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New Factor to Characterize Mechanism of "Strong Column-Weak Beam" of RC Frame Structures*

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Abstract: Most reinforced concrete (RC) frame structures did not achieve the "strong column-weak beam" failure mode in recent big earthquakes, resulting in a large number of casualties and significant property loss. To deal with this serious problem, a new column-beam relative factor was proposed to characterize the relative yield situation of column ends and beam ends. By limiting the column-beam relative factor, RC frame structures could achieve the "strong column-weak beam" failure mode under the excitation of strong ground motions. The limit values of column-beam relative factor were calculated, analyzed and verified by using structural simulation models for corner columns in the bottom story of structures, which are destroyed most seriously in earthquakes. The results show that the limit values should be analyzed under bi-directional ground motion and with different axial compression ratios of columns. The peak ground acceleration (PGA) of ground motions has no significant effect on the limit values, while the type of strong ground motions has a significant effect on the limit values.

Keywords: RC frame structure; strong column-weak beam; limit value of column-beam relative factor; moment magnification factor at column end

Many damage investigations indicate that most of reinforced concrete (RC) frame structures do not meet the expectation of seismic design codes to achieve the "strong column-weak beam" failure mode, which requires that beam ends should yield before column ends during earthquakes. On the contrary, there are many cases that column ends yield first or beam ends and column ends yield at the same time, which might lead to the overall structure collapse, causing serious casualties and huge economic loss (Fig. 1)[1-7].

Some researchers have analyzed the reasons for the phenomenon that most RC frame structures did not achieve the "strong column-weak beam" failure mode and summarized many influencing factors, such as slabs and over distribution of beam reinforcement[8-14]. According to the research of these factors, fruitful results have been achieved. However, there are too many complicated influencing factors and the "strong column-weak beam" problem is not considered from the most essential aspect in all the analyses of these factors. In addition, some researchers have done much work on the moment magnification factor at column ends in the code for seismic design of buildings[15] and suggested that increasing this factor could achieve the "strong column-weak beam" failure mode[16-18]. However, from the aspects of architectural aesthetics and the convenience in use, designers always want columns to be thinner, which results in difficulties in improving the moment capacity of column ends. Meanwhile, according to the principles of structural mechanics, members with larger stiffness will be assigned to larger moment. If the moment capacity of column ends is improved by increasing the cross-section, the stiffness of the columns will increase at the same time. Then the moment assigned to the column ends will increase corre-

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spondingly. It is not known whether the increase of moment capacity is enough for the columns to resist the increased moment to achieve the requirement that beam ends should yield before column ends.

Faced with such a variety of factors which influence the achievement of "strong column-weak beam" failure mode and the potential problem of increasing moment magnification factor at column ends in building codes, the most direct way to simplify and solve these problems is to study the mechanism of "strong column-weak beam". In this study, a new column-beam relative factor was proposed to characterize the relative yield situation of column ends and beam ends. By limiting the column-beam relative factor, RC frame structures could achieve the "strong column-weak beam" failure mode under the excitation of strong ground motions. The limit values of column-beam relative factor were also calculated, analyzed and verified by using structural simulation models.

(a) Overall collapse of a bottom story

(b) Damage of a column end

Fig. 1 Response of buildings to earthquakes

1 Definitions

1.1 Essential meaning of "strong column-weak beam" failure mode

In most earthquakes, the conclusion that RC frame structures did not achieve the "strong column-weak beam" failure mode is based on the intuitive judgment of the relative yield situation of beams and columns. Therefore, the general understanding of "strong column-weak beam", in fact, covers several levels of meanings.

