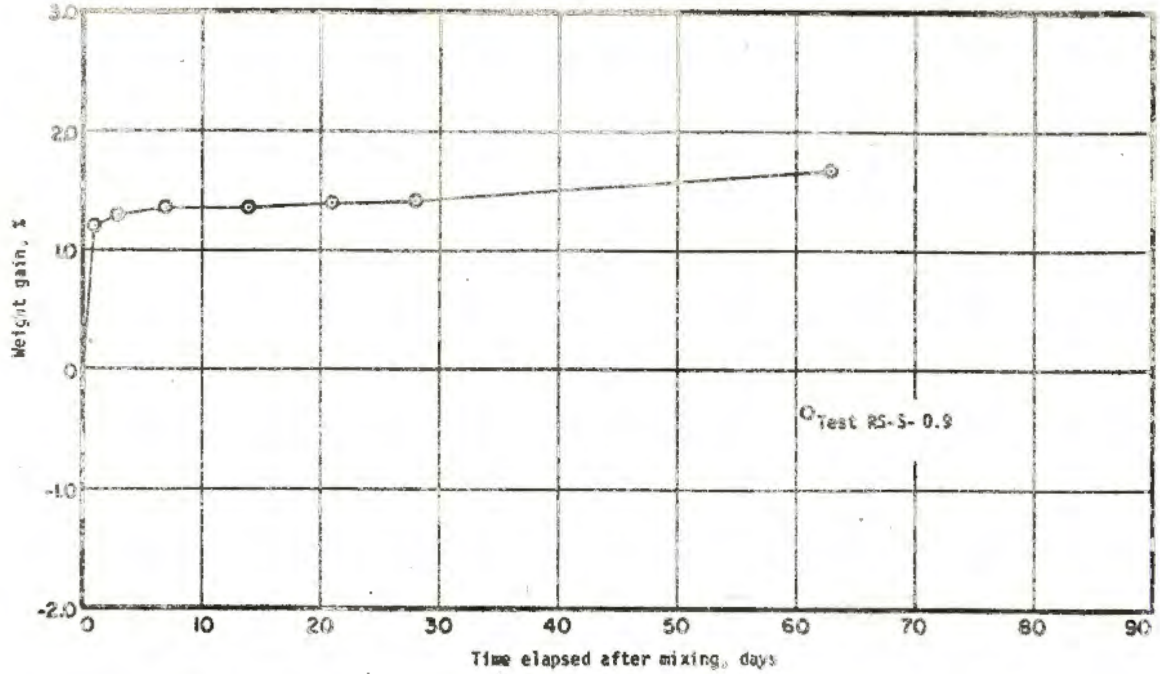


FIG. 5.10 WEIGHT GAIN FOR REGULATED-SET CEMENT CONCRETE WITH 0.9 PERCENT FIBER

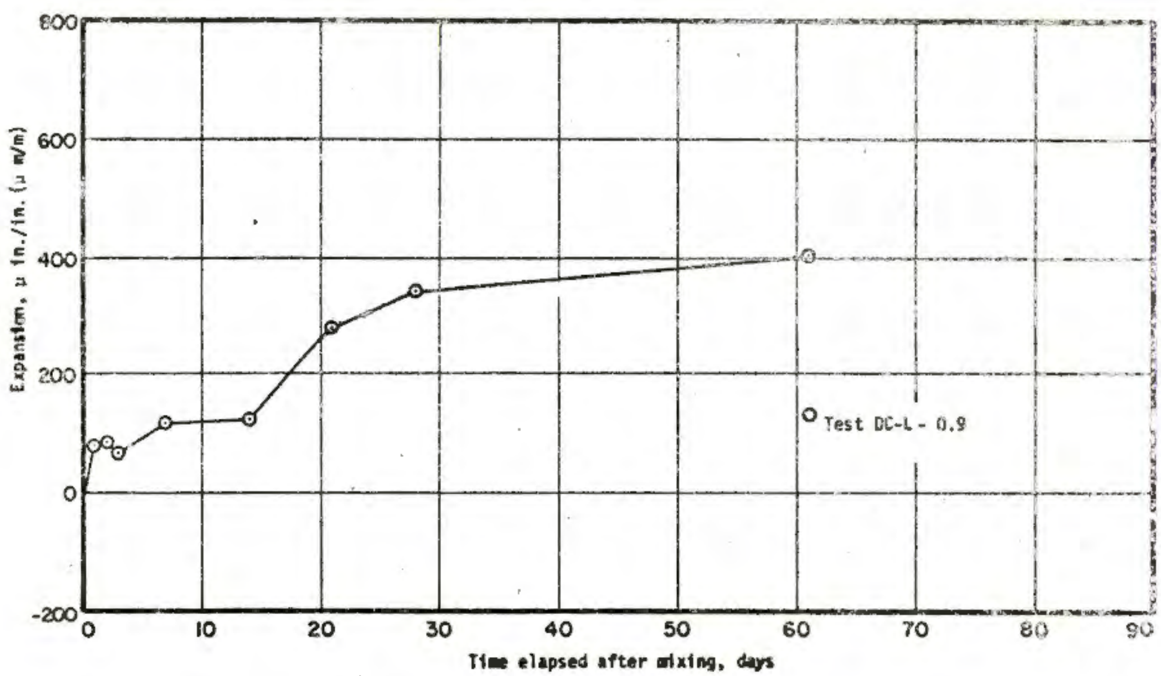


FIG. 5.11 EXPANSION FOR DURACAL CEMENT CONCRETE WITH 0.9 PERCENT FIBER

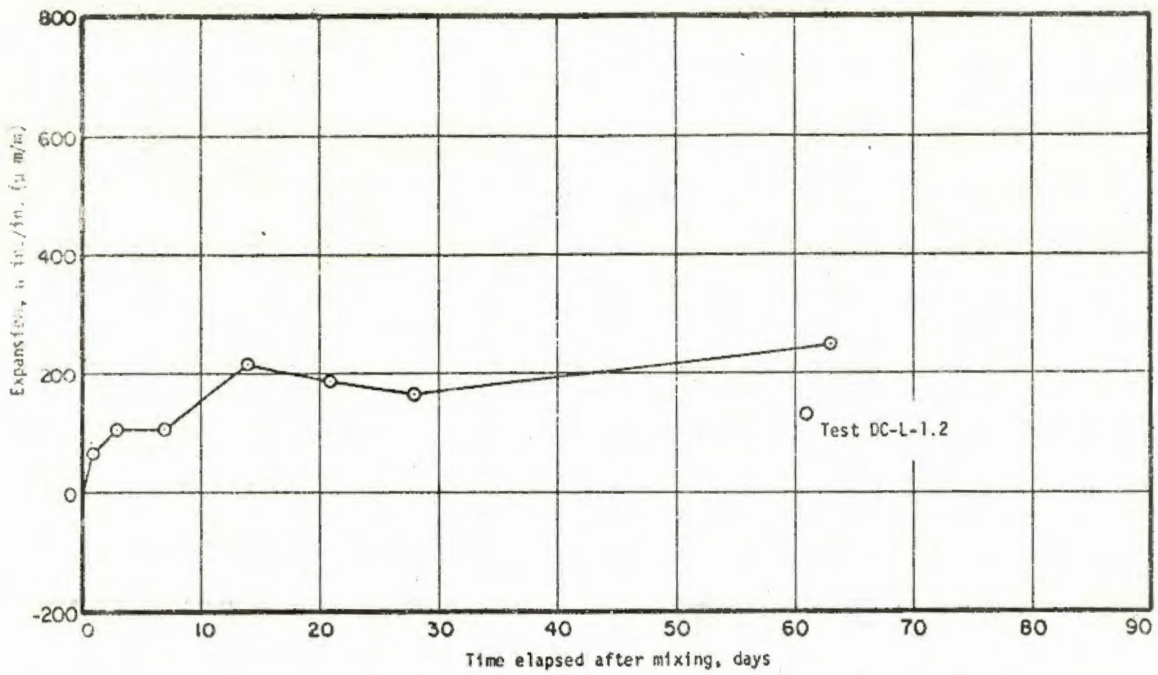


FIG. 5.12 EXPANSION FOR DURACAL CEMENT CONCRETE WITH 1.2 PERCENT FIBER

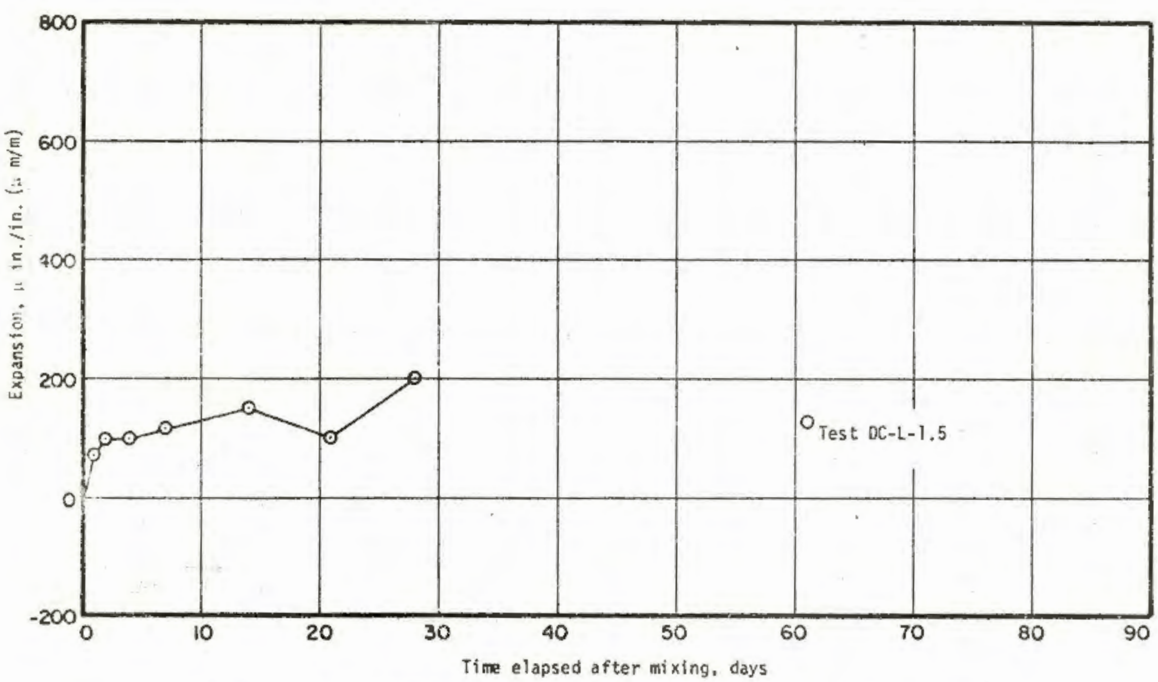


FIG. 5.13 EXPANSION FOR DURACAL CEMENT CONCRETE WITH 1.5 PERCENT FIBER

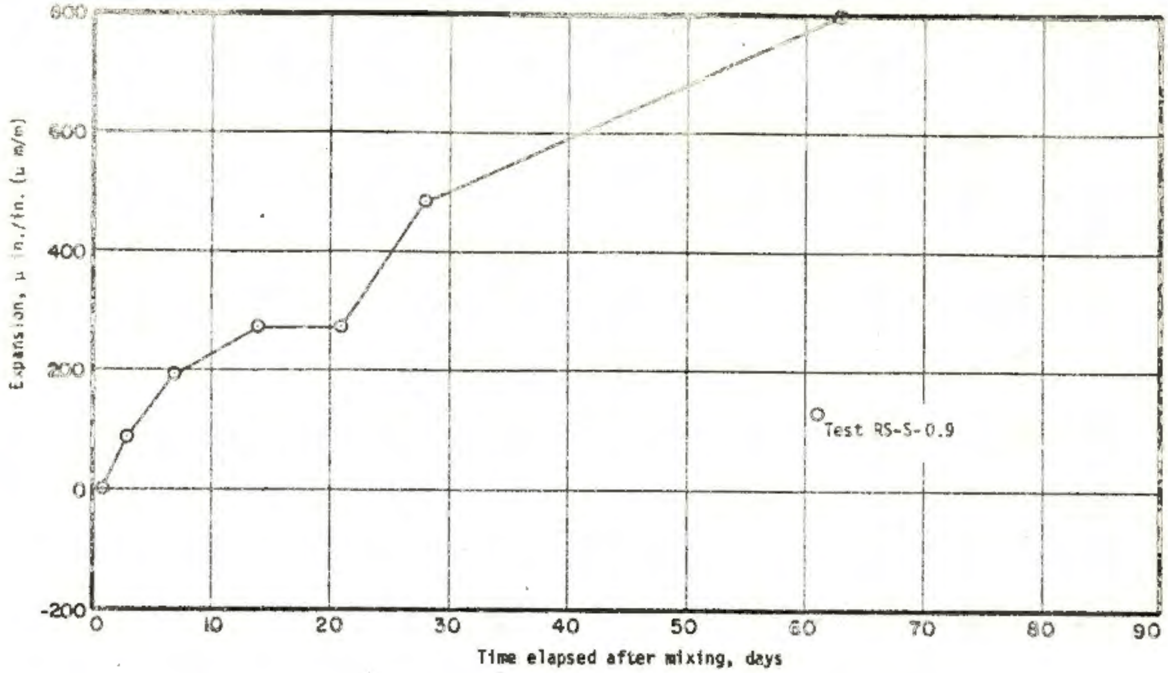


FIG. 5.14 EXPANSION FOR REGULATED-SET CEMENT CONCRETE WITH 0.9 PERCENT FIBER

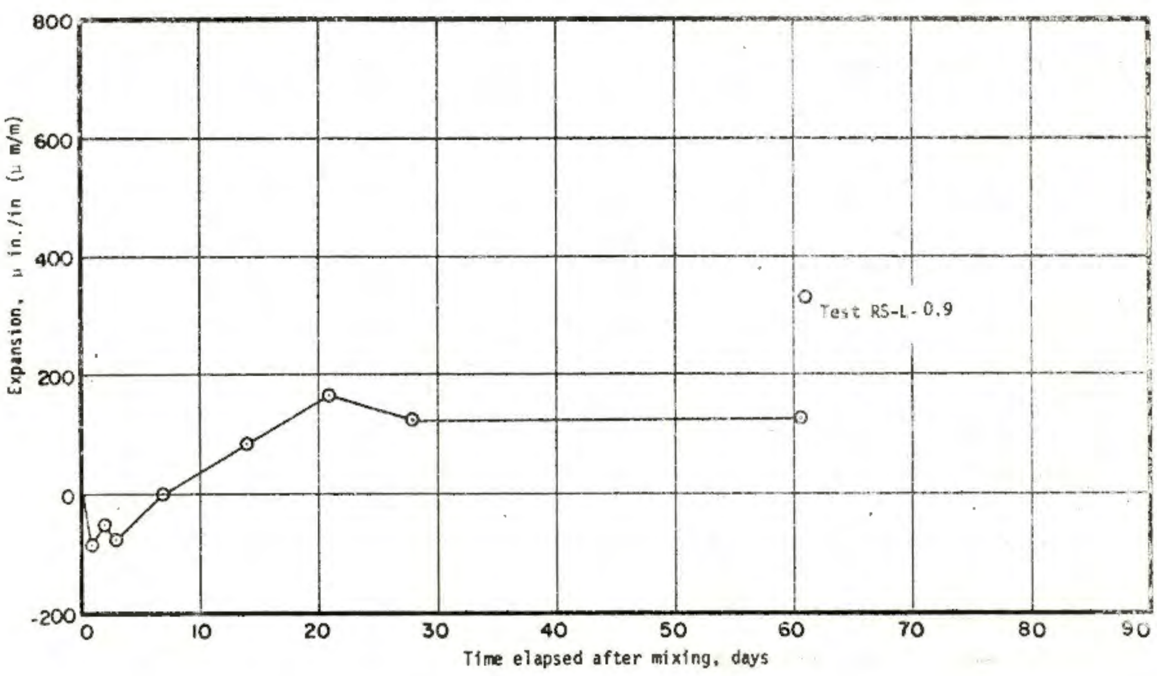


FIG. 5.15 EXPANSION FOR REGULATED-SET CEMENT CONCRETE WITH 0.9 PERCENT FIBER

TABLE 5.8

FLEXURAL STRENGTHS FOR SULFATE ENVIRONMENT TESTS

Mix designation	Flexural strength, psi (MPa), at time of test, days			
	14	21	28	60
DC-L-0.9	430 (2.95)	515 (3.55)	540 (3.72)	560 (3.86)
