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Service Load Deflections of High-Strength Fiber Reinforced Concrete Beams

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Abstract. Test results of eighteen beams of high-strength concrete of 85 MPa (12,300 psi) compressive strength are presented to study the influence of steel fibers and longitudinal tension and compression reinforcements on their immediate and long-term deflections. The results show that the addition of fibers in singly reinforced beams reduces immediate and long-term deflections. For beams with compression bars and fibers, the excessive amount of two reinforcements may have an unfavorable effect on the long-term deflections. The beams having the ACI Code's minimum longitudinal tension reinforcement showed much higher time-dependent deflection to immediate deflection ratio, when compared with that of the beams having about 50 percent of the balanced tension reinforcement. The theoretical method presented in the paper for predicting deflections considers the effect of steel fibers on material and section properties. The computed values for the tested beams and those of a parametric study indicate that the influence of steel fibers in increasing the moment of inertia of cracked transformed sections is most pronounced in beams having small amount of longitudinal tension reinforcement.

Keywords: beams (supports); deflection; effective moment of inertia; flexural rigidity; high-strength concrete; immediate (instantaneous) deflection; long-term (time-dependent) deflection; reinforced concrete; steel fibers.

Introduction

High-strength concrete (HSC) generally means concrete with uniaxial compressive strength in the range of 42 to 84 MPa (6000 to 12000 psi) or higher[1]. Compared with normal-strength concrete of 21 to 42 MPa (3000 to 6000 psi), HSC shows improvement in engineering properties and durability. The term "high-performance concrete" is, therefore, also used for high-strength concrete. HSC has been used in the columns for tall buildings, long span bridges and many other types of structure[1].

High-strength concrete exhibits brittle behavior in compression near failure. This behavior can be improved by adding steel fiber to concrete. Addition of fibers makes it a more homogeneous and isotropic material with improved post-peak behavior under compression[2].

Design with HSC may lead to member sizes less than those obtainable with normal-strength concrete. This reduction in size will decrease the self-weight, which can be particularly advantageous for long span members. Nevertheless, this reduction may create serviceability problem of excessive deflection due to reduced member stiffness. Design with high-strength fiber reinforced concrete (HSFRC) may lead to reductions in both member size and amount of longitudinal tension and compression reinforcements. The methods of predicting and controlling deflection in the current design codes, *e.g.,* ACI Building Code 318-95[3] are generally applicable to members of normal-strength concrete. The application of these methods to HSC and HSFRC beams needs to be examined.

The results of an experimental study by Paulson, Nilson and Hover[4] show that the creep coefficient and service load deflections of high-strength concrete beams are smaller than those of similar beams of normal-strength concrete. Nonetheless, the influence of compression steel reinforcement on time-dependent deflections is noted to be less significant for HSC beams than for normal-strength concrete beams.

Swamy and Al-Ta'an[5], Lim, Paramasivan and Lee[6], Patton and Whittaker[7], and Alsayed[8] have reported experimental and theoretical studies on fiber reinforced concrete beams of normal-strength concrete. The results of those studies showed that the addition of fibers increases the member stiffness and decreases the immediate deflection of beams. However, information on the time-dependent deflection behavior of fiber reinforced concrete (normal and high-strength) beams is presently lacking.

To study the service load deflection behavior of high-strength concrete beams with and without steel fibers (HSC and HSFRC), the authors have tested eighteen beams, with concrete compressive strength of 85 MPa (12300 psi), under sustained load in two series BS and BS(L). Keeping beam dimensions and span length constant, the volume of steel fibers, V_f, tension reinforcement ratio, ρ, and the compression to tension steel ratio, ρ' / ρ are the variables considered in the experimental program. Test results for the beams in series BS have been presented elsewhere[9]. This paper presents and discusses the results of all the eighteen beams in two series to cover the whole range of variables considered in the study. The paper also presents the computational procedure developed for computing the moment of inertia and deflection of fiber reinforced concrete beams. Based on the experimental results, a modified formula is suggested for the long-term deflection multiplier, λ, of the ACI Code.

Experimental Program

Eighteen beams tested under sustained load in the two series, BS and BS(L), were simply supported and subjected to a symmetrical two-point loading. The beam dimensions, reinforcement layout and loading arrangement are shown in Fig. 1. Three values of volume of steel fibers, V_f, (0, 0.75 and 1.5 percent), two values of tension reinforcement ratio, ρ (0.0125 and 0.0045), and three values of compression to tension steel ratio, ρ (0, 0.5m and 1.0) were used as variables to investigate their effect on immediate and long-term deflections. The beam designations and details of longitudinal reinforcement and steel fiber content for both series are given in Table 1.

The ratio of tension reinforcement ρ provided in the beams of series BS is 0.0125, which is about 50 percent of the balanced steel ratio considering longitudinal steel (without fibers) only. The ratio ρ in the beams of the second series BS(L) is 0.0045, which is higher than the minimum required by the previous ACI Building Code 318-89[10], but is slightly less than the minimum required by the current ACI Building Code 318-95[3]. An experimental study by Wafa and Ashour[11] shows that the provision for minimum tension reinforcement ratio in the previous code is inadequate for high-strength concrete, while the provision in the current code is conservative for all concrete compressive strength values.

Minimum tension reinforcement for beams is required to avoid sudden and brittle failure due to rupture of steel immediately upon concrete tension cracking. The minimum tension reinforcement in the ACI Code ensures that the reinforced concrete section has the flexural strength not less than that of the corresponding plain concrete section computed from its modulus of rupture. The beams with the minimum reinforcement will, therefore, be essentially uncracked under service loads. However, cracks may develop in these beams under sustained service loads due to the effect of creep and shrinkage. These cracks will affect the flexural stiffness and long-term deflections. In the case of HSFRC beams with the minimum reinforcement the effect of fibers also needs to be investigated.

The beams were cast and tested in the Structural Engineering Laboratories of King Abdulaziz University, Jeddah. Fig. 2 shows a general view of the testing arrangement.

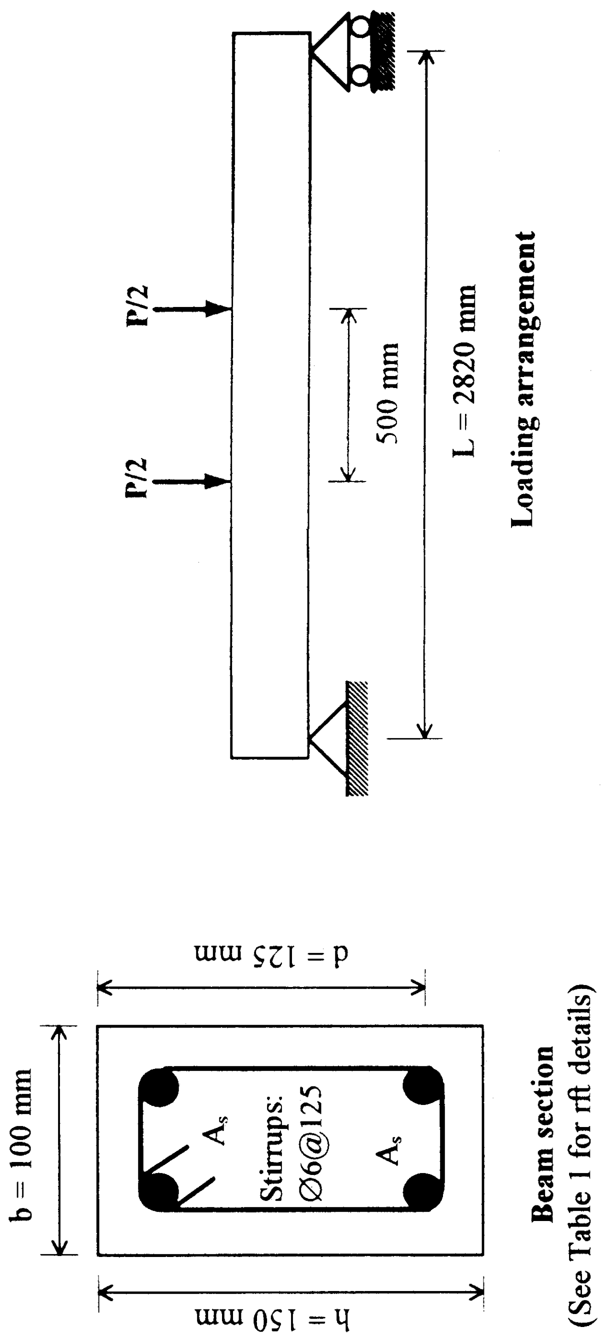


FIG. 1. Beam and loading details for series BS and BS(L).

