Numerical analysis of hysteretic behavior for RAC structure under earthquake loading

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\textbf{ABSTRACT}

The hysteretic properties of RAC structure subjected to different strain rates are analyzed, and the displacement ductility is assessed. It is revealed that, under earthquake wave excitation with the increasing strain rate, the lateral stiffness, the bearing capacity, and the energy dissipation increases progressively. However, the ductility of RAC structure decreases gradually with the increasing strain rate. The rate-dependent capacity curve model of RAC structure is suggested. The damage evolution of RAC structure subjected to different strain rates is investigated. Furthermore, the dynamic damage model of RAC structure in which the strain rate effect is taken into account is proposed.

\textbf{1. Introduction}

Recycling building waste was first tried in Germany after the Second World War (Khalaf and Devenny 2004). Since then, a great deal of research has been carried out to develop RAC worldwide. The material property of recycled aggregate (Wang, Hsiao, and Wang 2012; Bravo et al. 2015), the mechanic performance of RAC under the static loadings (ACI Committee 555, 2002), and the constitutive relationship of stress-strain of RAC under the static loadings (Xiao, Li, and Zhang 2005; Suryawanshi, Singh, and Bhargava 2018) are experimentally studied and theoretically analyzed. It has been found that there is a slight difference in mechanical properties between recycled aggregate concrete (RAC) and natural aggregate concrete (NAC) because of the variability and randomness of waste concrete from disparate sources. However, for RAC it is still sufficient for practical applications in civil engineering. This is further confirmed by recent experimental studies and theoretical analysis of the mechanical performance and structural behavior of RAC (Xiao et al. 2012; Goksu et al. 2019). It may be noted that most of the present studies on RAC were carried out under static or quasi-static loadings, and there is little evidence of numerical model with the inclusion of strain rate effect finding a place in the seismic analysis of framed structures made of recycled aggregate concrete. Concrete is a typical rate-dependent material and the corresponding strength, stiffness, and ductility (or brittleness) are affected by loading strain rates. The strain rate at critical sections from the dynamic loads may be up to $10^{-1}/s$ for the reinforced concrete structure subjected to strong earthquake ground motion excitations (Bischoff and Perry 1991). The material properties and the structural behaviors at dynamic loading will be different from those at static loading (Zhu and Zhang 2013; Zhang and Zhao 2014). An investigation of the seismic behavior of reinforced concrete (RC) beam-column joints under dynamic loadings with strain rates of $10^{-3}/s$, $10^{-4}/s$, and $10^{-5}/s$ was carried out by Wang, Fan, and Song (2015). Experimental investigation of dynamic tensile and compressive tests by Xiao, Li, and Lin (2008) indicates that the tensile and compressive strengths of concrete increase with the increasing loading rate. A uniaxial compressive experimental investigation on the modeled RAC under different strain rate carried out by Xiao, Yuan, and Li (2014), demonstrates that the peak stress and elastic modulus increase with increasing strain rate. However, the distribution of the constitutive relationship curves under different strain rate is similar.

For the nonlinear analysis of reinforced concrete structures, a variety of models have been considered (Cardone, Perrone, and Sofia 2013; Kyriakides et al. 2015; Xu et al. 2018; Peng et al. 2018). These range from refined and complex local models to simplified global models. This study concentrates on the discrete finite element models that within limits provide satisfactory response predictions with moderate numerical effort compared to refined finite element models and more realistic and useful than the simplified models (Taucer, Spacone, and Filipou 1991; Zeris and Mahin 1991; Shao, Aval, and Mirmiran 2005).

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Some strain-rate-dependent material models for natural aggregate concrete (NAC) have been investigated in recent years (Zhang et al. 2018; Li, Li, and Li 2019). The constitutive model in the form of three branches for NAC confined by transverse hoops was derived from dynamic tests by Kent and Park (1971), and Scott, Park, and Priestley (1982). A multiplying factor of 1.25 was proposed to the peak stress, the peak strain, and the slope of the falling branch as well to account for the strain rate effect. A modified nonlinear concrete material model suggested by Tedesco et al. (1997) was employed in the ADINA finite-element computer programs by accounting for the high strain rate effects ranging from $10^{-7}$/s to $10^3$/s.

