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Study on seismic damage of SRHSC frame columns

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The damage evolution in steel reinforced high strength concrete (SRHSC) frame columns was studied based on the test results of cyclic reversed loading experiment of 12 frame column specimens with various axial compression rations, stirrups ratios, steel rations and loading histories. The variation law of the ultimate bearing capacity, ultimate deformation and ultimate hysteretic energy dissipation of specimens under different loading protocols was obtained. The seismic damage characteristics, as well as strength and stiffness degradation, of SRHSC frame columns were analyzed. Based on the analysis of the nonlinear double parameters combination of deformation and energy, a damage model that can well reflect the mechanical characteristics of members subjected to a horizontal earthquake action was established by considering the effects of the number of the loading cycles on the ultimate resistance capacity (ultimate deformation and ultimate energy dissipation capacity) of members, and the loading history on damage, etc. According to the test results, the related parameters of the damage model were proposed. Finally, the damage model proposed was validated by the test results. Results indicated that the proposed damage model is theoretically more reasonable and can accurately describe the seismic damage evolution of the SRHSC frame columns. The results also can be used as a new theoretic reference for the establishment of damage-based earthquake-resistant design method of SRHSC members.

SRHSC frame columns, experimental study, damage analysis, damage model, model parameters

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1 Introduction

It is generally necessary to permit a certain degree of damage in the practical earthquake-resistant design. Otherwise, the design would be too costly [1]. Therefore, the related scholars of the world pay more attention to how to establish an ideal damage model in order to reasonably assess the damage degree of the structures or members under earthquake. The present research shows that the ideal damage model should not only have a relatively systematic theory but also objectively reflect the damage degree of the structures or members under earthquake.

A number of significant seismic damage models have been proposed by many researchers in recent decades. Owing to the complexity of the damage mechanism, there are some differences among the models. These damage models can be categorized into three kinds: energy-based [2, 3], deformation-based [4–6], and deformation and energy-based [7, 8].

Seismic damage analysis shows that the earthquake is a phenomenon of the reciprocation action and short duration of time [9]. The damage of the structures or members under earthquake is caused by deformation and dissipated energy. So a two-parameter expression, namely a linear combination of the damage with maximum deformation and dissipated energy is adopted in most of the seismic damage models. Such models not only reveal strength and stiffness

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degradation of the structures or members but also reflect the effect of three earthquake elements (amplitude, duration time, spectrum characteristics) on damage. Based on a lot of experimental data of reinforced concrete beams and columns, one of the best-known local damage models proposed by Park and Ang was used in a systematic regression analysis, it can be expressed as

$$D = \frac{\Delta_{\max}}{\Delta_u} + \frac{\beta}{F_y \Delta_u} \int \mathrm{d}E, \quad (1)$$

where $\Delta_{\max}$ is the maximum deformation by members under earthquake, Δ_u is the ultimate deformation under monotonic loading, F_y is the calculated yield strength, dE is the incremental absorbed hysteretic energy, and β is the non-negative parameters.

Other well-known damage models composed by displacement and strength were presented by Bracci et al. [10–14]. The models proposed by Park-Ang and by Bracci et al. provided a significant reference for the damage analysis of reinforced concrete (RC) structures and steel structures. However, there are still some major drawbacks of these methods.

(i) The theoretical basis of the weight of maximum plastic deformation is taken as $1+\beta$ is inadequate. Therefore, the result of eq. (1) is unequal to 1 when the structures or members are completely damaged under monotonic loading. At the same time, eq. (1) is only applied to elastic-perfectly-plastic system. (ii) Park-Ang model only considers the effect of maximum plastic deformation while neglecting other effects of inelastic deformation on cumulative damage. Meanwhile, it does not reflect the test fact that a larger plastic deformation has a greater effect on damage. (iii) The damage model proposed by Park and Ang just referred to the relationship of the limit hysteretic energy and the maximum deformation, but neglected the effect of the loading path. Therefore, the results achieved from the above mentioned models are not in accordance with the test results. (iv) The damage models are combined with a simple linear combination lacking of the accurate theoretical basis. A non-linear combination model is relatively more reasonable. (v) With the difficulty in determining the non-negative parameter β, Park and other scholars have given the empirical formula for β, which has a greater statistical discreteness.

In order to overcome the drawbacks mentioned above, some scholars have put forward various forms of two-parameter damage models in recent years [9]. For example, the weight of the largest plastic deformation is equal to 1 and other weights are equal to β in a damage model proposed by Usami et al. [15, 16]. This model can reduce the discreteness of calculation results and consider the effect of inelastic deformation on the cumulative damage. Wang et al. [11] analyzed the relationship of the displacement ductility factor and the limit hysteretic energy based on the existing research. And then, a modified Park-Ang model was proposed by considering the effect of loading path on the limit of hysteretic energy. These models mentioned above can make up for the deficiencies of Park-Ang model to a certain degree, but there are still some deficiencies. For example, these models are only suitable for the ideal elastic-plastic systems.

The experimental results have shown that the limited resisting ability of members (ultimate deformation and ultimate energy dissipation capacity) degrades with the increase of the number of the loading cycles, and the max number of the loading cycles corresponding to the structure failure also decreases with the degradation of the limited resisting ability of the members [17]. Therefore, it is shown that there is a certain dynamic relationship in them which cannot be reflected in the existing damage models. In addition, most of the current damage models are proposed for RC structures and steel structures, and the suitability for steel reinforced concrete (SRC) structures needs to be further investigated.