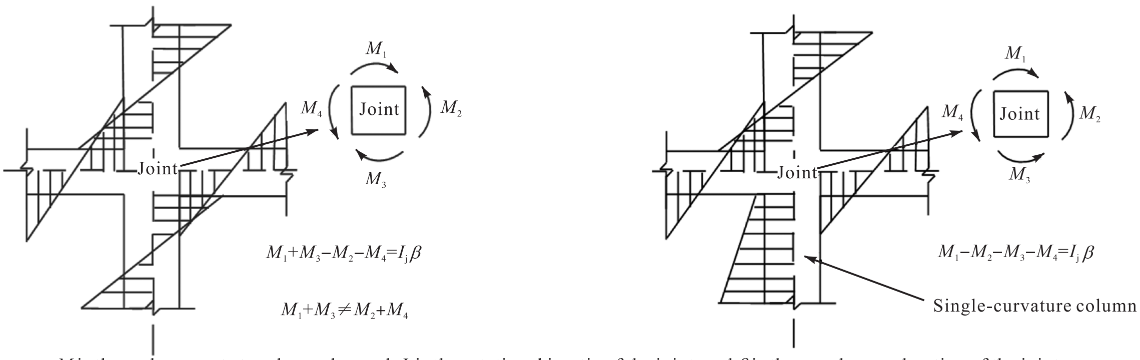
Firstly, from the perspective of structural members, the member subjected to a strong ground motion that does not yield is "strong"; in contrast, the member subjected to a strong ground motion that yields is "weak". Therefore, so-called "strong" should be considered from two aspects. One is the member's own capacity, i.e., the ultimate moment capacity of the member; the other one is the member's assignment, i.e., the maximum real bending moment during the entire process of the actual earthquake. The member with strong capacity and big assignment would also yield first.

It is important to note that the assignment of members in dynamic analysis is essentially different from that in static analysis of structures. In static analysis, it can be considered that the sum of moments at beam ends is equal to the sum of moments at column ends at the same joint by simplifying the structure. The moment capacity of column ends is larger than that of beam ends by multiplying with a moment magnification factor, so the structure can achieve the "strong column-weak beam" failure mode. However, in dynamic analysis, the rotational inertia of joints cannot be ignored, the inflection point of columns may be out of the intermediate position of columns, and there may even be single-curvature columns. The sum of moments at column ends is no longer equal to the sum of moments at beam ends at the same joint (Fig. 2)[19]. The assignment of members in the dynamic analysis and that in the static analysis are different. Therefore, considering the assignment of the members in dynamics is necessary.

In fact, the analysis of the capacity and assignment is the most essential aspect to deal with the problem of "strong column-weak beam". The effects of all the influencing factors in Refs. [8]—[14] can be performed as the change of the capacity and assignment of members.

Secondly, from the perspective of one joint, the real distinction between "strong column-weak beam" and "strong beam-weak column" is which member yields first, the column or the beam.

Obviously, in the concept of "strong column-weak beam" or "strong beam-weak column" failure mode, only the relative yield condition of the beam end and the column end at one joint is considered, while the overall failure mode of the structure can be reflected through the conditions of all the joints together. The achievement of "strong column-weak beam" failure mode should be based on the achievement that the beam yields before the column at each joint, and then the yield time of all the beams may be controlled macroscopically to make sure that all the beams yield before all the columns. Thus, the structures can achieve pure "strong column-weak beam" failure mode. Of course, this is the most ideal situation.



M is the real moment at each member end, I_j is the rotational inertia of the joint, and β is the angular acceleration of the joint.

Fig. 2 Schematic diagram of moments in dynamic analysis

However, in the past damage, it is difficult for a structure to achieve the pure "strong column-weak beam" failure mode. Some mixed yield modes should also be acceptable, but the overall collapse of one story should be basically avoided. Therefore, effective design methods and design parameters are required in design to have a clear understanding of the yield condition and the safety degree of structures, and make sure that the yield mode can be controlled in the acceptable manner.

