Compressive behavior of steel fiber reinforced- recycled coarse aggregate concrete with equivalent compressive strength

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Abstract: Experiments were carried out on over 100 specimens to characterize the compressive behavior of steel fiber reinforced-recycled coarse aggregate concrete (SFRCAC). Emphasis was placed on the combined effect of steel fibers (SFs) and recycled coarse aggregate (RCA) on the axial compressive behavior of SFRCAC with equivalent compressive strength. Test results indicate that with the addition of SFs, Young's modulus and stress-strain curves of SFRCAC are similar with natural coarse aggregate (NCA) concrete, but the critical strain has significant increase with increase of SFs content and RCA replacement ratio. Regression formula of compressive strength, Young's modulus, critical strain and constitutive model of SFRCAC were proposed.

Keywords: Recycled coarse aggregate; Steel fibers; Compressive strength; Critical strain; Stress-strain curve

1. Introduction

Recycling construction and demolition waste were initiated during the Second World War due to the difficulty of acquiring natural materials. Since then, extensive research

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has been conducted worldwide to develop the production of recycled materials, promoting sustainable methods of construction. One of the most effective approaches is to use waste concrete from construction and demolition as coarse aggregate in fresh concrete. Available studies have proved that the recycled coarse aggregate concrete (RCAC) is a potential solution to minimize the consumption of natural aggregate resources [1-6]. However, so far RCAC has been only permitted for use in non-structural applications such as roadway base since it is regarded as inferior to natural coarse aggregate concrete (NCAC) in terms of its structural properties. Available studies have illustrated that recycled coarse aggregate (RCA) has higher porosity and water absorption than natural coarse aggregate (NCA), so compared to NCAC with same mixture proportion the compressive strength and Young's modulus of RCAC may decrease 20-25%, critical strain may increase 10-20% [7-11].

Before RCAC is used in structural members, it must be ensured that the RCAC has satisfactory mechanical properties in comparison with NCAC. Steel fibers (SFs) have been recognized as reinforcing materials that could significantly improve the mechanical performance of concrete structural components [12-15]. Recently many studies have focused on steel-fiber reinforced recycled coarse aggregate concrete (SFRCAC). Previous research has shown that: (i) SFs could both increase the mechanical strength and modify the fracture process and toughness of RCAC, the behavior of the SFRCAC under compression becomes similar to that of steel fiber reinforced natural coarse aggregate concrete (SFNCAC) [16]; (ii) SFs can prevent and reduce the development of inherent micro-defects in RCAC [17]; and (iii) cost savings

are significant for an optimum combination of RCAC and SFs due to quantified environmental benefits of recycled aggregates[18]. Therefore, SFRCAC has great potential for application in structural members if a balance between SFs content and RCA replacement ratio is achieved for the optimal mechanical performance of concrete, which will greatly improve the use of RCA in structural application. The effect of SFs on properties of RCAC, such as failure mode, compressive strength (f_c), Young's modulus (E_c), critical strain (ε_c) and stress-strain relationship may be different from NCAC. However, there is a lack of research pertaining to the coupling effect of RCA and SFs on SFRCAC under compression.

The highlight of this paper is all specimens have equivalent cubic compressive strength (f_{cu}) although the addition of RCA replacement ratio (r_g) and SFs volume content (v_f) are different. The objectives of this study are: (1) to evaluate the effect of r_g and v_f on the f_{cu} , f_c , E_c and ε_c of SFRCAC; (2) to quantify the effect of r_g , v_f and their combined effect on the stress-strain relation of RCAC; (3) to characterize the crack pattern of SFRCAC specimens under compression. This investigation will be of significance for engineering practice and will provide a necessary data base for the further study of design methodology of RCAC structures. The experimental program and the result of the study are outlined in the following sections.