According to the analysis mentioned above and the test results acquired by experimental study on damage of SRHSC frame columns under low cycle reversed loading, the seismic damage characteristics of SRHSC frame columns are analyzed. Based on the nonlinear double parameters with the combination of deformation and energy, the damage model which can well reflect the mechanical characters of members subjected to horizontal earthquake action has been established by reasonably considering the effects of the number of the loading cycles on ultimate resistance capacity (ultimate deformation and ultimate energy dissipation capacity) of members and the loading history on damage etc. The study results will provide the test data for the seismic damage assessment and the establishment of damage-based earthquake-resistant design method of SRHSC members.

2 Experimental program

2.1 Test specimens

A total of twelve SRHSC frame columns specimens were tested. The test specimens had a rectangular cross section of 150 mm× 210 mm. All specimens were longitudinally reinforced with four 10 mm diameter reinforcements. In order to guarantee the bond power between steel and concrete which can avoid splitting failure, the author adopted 20 mm as the net cover to the stirrups. The concrete strength grade and shear ratio λ were equal to C80 and 3.0, respectively. The variable study included compression ratios n (n=0.2, 0.4, 0.6), stirrups ratios ρ_v (ρ_v=0.8%, 1.1%, 1.4%), steel rations ρ_s (ρ_s=4.6%, 5.7%, 6.8%) and three different loading histories, respectively. The configurations of the cross sections are shown Figure 1. A summary of the detail for each specimen is given in Table 1.

Table 1 Design parameters of specimens

Specimen	Steel type	n	ρ_s	ρ_v	Stirrups type	Loading history
SRHSC-1	I 14	0.4	6.8%	0.8%	Φ6@110	Monotonic loading
SRHSC -2	I 14	0.4	6.8%	0.8%	Φ6@110	Constant amplitude loading
SRHSC -3	I 14	0.4	6.8%	0.8%	Φ6@110	Mixed-loading
SRHSC -4	I 14	0.4	6.8%	0.8%	Φ6@110	Mixed-loading
SRHSC -5	I 14	0.4	6.8%	0.8%	Φ6@110	Mixed-loading
SRHSC -6	I 14	0.4	6.8%	0.8%	Φ6@110	Variable amplitude loading
SRHSC -7	I 14	0.2	6.8%	0.8%	Φ6@110	Variable amplitude loading
SRHSC -8	I 14	0.6	6.8%	0.8%	Φ6@110	Variable amplitude loading
SRHSC -9	I 10	0.4	4.6%	0.8%	Φ6@110	Variable amplitude loading
SRHSC -10	I 12	0.4	5.7%	0.8%	Φ6@110	Variable amplitude loading
SRHSC -11	I 14	0.4	6.8%	1.1%	Φ6@80	Variable amplitude loading
SRHSC -12	I 14	0.4	6.8%	1.4%	Φ8@120	Variable amplitude loading

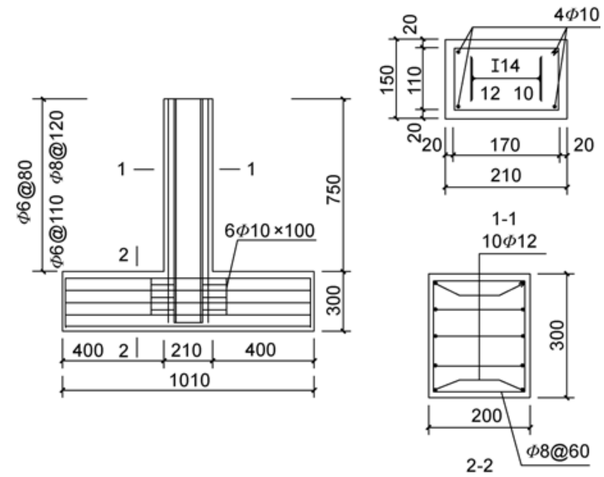

Figure 1 Sectional dimensions and distributed steels of specimens.

2.2 Material properties

The concrete mix was designed for the compressive cube strength f_{cu}, at 28 days of approximately 80 MPa. The mix proportions were: P.O 52.5R cement (450 kg/m^3); sand (544 kg/m^3); coarse aggregate (1156 kg/m^3); water (156 kg/m^3); water reducing agent (12 kg/m^3); wollastonite (30 kg/m^3); flyash (120 kg/m^3).

For each batch of concrete mixtures, three 150 mm × 150 mm × 150 mm standard cubes were also cast and maintained under the same conditions of the related specimens. Table 2 lists the measured average cube compression strength f_{cu}, average axial compression strength f_c, and modulus of elasticity E_c.

Tension tests were carried out to determine the material properties of the steel and rebars. Table 3 lists the measured average yield strength f_y, tensile strength f_u, and modulus of elasticity E_s.

2.3 Test setup

The test was carried out in the Education Ministry Structural and Seismic Laboratory of Xi'an University of Architecture and Technology. Cantilever beam was adopted in loading. Firstly, the constant vertical force was provided on the top of SRHSC frame column specimen by a hydraulic jack. And then, lateral cyclic and monotonic loads were applied by MTS actuator, respectively. A specimen was fixed in place by tie-down screw rod. The test floor bearing force system was L-shaped reaction wall. The test data were collected by 1000 channels 7V08 data acquisition instrument, and the process of the whole test was controlled by MTS electro-hydraulic servo structural test system and computer. The horizontal displacement was measured by two linear variable displacement transducers (LVDTs) at the tip and mid-span of columns with ranges of 500 and 300 mm, respectively. The test setup is shown in Figures 2 and 3.

