RELIABILITY ANALYSIS ON BEARING CAPACITY OF FRAME STRUCTURES WITH RECYCLED AGGREGATE CONCRETE

Jianzhuang Xiao [1, 2*], Kaijian Zhang [1], Tao Ding [1], Hongda Zhang [1]
[1] *Department of Building Engineering, Tongji University, Shanghai, P.R. China.*
[2] *State Key Lab. for Disaster Reduction in Civil Eng., Tongji University, Shanghai, P.R. China.*

ABSTRACT

The reliability of bearing capacity for recycled aggregate concrete (RAC) frame structure is theoretically analyzed in this paper. This investigation concludes that the failure pattern and failure mode of RAC frame structure is almost the same as that of natural aggregate concrete (NAC) frame structure. However, due to the increase of compressive strength discreteness of RAC, which results in the increase of resistance discreteness of RAC columns and beams, the bearing capacity reliability of RAC frame structures decreases. The results show that the reliability index of RAC frame is slightly lower than that of NAC frame and it is feasible to apply RAC in practical engineering.

KEYWORDS

Recycled aggregate concrete (RAC), natural aggregate concrete (NAC), frame structure, bearing capacity, reliability analysis

INTRODUCTION

As a green material, recycled aggregate concrete (RAC) has been applied in practical engineering widely, many researchers have undertaken a lot of work about RAC material and components. The compressive, tensile and shear strengths of RAC are generally lower than those of natural aggregate concrete (NAC) and the modulus of elasticity of RAC generally reduces as the replacement ratio of recycled coarse aggregate (RCA) increases (Xiao *et al.* 2012). Experiments indicate that there is no obvious difference between NAC and RAC beams while the deformation and width of cracks of RAC are larger compared with those of NAC beams (Ishill 1998). Shear capacity of RAC beam is lower than that of NAC beam and decreases with the replacement ratio of RCAs increase (Han *et al.* 2001, Xiao and Lan 2004). The cracking moment and ultimate moment of RAC beams are the same as NAC beams and stirrup spacing has large influence on the development of crack and shear capacity (Belén and Fernando 2006). In pre-stressed RAC beams, the deformations of beams increase with the replacement ratio of RCA (Dolara *et al.* 1998). In the case of RAC columns, RAC filled steel tube columns have the same failure mode and the stiffness degradation compared with NAC filled steel tube columns (Yang and Han 2006). As the replacement percentage of RCAs increases, the maximum axial load capacity decreases by approximately 6–8% compared to columns with natural coarse aggregate (Choia and Yun 2012). Energy-dissipating ability of beam-column joints is almost the same between RAC and NAC frame joints (Corinaldesi and

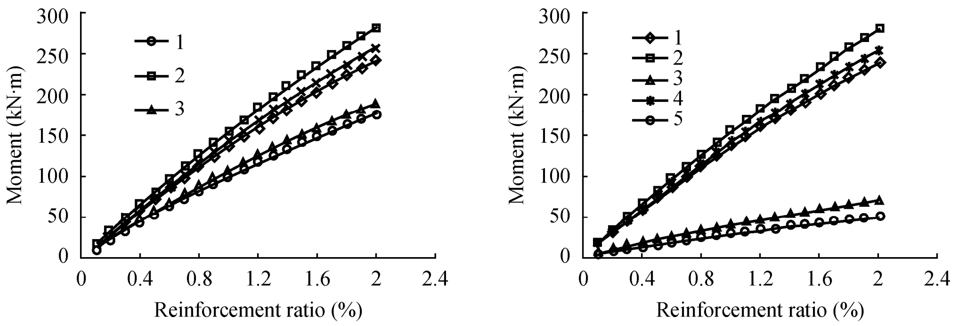
* Corresponding author: Tel: (+86)021- 65982787, Email: jzx@tongji.edu.cn

Moriconi 2003). Furthermore, the seismic performance decreases with the replacement ratio of RCAs increase (Xiao and Zhu 2004). The experimental results showed that the behavior under cyclic loading of the concrete made with 30% replacement of recycled coarse aggregates is quite similar to ordinary concrete (Letelier and Moriconi 2014). The general seismic behavior of a frame structure declines with an increase of the RCAs replacement percentage, however, the frame structure with a higher ratio of RCA still behaves well enough to resist an earthquake attack (Xiao, *et al.* 2006). It is feasible to apply and popularize RAC frame structures less than six stories high in seismic regions (Xiao *et al.* 2012).

However, there are few studies about RAC structural reliability in literature. The discrete degree of RAC structure will be increased due to its different sources, aging and crushing process of RCAs. More attention should be paid to the reliability of RAC structures. In this paper, the structural reliability of RAC frame is theoretically analyzed.