Notation of abbreviations and symbols

NCA	Natural coarse aggregate
NCAC	Natural coarse aggregate concrete
RCA	Recycled coarse aggregate
RCAC	Recycled coarse aggregate concrete
SFs	Steel fibers
SFNCAC	Steel fiber reinforced natural coarse aggregate concrete
SFRCAC	Steel fiber reinforced recycled coarse aggregate concrete
d_f	Diameter of steel fibers
l_f	Length of steel fibers
λ_f	Characteristic coefficient of steel fiber
v_f	Steel fibers volume content, %
r_g	Recycled coarse aggregate replacement ratio, %
f_c	Prism compressive strength, MPa
f_{cu}	Cube compressive strength, MPa
E_c	Young's modulus of concrete, GPa
d_c	Critical deformation, mm
E_0	Tangent modulus at the origin point, GPa
E_p	Secant modulus at the peak point, GPa
l_0	Gauge length
$W_{1,0}$	Compression work
$R_{e,1.0}$	Toughness index
ε_c	Critical strain corresponding to the peak stress of SFRCAC
ε_0	Critical strain corresponding to the peak stress of NCAC

2. Experimental Program

2.1 Materials and Mixture Proportion

Portland cement (P.O 42.5) was used in all mixtures. RCA was made of waste mix-ready concrete specimens obtained from a concrete testing station; the strength and age of these waste concrete were unknown. The waste concrete was crushed through a jaw crusher, and then sieved. The NCA was crushed limestone. Detailed properties of RCA and NCA are shown in Table 1. Compared to NCA, RCA had lower specific gravity, higher water absorption and higher porosity. The particle size

distribution of coarse aggregates obtained as result of the sieve analysis is presented in Fig.1. The fine aggregate was river sand with a fineness modulus of 2.67 and apparent density of 2556 kg/m^3 . The water reducing agent was polycarboxylate plasticizer, the optimum dosage is 1% of cement (by weight) and the water-reducing ratio is 25%. The plasticizer was used to ensure all the concrete mixtures had similar slump. All aggregates were used in the air-dry condition. The SFs were hooked at both ends, and has tensile strength of 1000 MPa, mean diameter (d_f) of 0.6 mm, mean length (l_f) of 30.5 mm, and aspect ratio (l_f / d_f) of 54.6.

Fig.1. Particle size distribution of coarse aggregates

Table 1

Physical Property of the RCA and NCA

Aggregate type	Apparent density (kg/m^3)	Bulk density (kg/m^3)	Water absorption (wt %)	Crush index (%)	Porosity (%)
RCA	2640	1412	4.85	17.7	50.3
NCA	2814	1630	1.40	8.80	44.3

RCA replacement ratio (r_g) was defined as the mass ratio of RCA to overall coarse aggregate, where $r_g = 0, 30\%, 50\%, 100\%$ respectively. SFs volume content (v_f) was taken as 0, 0.5%, 1%, 1.5%, 2% respectively. Eight group specimens in this research achieved same target compressive strength ($f_{cu} = 45 \text{ MPa}$) by adjusting the mixture

design. Details of mixture proportion are listed in Table 2.

Table 2

Mix proportions of SFRCAC

Group	w/c	Water (kg/m ³)	Cement (kg/m ³)	NCA (kg/m ³)	RCA (kg/m ³)	Fine aggregate (kg/m ³)	SFs (kg/m ³)	Water reducer (kg/m ³)
CR0F1	0.41	166	405	1121	0	740	78	4.05
CR30F1	0.40	169	422	735	315	737	78	4.22
CR50F1	0.39	170	437	506	506	737	78	4.37
CR100F1	0.365	175	480	0	938	747	78	4.79
CR50F0	0.39	158	406	553	553	769	0	4.06
CR50F0.5	0.39	164	421	527	527	749	39	4.21
CR50F1.5	0.39	177	454	506	506	754	117	4.54
CR50F2	0.39	183	469	473	473	725	156	4.69

Note: CR50F1 means the specimen with $r_g = 50\%$ and $v_f = 1\%$.