DC-L-1.2	520 (3.56)	520 (3.56)	600 (4.14)	625 (4.31)
DC-L-1.5	635 (4.36)	595 (4.01)	600 (4.14)	---
RS-L-0.9	960 (6.62)	920 (6.34)	880 (6.07)	945 (6.52)
RS-S-0.9	1060 (7.31)	1110 (7.65)	960 (6.62)	990 (6.83)

for mix RS-S-0.9 was 0.08 percent. No external deterioration such as rounding of corners or cracking is evident in any of the specimens stored in the sulfate environment. Specimen weights are increasing at a near-constant rate.

5.4.4 SUMMARY

A series of sulfate environment tests has been initiated for regulated-set and Duracal cement concretes with 0.9, 1.2 and 1.5 volume percent steel fiber reinforcement. At present half of the series has been cast and the balance will be cast in the near future. The current tests should be continued to obtain data past the point of marked specimen deterioration. It is estimated that this will require 12 to 18 months of further exposure.

CHAPTER 6

PUMPING FIBROUS CONCRETE

6.1 INTRODUCTION

Placement of fibrous concrete by pumping is a complex problem. Special attention should be given to certain parameters regarding aggregate shape, maximum size, and gradation; and cement paste content. Basic problems must be eliminated before a fibrous concrete mix can be considered pumpable.

Two basic factors that control pumpability are aggregate gradation and cement paste content. It has been reasoned that if an aggregate-fiber mix is formulated to contain minimum voids a pumpable mix may result with appropriate addition of cement and water. Much laboratory time has been devoted to relating aggregate void volume with pumpability. Once a specific aggregate gradation has been selected for each fiber content, the mix may be proportioned to include cement and water. Kempster (1969) has suggested that cement content should be at least