1.2 Column-beam relative factor

Combined with different levels of the meanings about the "strong column-weak beam" failure mode, $M_{r,max}/M_u$ is introduced to elaborate these meanings. $M_{r,max}/M_u$ is the ratio of the maximal real bending moment distributed at one member end in a certain period of time to the ultimate moment capacity of the member end. This parameter focuses on the assignment and capacity of one member. The member end will be more intact if the parameter is smaller; in contrast, the member end will be destroyed more heavily if the parameter is larger. The key of the "strong column-weak beam" problem is which member end yields first, the column end or the beam end. So the ratio of $M_{cr,max}/M_{cu}$ ($M_{r,max}/M_u$ of the column end) to $M_{br,max}/M_{bu}$ ($M_{r,max}/M_u$ of the beam end) is proposed (i.e., $(M_{cr,max}/M_{cu})/(M_{br,max}/M_{bu})$) to compare the yield tendency of the column end and the beam end connected with the same joint. The beam end will incline to yield first if the ratio $(M_{cr,max}/M_{cu})/(M_{br,max}/M_{bu})$ is small; the column end will incline to yield first if the ratio $(M_{cr,max}/M_{cu})/(M_{br,max}/M_{bu})$ is large. $(M_{cr,max}/M_{cu})/(M_{br,max}/M_{bu})$ was named as the column-beam relative factor (i.e., $\eta_{c\text{-}b}$) in this study. It can be summed up as follows: when a column end and a beam end connected with the same joint are analyzed, in the period of time from the beginning of one ground motion to the time when one member (the column or the beam) yields first, the column-beam relative factor is defined as follows:

$$\eta_{c\text{-}b}=(M_{cr,max}/M_{cu})/(M_{br,max}/M_{bu}) \tag{1}$$

where $M_{cr,max}$ is the maximal real bending moment at the column end in the corresponding time; M_{cu} is the ultimate moment capacity at the column end calculated with actual distribution of reinforcing steel bars; $M_{br,max}$ is the maximal real bending moment at the beam end in the corresponding time; M_{bu} is the ultimate moment capacity at the beam end calculated with actual distribution of reinforcing steel bars.

An example of the calculation process of the column-beam relative factor is demonstrated as follows. For the requirement of microscopic analysis, solid element was adopted in ABAQUS. In order to analyze the most adverse situation, the model with single story and single span was selected to simulate the bottom story of the structure. The span of the model is 4 m. The story height of the model is 3 m. The grade of concrete is C20, and the grade of the longitudinal reinforcement and the transverse reinforcement is HRB335. The size and the reinforcement of the members are shown in Tab. 1 and Fig. 3. The thickness of the slab reinforced with Φ8 @ 150 is 100 mm. The dead load except the self-weight of the slab is 0.8 kN/m^2. The live load of the slab is 2 kN/m^2. The dead weight of the wall is 7.98 kN/m acting on the beam. The axial compression ratio of the columns is 0.4. In order to prevent the stress concentration on the tops of the columns, the column axial forces were transformed into coupling mass points on the tops of the columns. The model was meshed as shown in Fig. 4. El Centro strong ground motion was selected. The original data of the acceleration are shown in Fig. 5 and the corresponding acceleration response spectra are shown in Fig. 6. The peak ground acceleration (PGA) of the two horizontal directions was adjusted to 400 cm/s^2 (Z axis) and 340 cm/s^2 (X axis) in the analysis.

Tab. 1 Dimensions and reinforcement of members

Member	Dimension/ (mm × mm)	Cross-sectional area of each longitudinal reinforcement/mm^2	Stirrup
Beam	200 × 400	135	Φ8@100/200
Column	300 × 300	135	Φ8@100/200