2.2 Specimen Preparation

SFRCAC was mixed through a shaft mixer. First, all aggregates and SFs were put together and mixed for 2 minutes to ensure the SFs were uniformly dispersed. Then cement was added and mixed for another minute. Finally, water and water reducing agent were added together and mixed for another 2 min. No segregation or bleeding of concrete or balling of SFs was observed in any of the eight mixtures.

The slump of fresh concrete was tested right after the mixing process. For each group, three $150\text{mm} \times 150\text{mm} \times 150\text{mm}$ cubic specimens were cast for test of f_{cu} , nine $150\text{mm} \times 150\text{mm} \times 300\text{mm}$ prism specimens were cast for tests of f_c , E_c and stress-strain curves. All specimens were cast using steel moulds and put on a vibration table for 20 seconds to ensure compaction, then demoulded after 24 hours and cured in a moisture room at approximately 95% relative humidity (RH) and 20°C temperature. The tests were performed at 28 days age.

2.3 Test procedures

All the tests were carried on a servo-hydraulic closed-loop testing machine with capacity of 3000kN. First, cube specimens and prism specimens were tested at a loading rate of 0.6 MPa/s to failure to obtain f_{cu} and f_c . Three specimens for each group were used and their mean value was taken as the test result of this group. Secondly, prism specimens were tested for E_c and stress-strain curves following Chinese Standard GB/T50081[19]. Two displacement transducers were attached to the opposite sides of the test section of the prism specimen to measure the deformation. The measured load and deformation were recorded with a data acquisition system at a rate of once per second. The test setup is shown in Fig.2. The compression specimens were preloaded to ensure axial loading. Load climbed steadily with rate of 0.6 MPa/s from zero to $f_c / 3$ and then unloaded with the same load rate to 0.5MPa, forming one loading-unloading cycle. More than three loading-unloading cycles were applied on the specimens. The preload should be ended when the difference of values between the two opposite displacement transducers was less than 15%. After the preloading process, the specimen was loaded at the same load rate to failure.

There had three stages for the test of complete stress-strain curve, that is, the same pre-loading method with the Elastic modulus test and the preload stress below $0.4f_c$ to ensure the axial compression; then loaded to the ultimate load with force control at a rate of 0.6 MPa/s; finally changed to displacement control at a rate of 0.1mm/min to obtain a stable descending stage. The whole testing process stopped when the load dropped to 30% of the peak load.

Fig.2. Test setup

3. Test Results and discussion

The obtained slump, f_{cu} , f_c , E_c , ε_c and critical deformation (d_c , which is the deformation corresponding to the peak load on the load-deformation curves) of each group are listed in Table 3. Each value in the table is an average of the test results from three specimens.

Table 3

Test results

Group	Slump/mm	f_{cu} /MPa	f_c /MPa	d_c /mm	E_c /GPa	$R_{e,1.0}$	$\varepsilon_c (\times 10^{-3})$
CR0F1	64	45.4	37.4	0.305	33.0	0.736	2.03
CR30F1	58	46.0	40.2	0.343	34.6	0.671	2.29
CR50F1	59	46.8	41.7	0.367	38.4	0.602	2.45
CR100F1	53	47.2	43.6	0.436	37.8	0.598	2.91
CR50F0	65	44.9	34.5	0.272	29.7	0.422	1.81
CR50F0.5	62	45.6	38.0	0.319	34.0	0.529	2.13
CR50F1.5	55	48.1	43.2	0.386	41.9	0.653	2.57
CR50F2	49	49.9	44.6	0.428	42.9	0.657	2.85

Note: $R_{e,1.0}$ stands for toughness index.