TABLE 1. Beam designation and reinforcement details.

Series	Beam designation	Fibers volume Vf (percent)	Longitudinal reinforcement	
			Tension	Compression
1	2	3	4	5
BS	BS - 0 - 0 - A	0	2 ϕ 10	
	BS - 0 - 0.5 - A	0	2 ϕ 10	1 ϕ 10
	BS - 0 - 1.0 - A	0	2 ϕ 10	2 ϕ 10
	BS - 0.75 - 0 - A	0.75	2 ϕ 10	
	BS - 0 .75 - 0.5 - A	0.75	2 ϕ 10	1 ϕ 10
	BS - 0.75 - 1.0 - A	0.75	2 ϕ 10	2 ϕ 10
	BS - 1.5 - 0 - A	1.5	2 ϕ 10	
	BS - 1.5 - 0.5 - A	1.5	2 ϕ 10	1 ϕ 10
	BS - 1.5 - 1.0 - A	1.5	2 ϕ 10	2 ϕ 10
BS(L)	BS - 0 - 0 - L	0	2 ϕ 6	
	BS - 0 - 0.5 - L	0	2 ϕ 6	1 ϕ 6
	BS - 0 - 1.0 - L	0	2 ϕ 6	2 ϕ 6
	BS - 0.75 - 0 - L	0.75	2 ϕ 6	
	BS - 0.75 - 0.5 - L	0.75	2 ϕ 6	1 ϕ 6
	BS - 0.75 - 1.0 - L	0.75	2 ϕ 6	2 ϕ 6
	BS - 1.5 - 0 - L	1.5	2 ϕ 6	
	BS - 1.5 - 0.5 - L	1.5	2 ϕ 6	1 ϕ 6
	BS - 1.5 - 1.0 - L	1.5	2 ϕ 6	2 ϕ 6

Note: All bar sizes in mm (1 mm = 0.03937 in).

Materials

10-mm diameter reinforcing bars used in series BS have yield strength of 412 MPa (60,000 psi). 6-mm diameter bars used in series BS(L) have no definite yield region. For these bars the apparent yield strength, measured at a strain of 0.0035 is 420 MPa (60,900 psi), and ultimate strength is 575 MPa (83,400 psi). Hook ended steel fibers, made of mild carbon steel, have yield strength of 1130 MPa (159,500 psi). The fibers have an average length of 60 mm (2.36 in.), nominal diameter 0.8 mm (0.03 in.), and the aspect ratio 75.

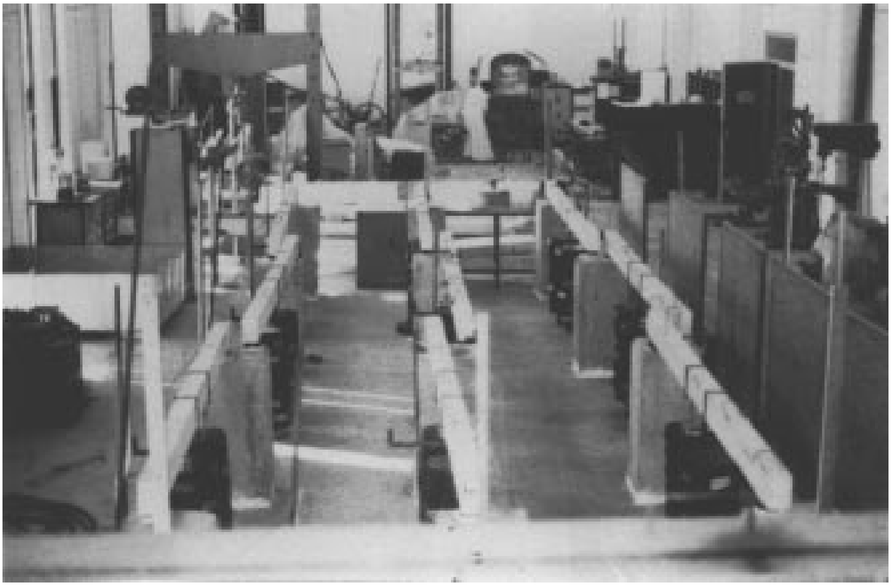


FIG. 2. General view of testing arrangement.

The concrete had a nominal 28-days compressive strength, f_c', equal to 85 MPa (12,300 psi). The required strength was obtained by using a concrete mix proportion of 1:1:2 (cement: sand: aggregate) with Type I cement, 9.5 mm (3/8 in.) crushed stone (Basalt) and desert sand having a fineness modulus of 3.1. Densified silica fume (20 percent of weight of cement), a water-cement ratio of 0.23 and a superplasticizer were used to obtain the desired high strength and a slump of 45 mm (1.8 in.). With these mix proportions and the steel fibers volume of 0 (no fibers), 0.75 and 1.5 percent three concrete mixes were produced to cast three beams in each series. Sufficient number of cylinders and prisms were cast from each mix to determine the mechanical properties and shrinkage and creep characteristics.

Beam Tests

The beams in both series were tested at the age of 28 days. The sustained load for the beams in series BS was 7.0 kN applied through two points. This value of sustained load is about 50 percent of the average load corresponding to the initiation of yielding determined experimentally from companion beams[9]. The sustained load for the beams in series BS(L) was 3.2 kN. This load is about 60 percent of the load corresponding to the moment capacity ignoring the effect of steel fibers.

The sustained load was incrementally applied through two points (Fig. 1) by using cast iron weight blocks. Mechanical dial gauges were fixed at midspan to measure deflections. The deflection reading immediately after applying the full sustained load was recorded as the immediate, initial or short-term deflection (at time t = 0). The sustained load was applied for 180 days, and during this period deflection readings were recorded at regular intervals. The beams were regularly inspected for any cracking under sustained load. After 180 days the load was removed and deflection recovery was noted for the next 5 to 7 days.

Test Results and Discussion

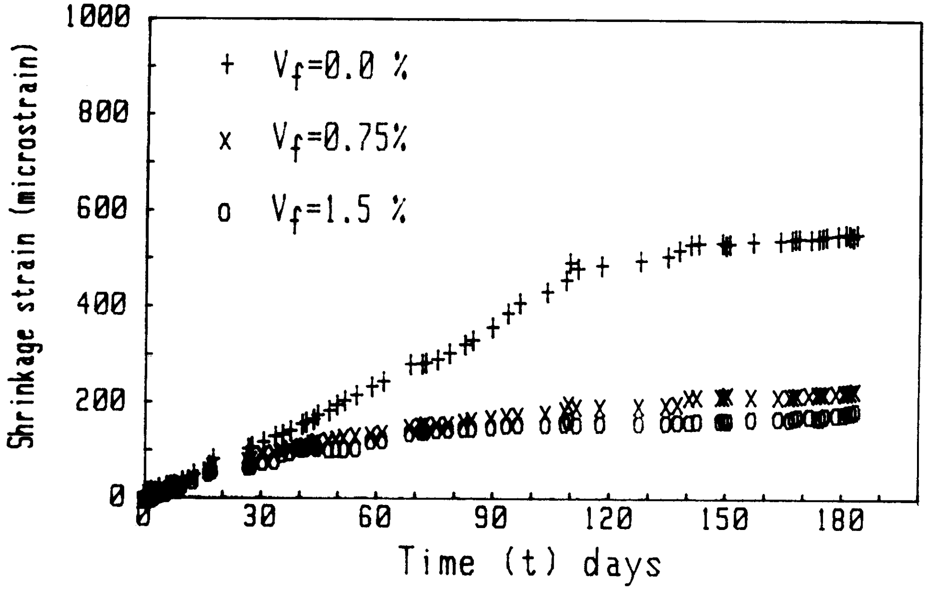
Table 2 shows the test results for modulus of rupture, splitting strength and compressive strength at 28 days and for compressive strength at 210 days for the three concrete mixes from each series. Fig. 3 shows the shrinkage and creep strain curves for the three concrete mixes used in the series BS. These curves were obtained from the test results presented earlier[9]. Fig. 3 shows that the effect of adding steel fibers to high-strength concrete is more pronounced in reducing the drying shrinkage strains than the creep strains.