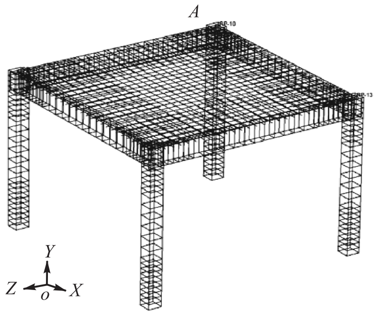


Fig. 3 Reinforcement of the model

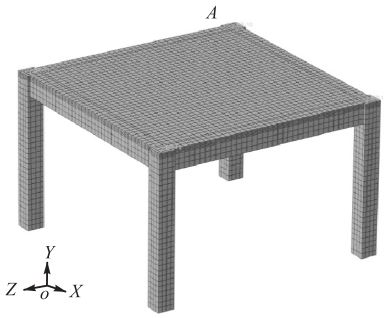


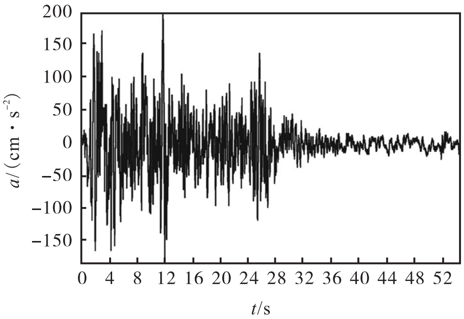
Fig. 4 Meshing situation of the model

The concrete damaged plasticity model in the ABAQUS software was adopted to describe the stress-strain relationship of the concrete (Fig. 7). The input values were calculated according to the appendix C.2 of the Code for Design of Concrete Structures[20]. The damage parameters were obtained by the energy equivalence principle. The mesh types of the concrete and reinforcement are C3D8R and T3D2 respectively. The double oblique line model was adopted to describe the stress-strain relationship of the reinforcement.

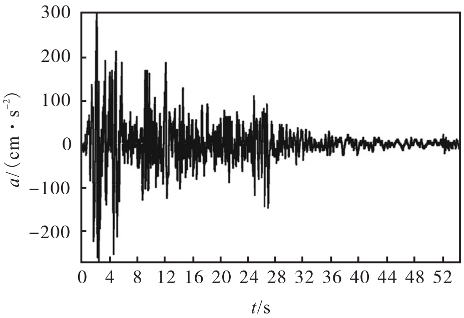
The beam and the column connected with joint A in the YoZ plane (Fig. 3) were selected as examples. The time-history analysis shows that the first plastic hinge is formed at the beam end at 1.52 s. Fig. 8 shows the time-history of the moment at the column end and the beam end. In the period of 0—1.52 s, the maximal real bending moment is 51.967 kN·m at the column end and 53.314 kN·m at the beam end.

The moment capacities at the beam end and the column end were calculated according to design codes. All the values used are design values. The moment capacity is 35.942 kN·m at the column end and 28.350 kN·m at the beam end.

With all the data substituted into Eq. (1), the column-beam relative factor is 0.7.

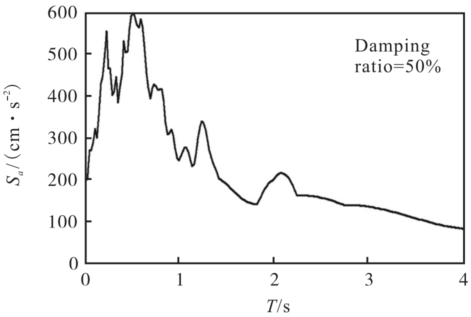


(a) X axis

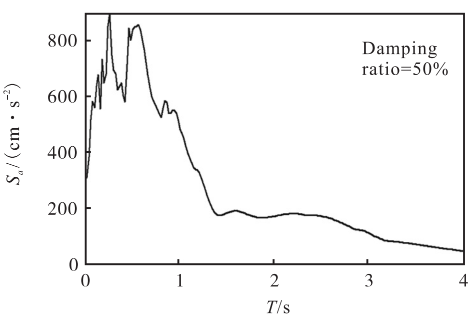


(b) Z axis

Fig. 5 Two horizontal components of acceleration time-history for 1940 El Centro earthquake



(a) X axis



(b) Z axis

Fig. 6 Two horizontal acceleration response spectra for 1940 El Centro earthquake