3.1 Load-deformation curves

Load-deformation curves of SFRCAC obtained from the tests are shown in Fig.3. It

can be seen that the load-deformation curves of SFRCAC mainly consist of three characteristic parts. The first part of curves is the linear portion, which means the material property submits to Hooke's law within the limit of proportionality. The second part of the curves is the nonlinear portion of the ascending branch which terminates at the peak point, the slope of curve decreases continuously at this stage. The descending branch is the third part of curve, the load reduces continuously with the increase of deformation at this stage. It is also found from the test that the shape of SFRCAC load-deformation curve is similar to that of SFNCAC, this similar result can be found in previous research[16], but, the key points on the curves are different with the change of v_f and r_g .

The complete load-deformation curves of SFRCAC with different r_g and same v_f of 1% are compared in Fig.3(a). It can be seen from the figure that both peak load and critical deformation increase progressively with the increase of r_g . Higher r_g could lead to higher d_c , which is consistent with published results[20, 21]. When $r_g=0$, the ascending portion is steeper and transition area is wider compared to the other three curves, which indicates that the compressive energy dissipation capacity of concrete decreases with introducing RCA.

Load-deformation curves of SFRCAC with different v_f and same r_g of 50% are shown in Fig.3 (b). It can be seen from the figure that with the increase of v_f , both peak load and critical deformation of SFRCAC increase significantly. The ascending branches of these curves are similar but the slope of descending branches are somewhat less steep with the increase of v_f . This means that the compressive energy

dissipation capacity of concrete increases with the increase of v_f , the addition of SFs can significantly improve the post-peak deformability of RCAC.

The most significant effect of adding SFs is to improve the toughness of concrete. Compression work ($W_{1.0}$) and toughness index ($R_{e,1.0}$) can be used to evaluate the axial compression toughness of steel fiber reinforced concrete[22], where $W_{1.0}$ is defined as the area under the load-deformation curve with the deformation range from 0 to $l_0 \times 1.0\%$, l_0 is gauge length ($l_0 = 150mm$ in this study). $W_{1.0}$ is an indicator of the energy absorption capacity of specimen. $R_{e,1.0}$ is defined by the following equation:

$$R_{e,1.0} = \frac{W_{1.0}}{F_{\max} \times l_0 \times 1.0\%} \quad (1)$$

Where, $F_{\max}$ is peak load.

In this study, $R_{e,1.0}$ was calculated to measure the reinforcement effect of SFs, the calculating results of $R_{e,1.0}$ for each group is listed in Table.3. When v_f increases from 0 to 2% , $R_{e,1.0}$ increases from 0.422 to 0.657, increasing ratio reaches 55.6%, which indicates SFs can remarkably improve the toughness and energy absorption capability of SFRCAC. Toughness and behavior of SFRCAC under compression becomes similar to those of SFNCAC and similar results were reported[16]. But when $v_f = 1\%$ and r_g increases from 0 to 100% , $R_{e,1.0}$ decreases from 0.736 to 0.598, this indicates that the toughness and energy absorption capability of SFRCAC degrade with the addition of RCA.

Fig.3 Load-deformation curve of SFRCAC; (a) with different RAC replacement ratio ($v_f=1\%$); (b) with

different SFs volume content ($r_g=50\%$)

3.2 Compressive strength

The compressive strength is the peak stress obtained during the compressive test on specimens. f_{cu} and f_c obtained from the tests are listed in Table 3, both f_{cu} and f_c increases with the increase of v_f and r_g .

When $r_g=50\%$, f_{cu} increases from 44.9MPa to 49.9MPa and f_c increases from 34.5MPa to 44.6MPa with the increase of v_f from 0 to 2%, increasing ratio is 11% and 29%, respectively. With the reinforcement of SFs, the RCAC and NCAC have the similar compressive behavior, SFs have better effect on RCAC than that on NCAC, which is similar to previous research[16].

When $v_f=1\%$, f_{cu} increases from 45.4MPa to 47.2MPa and f_c increases from 37.4MPa to 43.6MPa with the increase of r_g from 0 to 100%, increasing ratio is 6% and 16%, respectively. This phenomenon is different from the previously published results that compressive strength had decreasing trend with the increase of RCA [9, 20,23]. There are probably two reasons which can explain the results of this research.