STATISTICAL ANALYSES ON RESISTANCE AND LOAD EFFECT OF FRAME BEAMS AND COLUMNS

Resistance of beams with size of 250mm×500mm and columns with size of 400mm×400mm are statistical analyzed while the concrete strength grade is C30 according to Chinese code (Zhang 2007). Under 100% permanent load and 100% persistent live load, the variation of design capacity $M_{S,d}$, resistance M_R and load effect M_S for both NAC and RAC beams with different reinforcement ratios are analyzed and compared, which can be seen in Figure 1 and Figure 2. The same analysis has also been undertaken to both NAC and RAC columns, which can be seen in Figure 3 and Figure 4. In order to investigate the dispersion degree of resistance M_R and load effect M_S, the average of resistance μ_{M_R}, the average of load effect μ_{M_S} and the value of average minus the standard deviation $\mu_{M_R}-\sigma_{M_R}$ and $\mu_{M_S}-\sigma_{M_S}$ are shown in Figure 1~Figure 4.

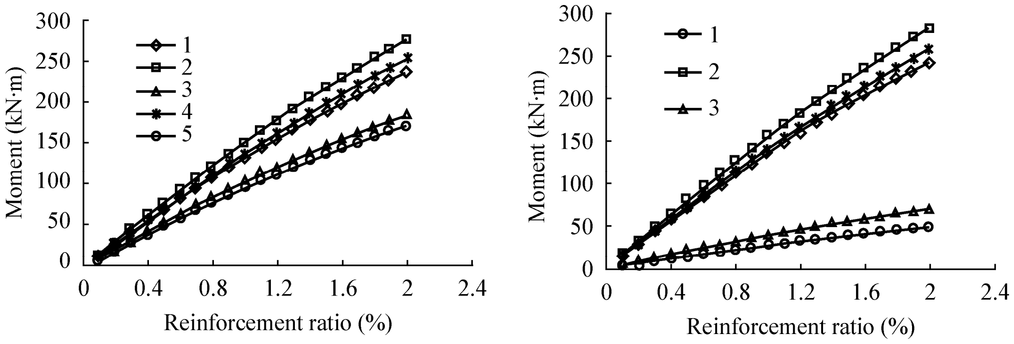


(a) 100% permanent load

(b) 100% persistent live load

Figure 1. Resistance and load effect of NAC beams

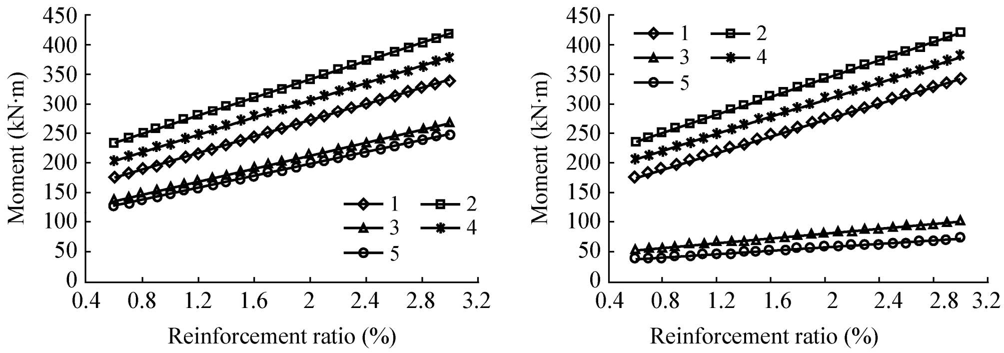
(1- $M_{S,d}$, 2- μ_{M_R} , 3- μ_{M_S} , 4- μ_{M_R} - σ_{M_R} , 5- μ_{M_S} - σ_{M_S} , the same hereafter)



(a) 100% permanent load

(b) 100% persistent live load

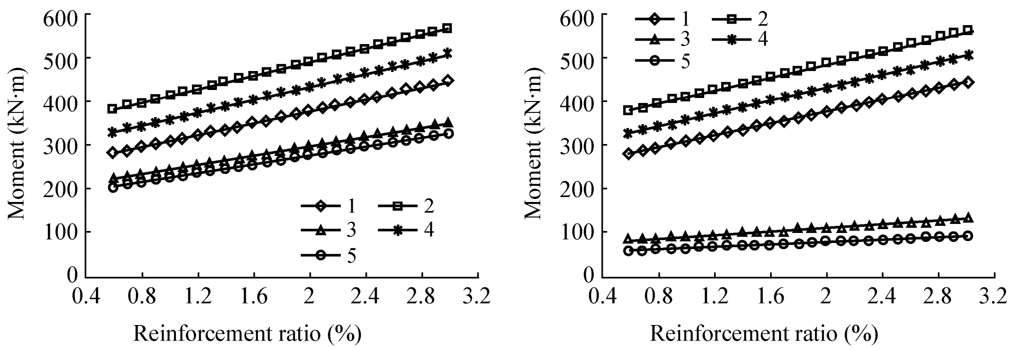
Figure 2. Resistance and load effect of RAC beams