In this study, on the basis of the observed stress-strain behavior in these dynamic tests and the previous conclusions, a strain-rate-dependent material model for confined recycled aggregate concrete (CRAC) is developed by accounting for the strain rate effect, the hoop reinforcement confinement and recycled coarse aggregate (RCA) replacement ratio influences. A one-fourth scaled, two-bay, two-span and 6-storey RAC 3-dimensional spatial frame is modeled based on a general purpose nonlinear analysis program (Mazzoni et al. 2006) for the dynamic analysis of complete three-dimensional structural systems. The typical hysteretic behavior and damage assessment on the RAC frame structure subjected to different strain rates are analyzed, and some significant differences induced by the strain rate effect are discussed.

2. Strain-rate-dependent material model

2.1. Strain-rate-dependent model of concrete

The dynamic tests of recycled aggregate concrete confined by overlapping transverse hoops were carried out in an MTS 815 concrete testing system (Figure 1-3). The shape and trend of the compressive stress-strain curves of recycled aggregate concrete (RAC) in the dynamic and quasi-static loading conditions are in accord with those of natural aggregate concrete (NAC). A series of typically controlled longitudinal compressive strain rates of $10^{-5}$/s-$10^{-1}$/s were employed in these tests. The influences of the strain rate, the hoop reinforcement, and the recycled coarse aggregate (RCA) replacement ratio on the behavior of stress-strain of recycled aggregate concrete (RAC) were analyzed thoroughly.

The strain rate effect can be represented by the ratio of the dynamic response to the quasi-static response, known as the dynamic increase factor (DIF), which is used to describe the rate dependency of the mechanical indices for RAC. According to the previous research (Wang and Xiao 2017), the DIF empirical models are expressed as functions of strain rate. Through regression analysis of experimental data from the dynamic tests with the MATLAB program, DIF for the compressive peak stress and the compressive peak strain are developed by the authors to depict the enhancement of the compression strength and the critical strain of RAC as shown in the equations (1) through (3).

\[ \varepsilon = 10^{-5}/s \]

\[ \varepsilon = 10^{-5}/s \]

\[ \varepsilon = 10^{-5}/s \]

Figure 1. Experimental curves under different RCA replacement ratios.

Figure 2. Experimental curves for the test samples under various strain rates.
stand for the compressive peak stress and are the DIFs for the compressive peak strain. The CIF empirical models are introduced. An expression of RIF upon the compressive peak strain is suggested by the authors. The proposed RIF model derived from experimental data are established and expressed in Eq. (6). It should be noted that the changing trend of the compressive peak stress under various RCA replacement ratios is inconspicuous.

$$F_{R} = \kappa R + 1.0$$

where $F_{R}$ stands for the RIF of the compressive peak strain of RAC; $R$ is the replacement ratio of RCA (%); $\kappa$ denotes the RIF model parameter, which is equal to 0.1965 according to the regression analysis.

On the basis of the analysis of the rate-dependence of the mechanical behavior of RAC, the Kent-Scott-Park model (Kent and Park 1971; Scott, Park, and Priestley 1982) is adapted for RAC by applying the factors of DIF, CIF, and RIF to the compressive peak stress and the compressive peak strain. The material model is illustrated in Figure 4. The successive degradation of the stiffness of both reloading and the unloading curves is included because of the increasing values in compressive strain, the tension stiffness, and the hysteretic effects.
response under seismic conditions (Yassin and Hisham 1994). The compressive peak stress and the compressive peak strain in the material model are expressed in the following forms, respectively.

\[ f_{dc0} = k_{fc} \cdot f_{c0} \]  

\[ \varepsilon_{dc0} = k_{fc} \cdot \varepsilon_{c0} \]  

where, \( f_{dc0} \) and \( \varepsilon_{dc0} \) stand the compressive peak stress and the corresponding strain of confined recycled aggregate concrete (CRAC) under the dynamic loading conditions.

According to the results by earlier researchers (Malvar and Ross 1998; Xiao, Li, and Lin 2008), DIF of the tensile peak strain, ie, the ratio of the tensile peak strain at dynamic loading rate to that at quasi-static loading rate is assumed to be equal to 1.0 in the present work.