One reason is that a new mix proportion method was used here to ensure the 8 group specimens had similar compressive strength[24], at the same time RCA provides better hydration in the interfacial transition zone (ITZ) due to the high water absorption. The other reason is that specimens in this paper are reinforced by SFs. SFs not only can provide the bridging effect to prevent the micro-crack expansion but also can promote hydration reaction of surrounding paste due to the fact that reaction heat can be conducted out faster. The cement matrix microstructure of RCAC with different SFs content is shown in Fig.4, which shows that the hydrated reaction is more complete with higher SFs content.

Fig.4 Representative SEM images of SFRCAC (a) $v_f = 0\%$; (b) $v_f = 1\%$; (c) $v_f = 2\%$

In some countries, such as China, f_{cu} is usually required in the building standards and is the basic value to determine the concrete strength [19]. However, f_c is usually used in engineering design. So it is necessary to establish a theoretical relation between f_c and f_{cu} .

Test results of f_c and f_{cu} from this paper and previous research [20, 21, 25-28] are drawn as scatter points in Fig.5. Obviously, the test results between f_c and f_{cu} is similar linearity. A formula between f_c and f_{cu} is proposed through the regression of test data, as flowing:

$$f_c = 0.86f_{cu} - 0.18 \quad (2)$$

The value of f_c calculated by Equation (2) based on the experimental value of f_{cu} from previous research [20, 21, 25-28] on recycled concrete aggregate was compared with the experimental value of f_c in Fig.6. Although mix proportion designs of specimens are different from the different experiments, the calculated value matches the experimental value well. Equation (2) can be used to calculate f_c of SFRCAC.

Fig.5. Relationship between f_c and f_{cu} Fig.6. Value compare of f_c between test and calculated.

3.3 Young's modulus

Young's modulus is an important property of concrete and is extensively used to calculate deformation of structures. Young's modulus was evaluated according to GB/T50081-2002 (China Standard)[19] as follows:

$$E_c = \frac{f_{c2} - f_{c1}}{\varepsilon_{c2} - \varepsilon_{c1}} \quad (3)$$

Where, $f_{c1} = 0.5MPa$, $f_{c2} = f_c / 3$; and $\varepsilon_{c1}, \varepsilon_{c2}$ are the compressive strain corresponding to f_{c1}, f_{c2} , respectively. Test results of E_c are listed in Table 3.

When $v_f = 1\%$, E_c increases from 33 GPa to 37.8 GPa with the increases of r_g from 0 to 100%, with the maximum value of 38.4 GPa at $r_g = 50\%$. This is attributed to the fact that the particles of RAC are somewhat finer than that of NAC in this study,

as shown in Fig.1. With the partial replacement of NCA with RCA, the grading of coarse aggregate can be improved, and thus, a denser concrete is achieved. As a result, RCAC exhibits a higher Young's Modulus than that without RCA and its maximum value appears at $r_g=50\%$. Previous studies show that the effects of RCA on E_c are different. No significant change of E_c was observed by substitution of RCA for NCA [16]. E_c of RCAC decreased with the increase of r_g , when $r_g=100\%$, the reduction ratio of E_c reached to 23.6% [20].

SFs has significant reinforcing effect on E_c . When $r_g=50\%$, with the increase of v_f from 0 to 2%, E_c increases from 29.7 GPa to 42.9 GPa, E_c / f_{cu} ranges from 0.66 to 0.871, achieving its maximum value of 0.871 at $v_f=1.5\%$. The results are similar with that by Marčiukaitis et al. ($l_f / d_f = 50$) [12]. This indicates that the effect of v_f on E_c of SFRCAC is not the more the better, but instead has an optimum content, in this study the best SFs content is 1.5%.