(a) 100% permanent load

(b) 100% persistent live load

ξ=0.2



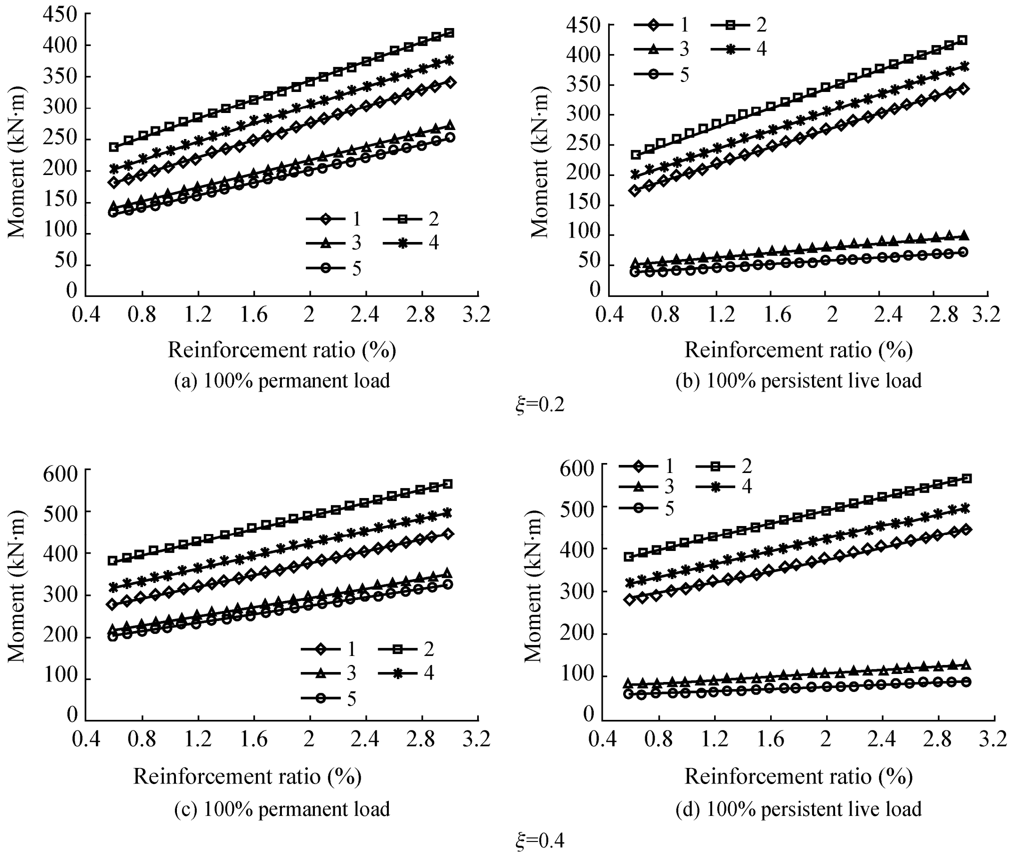
(c) 100% permanent load

(d) 100% persistent live load

ξ=0.4

Figure 3. Resistance and load effect of NAC columns

(ξ , the relative height of compression zone of concrete, the same hereafter)



(a) 100% permanent load (b) 100% persistent live load

ξ=0.2

(c) 100% permanent load (d) 100% persistent live load

ξ=0.4

Figure 4. Resistance and load effect of RAC columns

From Figure 1 and Figure 2, it can be concluded that resistances and load effects have no obvious difference between NAC and RAC beams no matter under 100% permanent load or 100% persistent live load. Compared with 100% permanent load, the average of load effect decrease, whereas both the standard deviation and variation increase under the condition of 100% persistent live load, irrespective of NAC beams or RAC beams. Figure 3 and Figure 4 show that columns have the same tendency on statistical parameters of load effect as that of beams. This paper also studied the capacity trend of columns with different relative heights of compression zone and it shows that with the increase of relative height of compression zone, the bearing capacity of columns increases. Only the results with the relative height of compression zone being 0.2 and 0.4 are shown in Figure 3 and Figure 4.