Table 2 Properties of concrete

Concrete strength grade	f_{cu} (MPa)	f_c (MPa)	E_c (MPa)
C80	83.89	75.49	42042

Table 3 Properties of steel and rebars

Steel	Model number	f_y (MPa)	f_u (MPa)	E_s (MPa)
Steel shape	Flange	319.7	491.5	2.07×10^5
	Wed	312.4	502.5	2.07×10^5
Longitudinal reinforcements	Φ10	386.3	495.7	2.06×10^5
Stirrups	Φ6	397.5	438.0	2.07×10^5
	Φ8	354.5	457.3	2.07×10^5

2.4 Loading histories

Each specimen was tested by applying the axial gravity load and keeping it to be constant during the process of the test. The lateral force P, simulating the seismic loading, was then applied under displacement control with a hydraulic actuator placed at height L above the base of the column.

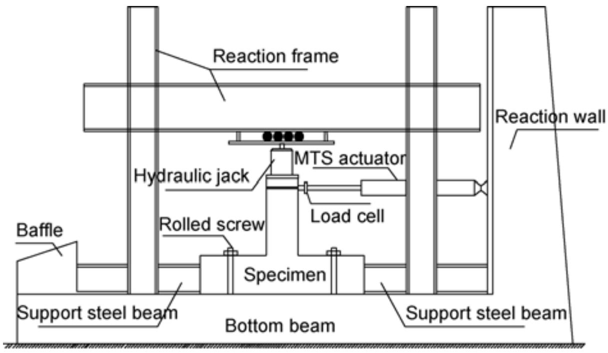

Figure 2 Test setup.

Figure 3 Experimental equipment.

The loading history of each specimen is listed in Table 1. In this test, the monotonic loading was carried out for specimen SRHSC-1; the displacement controlled cyclic loading and the monotonic loading (mixed loading) were carried out for specimens SRHSC-3~SRHSC-5; the tests of the rest specimens were carried out by displacement controlled cyclic loading. The specific loading histories are shown in Figure 4.

(i) The constant amplitude displacement controlled cyclic loading: the specimen was subjected to the constant displacement amplitude Δ, corresponding to the displacement ductility of μ=2.0 in successive cycles, where $\mu=\Delta/\Delta_y$, and Δ_y is the calculated lateral displacement of the column when the extreme longitudinal bar reaches yield. The loading history is shown in Figure 4(a).

(ii) Mixed loading: The specimens were subjected to one cycle at the displacement ductility of μ=0.2, 0.4 0.6, 0.8, and three successive cycles at the displacement ductility of μ=1.0, 2.0, and 3.0. Then the monotonic loading was adopted as long as the specimens' failure exists. The load was increased by 5% of the test yield load. The loading history is shown in Figure 4(b).

(iii) Variable amplitude displacement controlled cyclic loading: the specimens were subjected to one cycle at the displacement ductility of μ=0.2, 0.4 0.6, 0.8, and three successive cycles at the displacement ductility of μ=1.0, 2.0, 3.0, … in successive sets of cycles. The loading history is shown in Figure 4(c).

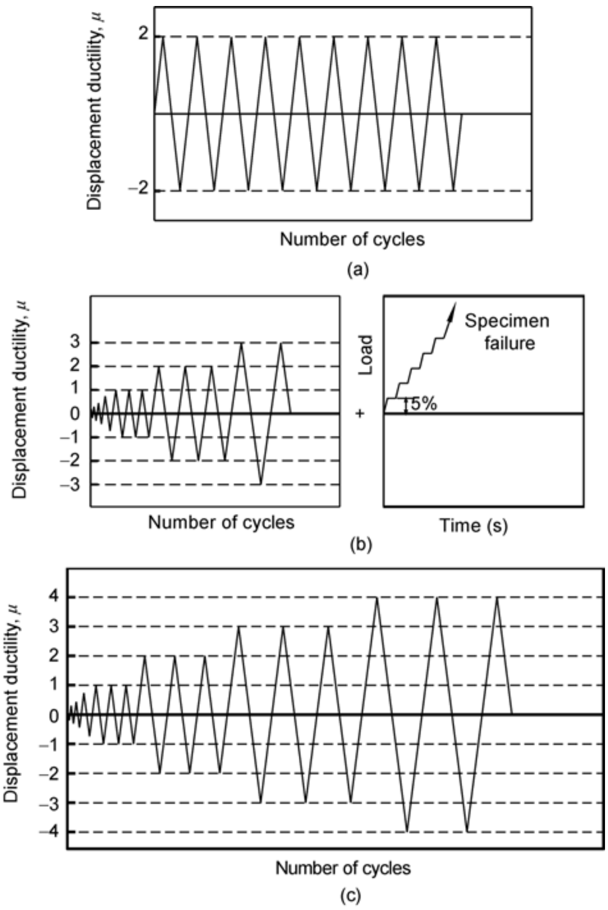

Figure 4 Loading history of specimens. (a) The constant amplitude loading; (b) the mixed-loading; (c) the variable amplitude loading.

3 Damage analysis of SRHSC frame columns

3.1 Strength and stiffness degradation

The test results show that the local damage, even the global damage of SRHSC members induced by earthquake displayed mainly at macro cracks of concrete, concrete crushing, steel flange and web, longitudinal and transverse reinforcement yielding and the bond-slip between steel and concrete, etc. All of these gradually caused the deterioration of mechanical properties of members including strength, stiffness, etc. Therefore, the damage degree and development can be reflected by the theoretical description of strength and stiffness degradation of members.