equal to the void volume. Laboratory tests for pumpability have been initiated with minimum void mixes. The laboratory apparatus shown in Fig. 6.1 allows study of concrete as it remolds itself, such as would be required in passing reducers. Mixes which pass the laboratory apparatus are believed to be pumpable.

6.2 CURRENT KNOWLEDGE

6.2.1 AGGREGATES

Coarse aggregate should be subangular and preferably rounded. Fine

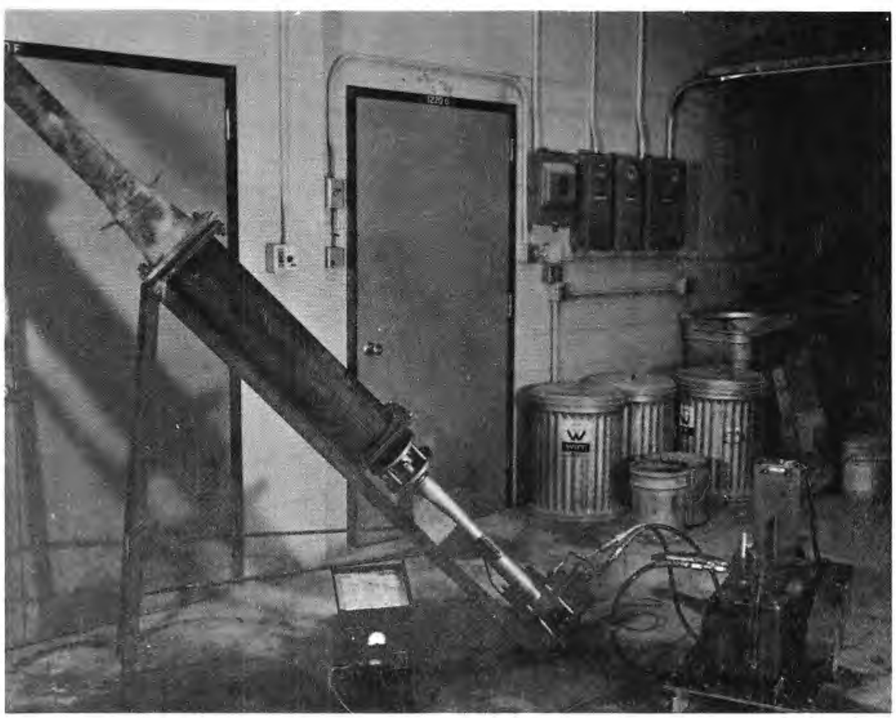


FIG. 6.1 LABORATORY PUMPING APPARATUS

aggregate must include a sufficient amount of fines and have a smooth gradation curve. A suggested gradation for fine aggregate has been cited by ACI Committee 304 (1973). This gradation is indicated in Fig. 6.2. Laboratory investigation has confirmed that a sand consisting of the recommended gradation is fairly dense and produces a low void aggregate mix when combined with coarse aggregate.