2.2. Strain-rate-dependent model of steel

Based on experimental studies and numerical analyses, some empirical formulae of DIF of reinforcing steel are developed. DIF formula for reinforcing steel derived by CEB (1993) is adopted and recalled here. The DIF formula proposed by CEB (1993) for reinforcing steel is valid for strain rates at a constant range of approximately \( 5 \times 10^{-5}/s < \varepsilon_s < 10/s \), which is given as follows

\[ k_{sy} = \frac{f_{yd}}{f_y} = 1.0 + \left( \frac{6.0}{f_y} \right) \ln \left( \frac{\dot{\varepsilon}_s}{\dot{\varepsilon}_{s0}} \right) \]  

\[ k_{su} = \frac{f_{ud}}{f_u} = 1.0 + \left( \frac{6.0}{f_u} \right) \ln \left( \frac{\dot{\varepsilon}_s}{\dot{\varepsilon}_{s0}} \right) \]  

where \( k_{sy} \) and \( k_{su} \) are the DIF of the yield and ultimate strength of reinforcing steel, respectively, \( f_y \) and \( f_u \) represent the quasi-static and dynamic yield strength of steel in MPa, respectively, \( \dot{\varepsilon}_s \) and \( \dot{\varepsilon}_{s0} \) denote the quasi-static and dynamic ultimate strength of steel in MPa, respectively, \( \dot{\varepsilon}_{s0} \) means the quasi-static strain rate of steel.

The proposed material model (Hognestad 1951; Filippou, D’Ambri, and Issa 1992) for the steel bar is shown in Figure 5, and the effect of the strain rate has been taken into account in this hysteretic model by applying the dynamic increase factors \( k_{sy} \) and \( k_{su} \) which are derived by CEB (1993). Thus, the stress-strain relation of the hysteretic model is modified by considering the dynamic increase factor \( k_{sy} \) in the yield strengths \( (s_{1p}, s_{1n}) \) and the critical strains at the yield strengths \( (e_{1p}, e_{1n}) \), as well as \( k_{su} \) in the ultimate strengths \( (s_{3p}, s_{3n}) \). The other characteristic parameters are assumed to be unchanged.

3. Numerical modeling strategy

3.1. Finite Element Model (FEM) of RAC frame structure

The tested model is a two-bay and two-span and six-storey frame structure regular in plan and elevation, and is designed according to Chinese building standard (GB 50011, 2016). The RAC frame model is \( 2175 \times 2550 \) mm in plan and has a constant storey height of 750 mm. The thickness of the slab of the frame model is 30 mm. The details of the general geometry are shown in Figure 6(a and b). Ordinary Portland cement with a 28 days nominal compressive strength grade of 42.5 MPa is used. The fine aggregates are river sand with nominal particle diameter not greater than 5 mm. The coarse aggregates used are RCAs with a particle diameter of between 5 mm and 10 mm. The recycled aggregate concrete mixture of nominal strength grade C30, and with slump value

![Figure 4. Modified Kent-Scott-Park model.](image-url)
in the range of 180–200 mm, is proportioned with the recycled coarse aggregate replacement ratio equal to 100% (ie, the ratio of the recycled coarse aggregate mass to the mass of all the coarse aggregate). An overview of the model after installation on the shaking table facility and the experimental set-up is shown in Figure 6(c).

The RAC frame specimen is idealized as a three-dimensional discrete numerical model (shown in Figure 7), and the frame beam and column members are modeled with the flexibility-based distributed-plasticity nonlinear fiber beam-column elements. For the element, it is subdivided into several control sections and each section is composed of a number of fibers. In the numerical implementation of the RAC numerical model, each nonlinear fiber beam-column element is subdivided into five integration points for the numerical simulation performing a good agreement with the experimental results. Each fiber is characterized by its area, material type, and position with respect to the section reference system. The material behavior of the element depends entirely on the fiber stress-strain relation of the fibers, which follow the unconfined and confined concrete as well as reinforcing steel models involving the strain rate effect mentioned in the above Section 2. Different concrete and steel material types can be specified for the fibers by varying the values of material parameters. Figure 8 (a-d) shows the details of the section modeling for the beam and column members, respectively. The method of modeling of the section was also used by Xu et al. (2018) in the seismic fragility analysis, and good simulation results were obtained.