STATISTICAL PARAMETERS AND CHECKOUT ON RESISTANCE OF BEAMS AND COLUMNS

Based on previous experience and investigation of structures (Zhang 2007), this paper calculated the statistical parameters of resistance of beams with different reinforcement ratios and columns with different reinforcement ratios (the relative height of compression zone ξ =0.2). The statistical parameters of resistance can be obtained from Figure1~Figure 4. For NAC components, the variation of bending moment is 0.10 (Yi *et al.* 2006). For RAC components, the variation of bending moment is larger than that of NAC components taking account of the increase of compressive strength discreteness of RAC. The results are summarized in Table 1 and Table 2. The resistance of components is normal distribution (Zhang 2007, Zhao *et al.* 2000).

Table 1. Resistance statistics of NAC and RAC beams

Component category	Reinforcement ratio (%)	Distribution pattern	μ(kN·m)	δ
NAC beams	1.0	Normal distribution	155.1	0.09
RAC beams	1.0	Normal distribution	155.0	⩾0. 09
NAC beams	1.2	Normal distribution	182.7	0. 09
RAC beams	1.2	Normal distribution	182.4	⩾0. 09
NAC beams	1.5	Normal distribution	221.8	0. 09
RAC beams	1.5	Normal distribution	221.5	⩾0. 09
NAC beams	1.8	Normal distribution	258.4	0. 09
RAC beams	1.8	Normal distribution	257.9	⩾0. 09

Table 2. Resistance statistics of NAC and RAC columns

Component category	Reinforcement ratio (%)	Distribution pattern	μ(kN·m)	δ
NAC column	0.6	Normal distribution	234.1	0.10
RAC column	0.6	Normal distribution	234.0	⩾0.10
NAC column	0.8	Normal distribution	249.5	0.10
RAC column	0.8	Normal distribution	249.5	⩾0.10
NAC column	1.5	Normal distribution	303.5	0.10
RAC column	1.5	Normal distribution	303.5	⩾0.10
NAC column	1.8	Normal distribution	326.6	0.10
RAC column	1.8	Normal distribution	326.6	⩾0.10

(The relative height of compression zone ξ =0.2)

CALCULATION METHODS OF STRUCTURAL RELIABILITY

Because of the large number of components and complex details, it is difficult to calculate the structural reliability accurately. As for ductile structures, Stevenson and Moses proposed a method by using the plastic hinge mechanism in 1970 (Stevenson and Moses 1970). Nine years later in 1979, Gorman and Moses simplified the method and concluded that structure with the smallest value of performance function will fail firstly. Some methods for calculating structural reliability are described in the following.

The Correlation of Performance Function

In fact, structures and components as well as the random variable acting on them are not independent but interrelated. Assuming there are only two random variables R and S, the average and standard deviation are μ_R, μ_S and σ_R, σ_S , then performance function can be obtained:

$$\begin{aligned} Z_i &= a_i R - b_i S \\ Z_j &= a_j R - b_j S \end{aligned} \tag{1}$$

Where, a_i, a_j, b_i and b_j are coefficients of random variables R and S. According to the theory of probability and statistics, covariance of Z_i and Z_j is:

$$\mathrm{Cov}(Z_i Z_j) = E(Z_i Z_j) - E(Z_i)E(Z_j) \tag{2}$$

Where:

$$
\begin{aligned}
E(Z_iZ_j) &= E(a_ia_jR^2 + b_ib_jS^2 - a_ib_jRS - b_ia_jRS) \\
&= a_ia_j(\sigma_R^2 + \mu_R^2) + b_ib_j(\sigma_S^2 + \mu_S^2) - a_ib_j\mu_R\mu_S - b_ia_j\mu_R\mu_S \\
E(Z_i)E(Z_j) &= (a_i\mu_R - b_i\mu_S)(a_j\mu_R - b_j\mu_S) \\
&= a_ia_j\mu_R^2 + b_ib_j\mu_S^2 - a_ib_j\mu_R\mu_S - b_ia_j\mu_R\mu_S
\end{aligned}
\tag{3}
$$

Thus it can be obtained:

$$
\mathrm{Cov}(Z_iZ_j) = a_ia_j\sigma_R^2 + b_ib_j\sigma_S^2 \tag{4}
$$

The correlation coefficient is:

$$
\rho_{Z_iZ_j} = \frac{\mathrm{Cov}(Z_iZ_j)}{\sigma_{Z_i}\sigma_{Z_j}} = \frac{a_ia_j\sigma_R^2 + b_ib_j\sigma_S^2}{\sigma_{Z_i}\sigma_{Z_j}} \tag{5}
$$