Combining a dense sand with a specific coarse aggregate, such as pea gravel, in different proportions results in different unit weights. These weights can be related to the void ratio of the combined aggregate. For various fiber contents different combinations of coarse and fine aggregate will constitute the low void volume mix. It is these minimum void ratio mixes that will produce pumpable mixes.

6.2.2 CEMENT CONTENT

Kempster (1969) has indicated that cement paste content must be equal to, if not in excess of, the aggregate void volume. If the cement content is less than the void volume, the pumping force cannot be properly transmitted, water will bleed out, and a segregation failure will result. If cement content is in great excess of void volume the presence of such a large amount of fines will cause the mix to drag in the pipeline with a resulting friction failure.

6.2.3 ADMIXTURES

Admixtures may diminish problems with failure. One type of additive is a phase thickener which increases water viscosity and produces increased

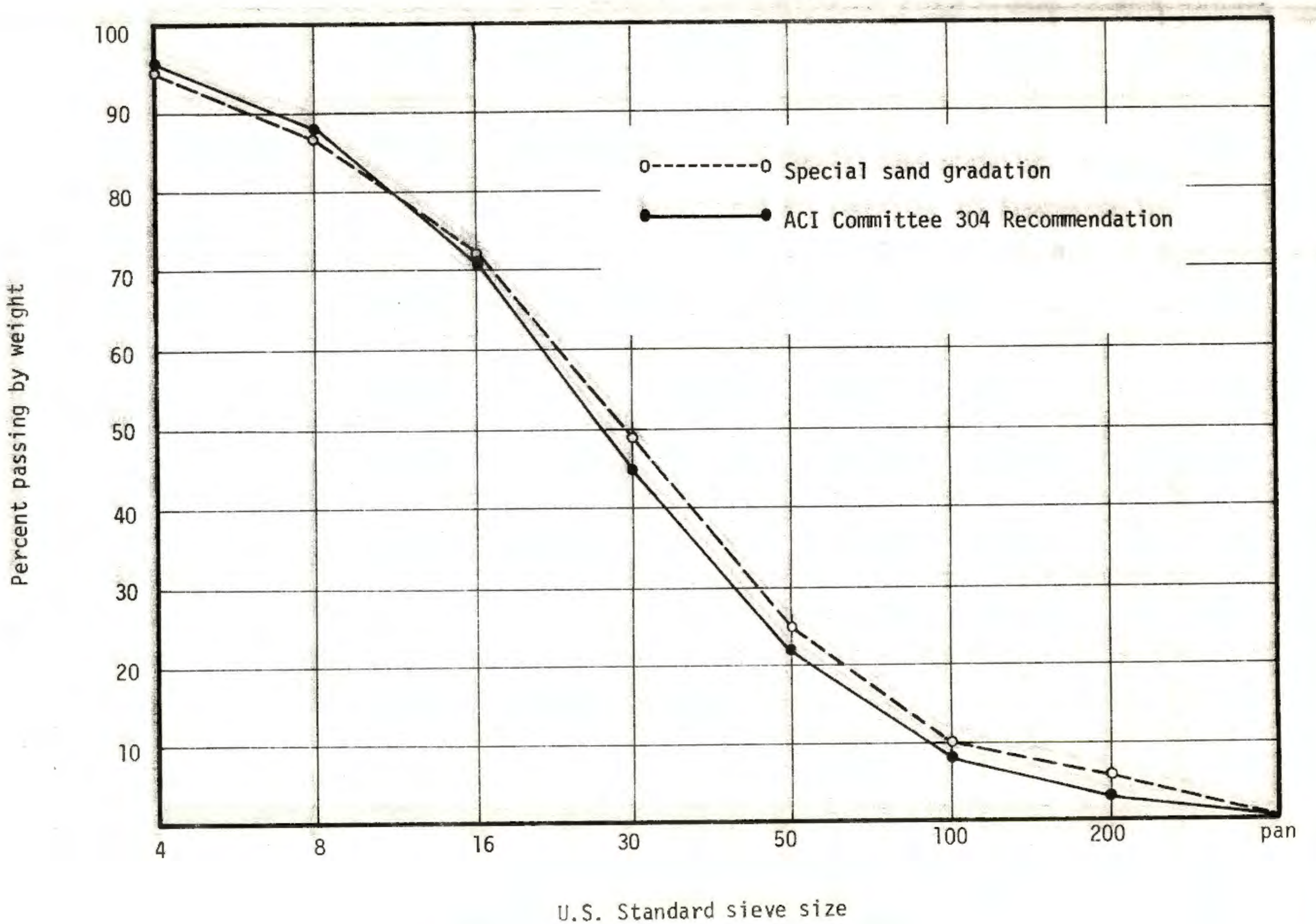


FIG. 6.2 FINE AGGREGRATE GRADATION

internal resistance and reduced frictional resistance of a concrete mass present in a pipeline. With phase thickeners the water phase is able to transmit pumping force more efficiently and thus reduce the tendency for segregation.