3.1.1 Strength deterioration

The hysteretic model proposed by Takeda showed that the strength deterioration of members was related to hysteretic energy dissipation. The test results also showed that the deterioration of the strength of frame columns mainly appeared after the concrete crushing and spalling. When the displacement ductility was equal to 2.0, the strength deterioration of specimens was obvious. So was the change of the shape of hysteretic loops.

In order to reflect the strength deterioration of specimens under cyclic loading, based on the test results of displacement controlled constant amplitude cyclic loading, the simplified relationship of strength deterioration and the hysteretic energy dispassion is shown in Figure 5. As one can see, from the first cyclic loading (loading path is ABCDEFG) to the *i*-th cyclic loading, the hysteretic energy of specimens changed from $E_{h,1}$ to $E_{h,i}$, and strength changed from P_1 to P_i, deterioration amplitude was ΔP. The relationship of strength and hysteretic energy can be obtained through the geometric relation shown in Figure 5. Thus, the strength deterioration can be reflected by using the energy dissipation ability under the present displacement amplitude.

3.1.2 Stiffness degradation

Similar to strength deterioration, based on the test results of variable amplitude cyclic loading, the simplified relationship of stiffness degradation and deformation was shown in Figure 6 (secant stiffness was adopted in Figure 6). In the

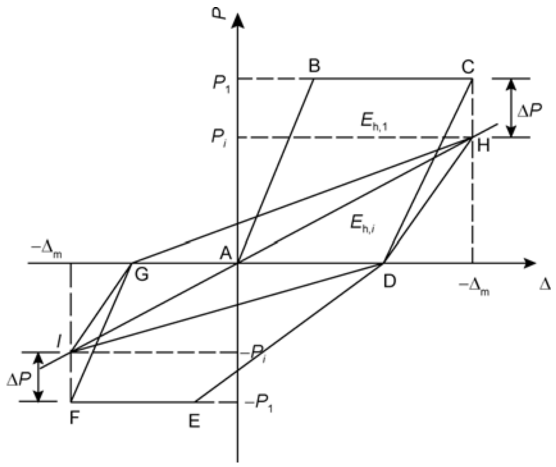

Figure 5 Simplified relation of strength deterioration and hysteretic energy dissipation.

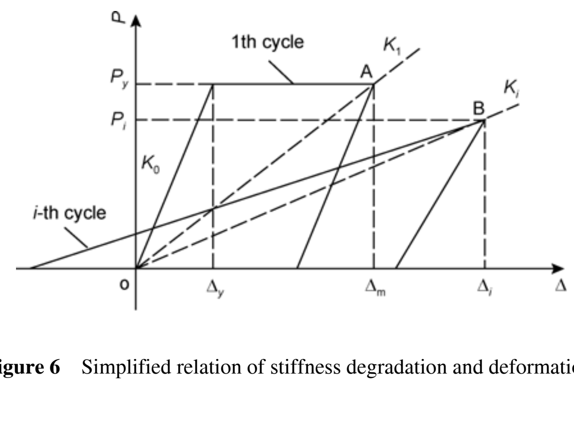

Figure 6 Simplified relation of stiffness degradation and deformation.

first positive cyclic loading with the horizontal displacement amplitudes Δ_m and Δ_i, the corresponding stiffness is K_1 (secant OA) and K_i (secant OB), respectively. It could be seen that the stiffness of specimens decreased continuously with the increase of horizontal displacement. Thus, the effect of variable amplitude displacement on stiffness can be reflected by using the deformation item.

The analysis above indicates that damage leads to strength and stiffness degradation which can be reflected through hysteretic energy and deformation. In this paper, based on the combination of energy and deformation, a new damage model will be put forward to assess seismic damage of SRHSC frame columns reasonably.

3.2 Ultimate capacity of frame columns

In this section the results of the mixed loading test were compared. The hysteretic load-displacement curves of specimens were shown in Figure 7. It was clearly observed that the ultimate deformation and ultimate energy dissipation capacity of specimens, which were subjected to the monotonic loading after different numbers of loading cycle the displacement ductility of μ=1.0, 2.0, 3.0 in turn, gradually degenerated with the damage accumulation of specimens caused by the increase of the number of loading cycles.

Based on the experimental results, the monotonic load-displacement curves of specimens subjected to different numbers of cycles of loading were shown in Figure 8. It can be shown that the ultimate capacity (ultimate deformation and ultimate energy) of frame columns gradually degenerated under monotonic loading after it experienced different numbers cycle loadings. As the number increases, accumulative damage of frame columns becomes serious and the ultimate capacity degenerates greatly. It reflects the residual ultimate capacity of frame columns after the earthquake.

In order to assess the damage degree of frame columns, according to the mention above, the variation relationship of cycle number and ultimate capacity will be considered into the damage model established in the next section.

3.3 Effect of loading histories

Loading history is one of the important influence factors for structural member damage [15]. In this section the results of the constant amplitude test of specimens SRHSC-2 were compared with the results of the variable amplitude test of specimen SRHSC-6. The load-displacement hysteretic curves were shown in Figure 7. The strength retrogression, stiffness deterioration, energy dissipated in each half-cycle versus half-cycle number, and cumulative energy versus half-cycle number were shown in Figure 9. It shows that the strength and stiffness degradation of specimens is increased obviously with the increase of displacement amplitude compared with the constant amplitude test of specimens.