When the random variables of the performance function are more than two (supposing the number of random variables *R* and *S* are *m* and *n*), then:

$$
\begin{aligned}
Z_i &= \sum_{p=1}^{m} a_{ip}R_p - \sum_{k=1}^{n} b_{ik}S_k \\
Z_j &= \sum_{p=1}^{m} a_{jp}R_p - \sum_{k=1}^{n} b_{jk}S_k
\end{aligned}
\tag{6}
$$

Where, a_{ip} , a_{jp} , b_{ik} and b_{jk} are coefficients of random variables R_p and S_k . Then the correlation coefficient is：

$$
\rho_{Z_iZ_j} = \frac{\sum_{l\in Z_iZ_j} a_{il}a_{jl}\sigma_{R_l}^2 + \sum_{f\in Z_iZ_j} b_{if}b_{jf}\sigma_{S_f}^2}{\sigma_{Z_i}\sigma_{Z_j}} \tag{7}
$$

$\sigma_{R_l}^2$ and $\sigma_{S_f}^2$ are variances of common resistance and load effects while a_{il} , a_{jl} , b_{if} and b_{jf} are coefficients of common random variables in Z_i and Z_j .

General Bounds and Narrow Bounds Method

In a serial system which consisting of *n* components, the following formula can be obtained:

$$
\begin{gathered}
P(\prod_{i=1}^{k+1} Z_i = 1) \geqslant P(\prod_{i=1}^{k} Z_i = 1)\cdot P(Z_{k+1} = 1) \\
(1 \leqslant i \leqslant n-1)
\end{gathered}
\tag{8}
$$

Through derivation, the upper and lower limit formula is described as (Ang and Wilson 1984):

$$
\max_{i=1,2,\cdots,n} P(Q_i = 1) \leqslant P_{f_s} \leqslant 1 - \prod_{i=1}^{n}(1 - P(Q_i = 1)) \tag{9}
$$

In the same way, failure probability for parallel system can be obtained:

$$
\prod_{i=1}^{n} P(Q_i = 1) \leqslant P_{f_p} \leqslant \min_{i=1,2,\cdots,n} P(Q_i = 1) \tag{10}
$$

Because the scope of general bounds is too wide, Ditleven deduced narrow bounds formula (Ditlevsen 1979).

$$P(E_1)+\max\left\{\sum_{i=2}^{n}\left[P(E_i)-\sum_{j=1}^{i-1}P(E_iE_j)\right],0\right\}$$

$$\leqslant P_f \leqslant \sum_{i=1}^{n}P(E_i)-\sum_{i=2}^{n}\max_{j<i}P(E_iE_j) \tag{11}$$

Generally, narrow bounds method is complex, but narrower failure probability scope can be usually obtained when the correlation coefficient is lower ($\rho < 0.6$).

Monte Carlo Method and PNET Method

The theoretical basis of Monte Carlo method is probability theory. Setting $x_1, x_2, \cdots, x_n$ are n independent random variables, if they are from the same matrix and have the same averages μ and variance σ^2 then, for any $\varepsilon > 0$ there is:

$$\lim_{n\to\infty}P\left(\left|\frac{1}{n}\sum_{i=1}^{n}x_i-\mu\right|\geqslant\varepsilon\right)=0 \tag{12}$$

If the probability of random event A is $P(A)$, and the frequency is $W(A)=m/n$ in n independent tests. For any $\varepsilon > 0$, there is:

$$\lim_{n\to\infty}P\left(\left|\frac{m}{n}-P(A)\right|<\varepsilon\right)=1 \tag{13}$$

When n is large enough, $\frac{1}{n}\sum_{i=1}^{n}x_i$ converges to μ and $W(A)=m/n$ converges to $P(A)$.

Ma and Ang proposed PNET method, which is a more accurate and approximate method to calculate structural reliability (Ang and Wilson 1984).

In the PNET method, using representative failure mechanisms replace all major failure mechanisms, the failure and safe probability of i^{th} mechanism are P_{f_i} and P_{r_i} , then the reliability of structural system is:

$$P_r=\prod_{i=1}^{m}(1-P_{f_i})=\prod_{i=1}^{m}P_{r_i} \tag{14}$$

The failure probability is:

$$P_f=1-P_r=\prod_{i=1}^{m}(1-P_{f_i})$$

$$=1-\prod_{i=1}^{m}P_{r_i} \tag{15}$$

When P_{f_i} is small, the above formula can be approximately written as:

$$P_f=\sum_{i=1}^{m}P_{f_i} \tag{16}$$

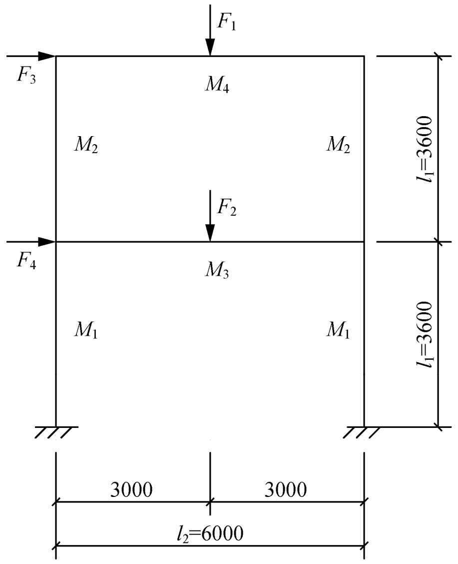


Figure 5. Model frame (Unit: mm)

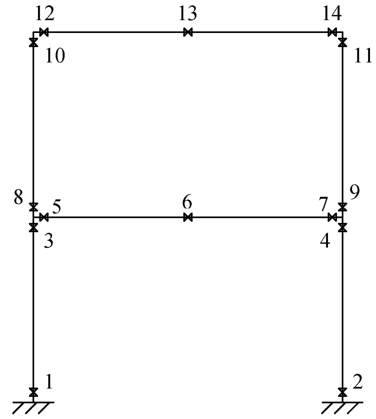


Figure 6. Possible plastic hinges formation

EXAMPLE ANALYSIS

According to the requirements of real engineering, this paper will analyze the reliability of the NAC and RAC frames shown in Figure 5 and investigate how the variation of components influences the structural reliability. Two examples are designed, in which beams with size of 250mm×500mm and columns with size of 400mm×400mm are chosen while the concrete strength grade is C30 according to Chinese code. Example 1 is ‘strong column-weak beam’ design and example 2 is ‘strong beam-weak column’ design. According to the results shown in Table 1 and Table 2, the statistical parameters of resistance (M_1, M_2, M_3, M_4) and external loads (F_1, F_2, F_3, F_4) (Zhao *et al.* 2000, Yi *et al.* 2006) are summarized and list in Table 3 and Table 4.

Table 3. Statistic parameters of external loads and bending moment resistance of example 1

Random variables	NAC frame		RAC frame	
	Mean	variation	Mean	variation
M_1	326.6	0.10	326.6	⩾0.10
M_2	303.5	0.10	303.5	⩾0.10
M_3	155.1	0.09	155.0	⩾0.09
M_4	182.7	0.09	182.4	⩾0.09
F_1	116.0	0.30	116.0	0.30
F_2	89.0	0.30	89.0	0.30
F_3	62.0	0.30	62.0	0.30
F_4	31.0	0.30	31.0	0.30

(The unit of resistance is $\mathrm{kN \cdot m}$ while external load is kN)

Table 4. Statistic parameters of external loads and bending moment resistance of example 2

Random variables	NAC frame		RAC frame	
	Mean	variation	Mean	variation
M_1	234.1	0.10	234.0	⩾0.10
M_2	234.1	0.10	234.0	⩾0.10
M_3	221.8	0.09	221.5	⩾0.09
M_4	258.4	0.09	257.9	⩾0.09
F_1	116.0	0.30	116.0	0.30
F_2	89.0	0.30	89.0	0.30
F_3	86.8	0.30	86.8	0.30
F_4	43.4	0.30	43.4	0.30

(The unit of resistance is $\mathrm{kN\cdot m}$ while external load is kN)

In single component, when the resistance of each section satisfies the condition of perfect correlation, plastic hinges that would possibly occur in frame are shown in Figure 6. The major failure mechanisms can be obtained (Ambartzmian, *et al.* 1998). Firstly, listed the performance function Z_i of each kind of failure mechanism and calculated the reliability index β_i and failure probability P_{f_i}. Then, sort the reliability index reversely according to their value. Finally, the correlation coefficients between each kind of mechanism are calculated according to Eq.(7). For both examples, the variations of RAC beam and column are 0.13 and 0.12 while for NAC beam and column are 0.10 and 0.09. The results of example 1 and example 2 are presented in Table 5—Table 8.

After above works have been done, different methods are used to calculate the reliability of frames.

(1) PNET method. Represent mechanism can be chosen after the correlation coefficient between every two failure mechanisms is known. Setting $\rho_0 = 0.7$ and the failure probability can be obtained according to Eq.(15) or (16).