An alternate pumping admixture is the wetting agent. These products act as water reducers allowing more fluid concrete with less water. The effect is marginal, however, in that only mixes that contain low voids will be favorably effected (Kempster 1969).

6.2.4 PUMPING EQUIPMENT

Conventional equipment may be used for pumping fibrous mixes if certain deficiencies are eliminated. At present, it seems apparent that pipeline configurations such as reducers and Y's may have to be eliminated or their usage reduced. The less remolding required the more likelihood of pumpability. In general, though, the major problem with pumping fibrous concrete lies not in the machinery but rather in the mix design.

6.3 TESTING PROGRAM

6.3.1 AGGREGATE TESTING

In order for a mix to be pumpable, the sand gradation should closely resemble the ACI Committee 304 recommendation. In laboratory tests, an artificial sand conforming to the ACI-304 gradation was formulated. Amounts retained on the seven major sieves were controlled to closely match the recommended gradation. Lesser amounts on the #8 and #16 sieves are to account for space taken by the steel fiber. Although not an aggregate, the fiber plays a major role in how well the aggregate-fiber array may compact.

The special sand was combined with varying amounts of fiber and coarse aggregate. Unit weight tests were performed in accordance with ASTM C-29. Mixes were rodded at one-third levels and the unit weights determined. No vibration was applied. Tests were conducted with 0, 0.9, 1.2 and 1.5 percent fiber by volume. The results of these tests appear in Table 6.1. A plot of void ratio vs. percent pea gravel for the four fiber contents appears in Fig. 6.3. Void ratios have been computed on the basis of specific gravity values obtained from laboratory analysis. Figure 6.3 indicates the significant difference in void ratio between fibrous and nonfibrous mixes. This difference in void content is believed to be a major reason for the difficulties encountered in pumping fibrous concrete.

The minimum values on the curve have been cited as near optimum. It has been suggested by Troxell, Davis and Kelly (1968) that the coarse to fine aggregate percentage at minimum voids does not give the densest mix. This is due apparently to a separation of aggregates by the cement paste. Hence it is most desirable to use a mix with a coarse to fine aggregate percentage slightly different, and in this case higher, than optimum by about 5 percent.

6.3.2 MIX DESIGN AND PUMPING TESTS

Mixes have been formulated using the minimum void principle. The void left by the aggregate can be filled by adding an appropriate amount of cement and water. It is impractical to obtain sufficient quantity of the sand with ideal gradation for use in concrete mixes so it was necessary to use the sand of Table 2.1, which is deficient in fines. In order to make this

TABLE 6.1

UNIT WEIGHT TEST DATA

Steel fiber, percent	Pea gravel, percent	Sand, lb (kg)	Pea gravel, lb (kg)	Steel fiber, lb (kg)	Unit weight, lb/cu ft (kg/m^3)	Void ratio, percent
0.0	60	25.0 (11.3)	37.5 (17.0)	--	125.0 (2002)	29.9
0.0	50	25.0 (11.3)	25.0 (11.3)	--	126.1 (2020)	28.2
0.0	40	25.0 (11.3)	16.7 (7.6)	--	126.1 (2020)	28.3
0.0	30	25.0 (11.3)	10.7 (4.8)	--	125.9 (2017)	28.5
0.0	25	30.0 (13.6)	10.0 (4.5)	--	123.9 (1985)	29.7
0.0	20	30.0 (13.6)	7.5 (3.4)	--	123.1 (1972)	30.3
0.0	0	28.7 (13.0)	-- --	--	116.8 (1871)	36.8
0.9	60	25.0 (11.3)	37.5 (17.0)	2.31 (1.05)	118.9 (1905)	39.7
0.9	50	25.0 (11.3)	25.0 (11.3)	1.84 (0.83)	122.8 (1967)	34.8
0.9	40	25.0 (11.3)	16.7 (7.6)	1.55 (0.70)	123.1 (1972)	34.5
0.9	30	25.0 (11.3)	10.7 (4.8)	1.35 (0.61)	123.2 (1973)	34.4
0.9	25	30.0 (13.6)	10.0 (4.5)	1.50 (0.68)	123.8 (1983)	32.9
0.9	20	30.0 (13.6)	7.5 (3.4)	1.40 (0.64)	123.3 (1975)	33.1
0.9	15	30.0 (13.6)	5.7 (2.6)	1.30 (0.59)	122.9 (1969)	33.5
1.2	60	25.0 (11.3)	37.5 (17.0)	3.20 (1.45)	117.5 (1882)	43.0
1.2	50	25.0 (11.3)	25.0 (11.3)	2.50 (1.13)	118.4 (1897)	40.1
1.2	40	25.0 (11.3)	16.7 (7.6)	2.10 (0.95)	121.4 (1945)	37.8
1.2	30	25.0 (11.3)	10.7 (4.8)	1.80 (0.82)	122.6 (1964)	36.0
1.2	25	30.0 (13.6)	10.0 (4.5)	2.10 (0.95)	123.0 (1970)	35.2
1.2	20	30.0 (13.6)	7.5 (3.4)	1.90 (0.86)	121.8 (1951)	35.9
1.2	15	30.0 (13.6)	5.7 (2.6)	1.80 (0.82)	122.8 (1967)	35.5
1.5	60	25.0 (11.3)	37.5 (17.0)	4.00 (1.81)	114.2 (1829)	48.1
1.5	50	25.0 (11.3)	25.0 (11.3)	3.30 (1.50)	117.3 (1879)	43.7
1.5	40	25.0 (11.3)	16.7 (7.6)	2.80 (1.27)	120.4 (1929)	40.2
1.5	30	25.0 (11.3)	10.7 (4.8)	2.40 (1.09)	120.1 (1924)	40.3
1.5	25	30.0 (13.6)	10.0 (4.5)	2.60 (1.18)	122.6 (1964)	36.3
1.5	20	30.0 (13.6)	7.5 (3.4)	2.40 (1.09)	122.0 (1954)	37.2