TABLE 2. Properties of concrete mixes.

Series	Concrete mix	Fiber volume (percent)	At 28 days			At 210 days
			Splitting strength (MPa)	Modulus of rupture (MPa)	Compressive strength (MPa)	Compressive strength (MPa)
1	2	3	4	5	6	7
BS	1	0	4.71	8.97	80.19	89.23
	2	0.75	6.07	9.84	80.87	90.95
	3	1.5	7.24	14.05	82.32	91.84
BS(L)	1	0			82.23	
	2	0.75			85.86	
	3	1.5			88.16	

1 Mpa = 145 PSI = 0.145 ksi.

The beams in series BS showed cracks on the application of service load. The number and length of surface (visible) cracks decreased with increase in V_f. Under sustained load, new cracks were noted within and outside the initial cracking regions. The length of cracks was also noted to increase with time. On the other hand, the beams in series BS(L) did not show any surface cracks immediately after the application of service load, indicating that the beams were un-



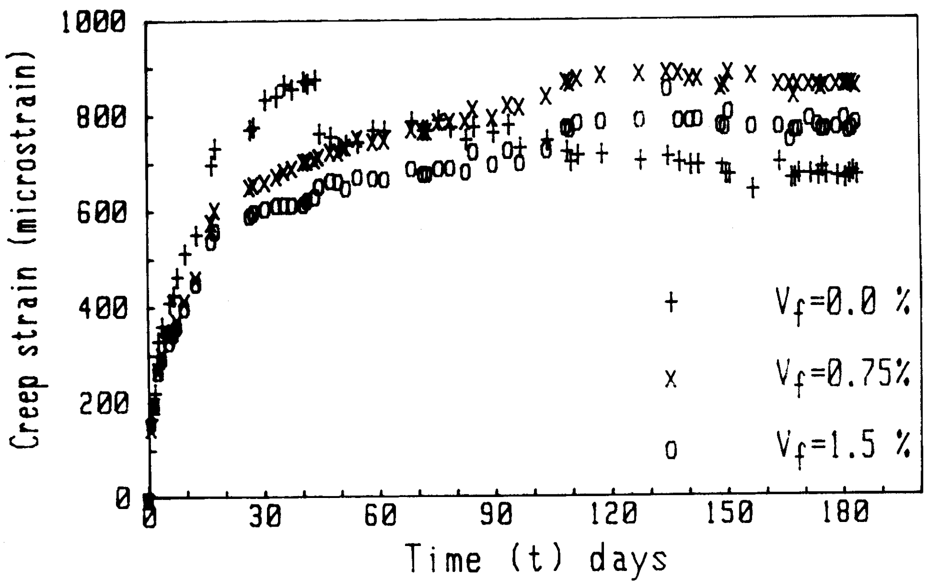


FIG. 3. Effect of steel fibers volume V_f on shrinkage and creep strains.

cracked at that load. However, cracks appeared in these beams under sustained load. These cracks affected long-term deflections of the beams, as will be discussed later.

To study the effect of steel fibers volume, V_f, and compression to tension steel ratio ρ' / ρ time versus total long-term (immediate plus time-dependent) deflection curves (t versus Δt curves) and time versus the ratio of time-dependent deflection to immediate deflection curves (t versus $(\Delta_t - \Delta_i) / \Delta_i$ curves) were drawn for the beams in both series. Figs. 4 and 5 show typical curves from both series. The results are discussed here by considering the immediate (initial) deflection, Δ_i, total long-term deflection at 180 days, $\Delta 180$, and the deflection ratio at 180 days, $(\Delta_{180} - \Delta_i) / \Delta_i$. Fig. 6 shows the effect of V_f on Δ_i, Δ_{180}, and the deflection ratio $(\Delta_{180} - \Delta_i) / \Delta_i$ for all the beams of both series. Fig. 7 shows the effect of ρ' / ρ on Δ_i, Δ_{180} and the deflection ratio $(\Delta_{180} - \Delta_i) \Delta_i$ for all the beams.

Considering the effect of V_f, Fig. 6 shows that the addition of fibers decreases the immediate and total long-term deflections for the beams in series BS. For this series, the maximum effect of the addition of steel fibers on Δ_i and Δ_{180} is noted for the singly reinforced beams ($\rho' / \rho = 0$). For the beams in series BS (L), increase in V_f from 0 to 0.75 percent shows appreciable decrease in immediate and total long-term deflections. However, with increase in V_f from 0.75 to 1.5 percent, immediate and total deflections increased. Fig. 6 also shows that for beams with compression reinforcement, increase in fiber content increased the deflection ratio $(\Delta_{180} - \Delta_i) / \Delta_i$ for both series, except for the singly reinforced beam in series BS(L).

Considering the effect of ρ' / ρ, Fig. 7 shows that for the beams having the same V_f, addition of compression reinforcement did not influence appreciably the immediate deflection in both series. The addition of compression reinforcement reduced the total long-term Δ_{180}, and the beneficial effect of compression reinforcement in reducing long-term deflections generally increased with increase in the ratio ρ' / ρ. Fig. 7 shows that for the beams in series BS, increase in the ratio ρ' / ρ from 0 to 0.5 reduced the deflection ratio $(\Delta_{180} - \Delta_i) \Delta_i$. However, no further improvement in the effectiveness of compression reinforcement in controlling the deflection ratio was noted in this series when ρ' / ρ was increased to 1. In series BS(L), the effectiveness of compression reinforcement is more pronounced for the beams without fibers ($V_f = 0$) as indicated in Fig. 7. Figs. 5 and 7(c) show that for the beams without fibers ($V_f = 0$), increase in the ratio ρ' / ρ also reduced the deflection ratio $(\Delta_t - \Delta_i) / \Delta_i$. For beams with fibers (HSFRC beams) in series BS(L), the deflection ratio increased with increase in the reinforcement ratio ρ' / ρ from 0 to 0.5 and then decreased for $\rho' / \rho = 1.0$. The results of series BS(L), therefore, augment the conclusion drawn from the series BS that for beams having steel fibers and compression reinforcement the behav-