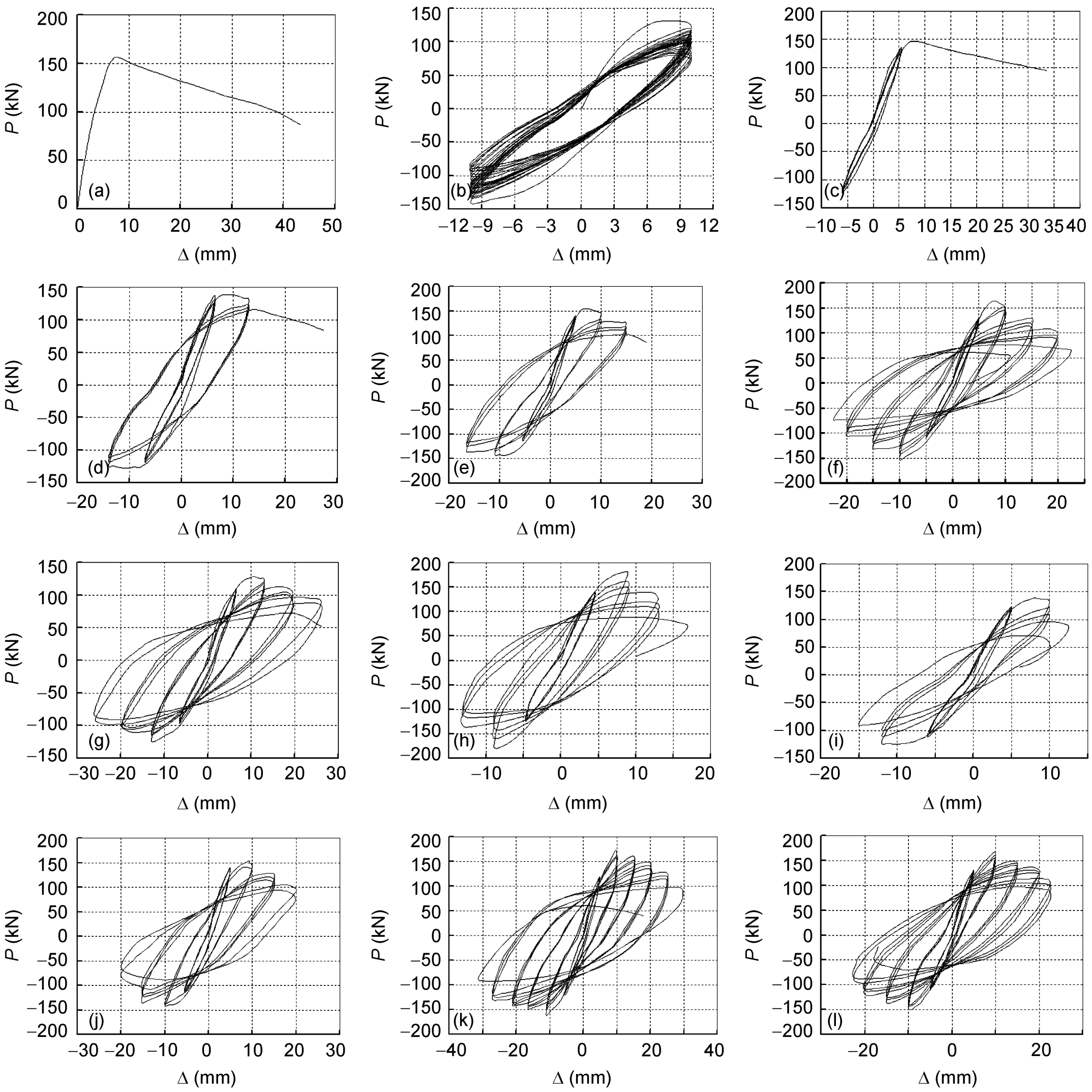

Figure 7 Hysteretic curves for specimens. (a) SRHSC-1; (b) SRHSC-2; (c) SRHSC-3; (d) SRHSC-4; (e) SRHSC-5; (f) SRHSC-6; (g) SRHSC-7; (h) SRHSC-8; (i) SRHSC-9; (j) SRHSC-10; (k) SRHSC-11; (l) SRHSC-12.

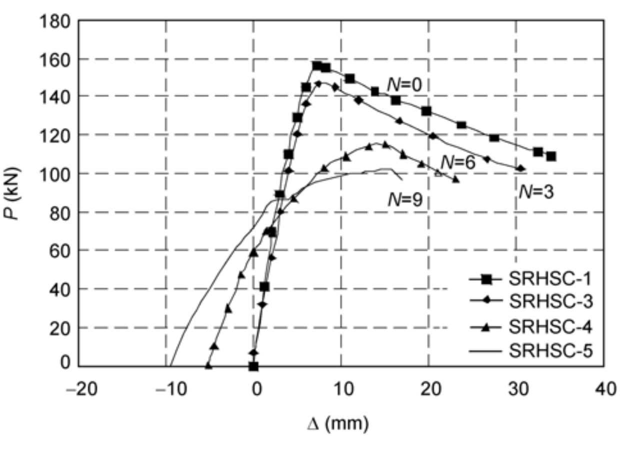

Figure 8 Monotonic load-displacement curves of specimens subjected to different numbers of cycles of loading.

The energy dissipation capacity of specimens is almost unchanged with the increase of the number of half-cycle under constant amplitude loading. On the contrary, the energy dissipation capacity of specimens is enhanced by the increase of the number of half-cycle under the variable amplitude loading. With the increase of the degree of cumulative damage, the horizontal load is reduced obviously. However, the energy dissipation capacity of specimens still has a certain amount of improvement after the peak load. With the failure of the specimen, the total cumulative energy of specimens under constant amplitude is larger than that of the variable amplitude for the same specimens.