### 3.2. Dynamic inputs

According to Code for Seismic Design of Buildings (GB 50011, 2016), Wenchuan earthquake wave (WCW, 2008, N-S) belonging to Type-II site soil was chosen.
Considering the spectral density properties of Type-II site soil, El Centro earthquake wave (ELW, 1940, N-S), Shanghai artificial wave (SHW) were also selected. Except that SHW is a 1-D wave, WCW and ELW are 2-D waves. The tests were carried out along with the numerical simulation with the main excitation in direction X of the RAC frame structure model. During earthquake motions with various intensities, the typical responses of the structure are influenced more significantly for excitation SHW than for others. Therefore, the structural responses under SHW excitation are mainly analyzed and discussed. Correspondingly, the time history, the standard acceleration response spectrum and FFT of SHW are illustrated as follows in Figure 9.

The gradual increasing amplitudes of base excitation were input successively in a manner of time-scaled earthquake waves with 0.00736 s intervals. Before and after each dynamic response time history analysis, modal analyses were performed to capture the dynamic characteristic parameters of the numerical model, including natural frequency and equivalent lateral stiffness. In order to consider the influence of the cumulative damage, in the numerical modeling, the earthquake waves are input in series. For
diminishing the influence of the free vibration after the ground motion excitation, a zero acceleration time history is put between the current loading case and the next loading case to assure that the initial velocity of the model structure under the next loading case is zero. The residual stress and deformation of the model structure at the current loading case are designated as the initial condition of the structure at the next loading case. Thus the influence of the cumulative damage the structure induced by different earthquake excitations is included using the dynamic input numerical simulation method. Take the excitation ELW in the 25th loading case (LC25) of the 1.170g test phase, for example, the earthquake ground acceleration time histories with and without cumulative damage influence are shown in Figure 10 as follows.

4. Dynamic nonlinear analysis

The typical responses of the structure which is subjected to WCW, ELW, and SHW excitations and modeled using the material model proposed in the above sections 2.1 and 2.2 are presented and analyzed in this section. Four different analytical models were developed and analyzed with the same modeling features, analytical procedure, solution algorithm, convergence criterion, and dynamic loading input method for the motivation to examine how the seismic behaviors of the RAC frame structure vary under earthquake excitation with different strain rates included. The investigators (Bischoff and Perry 1991) found that there no clear increase in strength up to a strain rate of about $5 \times 10^{-5}$/s. Here, the numerical model NM-1 with strain rate of $3.04 \times 10^{-5}$/s obtained from the shaking table tests (Wang and Xiao 2012) is
used as the reference model in this analysis, and the corresponding seismic responses are defined as the quasi-static responses and used as the reference values of the structural responses to compare with the numerical models (NM-2∼NM-4) with strain rate of $3.04 \times 10^{-3}/s$, $3.04 \times 10^{-2}/s$, $3.04 \times 10^{-1}/s$, respectively. The boundary values of strain rate for the material models are given to emphasize mainly that the indicative strain rates of reinforced recycled aggregate concrete during the response under earthquakes just fall within the prescribed range.

The initial analysis and comparison between the experimental and numerical results were provided elsewhere by the authors (Wang, Xiao, and Sun 2016), in which the accuracy of the numerical models suggested by the authors was checked and verified thoroughly. Taking the structural deformation as an example, the lateral storey displacement is illustrated in Figure 11. It is indicated that the numerical and experimental results agree well with each other, and the error between them is less than 7% in a high nonlinear phase.

Based on the previous conclusions (Wang, Xiao, and Sun 2016), in this paper, the hysteretic behaviors (i.e., the hysteresis loops, the stiffness degradation, the capacity curve model, and the energy dissipation-based damage model) of recycled aggregate concrete (RAC) frame structure suffered the strain rate effect have been investigated intensively.

### 4.1. Hysteresis loops

In this section, the Hysteresis loops of the RAC frame structure subjected to ground motion excitation and modeled using the numerical models proposed with different strain rates included are analyzed and plotted in Figure 12. By comparing and analyzing the hysteretic characteristics of the RAC frame structure with strain rate effect included, there are some significant conclusions are obtained and presented as follows.

First, for the four numerical models, the inclusion of strain rate makes little difference to the curve distribution of the base shear vs. roof displacement for the RAC structure. Before initial cracking of the RAC frame model, the force-displacement relation curves of the structure are of straight line basically. With the development of concrete cracking and non-elastic deformation of the structure progressively, the lateral stiffness, the loading, and the energy dissipation capacity of the structure degrade gradually, and the pinch effect of the hysteresis loops are more obvious.