SFs and RCA also have a combined effect on the Young's modulus of SFRAC which is similar to the effect on compressive strength. Based on the experimental data from this study and previous studies [7, 10, 16, 20, 23, 29, 30], the relationship between E_c and f_c are drawn as scatter points in Fig.7. Overall, E_c increases with the increase of f_c . A formula of E_c is proposed based on the regression of test data, as shown in Equation (4).

$$E_c = 0.88f_c - 0.7 \quad (4)$$

Equation (4) is suitable for predicting E_c of SFRCAC when f_c is less than 50MPa.

Fig.8 shows the comparison of the calculated value with test data. It can be seen from

the figure that the calculated results matches the experimental data well.

Fig.7. Relationship between f_c and E_c Fig.8. Value compare of E_c between test and calculated.

3.4 Critical strain

The critical strain (ε_c) is defined as the strain corresponding to the peak stress on the stress-strain curve, which could evaluate the deformability of specimen. Test results of ε_c are shown in Table.3. Generally ε_c increases with the increase of f_c . With similar f_c , both v_f and r_g can cause the increase of ε_c .

In order to analysis the effect of SFs and RCA on ε_c , the relationship between the increase ratio of ε_c and λ_f , r_g is shown in Fig.9, according to the test results, in which λ_f is character coefficient of SFs, $\lambda_f = v_f l_f / d_f$. The critical strain ratio, which is the critical strain of SFRCAC to that of NCAN having the same mix design as SFRCAC except the different r_g , was calculated based on test data originally from this paper and previous literatures.

It can be seen that ε_c increases with increase of r_g , the previous study also proved this conclusion [2]. The main reason for the increase of ε_c is that RCA has higher void and lower density. Quantity of cement paste matrix in RCAC is much higher than NCAC because of the inherent existence of old cement paste matrix in RCA. When r_g

reaches 100%, the increase ratio of critical strain were 5.5%[31], 20%[32], 20.5% [20], respectively. SFs have similar increasing effect on ε_c of RCAC and NCAN, but increasing effect on RCAC is higher than that on NCAN [26, 33-37]. Obviously, with the combined effect of SFs and RCA the increase rate of ε_c is bigger than the other, the biggest increase ratio of ε_c reaches 40% in this study.

Based on the test results from this paper, an empirical formula of ε_c is put forward to take into account effect of SFs and RCA, as shown in Equation (5):

$$\varepsilon_c / \varepsilon_0 = (1 + 0.37\lambda_f)(1 + 0.22r_g) \quad (5)$$

Where, ε_c is the critical strain of SFRCAC; ε_0 is the critical strain of NCAC; When ε_0 is known, ε_c can be estimated by Equation (5). Using the empirical formula of $\varepsilon_0 = (0.7 + 0.172\sqrt{f_c}) \times 10^{-3}$ [38], f_c is compressive strength of NCAC (in MPa), ε_c can be calculated by Equation(6):

$$\varepsilon_0 = (0.7 + 0.172\sqrt{f_c})(1 + 0.32\lambda_f)(1 + 0.22r_g) \times 10^{-3} \quad (6)$$

When f_c is known, ε_c can be estimated by Equation (6). Compare of ε_c calculated by Equation (5) and (6) with the test results is shown in Fig.10. It can be seen that the calculation results are in good agreement with test data.

Fig. 9. Relationship of $\varepsilon_c / \varepsilon_0$ with r_g and λ_g Fig.10. Value compare of ε_c between test and calculated.

3.5 Stress-strain relation

The basic character of the stress-strain curves are consistent with the load-deformation curves which have been analyzed in the previous section. In order to make the research of the stress-strain curve based on the same foundation, the stress-strain curves are transformed into the dimensionless form, as shown in Fig.11.