With the increase of displacement amplitude, it is also shown that the stiffness and strength degradations are more obvious and the stiffness degradation is slight. Reversely, the strength deterioration is still more obvious in the second

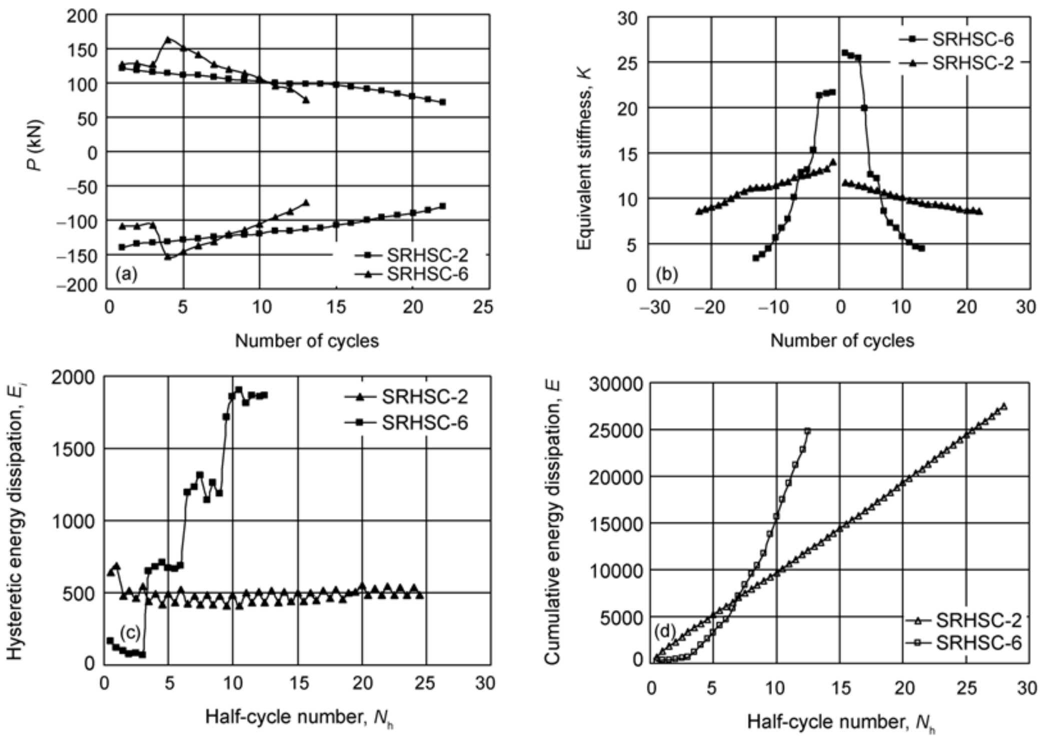

Figure 9 Comparisons between the results of the constant amplitude and the variable amplitude test. (a) Strength deterioration; (b) stiffness deterioration; (c) energy dissipated in each half-cycle versus half-cycle number; (d) cumulative energy versus half-cycle number.

and third cycles under the same displacement amplitude. The traditional damage models only consider the effect of maximum displacement on damage, neglecting the effect of the unelastic peak displacement on damage.

Based on the above, the effect of unelastic peak displacement will be also considered, as well as the effect of maximum displacement on damage. The damage model established in the next section is to simulate the action of earthquake factually.

4 Definition of the damage model

On the basis of the analysis above, the nonlinear double parameters damage model composed of deformation damage component D_Δ and cycle accumulation damage component D_c, which can well reflect the mechanical character change of SRHSC frame columns subjected to horizontal earthquake action, was established. At the same time, the interaction between deformation and energy accumulation damage was considered by introducing combination parameters γ in this model, which can be expressed as

$$D = (1-\gamma)D_\Delta + \gamma D_c, \tag{2}$$

where D_Δ and D_c are given by

$$D_\Delta = \sum_{j=1}^{N_1} \left(\frac{\varDelta_{\max,j} - \varDelta_y}{\varDelta_{u,i} - \varDelta_y} \right)^c, \tag{3}$$

$$D_c = \sum_{i=1}^{N_h} \left(\frac{E_i}{F_y(\varDelta_{u,i} - \varDelta_y)} \right)^c, \tag{4}$$

where $\varDelta_{\max,j}$ is the maximum unelastic deformation for the j-th half-cycle, N_1 is the number of half-cycles producing $\varDelta_{\max,j}$ firstly, $\varDelta_{u,i}$ is the ultimate deformation under a constant axial load and monotonic lateral load after i-th half-cycle, E_i is the hysteretic energy dissipation in the ith half-cycle, N_h is the number of half-cycles, γ is a combination parameter, and c is a test parameter.

According to the reasonable hypothesis proposed by Shen et al. [1], the cycle accumulation damage component D_c can be expressed as

$$D_c = \sum_{i=1}^{N_h} \left(\frac{E_i}{E_u} \right)^c, \tag{5}$$

where E_u is the ultimate dissipative capacity under monotonic loading.

In order to consider the effect of the number of loading cycles on the ultimate dissipative capacity of member, eq. (5) can be rewritten as

$$D_c=\sum_{i=1}^{N_h}\left(\frac{E_i}{E_{u,i}}\right)^c, \tag{6}$$

where $E_{u,i}$ is the ultimate dissipative capacity under a constant axial load and monotonic lateral load after i-th half-cycle, and depends on the number of loading cycles.