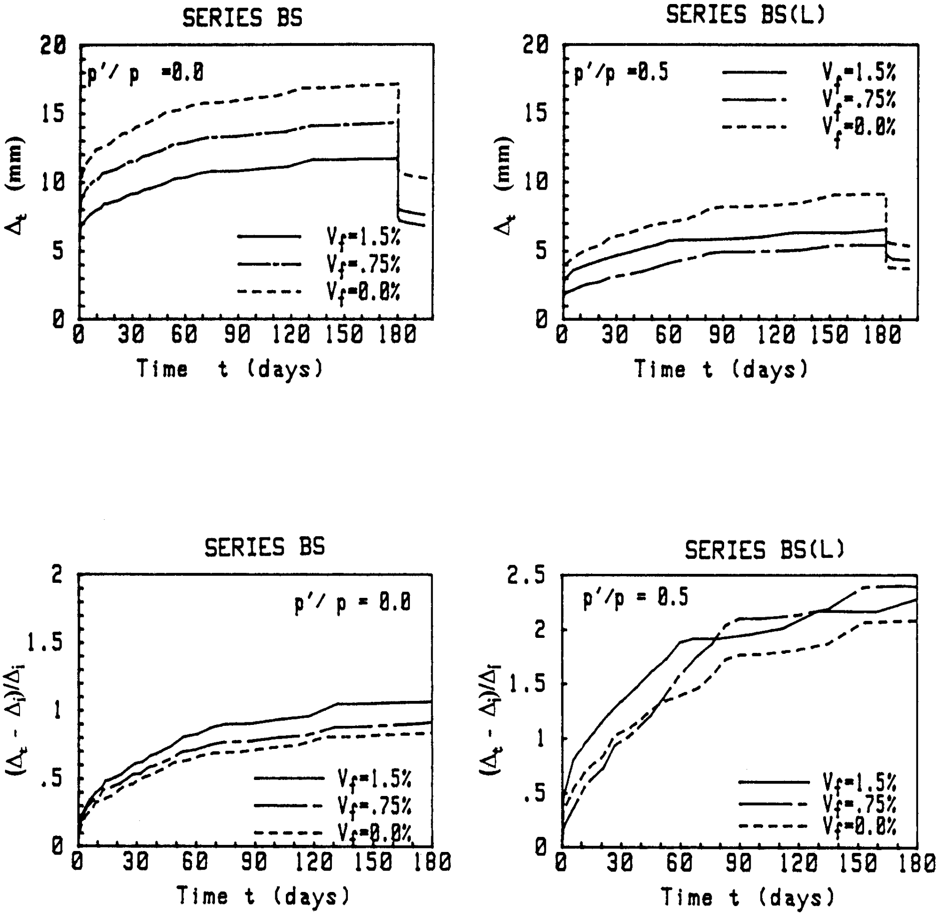


FIG. 4. Effect of V_f on the time vs. deflection [t vs. Δ_t] and time vs. deflection ratio [t vs. $(\Delta_t - \Delta_i) / \Delta_i$] curves with constant ρ' / ρ (typical results from both series).

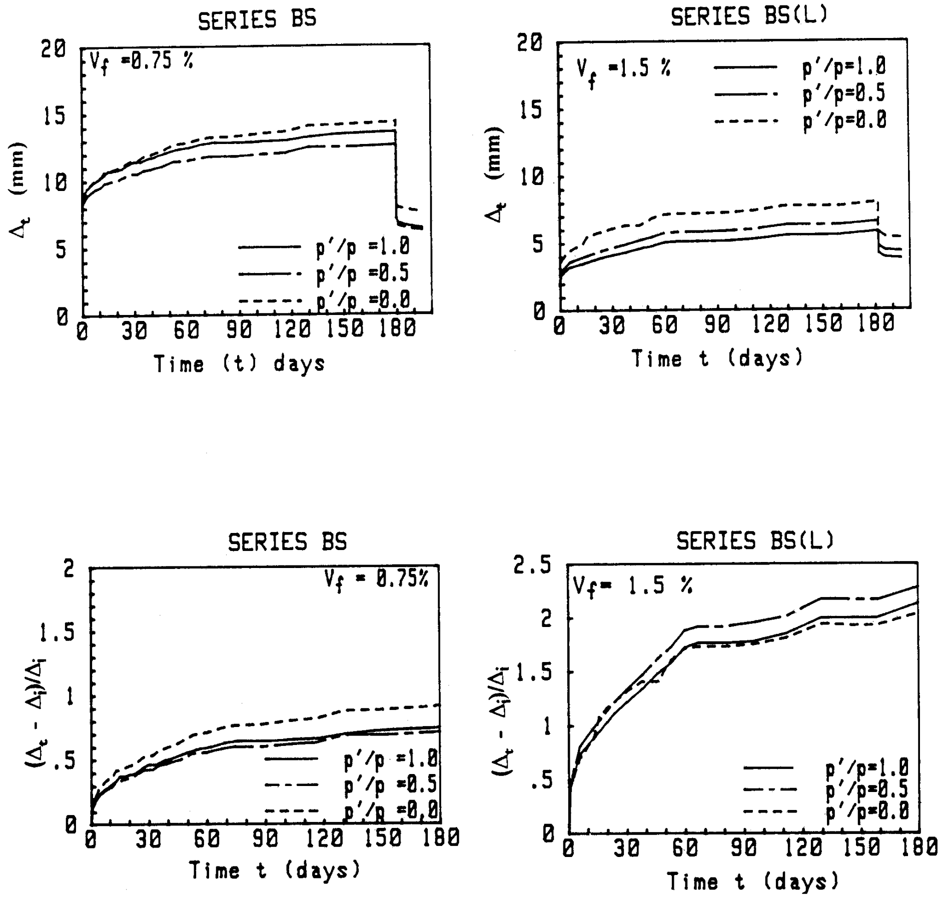
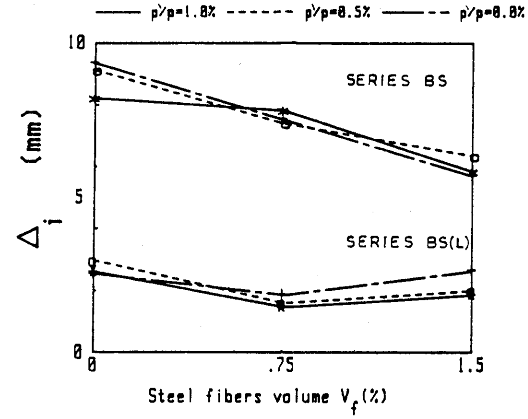


FIG. 5. Effect of ρ' / ρ on the time vs. deflection [t vs. Δ_t] and time vs. deflection ratio [t vs. $(\Delta_t - \Delta_i) / \Delta_i$] curves with constant V_f (typical results from both series).



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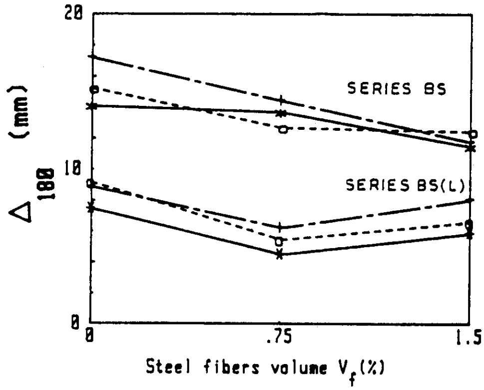

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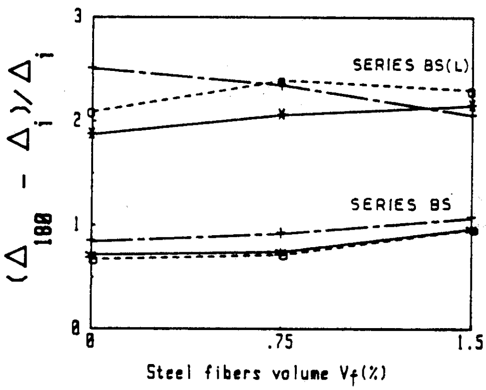

c

FIG. 6. Effect of V_f on immediate deflection Δ_i, long-term deflections at 180 days Δ_{180} and deflection ratio $(\Delta_{180} - \Delta_i) / \Delta_i$.