Second, based on the Hysteresis loops the overall lateral stiffness of the RAC frame structure is computed and plotted in Figure 12, and the computation values are listed in Table 1. From Figure 12 and Table 1, it is indicated that for the four numerical models (NM-1∼NM-4) the corresponding overall structural lateral stiffness degrades gradually with the increasing acceleration amplitude of ground motion. In addition, under the same earthquake intensity, the overall lateral stiffness increases with the augmenting strain rate amplitude for the RAC frame structure.

In general, based on the hysteretic characteristic analysis, it is revealed that under earthquake wave excitation with the increasing strain rate amplitude, the lateral stiffness, the bearing capacity and the energy dissipation of the RAC frame structure increases progressively.

### 4.2. Capacity curves

Based on the Hysteresis loops illustrated in Figure 12, the peak base shear force vs. the corresponding roof

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**Figure 11.** Comparison between the calculated and experimental displacement curves.
displacement in a series of earthquake time history excitations are plotted in Figure 13. Based on the information presented in Figure 13, the fitting curves for the base shear vs. the top floor displacement of RAC frame structure subjected to earthquake excitations with strain rate effect included are illustrated in Figure 13. The corresponding rational functional formulas derived from the fitting curves are expressed as follows:

\[ S_i(\Delta) = \frac{a_4 \Delta^2 + b_4 \Delta + c_4}{\Delta + d_3} + q_i \]  

(11)

Here, \( \Delta \) as the independent variable of the function is defined at the interval [0 90] with mm as the fundamental unit. \( S_i \) \((i = 1, 2, 3, 4) \) denotes the base shear for numerical models of NM-1 \((3.04 \times 10^{-5}/s)\), NM-2 \((3.04 \times 10^{-3}/s)\), NM-3 \((3.04 \times 10^{-2}/s)\) and NM-4 \((3.04 \times 10^{-1}/s)\) respectively. \( a_4, b_4, c_4, d_3, c_3, \) and \( q_i \) represent the function model parameters, \((i = 1, 2, 3, 4)\). The related information of the fitting function model for each numerical model is presented in Table 2.

It should be noted that the relationship between the base shear and the roof displacement is expressed under the test phases with PGAs from 0.066 \(g\) to 1.170 \(g\). From the load-bearing capacity fitting curve, the cracking loading point, the yield loading point, the maximum loading point as well as the ultimate loading point can be easily recognized. Generally, the capacity curve can reflect the variation of the lateral loading capacity of the structure, and the slope of the curve represents the overall lateral stiffness of the structure. The initial cracking point, the yield point, the maximum load point, and the ultimate load point are the key points of the capacity curves as shown in Figure 13. Likewise, the corresponding loads and displacements of the characteristic points are expressed as the cracking load \( P_c \) and the cracking displacement \( \Delta_c \); the yield load \( P_y \) and the yield displacement \( \Delta_y \), the maximum load \( P_m \) and the corresponding displacement \( \Delta_m \); the ultimate load \( P_u \) and the ultimate displacement \( \Delta_u \). The characteristic values of the capacity curves for four numerical models are presented in Table 3. It is

Figure 12. Hysteresis loops under dynamic loading with strain rate effect included.

| Test phase | \( K_4 \) (kN/mm) | \( K_3 \) (kN/mm) | \( K_2 \) (kN/mm) | \( K_1 \) (kN/mm) |
|------------|------------------|------------------|------------------|------------------|
| 0.185g     | 3.296            | 3.182            | 3.071            | 2.754            |
| 0.415g     | 1.909            | 1.853            | 1.802            | 1.713            |
| 0.550g     | 1.502            | 1.436            | 1.361            | 1.225            |
| 0.750g     | 1.0310           | 0.9901           | 0.9530           | 0.8796           |

Table 1. Overall lateral stiffness of RAC frame structure with strain rate included.
indicated that the bearing capacity of the RAC frame structure increases when the strain rate increases. The displacement ductility of the RAC frame structure with strain rate included is analyzed and assessed. The average roof displacement ductility coefficients are 3.53, 3.51, 3.38 and 3.16 for NM-1, NM-2, NM-3, and NM-4, respectively. The analysis shows that the ductility of RAC frame structure decreases gradually with the augmenting strain rate amplitude.