As shown in Fig.11(a), the ascending branches of stress-strain curves are very close for the specimens with different v_f . This indicates that the stiffness of SFRCAC before peak load is mainly affected by concrete matrix instead of SFs reinforcement. However, with the increase of v_f , descending branch of the curve becomes flatter, this proves that SFs leads to significant improvement in deformability after peak load, As shown in Fig.11(b), RCA has significant impact on the entire curve. At ascending portion, the stress-strain curve of concrete without RCA is steeper than that adding RCA. Although r_g is different, the other three curves almost overlap together before peak point. At descending stage, stress-strain curve becomes steeper when r_g increases from 0 to 50%, but the curve has little change when r_g increases from 50% to 100%.

Fig.11. Normalized stress-strain curve of SFRCAC obtained from the tests
(a) with different SFs content;(b) with different RCA replacement ratio

Mathematical character of typical normalized stress-strain curve including original

point ($x = 0, y = 0$), ascending portion ($0 \leq x < 1, \frac{d^2y}{dx^2} < 0$, curve slope decreased),

peak point ($x = 1, y = 1, \frac{dy}{dx} = 0$), inflection point of decreasing portion ($x > 1, \frac{d^2y}{dx^2} = 0$),

maximum curvature point of decreasing portion ($x > 1, \frac{d^3y}{dx^3} = 0$). These characters of

stress-strain curves of SFRCAC are similar with those of common concrete. So the

constitutive model of NCAC can be extended to SFRCAC. In order to obtain accurate

stress-strain fitting curve, many functional forms, such as polynomial, exponential

function, trigonometric functions, rational fraction, were used in previous

researches[12, 15, 39-42]. Two of these models were chosen to express the

stress-strain curve of SFRCAC.

One constitutive model[41]is as follows:

$$y = \begin{cases} \frac{\xi x - x^2}{1 + (\xi - 2)x}; 0 \leq x \leq 1 \\ \frac{x}{\psi(x-1)^2 + x}; x > 1 \end{cases} \quad (7)$$

Where, $x = \varepsilon / \varepsilon_c$, $y = \sigma / f_c$, $\left. \frac{dy}{dx} \right|_{x=0} = \xi = \frac{d\sigma / df_c}{d\varepsilon / d\varepsilon_u} \bigg|_{x=0} = \frac{E_0}{E_p}$, ξ is the ratio of tangent modulus at the origin point (E_0) to the secant modulus at the peak point (E_p) and a parameter related to the curvature of the ascending portion. ψ is the parameter related to descending branch, when $\psi = 0$, $y=1$, the curve is horizontal line, which means the concrete is totally plastic after peak point; when $\psi = \infty$, $y \equiv 0$, the curve is vertical line, which means the concrete is totally brittle after peak point. The bigger of the ψ value, the brittle of the concrete after peak point.

The advantage of Equation(7) is that parameters have clear physical meaning, but the functional form is complicated piecewise function. Comparatively constitutive model proposed by Ezeldin and Balaguru [42] is more brief,

$$y = \frac{\beta x}{\beta - 1 + x^\beta} \quad (8)$$

Where, $x = \varepsilon / \varepsilon_c$, $y = \sigma / f_c$, β is the only parameter determined by material.

The value of ξ , ψ and β is determined by experimental stress-strain curve of SFRCAC and the relationship between ξ , ψ , β and λ_f , r_g is shown in Fig.12. It can be seen that ξ increase with the increase of λ_f , rapidly decreases when r_g increase from 0 to 50%. Compared to λ_f , ξ is more sensitive to r_g . ψ and β increase with increasing of r_g and decreasing of λ_f .

Fig.12. Relationship of constitutive model parameters with λ_f and r_g

According to the above analysis, the regression equation of ξ 、 ψ 、 β is set up by using λ_f and r_g as variables, as follows.

$$\xi = 3.92 + 0.7\lambda_f - 3.2r_g, R^2 = 0.44 \quad (9)$$

$$\psi = 0.75 - 0.6\lambda_f + 0.78r_g, R^2 = 0.53$$

$$\beta = 2.15 - 0.53\lambda_f + 0.81r_g, R^2 = 0.44 \quad (10)$$

ξ 、 ψ and β can be calculated by Equation (9) and Equation (10), then the fitted stress-strain curve can be determined by Equation (7) and Equation (8). The comparison of fitted curves with experimental curves is shown in Fig.13. It can be seen that ascending portion estimated by the two models is very close, but the descending portion estimated by Equation (7) is more accurate. Overall, fitted curves matches well with experimental curves. The constitutive model of NCAN still can be used to estimate the stress-strain curves of SFRCAC by using appropriate parameter values.