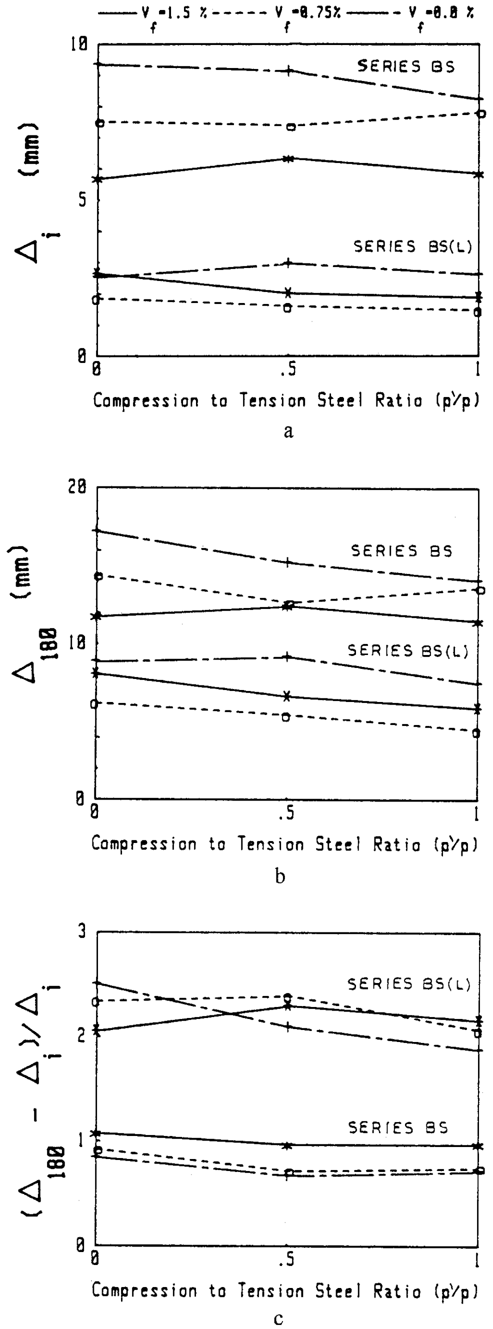

FIG. 7. Effect of ρ' / ρ on immediate deflection Δ_i, long-term deflections at 180 days Δ_{180} and deflection ratio $(\Delta_{180} - \Delta_i) / \Delta_i$.

ior depends on the amount of two types of reinforcement, and their excessive amount may have an unfavorable effect on the long-term deflection[9].

Figs. 6 and 7 indicate that the deflection ratios $(\Delta_{180} - \Delta_i) / \Delta_i$ for the beams in series BS(L) are appreciably higher than those for the beams in series BS. This is mainly due to the condition of the beams immediately after loading and under the effect of sustained load. In series BS(L) the beams were noted to be uncracked upon loading. A check of bending moments in the beams of series BS(L) under applied load also indicates these moments to be less than the theoretical cracking moment M_{cr}, based on the limiting extreme fiber tension stress equal to the modulus of rupture. However, as mentioned earlier, cracks developed under the influence of sustained load. These cracks seem to develop because of the extreme fiber tension strain reaching the limiting value under creep. The position of the nearest crack from the support in these beams indicated that it developed at a bending moment equal to about 50 percent of the theoretical cracking moment M_{cr}. These cracks reduced the stiffness of the beams, increased long-term deflection and the corresponding ratio $(\Delta_{180} - \Delta_i) / \Delta_i$ which is based on cracked/uncracked conditions. On the other hand, the beams in series BS were cracked upon loading. Although additional cracks developed in these beams under sustained loading, their ratio $(\Delta_{180} - \Delta_i) / \Delta_i$ is based on cracked/cracked conditions.

Theoretical Investigation

Immediate deflection for HSC and HSFRC beams under service load can be computed by the ACI Code[3] approach, using the formulas of elastic deflection with flexural rigidity equal to $E_c I_e$, where E_c is the short-term modulus of elasticity of concrete and I_e is the effective moment of inertia. However, the effect of steel fibers must be included in estimating the moment of inertia for HSFRC beams. The following procedure is, therefore, suggested for computing flexural rigidity $E_c I_e$ for HSC and HSFRC beams:

a. The modulus of elasticity, E_c for high-strength concrete is obtained by the formula proposed by ACI Committee 363[12].

b. The ACI Code[3] formula for I_e is modified by replacing the moment of inertia of gross concrete section, I_g, with the moment of inertia of the uncracked transformed section, I_u.

c. In computing the moment of inertia for transformed cracked and uncracked sections, the transformed area of fibers in the tension zone is considered, but their effect in the compression zone is neglected. Fig. 8 shows the proposed transformed cracked and uncracked sections for beams having longitudinal bars and steel fibers[9].

d. The cracking moment M_{cr}, to be used in the formula for I_e, is computed by assuming the modulus of rupture equal to 0.7 $\sqrt{f_c'}$ (MPa), where f_c' is the compressive strength of plain concrete ($V_f = 0$).

The orientation factor α (Fig.8), which defines the effective number (area) of fibers per unit of beam cross section, depends on the length of fibers and section dimensions. The values of α suggested by Soroushian and Lee[13] for different geometric conditions have been used in the study.

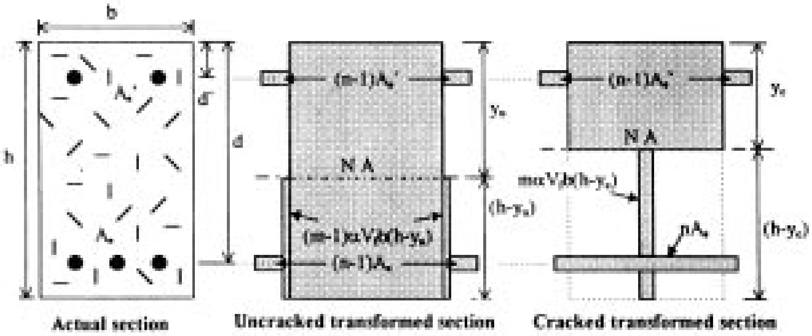

FIG. 8. Transformed uncracked and cracked sections for beams having longitudinal bars and steel fibers.

The total long-term deflection Δ_t at any time t is the sum of the immediate deflection Δ_i and the additional (time-dependent) deflection due to the effect of shrinkage and creep under sustained load. In the ACI Code[3], the additional deflection is obtained by multiplying the immediate deflection by a factor λ. On the basis of the results of beams in series BS, the following modified formula for λ has been suggested[9] for HSC and HSFRC:

$$\lambda = \frac{\mu_m \alpha_m \xi}{1 + 50\mu_s \alpha_s \rho} \tag{1}$$

where, ζ is the ACI Code's time-dependent factor for sustained load and ρ' is the compression steel ratio. μ_m and μ_s are material and section modifiers, respectively, for HSC. α_m and α_s are material and section modifiers, respectively, for steel fibers in HSC. The modifiers μ_m and μ_s were initially introduced by Paulson *et al.*[4] in the code equation for high-strength concrete beams without fibers.

Based on the results of beams in series BS, the following formulas have been suggested for μ_m, μ_s, α_m and α_s[9]:

$$\mu_m = 1.29 - 0.0000755\ f_c'\ \text{(psi)} \tag{2}$$

$$\mu_s = 1.48 - 0.000107\ f_c'\ \text{(psi)} \tag{3}$$

$$\alpha_m = 1 - 40\ V_f + 4000\ V_f^2 \tag{4}$$

$$\alpha_s = 1 - 40\,V_f \tag{5}$$

An attempt was made to further modify Eqs. (1) - (5) by including the results of the second series BS(L). However, considering that the range of values for V_f and f'_c for the beams in the two series was the same and that the high ratios of $(\Delta_t - \Delta_i) / \Delta_i$ observed in series BS(L) were not due to these variables, no modification in these equations was made.