4.3. Accumulated damage

A dynamic analysis is performed to quantify the accumulated damage of the RAC frame structure subjected to earthquake loadings with different strain rates included. Based on the energy dissipation and damage mechanics theory, the accumulated damage is calculated with the following formulas.

\[
D = 1 - \frac{W_e + W_p}{W} = 1 - \frac{\int_{-\Delta_0}^{\Delta_0} f_1(\Delta) d\Delta + \int_{-\Delta_1}^{\Delta_1} f_2(-\Delta) d\Delta}{K_0\Delta_1^2}
\]

(12)

\[
W = W_e + W_p + W_D
\]

(13)

where \(D\) denotes the damage index; \(W\) represents work done by external force under ideal condition; \(W_e, W_p\) and \(W_D\) stand for structural elastic deformation energy, plastic deformation energy, and damage dissipation energy respectively; \(\Delta_0\) and -\(\Delta_0\) stand for the corresponding deformation at peak load points in positive and negative loading, respectively; \(\Delta_1\) represents the residual deformation in negative loading under cycle \(i-1\); \(\Delta_1\) represents the residual deformation in positive loading under cycle \(i\); \(K_0\) is the initial lateral stiffness of the structure before subjected to damage; \(f_1(\Delta)\) and \(f_2(-\Delta)\) denote force-displacement functions under positive and negative loading respectively.

Based on the hysteresis loops obtained in the above section, the damage evolution of the RAC frame structure subjected to earthquake excitation SHW with different strain rates included is analyzed and calculated as shown in Table 4. It can be concluded that the damage degree of the RAC frame structure decreases with the augmenting strain rate amplitude. Taking 0.415 g test phase for an example, the damage index is 0.447 for model NM-1 with the strain rate of \(3.04 \times 10^{-5}\) /s. However, compared with

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**Table 2. Parameters of the equations for fitting function models of capacity curves.**

| Parameter | \(a_i\) | \(b_i\) | \(c_i\) | \(d_i\) | \(q_i\) | Curve equations |
|-----------|--------|--------|--------|--------|--------|----------------|
| \(-1.142\) | 197.1  | 24.77  | 43.81  | -0.57  | \(S_1\) |
| \(-1.121\) | 199.7  | 31.05  | 45.23  | -0.72  | \(S_2\) |
| \(-1.328\) | 224.8  | 37.46  | 49.37  | -0.76  | \(S_3\) |
| \(-1.620\) | 259.9  | 58.49  | 59.04  | -0.99  | \(S_4\) |

**Table 3. Characteristic parameters of the capacity curve of RAC frame structure.**

| Parameters | NM-4 | NM-3 | NM-2 | NM-1 |
|------------|------|------|------|------|
| \(P_c\) (kN) | 36.898 | 39.159 | 38.865 | 39.388 |
| \(P_y\) (kN) | 68.537 | 66.017 | 63.331 | 61.437 |
| \(P_m\) (kN) | 81.899 | 79.578 | 76.894 | 74.462 |
| \(P_{u}\) (kN) | 69.614 | 67.641 | 66.263 | 63.293 |
| \(\Delta_c\) (mm) | 10.638 | 11.389 | 11.404 | 12.033 |
| \(\Delta_y\) (mm) | 27.860 | 26.548 | 25.597 | 25.359 |
| \(\Delta_m\) (mm) | 54.600 | 54.400 | 54.500 | 53.400 |
| \(\mu\) | 88.028 | 89.754 | 90.000 | 89.564 |

**Table 4.**
Table 4. Damage index of the RAC frame structure with strain rate effect included.

| PGA  | Damage index | Damage index | Damage index | Damage index |
|------|--------------|--------------|--------------|--------------|
| 0.066g | 0.0 | 0.0 | 0.0 | 0.0 |
| 0.130g | 0.276 | 0.222 | 0.199 | 0.159 |
| 0.185g | 0.406 | 0.351 | 0.288 | 0.202 |
| 0.370g | 0.432 | 0.378 | 0.376 | 0.376 |
| 0.415g | 0.447 | 0.425 | 0.412 | 0.403 |
| 0.550g | 0.472 | 0.453 | 0.445 | 0.413 |
| 0.750g | 0.601 | 0.567 | 0.545 | 0.529 |

Based on the information presented in Table 4, the fitting curves are plotted in Figure 14. The corresponding fitting exponential function model derived from the information shown in Figure 14 for each fitting curve is described and expressed as follows:

\[
D = (PGA) = \eta(\dot{e}) (0.3038e^{0.9094PGA} - 1.648e^{-24.71PGA})
\]

Here, \(PGA\) as the independent variable of the function is defined at the interval \([0.066g, 0.750g]\) with \(g (9.8N/kg)\) as the basic unit. \(D\) indicates the overall dynamic damage index. \(\eta\) stands for the dynamic influence factor of the structural damage.