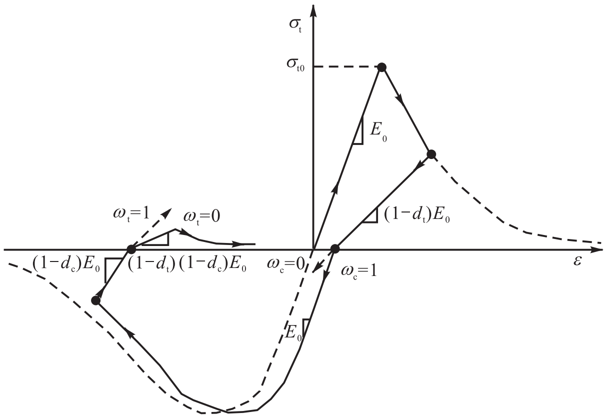


Fig. 7 Stress-strain relationship of concrete

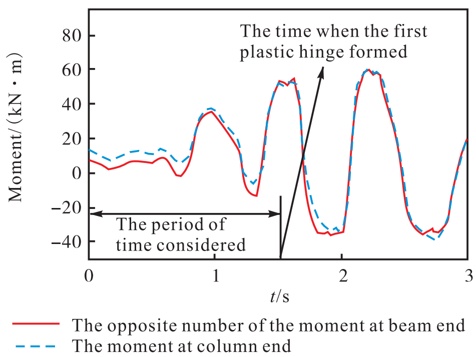


Fig. 8 Time-history of moment

1.3 The limit value of column-beam relative factor

Although the column-beam relative factor corresponds to the relative yield situation of a column end and a beam end, the column and the beam cannot be controlled to achieve the "strong column-weak beam" failure mode by the column-beam relative factor. However, an array of column-beam relative factors can be obtained by changing the relative strength of the column end and the beam end, and certain regularity of the column-beam relative factors is found. There is a critical value for all the column-beam relative factors corresponding to the "strong column-weak beam" mode or the "strong beam-weak column" mode. The column and the beam will be the "strong column-weak beam" mode if the column-beam relative factor is smaller than the critical value; otherwise, the column and the beam will be the "strong beam-weak column" mode. So this critical value can be used as the control parameter to achieve the "strong column-weak beam" failure mode for RC frame structures. It is named as the limit value of column-beam relative factor ($\eta_{\text{c-b,lim}}$). It can be obtained as follows.

For one joint under certain circumstances, with only the relative strength of beam ends and column ends changing in one structure, the relative yield situations of a column end and a beam end at the joint can be observed, and a number of $\eta_{\text{c-b}}$ can be calculated for the changes. $\eta_{\text{c-b,lim}}$ is the largest among the $\eta_{\text{c-b}}$ corresponding to the "strong column-weak beam" mode.

The limit value of column-beam relative factor is a limiting parameter ensuring that the plastic hinge should form at the beam end first. If the "strong column-weak beam" mode is required, the column-beam relative factor must be smaller than the corresponding limit value of column-beam relative factor. Thus, it is safer to take the largest among the $\eta_{\text{c-b}}$ corresponding to the "strong column-weak beam" mode as the limit value of the column-beam relative factor.

In addition, the column-beam relative factor can also be adjusted to achieve the expected failure mode. The smaller the $\eta_{\text{c-b}}$ than the $\eta_{\text{c-b,lim}}$, the stronger the column will be; the greater the $\eta_{\text{c-b}}$ than the $\eta_{\text{c-b,lim}}$, the stronger the beam will be. The relatively strong degree of members depends on the gap between the $\eta_{\text{c-b}}$ and the $\eta_{\text{c-b,lim}}$. It can be more economical when taking advantage of the limit value of column-beam relative factor to adjust the failure mode of structures numerically.

Under the given conditions, the calculation process of the limit value of column-beam relative factor is summarized in Fig. 9.

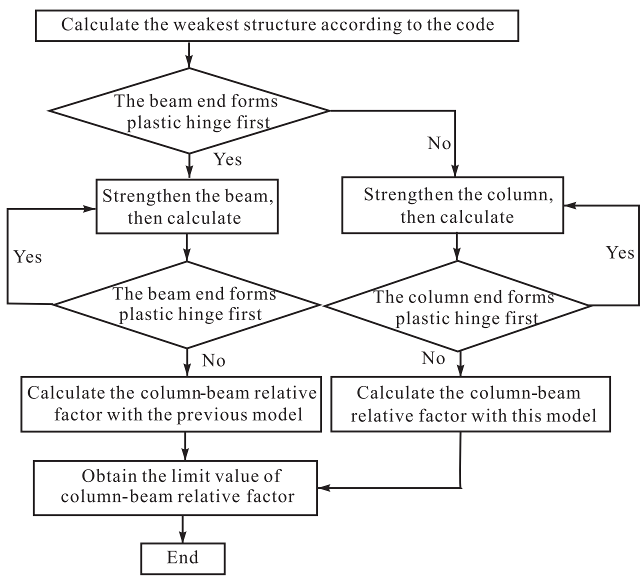


Fig. 9 Calculation process of the limit value of column-beam relative factor under a given condition