Table 5. The failure probability of RAC frame in example 1

Failure mechanisms	Plastic hinges	β_i	P_{f_i}
1	12,13,14	2.808	2.496E−03
2	5,6,7	3.232	6.153E−04
3	1,2,6,7,13,14	3.302	4.798E−04
4	1,2,6,7,12,14	3.841	6.118E−05
5	1,2,6,7,11,13	4.298	8.602E−06
6	1,2,6,7,11,12	4.438	4.542E−06
7	8,9,13,14	4.478	3.759E−06
8	10,11,13	4.524	3.033E−06
9	1,2,6,7,10,11	4.797	8.071E−07
10	8,9,11,13	4.907	4.622E−07
11	1,2,3,4	4.996	2.925E−07
12	8,9,10,11	5.779	3.749E−09

Table 6. The correlation coefficient between failure mechanisms of RAC frame in example 1

Failure mechanisms	1	2	3	4	5	6	7	8	9	10	11	12
1	1.00	0.00	0.57	0.14	0.45	0.07	0.80	0.78	0.00	0.53	0.00	0.00
2	0.00	1.00	0.46	0.55	0.46	0.54	0.00	0.00	0.52	0.00	0.00	0.00
3	0.57	0.46	1.00	0.88	0.93	0.84	0.68	0.45	0.76	0.48	0.55	0.22
4	0.14	0.55	0.88	1.00	0.84	0.97	0.37	0.07	0.90	0.26	0.66	0.26
5	0.45	0.46	0.93	0.84	1.00	0.88	0.73	0.58	0.88	0.70	0.55	0.52
6	0.07	0.54	0.84	0.97	0.88	1.00	0.41	0.15	0.98	0.39	0.66	0.44
7	0.80	0.00	0.68	0.37	0.73	0.41	1.00	0.89	0.42	0.90	0.14	0.58
8	0.78	0.00	0.45	0.07	0.58	0.15	0.89	1.00	0.21	0.89	0.00	0.53
9	0.00	0.52	0.76	0.90	0.88	0.98	0.42	0.21	1.00	0.49	0.63	0.59
10	0.53	0.00	0.48	0.26	0.70	0.39	0.90	0.89	0.49	1.00	0.12	0.83
11	0.00	0.00	0.55	0.66	0.55	0.66	0.14	0.00	0.63	0.12	1.00	0.14
12	0.00	0.00	0.22	0.26	0.52	0.44	0.58	0.53	0.59	0.83	0.14	1.00

Table 7. The failure probability of RAC frame in example 2

Failure mechanisms	Plastic hinges	β_i	P_{f_i}
1	1,2,3,4	2.910	1.810E−03
2	1,2,6,7,10,11	3.099	9.706E−04
3	1,2,6,7,11,12	3.244	5.899E−04
4	1,2,6,7,12,14	3.290	5.015E−04
5	1,2,6,7,11,13	3.395	3.431E−04
6	1,2,6,7,13,14	3.402	3.350E−04
7	8,9,11,13	4.047	2.597E−05
8	8,9,10,11	4.059	2.462E−05
9	12,13,14	4.229	1.174E−05
10	8,9,13,14	4.271	9.717E−06
11	5,6,7	4.655	1.619E−06
12	10,11,13	4.690	1.368E−06

Table 8. The correlation coefficient between failure mechanisms of RAC frame in example 2

Failure mechanisms	1	2	3	4	5	6	7	8	9	10	11	12
1	1.00	0.68	0.69	0.68	0.61	0.58	0.28	0.36	0.00	0.28	0.00	0.00
2	0.68	1.00	0.98	0.94	0.90	0.81	0.51	0.65	0.00	0.43	0.53	0.11
3	0.69	0.98	1.00	0.98	0.91	0.87	0.48	0.56	0.10	0.48	0.54	0.11
4	0.68	0.94	0.98	1.00	0.90	0.91	0.44	0.46	0.19	0.51	0.53	0.11
5	0.61	0.90	0.91	0.90	1.00	0.95	0.73	0.59	0.41	0.73	0.48	0.49
6	0.58	0.81	0.87	0.91	0.95	1.00	0.63	0.39	0.56	0.76	0.46	0.47
7	0.28	0.51	0.48	0.44	0.73	0.63	1.00	0.78	0.59	0.90	0.00	0.83
8	0.36	0.65	0.56	0.46	0.59	0.39	0.78	1.00	0.00	0.54	0.00	0.35
9	0.00	0.00	0.10	0.19	0.41	0.56	0.59	0.00	1.00	0.82	0.00	0.84
10	0.28	0.43	0.48	0.51	0.73	0.76	0.90	0.54	0.82	1.00	0.00	0.83
11	0.00	0.53	0.54	0.53	0.48	0.46	0.00	0.00	0.00	0.00	1.00	0.00
12	0.00	0.11	0.11	0.11	0.49	0.47	0.83	0.35	0.84	0.83	0.00	1.00