The inclusion of strain rate makes little difference to the damage index curve trend of the RAC frame structure under different earthquake excitations. However, for the model, the amplitude quantities for different loading cases by varying strain rates apparently suffer a significant influence. Analyzing the damage index vs. PGA curves, it can be concluded that the overall damage index increases sharply with the increasing input acceleration amplitudes at the test phases with the PGAs from 0.130\(g\) to 0.185\(g\), however, increases flatly after the PGA of 0.415\(g\). The structure suffers more severe damage for the case of strain rate equaling to 3.04\(×10^{-5}/s\) than for others. Moreover, the overall damage indexes vary significantly in the trend of decreasing with the increasing strain rate of RAC. The dynamic damage model proposed provides important technical support for seismic design and engineering application of RAC structures, and helpful for the comprehensive understanding of damage evolution mechanism of RAC frame structure under dynamic loading with strain rate effect included.

In general, under different strain rates, the damage developing for the RAC frame structure is also not the same. With the increasing strain rate, the lateral
stiffness and the load-bearing capacity increases, on the other hand,  the inter-storey drift, the displacement ductility and the accumulative damage of the RAC frame structure decrease.

5. Summary and conclusions

Based on both RAC material model and reinforcement material model proposed with strain rate effect included, adopting a finite element and numerical simulation method, the dynamic nonlinear behaviors of the RAC frame structure are analyzed and discussed. Some main conclusions are obtained and given as follows:

1. The shape and trend of the compressive stress-strain curves of RAC in the dynamic and quasi-static loading conditions are in accord with those of NAC. That is, the experimental curves exhibit well-defined peak loads, as well as smoothly and continuously descending branches after reaching the peak load.

2. The DIF models for the compressive peak stress and the compressive peak strain are developed as a function of strain rate to depict the changing laws of the compressive strength and the critical strain of RAC. The CIF models for the compressive peak stress and the corresponding strain are proposed. The volume ratio of the stirrups, the yield strength of the stirrups, the type of the stirrups, the stirrup spacing, and the compressive strength of RAC are considered in the relationships described in the CIF models. Through regression analysis of experimental data from the dynamic tests, the RIF model of the compressive peak strain is suggested to describe the effect of the RCA replacement ratio on the mechanical behaviors of RAC.

3. The Hysteresis loops of the RAC frame structure subjected to different strain rates are analyzed. Based on the hysteretic characteristic analysis, it is revealed that, under earthquake wave excitation with the increasing strain rate, the lateral stiffness, the bearing capacity, and the energy dissipation increases progressively.

4. The rate-dependent capacity curve model of RAC frame structure is suggested and express in Eq. (11), and the displacement ductility is assessed. The average roof displacement ductility coefficients for strain rates of $3.04 \times 10^{-5}/s$, $3.04 \times 10^{-3}/s$, $3.04 \times 10^{-2}/s$, $3.04 \times 10^{-1}/s$ are 3.53, 3.51, 3.38 and 3.16, respectively. The ductility of RAC frame structure decreases gradually with the increasing strain rate.

5. The damage evolution of RAC frame structure subjected to different strain rates is investigated. The overall damage indexes vary significantly in the trend of decreasing with the increase of the strain rate. Furthermore, the dynamic damage model of RAC frame structure is proposed and expressed in Eq. (12). The dynamic damage model proposed provides important technical support for seismic design and engineering application of RAC structures, and helpful for the comprehensive understanding of damage evolution mechanism of RAC frame structure under dynamic loading with strain rate effect included.

Disclosure statement

No potential conflict of interest was reported by the authors.

Funding

This work was supported by the China Postdoctoral Science Foundation [2014MS50247, 2015T80449]; National Natural Science Foundation of China [51608383]; Key Projects of Science & Technology Pillar Program of Henan Province [152102310027].

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