2 Influencing factors of the limit value of column-beam relative factor

In practice, to derive a more versatile limit value of column-beam relative factor, the influencing factors of the limit value of column-beam relative factor should be studied. The bottom story of the most simple frame structures shown in Section 1.2 was selected to be an example. The column and the beam connected with joint *A* in *YoZ* plane (Fig. 3) were still selected to be the analysis targets. Some factors, such as the axial compression ratio of columns, input method of ground motions, PGA of ground motions and type of ground motions, were considered and the law of the limit value of column-beam relative factor was summarized.

All the strong ground motions adopted for the analysis are shown in Tab. 2. The PGA was adjusted to 1∶0.85 (*Z* axis∶*X* axis) in the two horizontal directions (*Z* axis and *X* axis are shown in Fig. 3) of each strong ground motion according to the clause explanation 5.1.2 of the Code for Seismic Design of Buildings[15]. All the strong ground motions are bi-directional ground motions except GM-1′, which is a uni-directional ground motion.

2.1 Axial compression ratio

The limit values of column-beam relative factor calculated with different axial compression ratios from 0.15 to 0.5 under GM-1 in Tab. 2 are shown in Fig. 10, which are different with different axial compression ratios. Therefore, the axial compression ratio must be taken into consideration in the discussion of the limit value of column-beam relative factor.

Tab. 2 Input strong ground motions

Number	Strong ground motion	Adjusted PGA/(cm · s^{-2}) (*Z* axis∶*X* axis)
GM-1	1940, El Centro-Imp Vall Irr Dist, El Centro	400∶340
GM-1′	1940, El Centro-Imp Vall Irr Dist, El Centro	400 (*Z* uni-directional)
GM-2	1940, El Centro-Imp Vall Irr Dist, El Centro	620∶527
GM-3	1940, El Centro-Imp Vall Irr Dist, El Centro	220∶187
GM-4	1952, Taft, Kern County	400∶340
GM-5	1971, Castaic Oldbridge Route, San Fernando	400∶340
GM-6	2008, Wolong, Wenchuan earthquake	400∶340

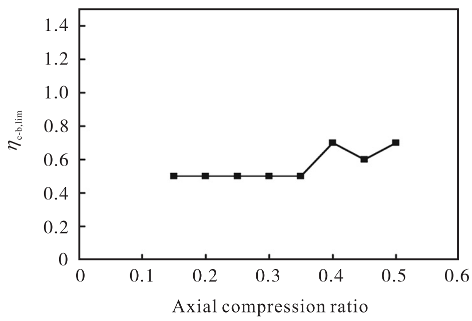


Fig. 10 Relationship between $\eta_{c\text{-}b,lim}$ and axial compression ratio under GM-1

2.2 Input method of ground motions

In order to analyze the effect of the input method of ground motions on the limit value of column-beam relative factor, GM-1 and GM-1′ in Tab. 2 were selected as input excitations. In Fig. 11, through comparing the limit values of column-beam relative factor of the models with different axial compression ratios, it is found that the limit values with the uni-directional ground motion are greater than those with the bi-directional ground motion, showing that the requirement for structures to achieve the "strong column-weak beam" failure mode is stricter with the bi-directional ground motion, namely, bi-directional seismic input is more unfavorable to structures. Therefore, in the following analysis, the limit values of column-beam relative factor discussed in this paper were all obtained by using bi-directional ground motion input.