Effect of steel fibers on moment of inertia

Table 3 shows the moment of inertia of uncracked transformed section, I_u and moment of inertia of the cracked transformed section, I_{cr}, for all the eighteen beams of series BS and BS(L) computed by the above-mentioned procedure. The values in Table 3 show that steel fibers do not have any notable effect on the moment of inertia of uncracked transformed section I_u for the beams in both series. The same is true for the compression steel ratio ρ' on I_u and I_{cr}. However, the inclusion of steel fibers significantly increases the moments of inertia of the cracked transformed sections I_{cr}, with the increase being more pronounced in the lightly reinforced (r = 0.0045) beams of series BS(L). This increase in moment of inertia is due to the decrease in the neutral axis depth that increases the tension zone and, therefore, the contribution of steel fibers.

TABLE 3. Computed moments of inertia for transformed sections of beams of series BS and BS(L).

Beam designation	Uncracked I_u ($\times 10^4$ mm^4)	Cracked I_{cr} ($\times 10^4$ mm^4)
1	2	3
BS - 0.0 - 0.0 - A	2978	826
BS - 0.75 - 0.0 - A	3004	939
BS - 1.5 - 0.0 - A	3030	1043
BS - 0.0 - 0.5 - A	3070	831
BS - 0.75 - 0.5 - A	3098	948
BS - 1.5 - 0.5 - A	3125	1055
BS - 0.0 - 1.0 - A	3159	836
BS - 0.75 - 1.0 - A	3188	955
BS - 1.5 - 1.0 - A	3216	1066
BS - 0.0 - 0.0 - L	2876	367
BS - 0.75 - 0.0 - L	2905	524
BS - 1.5 - 0.0 - L	2933	663

TABLE 3. Contd.

Beam designation	Uncracked I_u ($\times 10^4$ mm^4)	Cracked I_{cr} ($\times 10^4$ mm^4)
BS - 0.0 - 0.5 - L	2909	367
BS - 0.75 - 0.5 - L	2939	525
BS - 1.5 - 0.5 - L	2962	665
BS - 0.0 - 1.0 - L	2942	367
BS - 0.75 - 1.0 - L	2972	525
BS - 1.5 - 1.0 - L	3001	666

Note: For all beams, $I_g = 2812.5 \times 10^4$ mm^4 (1×10^4 mm^4 = 0.024 in^4).

To study the effect of tension reinforcement ρ, volume of steel fibers V_f, and orientation factor α on the computed values of I_{cr} for HSFRC beams, a parametric study was carried out. Figs. 9 and 10 show the results of this parametric study based on the properties of steel fibers, materials and section dimensions (Fig. 1) used in the experimental program. Fig. 9 shows that for any value of ρ, increase in V_f increases the ratio of I_{cr} for sections with fibers and longitudinal steel to I_{cr} for beams with longitudinal steel only. For sections with low longitudinal tension steel ratio ρ, the addition of fibers increases the moment of inertia appreciably, but the effectiveness of fibers decreases as ρ increases. Fig. 10 shows that the contribution of steel fibers in increasing moment of inertia I_{cr} may be more effective in thin (shallow) members than in relatively deep members.

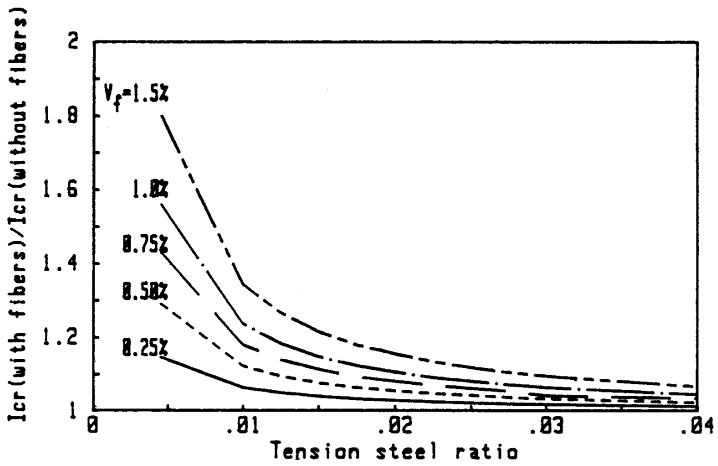

FIG. 9. Effect of fiber volume V_f on moment of inertia of cracked transformed section (b =100 mm, h = 150 mm, ρ' = 0).

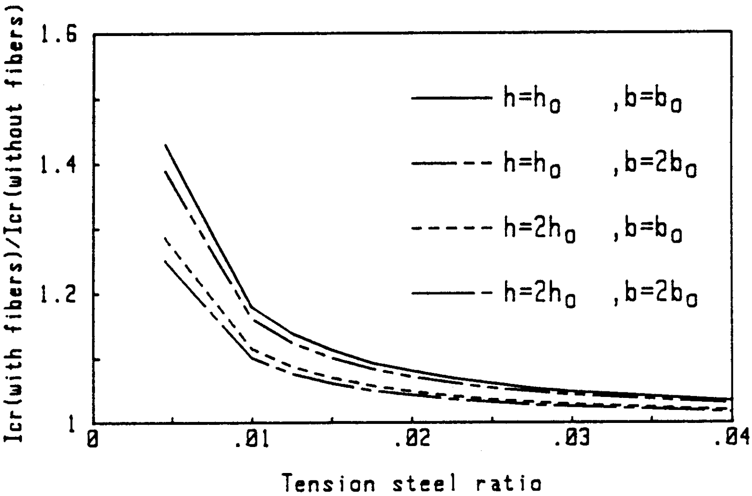

FIG. 10. Effect of beam size (section dimensions) on moment of inertia of cracked transformed section (b_0 = 100 mm, h_0 = 150 mm, V_f = 0.75 percent, ρ' = 0).

Comparison of experimental and predicted deflections

Using the above procedure and the values of I_u and I_{cr} shown in Table 3, effective moment of inertia I_e, immediate deflection Δ_i, and total long-term deflection Δ_t, at different time, were computed. In computing Δ_t, the effect of the self-weight of beams was also included. The ratio of cracking moment to the maximum span moment, M_{cr}/M_a, for the beams in series BS(L) is greater than 1 and therefore the deflections for these beams were computed by using I_e equal to I_u.

A comparison of observed and predicted time versus total long-term deflection curves (t versus Δ_t curves) for the beams in series BS has been already presented[9]. This comparison shows that the proposed procedure gives a good estimate of immediate and long-term deflections for series BS.

Table 4 presents a comparison of experimental and predicted deflections Δ_i and Δ_{180}, and deflection ratios for the beams from both series. Table 4 shows that the proposed procedure gives a good prediction of immediate deflections for the beams in both series. The method, however, grossly underestimates the long-term deflections of the beams in series BS(L), indicating that the use of immediate deflection based on uncracked section may not be valid for the beams that are initially uncracked but develop cracks under sustained loading. For these beams the values of cracking moment ($M_{cr(exp)}$) under sustained loading were obtained from the observed position of cracks and the bending moment diagram. These values of cracking moment and the vales of I_u and I_{cr}

from Table 3 were then used to compute the effective moment of inertia I_e and deflections for the beams in series BS(L). Fig. 11 shows time versus total deflection curves for these beams These curves indicate that although deflections at initial stage of sustained loading are overestimated, the adopted method gives a good estimate of long-term deflections after about 90 days for all the beams.

TABLE 4. Comparison of measured and predicted immediate and long-term deflections for beams of series BS and BS(L).