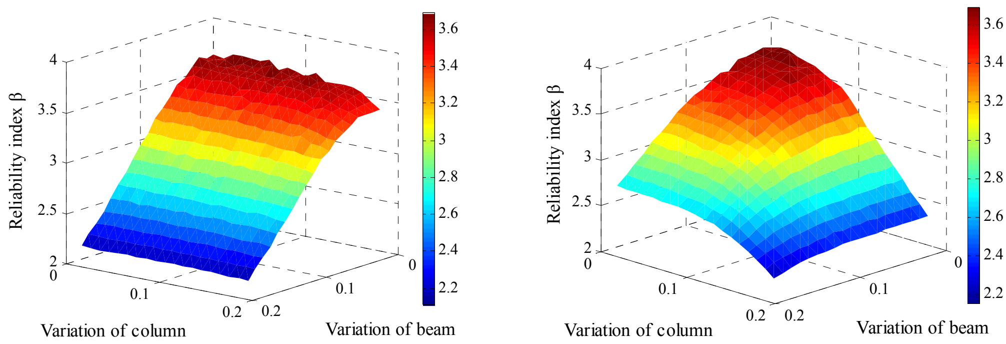
(2) Monte Carlo method. Because of the failure probability being low and taking account of the calculating capacity of computer, random sampling frequency $n = 10^6$ is chosen finally.

The reliability of RAC frame can be calculated using above methods. For NAC frame, the same methods and steps are also used to calculate the reliability. And the results are shown in Table 9.

Table 9. Results of calculation on frame failure probability

Examples		PNET method		Monte Carlo method	
		P_f	β	P_f	β
Example 1	NAC frame	1.273E-3	3.018	1.186E-3	3.039
	RAC frame	3.592E-3	2.688	3.287E-3	2.718
Example 2	NAC frame	4.566E-4	3.316	9.450E-4	3.107
	RAC frame	2.820E-3	2.768	2.937E-3	2.755

The results show that the reliability index of frame structure with the failure mode in which the plastic hinges occurred at the mid and end of beam is the lowest in example 1. In example 2, the failure mode in which the plastic hinges occurred at the end of column is the lowest. Table 9 shows that the results are almost the same when using different methods and the reliability of RAC frame is lower than that of NAC frame. Then only Monte Carlo method is used to calculate the structural reliability when the variation of components is changing. In order to investigate how the variation of beam and column influences the structure reliability respectively, example 1 is 'strong column-weak beam' design while example 2 is 'strong beam-weak column' design (as have been mentioned above). The reliability index is calculated for each example and the results are shown in Figure 7. (x, y, z coordinate axis denote variation of beam, variation of column and reliability index.)



(a) Reliability index of example 1

(b) Reliability index of example 2

Figure 7. Reliability index of frame structure

From Figure 7, the reliability index of frame structure can be easily obtained. Figure 7(a) shows that the reliability index decreases with the increase of beam variation. But it has almost no change with the increase of column variation. It indicates that the failure mode of example 1 is beam mechanism. Figure 7(b) shows that both the variation of beam and column has obvious influence on the structural reliability in example 2, which indicates that the failure mode of example 2 is beam-column mechanism. As for RAC frame, the variation of components is larger than that of NAC frame, which results in the decrease of structural reliability. If the variation of RAC components is slightly larger than that of NAC components, and the calculate results show that NAC and RAC frames have the same failure mechanisms, the reliability index of RAC frame is slightly lower than that of NAC frame, which indicates that the decrease of RAC strength variation may guarantee the reliability index of RAC frame.

CONCLUSIONS

The follow conclusions can be drawn from this theoretical analysis:

(1) The major failure modes are dependent on the external loads and resistance of components. If the frame structure is 'strong column-weak beam' design, the beam mechanism will be the lowest one and the structural reliability is more sensitive to the variation of beams.

(2) The discrete degree of RAC components resistance will increase because of the high discrete degree of strength and that further results in the decrease of reliability of RAC frame. Moreover, the average resistance of beam and column are lower than those of NAC components, which also contributes to the decrease of the reliability of RAC frame.

(3) Reliability for bearing capacity has no large difference between NAC and RAC frames which are under the combined action of horizontal and vertical loads. Therefore, it is feasible to apply RAC in practical engineering if the reliability of RAC structures is guaranteed and has no much decrease compared to that of NAC structures.

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