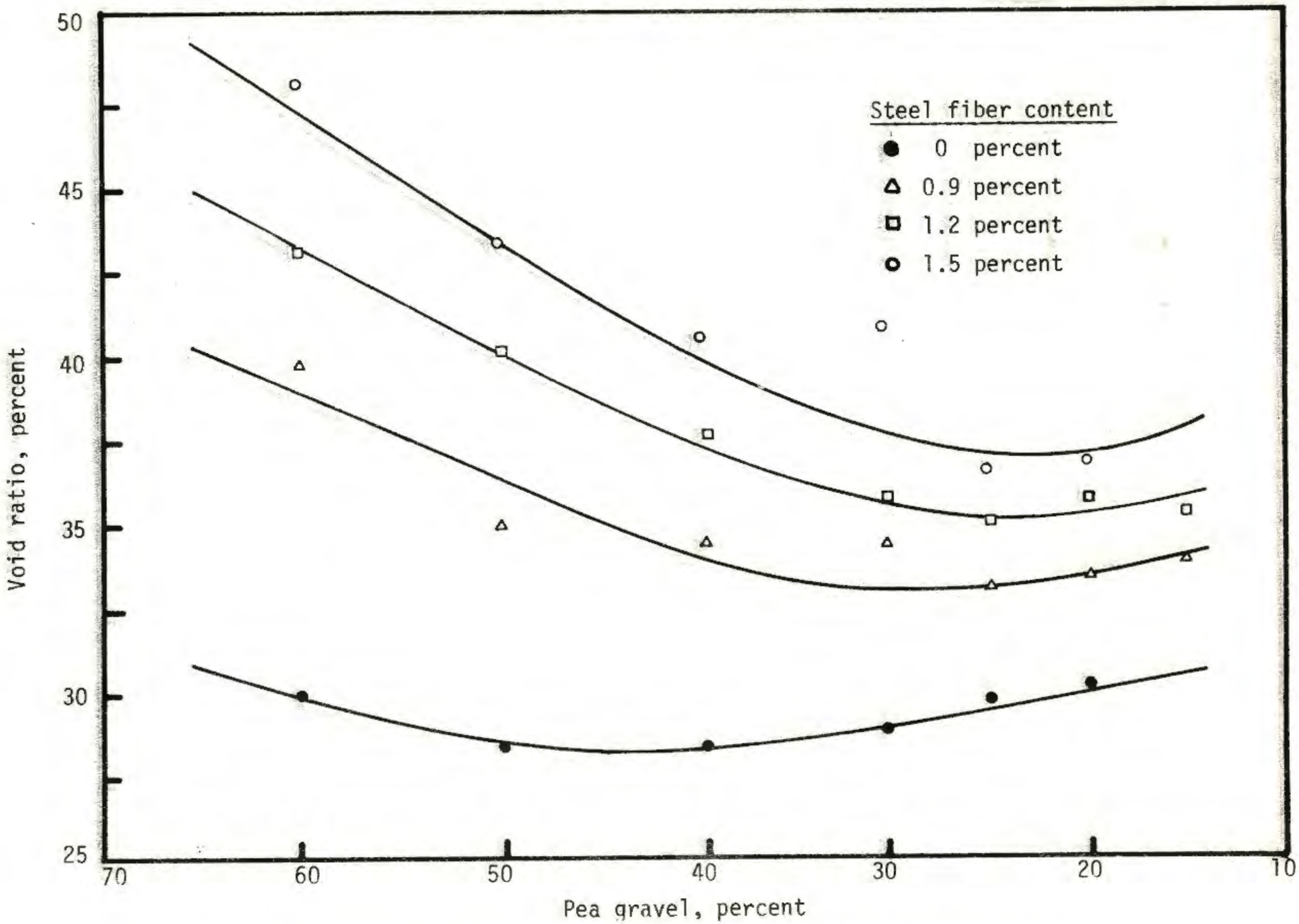


FIG. 6.3 VOID RATIO VS. PERCENT PEA GRAVEL FOR DIFFERENT FIBER CONTENTS

sand most resemble the special mix and desirable for use, fly ash was added in an amount of about 6 percent of the fine aggregate weight. Laboratory pumping tests have been performed using this sand and additional amounts of fly ash.

Some mixes have shown promising ability to conform to the testing apparatus. The inconsistency of results to date precludes the formulation of exact mix designs. Although the results are inconclusive three valuable concepts have appeared. Perhaps the most important is that the aggregates must be combined for minimum voids, or segregation failure will occur. The addition of fly ash greatly increases mix cohesiveness. If a mix will not pump at low slump, addition of some water may increase its pumpability to a point, after which any more addition of water causes rapid segregation.