Beam designation	Δ_i (mm)			Δ_{180} (mm)		
	Measured	Predicted	Ratio*	Measured	Predicted	Ratio*
1	2	3	4	5	6	7
BS - 0.0 - 0.0 - A	9.33	7.95	0.85	17.16	15.09	0.88
BS - 0.75 - 0.0 - A	7.5	7.23	0.96	14.38	13.24	0.92
BS - 1.5 - 0.0 - A	5.67	6.67	1.18	11.73	14.46	1.23
BS-0.0-0.5-A	9.11	7.65	0.84	15.2	14.11	0.93
BS - 0.75 - 0.5 - A	7.38	6.96	0.94	12.65	12.48	0.99
BS-1.5-0.5-A	6.33	6.43	1.02	12.41	13.49	1.09
BS - 0.0 - 1.0 - A	8.2	7.36	0.90	14.04	13.23	0.94
BS - 0.75 - 1.0 - A	7.8	6.71	0.86	13.6	11.83	0.87
BS - 1.5 - 1.0 - A	5.82	6.2	1.07	11.44	12.39	1.08
BS - 0.0 - 0.0 - L	2.52	1.63	0.65	8.83	3.1	0.35
BS - 0.75 - 0.0 - L	1.85	1.62	0.88	6.18	2.97	0.48
BS - 1.5 - 0.0 - L	2.63	1.6	0.61	8.02	3.47	0.43
BS - 0.0 - 0.5 - L	2.96	1.62	0.55	9.14	3.04	0.33
BS - 0.75 - 0.5 - L	1.6	1.6	1.00	5.42	2.91	0.54
BS - 1.5 - 0.5 - L	2	1.59	0.80	6.59	2.89	0.44
BS - 0.0 - 1.0 - L	2.59	1.6	0.62	7.43	2.98	0.40
BS - 0.75 - 1.0 - L	1.46	1.59	1.09	4.47	2.87	0.64
BS - 1.5 - 1.0 - L	1.86	1.57	0.84	5.84	3.38	0.58

*Ratio of predicted to measured deflections
1 mm = 0.03937 in.

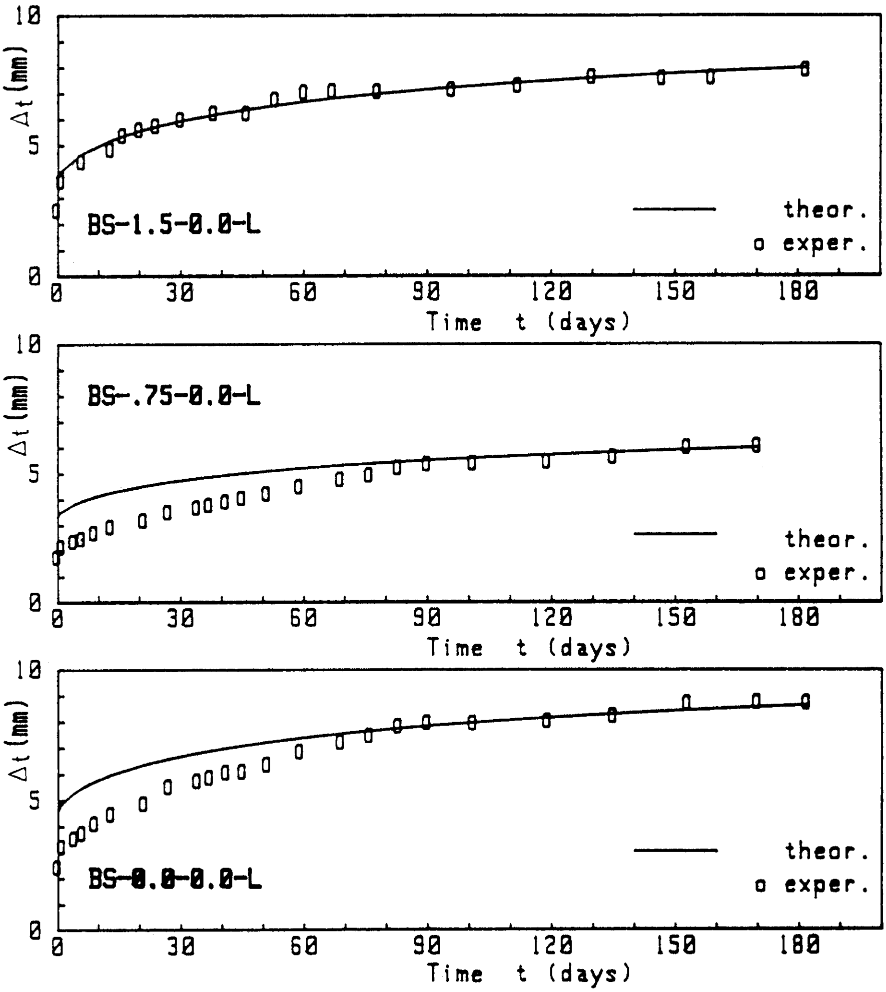

FIG. 11. Comparison of observed and predicted time vs. deflection curves for beams in series BS(L).

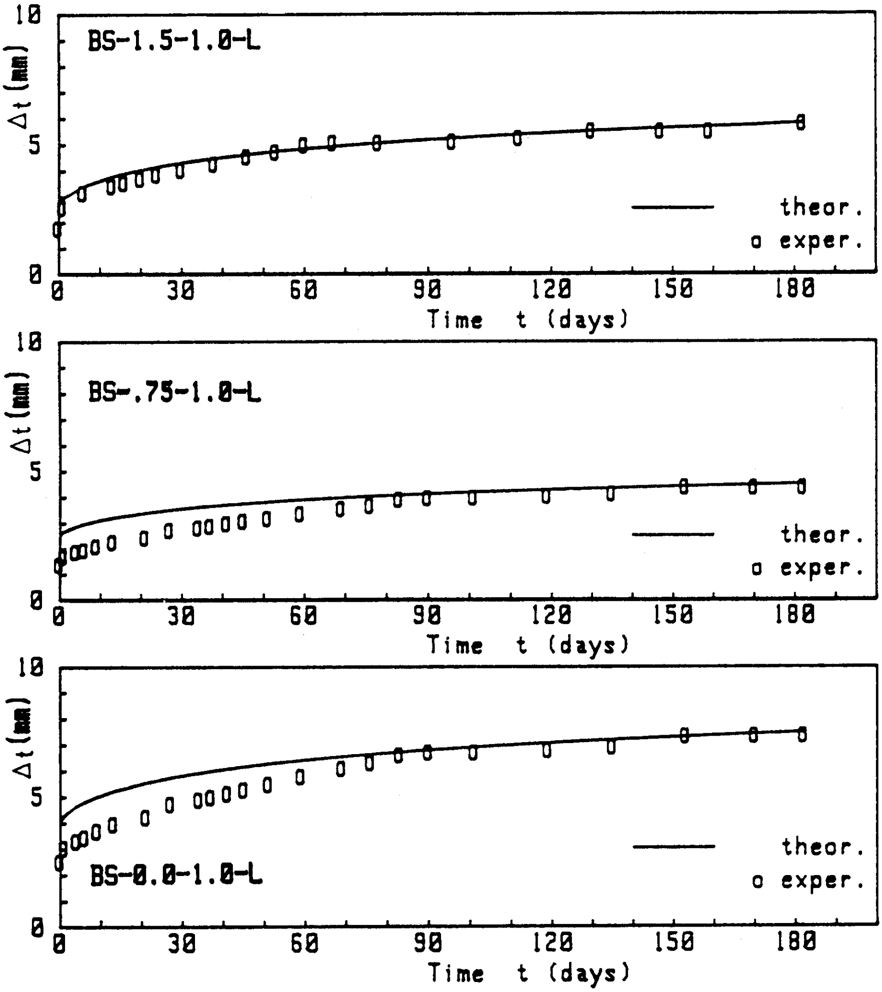
BS-1.5-1.0-L
Δt (mm)
theor.
o exper.
Time t (days)
BS-.75-1.0-L
Δt (mm)
theor.
o exper.
Time t (days)
BS-0.0-1.0-L
Δt (mm)
theor.
o exper.
Time t (days)

FIG. 11. Contd.