Fig.13.Comparison of calculated normalized stress-strain curve of SFRACAC with that from test

3.6 Failure mechanism

Compared to RCA, SFs has significant effect on the failure mode of SFRCAC. For the Group CR50F0, $r_g = 50\%$ and $v_f = 0$, no visible cracks were observed in the early stage of loading. When approaching to the peak load, one crack or several longitudinal parallel cracks started to appear in the central portion of the specimen and then propagated quickly. Finally, specimen crushed locally and cracked, hardly maintained its integrity, the failure process was similar to NCAC.

For specimens with the SFs, with the increase of load, the first crack formed in the middle of specimen due to lateral expansion, and followed by other cracks appearing nearby. These cracks were parallel to the direction of force. At this moment, steel fibers perpendicular to the loading direction intercepted these cracks and offered some resistance to their growth. Later, diagonal cracks appeared in upper or lower portion of specimen. With the increase of load, diagonal cracks broadened and extended continuously, led to them connect with the cracks at the middle parts. At the same time, many small vertical cracks grew in the middle parts, resulting in the peeling of the surface of specimen. Finally, cracks extended to the other side of specimen. Loud noises were heard when SFs was being pulling out of the cement matrix at this stage. When the stress was beyond its peak value, the specimen did not break suddenly, maintained a good integrity at the end of testing.

After the compressive test, the angle of the main diagonal cracks was measured. The typical crack patterns of four samples with the same SFs volume content ($v_f = 1\%$) are shown in Fig.14. The angle of the principal diagonal crack for specimens CR0F1 ($r_g = 0$), CR30F1 ($r_g = 30\%$), CR50F1 ($r_g = 50\%$) and CR100F1 ($r_g = 100\%$) was about 65° ,

75°, 85° and 90° respectively. It is clear that the angle of main diagonal cracks increases with the increase of the RCA replacement ratio.

Fig.14. Typical crack pattern of specimens

4. Conclusions

The mechanical characteristics of concrete with different RCA replacement ratio and different SFs volume content have been experimentally studied through a compressive test. Based on the extensive test data and analysis, the following conclusions can be drawn:

1. Through reasonable mix proportion design and reinforcement with steel fibers, RCAC with different RCA replacement ratio can have similar compressive strength. Compared to NCAC, steel fibers have better reinforcing effect on RCAC. For the SFRCAC with equivalent compressive strength, its toughness index increases with steel fiber content and decreases with RCA replacement ratio.
2. With the equivalent compressive strength, Young's Modulus of SFRCAC achieves maximum value at $r_g = 50\%$, the best SFs content is 1.5%. A

calculation model of Young's Modulus is proposed by regression of experimental data.

3. The critical strain of SFRCAC significantly increases with the combined effect of SFs and RCA, the biggest increase ratio reaches to 40%. An empirical formula is proposed to estimate the critical strain of SFRCAC.
4. The mathematical character of stress-strain curves of SFRCAC is similar with that of NCAC. The constitutive model of NCAN still can be used to estimate the stress-strain curves of SFRCAC by using appropriate parameter values. Stress-strain curves of SFRCAC can be estimated through Equation (7) to (10).
5. The failure of RCAC specimen is brittle with localized crushing. With the addition of SF, the RCAC specimen exhibits ductile failure and maintains their integrity during testing. Angles between the major diagonal crack and horizontal line increase with the increase of RCA replacement ratio.

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