6.3.3 CONCLUSIONS

Although tests have been made with fiber contents of up to 1.8 percent, the inconsistency of results has made it impossible to determine which variable causes the failures. With three or four critical constituents concrete pumping is a complex problem. Once reasonable results are available, full scale pumping tests may be necessary to confirm pumpable mix designs. With consistent results it will be possible to cite boundary conditions for pumpable concrete which relate aggregate gradation, cement content, fly ash content, and use of pumping aids.

6.4 GENERAL DISCUSSION

Void volume is the most important concept in concrete pumping.

The ability of a concrete slug to move through the pipeling is dependent on void volume and cement content. An optimum value of voids will result from a certain combination of fine and coarse aggregate.

Different natural sands may be made useful for pumping by adding certain amounts of fines. For the smallest sizes fly ash may be included. For particles in the #100 to #200 sieve size range, a beach or dune sand may be added to fill in an otherwise gap-graded aggregate.

Laboratory pumping tests are a good measure of the pumpability of the trial mixes. With the present apparatus it is possible to readily distinguish pumpable mixes. Future testing will include a back pressuring device to more closely simulate pipeline conditions with the laboratory apparatus. Fiber percentages of up to 1.8 percent have been tested with marginal success. Refinements in mix designs will most likely lead to a finalized and workable range of concrete mixes suitable for pumping.

CHAPTER 7

SUMMARY

The preceding sections of this chapter have reported considerable activity in the evaluation of concrete materials for tunnel liners.

Research has focused on the properties of fresh mixes and hardened concrete specimens made from steel fiber reinforced, regulated-set cement and Duracal cement concretes. Various mix designs were formulated for purposes of testing.

Time-dependent behavior of the experimental concretes has been investigated in terms of strength-time and creep behavior. Strength levels of steel fiber reinforced, Duracal cement concrete are lower than that for regulated-set cement concretes of similar proportions and age. The early strength behavior of steel fiber reinforced Duracal cement concrete may determine suitability for certain applications.

Observed creep behavior of the experimental concretes was nearly identical, although the mixes differ in cement type and water-cement ratio. No information was obtained to distinguish creep behavior of steel fiber reinforced, quick setting cement concrete from plain concrete.

The behavior of fiber reinforced concrete subjected to axial, flexural, and combined loads was observed. Data from these observations indicate that Poisson's ratio was not influenced by cement type, fiber content or fiber orientation, and may be taken as 0.14. Strength and Young's modulus are dependent on each of these quantities. Tests of beam-column specimens have resulted in ultimate interaction diagrams for steel fiber reinforced, Duracal cement concrete, as well as stress block constants for ultimate flexural analysis. Generalized tensile stress-strain relationships have been obtained by applying numerical techniques to data generated from tests of large beam specimens. A size effect was observed in flexural strengths obtained from large and small beam specimens.

Durability studies were continued and initiated for a variety of special environments. The effect of fibrous reinforcement on coefficient of permeability is unclear from these studies. Duracal cement concretes were most permeable, regulated-set cement concretes least permeable and type 1 cement concretes intermediate between the experimental concretes. Duracal cement exhibits a 15 percent strength reduction from leaching. Duracal cement concretes tend to be slightly expansive before the onset of shrinkage. Regulated-set cement concretes exhibit more shrinkage than do type 1 cement concretes. A decline was evident in the flexural and compressive strength of steel fiber reinforced Duracal cement concrete specimens revibrated at intervals after initial compactive effort, although the decline did not significantly affect overall strength. No significant difference was detected in the fiber distribution of revibrated and control specimens. Performance of steel fiber reinforced Duracal cement concrete in a high temperature environments has suggested that for this concrete, environment temperature is more important than time of exposure. Further study is necessary to define the characteristics in more detail. Sulfate resistance of steel fiber reinforced Duracal cement and regulated-set cement concrete has been initiated with study of weight gain, expansion, and flexural strength of beam specimens immersed in test and control environments. It is estimated that 12 to 18 months of further observation will be required before trends regarding severe deterioration will be evident.

A significant effort has been expended in the determination of factors which influence the pumpability of steel fiber reinforced concrete. Aggregate gradation and cement paste content were the parameters given major consideration. A special sand gradation was formulated, based on ACI Committee 304 recommendations, accounting for the presence of steel fiber reinforcement. The special sand was not available in sufficient quantity for use in fresh concrete mixes. Laboratory sand was used and fly ash added to supply deficient fines. Fibrous concrete mixes were designed from minimum-void aggregate fiber mixes and laboratory pumping tests begun. Tests have been performed on these trial mixes with up to 1.8 percent by volume of steel fiber reinforcement. Causes of pumping failures have been impossible to determine because of inconsistent results. Research continues on this topic with anticipated formulation of mix designs; it is expected that full scale pumping tests may be necessary to confirm pumpable mix designs.

The research reported in this chapter contributes extensive information to that previously obtained regarding the development of new concrete material for tunnel liners. Continuing studies are necessary for the completion of long-term tests; the resolution of current problems and questions; the search for, and continued evaluation of new materials; and the extension of research to further areas of interest.

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