Substituting eqs. (3) and (6) into eq. (2), the seismic damage model can be formulated as follows

$$D=(1-\gamma)\sum_{j=1}^{N_1}\left(\frac{\Delta_{\max,j}-\Delta_y}{\Delta_{u,i}-\Delta_y}\right)^c+\gamma\sum_{i=1}^{N_h}\left(\frac{E_i}{E_{u,i}}\right)^c. \tag{7}$$

Based on the test results, the parameters will be discussed and fixed in the damage model in the next section.

5 Definition of parameters

5.1 Ultimate energy dissipation capacity of members

Figure 10 gives the relation curve of the normalized ultimate energy dissipation $E_{u,i}/E_{u,0}$ and the normalized cycle cumulative energy dissipation from the analysis of the test results. It can be observed clearly that this curve can be expressed by assuming the normalized ultimate dissipative capacity. $\overline{E}_{u,i}$ is an exponential decay function of α, and it can be expressed as

$$\overline{E}_{u,i}=A+Be^{-\alpha}, \tag{8}$$

$$\alpha=\frac{\sum_{i=1}^{N_h}E_i}{E_{u,0}}, \tag{9}$$

where $\sum E_i$ is the cumulative hysteretic energy after the i-th half-cycle, $E_{u,0}$ is the ultimate energy dissipation capacity under monotonic loading directly, $\overline{E}_{u,i}$ is the normalized ultimate energy dissipation capacity under a constant axial load and monotonic lateral load after i-th half-cycle, and α is the normalized cycle cumulative energy dissipation under i-th half-cycle.

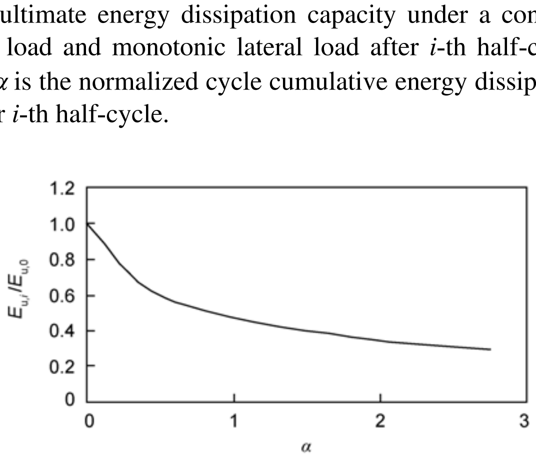

Figure 10 Relationship of normalized ultimate energy dissipation and normalized cycle accumulation energy dissipation.

5.2 Ultimate deformation capacity of members

Figure 11 gives the relation curve of normalized ultimate deformation and normalized cycle cumulative energy dissipation obtained by test results. It can be observed clearly that this curve can be expressed by assuming the normalized ultimate deformation capacity. $\overline{\Delta}_{u,i}$ is an exponential decay function of α, and it can be expressed as

$$\overline{\Delta}_{u,i}=A+Be^{-\alpha}, \tag{10}$$

where $\overline{\Delta}_{u,i}$ is normalized ultimate deformation capacity under a constant axial load and monotonic lateral load after the i-th half-cycle.

By regression analysis of the experimental data, A=0.46, B=0.54 for ultimate energy dissipation and A=0.76, B=0.24 for ultimate deformation are obtained respectively.

5.3 Maximum unelastic deformation $\Delta_{\max,j}$

The definition of maximum unelastic deformation is the maximum unelastic displacement amplitude experiencing firstly. Like the deformation damage component, the cycle accumulation damage component is also considered under positive half-cycles 1, 3, 5, 9, 11 and 17 (shown in Figure 12), while other positive half-cycles merely considered the cycle accumulation damage component (for example 7, 13, and

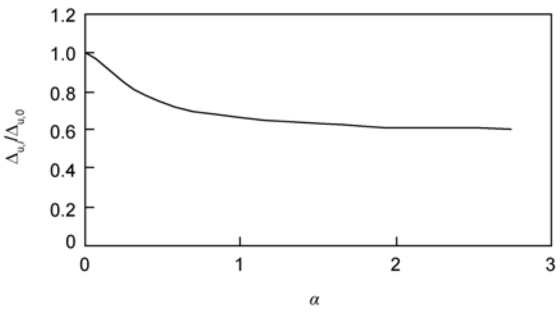

Figure 11 Relationship of normalized ultimate displacement with normalized cycle accumulation energy dissipation.

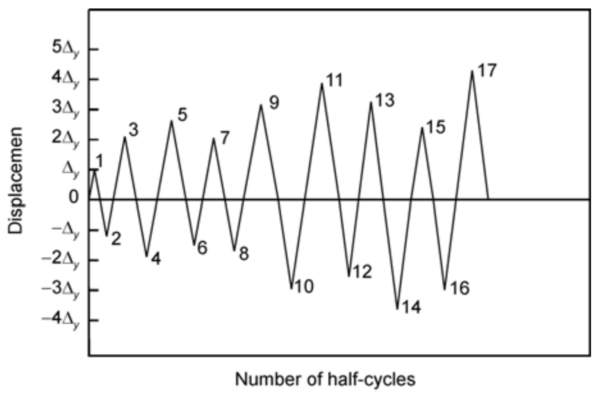

Figure 12 Definition of maximum unelastic deformation.

15). This can reflect the fact that the strength deterioration increases when the displacement amplitude grows gradually, which leads to more damage. Similar to the positive half-cycles, the opposite half-cycles have the same pattern.