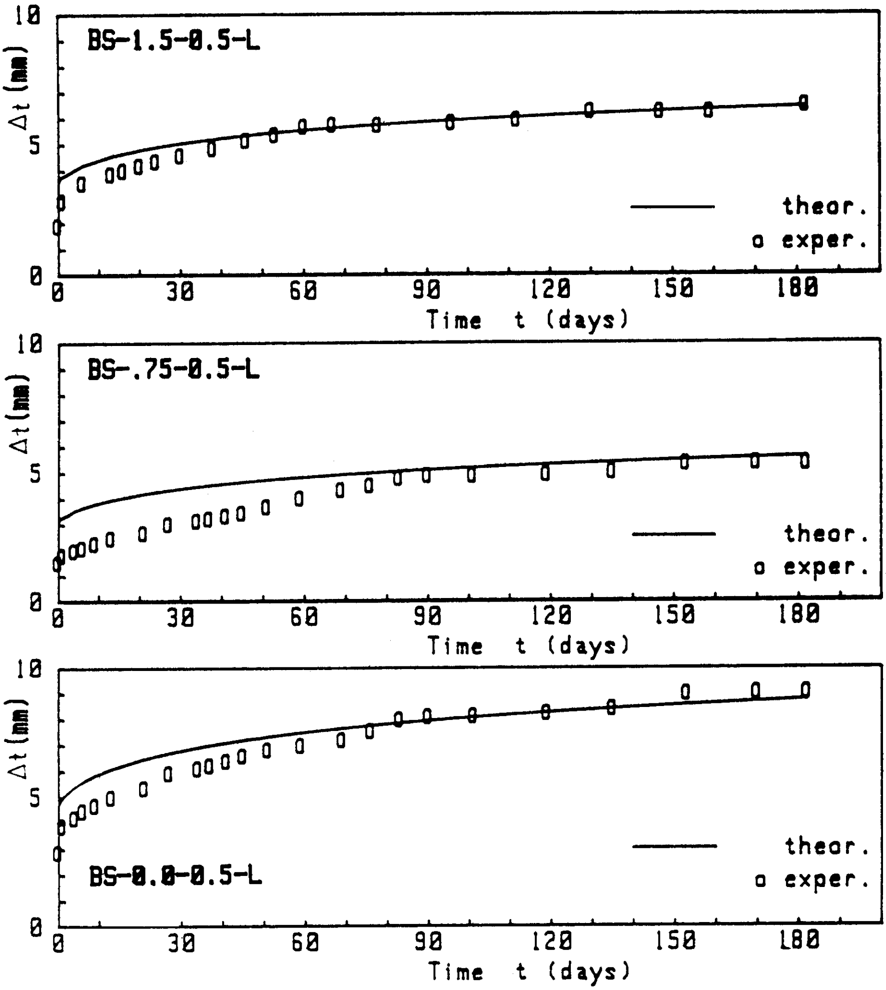
BS-1.5-0.5-L
Δt (mm)
theor.
o exper.
Time t (days)
BS-.75-0.5-L
Δt (mm)
theor.
o exper.
Time t (days)
BS-0.0-0.5-L
Δt (mm)
theor.
o exper.
Time t (days)

FIG. 11. Contd.

Conclusions

1. Addition of steel fibers to singly reinforced beams of high-strength concrete increases their flexural rigidity and reduces immediate and long-term deflections.

2. For high-strength beams with steel fibers and compression reinforcement, the long-term deflection behavior under sustained load seems to be quite complex. This behavior is affected by the amount of fiber volume and compression reinforcement; excessive amount of two reinforcements may have an unfavorable effect on the long-term deflections.

3. The beams having longitudinal tension reinforcement approximately equal to the ACI Code's minimum value indicated a much higher ratio of time-dependent deflection to immediate deflection compared with that of the beams having about 50 percent of the balanced tension reinforcement.

4. For HSFRC beams flexural rigidity and immediate deflection under service loads can be predicted by modifying the transformed sections to include the effect of steel fibers on the tension side.

5. Theoretical analysis of tested beams indicate that the addition of fibers does not increase appreciably the moment of inertia of uncracked transformed section but have a significant effect on the moment of inertia of cracked transformed section. These results and those of a parametric study further show that the influence of steel fibers in increasing the moment of inertia of cracked transformed sections is most pronounced in relatively thin members having small amount of longitudinal tension reinforcement.

6. The long-term deflection multiplier λ of ACI Code can be modified to include the effect of steel fibers on material and section properties.

7. For beams that are uncracked on loading and show gradual cracking under sustained load the use of immediate deflection based on uncracked section and multiplier λ may grossly underestimate long-term deflections. For such beams an estimate of long-term deflection, using the proposed procedure for computing the section properties and the multiplier λ, can be made if the cracking moment under sustained load is known.

Notations

A_s = Area of longitudinal tension reinforcement
A_s' = Area of longitudinal compression reinforcement
b = Width of beam section (also b_0)
h = Overall thickness of beam section (also h_0)
d = Effective depth
d' = Depth of compression reinforcement
E_c = Modulus of elasticity of concrete

f'_c = Compressive strength of concrete (at 28 days)
I_{cr} = Moment of inertia of cracked transformed section
I_e = Effective moment of inertia
I_g = Moment of inertia of gross concrete section ignoring reinforcement
I_u = Moment of inertia of uncracked transformed section
M_a = Maximum bending moment in the span
M_{cr} = Cracking moment
V_f = Volume of fiber, expressed as percentage of concrete volume
ζ = ACI Code time-dependent factor for sustained load
α = Fiber orientation factor
α_m = Material modifier for fibers in HSC
α_s = Section modifier for fibers in HSC
ρ = Longitudinal tension steel ratio $= A_s/bd$
ρ' = Longitudinal compression steel ratio $= A'_s/bd$
Δ_i = Immediate deflection
Δ_t = Total (immediate plus long-term) deflection at time t
Δ_{180} = Total deflection at 180 days
μ_m = Material modifier for HSC
μ_s = Section modifier for HSC
λ = Multiplier for additional long-term deflection

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انحرافات العوارض الخرسانية المسلحة ذات المقاومة العالية والمقواة بالألياف المعدنية بفعل أحمال التشغيل

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المستخلص . تقدم هذه الورقة نتائج اختبار ١٨ عارضة خرسانية عالية المقاومة (٨٥ ميجا باسكال) وذلك لدراسة تأثير الألياف المعدنية وتسليح الشد والضغط الطوليين على الانحرافات الفورية وطويلة المدى . وقد أظهرت النتائج أن إضافة الألياف المعدنية للعوارض الخرسانية المسلحة للشد تعمل على تقليل الانحرافات الفورية وطويلة المدى .

أما بالنسبة للعوارض ذات تسليح الضغط والألياف فإن الزيادة المفرطة لكلا التسليحين ربما تؤدي إلى تأثير غير مرغوب فيه على الانحراف طويل المدى . وأظهرت العوارض ذات التسليح الشدي الطولي الأدنى حسب المواصفات الأمريكية أن النسبة بين الانحراف الخاضع للزمن والانحراف الفوري ذات قيمة عالية مقارنة بنفس النسبة للعوارض ذات تسليح ٥٠٪ من التسليح الشدي المتوازن .

وتأخذ الطريقة النظرية لحساب الانحراف والمقدمة في هذه الورقة بعين الاعتبار تأثير الألياف المعدنية على خواص المادة والمقطع . كما أوضحت النتائج التي تم حسابها على العوارض المختبرة ونتائج الدراسة البارامترية أن تأثير الألياف المعدنية على زيادة عزم القصور الذاتي للمقطع المتشقق والمحول يكون أكثر بصورة ملحوظة بالنسبة للعوارض ذات كميات تسليح شدي صغير .