5.4 Combination parameter γ and test parameter c

To satisfy the theory request, the combination parameter γ and test parameter c were introduced to make D equal to 1 at complete failure point of members in eq. (7).

For the same specimens, variable amplitude cycle loading has more obvious strength deterioration and smaller cumulative energy dissipation compared with constant amplitude cycle loading. So γ under variable amplitude cycle loading is more than that under constant amplitude cycle loading when D is equal to 1 obtained by eq. (7). Because the earthquake is a variable amplitude, the variable amplitude test results were used to fix γ and c. At same time, the values of γ and c are conservative for the constant amplitude cycle loading.

The damage model proposed by Kumar et al. [15] also adopted γ and c and gave a more reasonable determination method. By analyzing test results for possible sets of γ and c values, it was found that the value of γ is within the range of 0.1–0.2, while c varies between 1 and 2 [15]. Based on the test results obtained by this time and prophase [17–19], it is indicated that the scatter in c is minimum when γ is equal to 0.15 by larger calculation results. Thereby, in this paper, γ is equal to 0.15.

According to the test results obtained by Zheng et al. [17] and Zhang [20], the test parameters c of each frame column specimen with the various concrete strength grades, shear span rations λ, axial compression rations n, steel rations ρ_s and stirrups ratios ρ_v can be obtained by eq. (7), from which D is equal to 1 when it is destroyed completely. The specific design parameters of each specimen for regression analysis are shown in Table 4.

To obtain the test parameter c of each specimen, their ultimate deformation and ultimate energy dissipation should be fixed firstly under monotonic loading. The FE analysis method was used to obtain the ultimate deformation and ultimate energy dissipation of each specimen. The existing research shows that the effect of concrete strength on member performance is not obvious [21]. At the same time, the multiple regression analysis indicates that the main effect factors on test parameter c are shear span rations λ, axial compression rations n, steel rations ρ_s and stirrups ratios ρ_v, while the effect of concrete strength is not obvious. Then the test parameter c can be expressed as

$$c = 5.69 + 0.87\ln\rho_v + 0.056\lambda + 10.46\rho_s - 2.1n. \tag{11}$$

6 Experimental verification of the damage model

In order to verify the rationality of the damage model proposed in here, the damage indexes of partial specimens tested by the previous researchers were calculated. The results are shown in Table 5. It can be seen that the mean value of damage indexes is equal to 0.997; the standard deviation and the coefficient of variation are all equal to 0.0035. The results verified that the damage model can fully describe the seismic damage evolvement process and degree of SRHSC frame columns under low cycle reversed loading.

7 Conclusions

From the above investigation, the following conclusions can be drawn:

Table 4 Design parameters of specimens for regression

Specimen	Concrete strength grade	λ	n	ρ_s (%)	ρ_v (%)
PSRHSC-1	C80	3.0	0.4	5.6	1.38
PSRHSC-2	C80	2.5	0.4	5.6	1.38
PSRHSC-3	C80	2.0	0.4	5.6	1.38
PSRHSC-4	C80	1.5	0.4	5.6	1.38
PSRHSC-5	C60	3.0	0.4	5.6	1.38
PSRHSC-6	C100	3.0	0.4	5.6	1.38
PSRHSC-7	C120	3.0	0.4	5.6	1.38
SRHSC-6	C80	3.0	0.4	6.8	0.8
SRHSC-7	C80	3.0	0.2	6.8	0.8
SRHSC-8	C80	3.0	0.6	6.8	0.8
SRHSC-9	C80	3.0	0.4	4.6	0.8
SRHSC-10	C80	3.0	0.4	5.7	0.8
SRHSC-11	C80	3.0	0.4	6.8	1.1
SRHSC-12	C80	3.0	0.4	6.8	1.4

Table 5 Damage indexes of specimens at failure point

Specimen	Design parameters					Damage index
	Concrete strength grade	λ	n	ρ_s (%)	ρ_v (%)	
PSRC-8	C80	1.5	0.2	5.6	1.38	0.985
PSRC-9	C80	1.5	0.4	5.6	1.38	0.996
PSRC-10	C80	1.5	0.6	5.6	1.38	1.016
PSRC-11	C100	1.5	0.4	5.6	1.38	0.997
PSRC-12	C120	1.5	0.4	5.6	1.38	0.993
PSRC-13	C80	2.0	0.4	5.6	1.26	1.009
PSRC-14	C80	2.0	0.4	5.6	1.72	0.986
PSRC-15	C80	2.0	0.4	4.7	1.26	0.995

(i) With the increase of the number of the loading cycle and the displacement amplitude, the damage of columns is cumulative, which makes strength, stiffness, ultimate deformation and ultimate energy dissipation capacity degrade gradually. Compared with the variable amplitude loading, the damage evolution is slower and the gross of cumulative energy is relatively larger under the constant amplitude loading.

(ii) The damage model proposed in this paper is a nonlinear double parameters damage model, which can fully describe the effects of loading histories on damage and number of loading cycles on ultimate deformation and ultimate energy dissipation capacity for SRHSC frame columns and reflect the variation of their mechanical properties under a horizontal earthquake action.

(iii) In the damage model proposed, the main effect factors on test parameter c are shear span rations λ, axial compression rations n, steel rations ρ_s and stirrups ratios ρ_v, while the effect of concrete strength is not obvious. The relationship of test parameter c and design parameters are given by the regression equation, which can be used for the seismic design of SRHSC members.

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