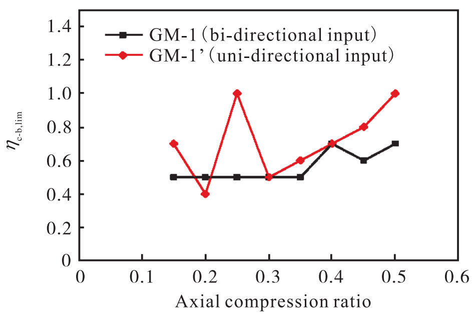


Fig. 11 Comparison of $\eta_{c\text{-}b,lim}$ under GM-1 and GM-1′

2.3 PGA of ground motions

GM-1, GM-2 and GM-3 in Tab. 2 were selected as input excitations to analyze the effect of PGA on the limit value of column-beam relative factor. The limit values of column-beam relative factor calculated under the same ground motion with different PGAs with each axial compression ratio are shown in Fig. 12. The limit values of

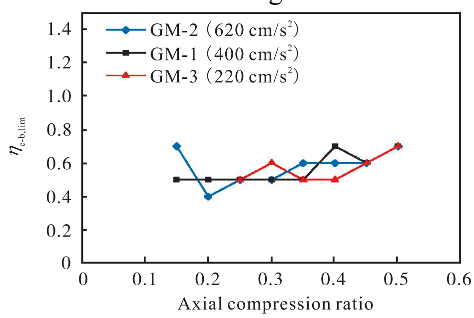


Fig. 12 Comparison of $\eta_{c\text{-}b,lim}$ under one strong ground motion with different PGAs

column-beam relative factor have little change with different PGAs.

2.4 Type of ground motions

Four different ground motions were adjusted to the same PGA, 400 : 340 cm/s^2 (*Z*-axis : *X*-axis). The limit values of column-beam relative factor calculated under GM-1, GM-4, GM-5 and GM-6 in Tab. 2 with each axial compression ratio are shown in Fig. 13. The limit values of column-beam relative factor calculated with different ground motions change greatly with each axial compression ratio.

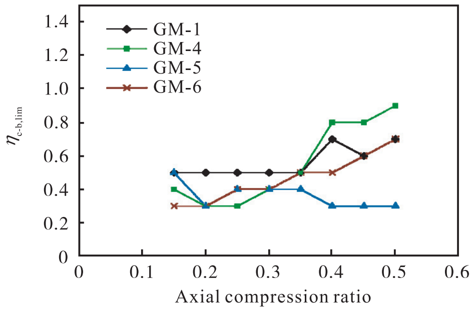


Fig. 13 Comparison of $\eta_{c\text{-}b,lim}$ under different strong ground motions with the same PGA

Therefore, statistical analysis with a large number of strong ground motions should be done to obtain the strong ground motions corresponding to the smallest value of $\eta_{c\text{-}b,lim}$ and the mean value of $\eta_{c\text{-}b,lim}$ for engineering applications.

3 Verification

To verify the $\eta_{c\text{-}b,lim}$, based on current codes, a single-span RC frame was designed for analysis. The height of each story was 3 m, and the span of each story was 4 m. The dimensions and the reinforcement of columns and beams are shown in Tab. 3. The thickness of the slab was 0.1 m. The grade of concrete was C20, and the grade of the reinforced bars was HRB335.

The dead load except the self-weight of the slab was 0.8 kN/m^2; the live load of the slab and the roof was 2 kN/m^2; the dead load except the self-weight of the roof was 2.9 kN/m^2; the weight of the wall acting on the beam was 7.98 kN/m; the axial compression ratio of the bottom columns was 0.5. The anti-seismic grade belonged to the second grade, and the seismic fortification intensity was Ⅷ (the corresponding PGA is 0.2*g*). The site classification was Ⅱ. The GM-1 in Tab. 2 was selected for verification. Nonlinear time-history analysis of the structure was carried out by using the ABAQUS software. The result shows that the column and the beam selected in the bottom story achieved the "strong column-weak beam" failure mode under the GM-1, and the column-beam relative factor was 0.5, which is smaller than the corresponding limit value of column-beam relative factor 0.7 in Fig. 10. The conclusions and the phenomenon fit well.

Tab. 3 Dimensions and steel reinforcement of columns and beams

Story number	Cross section dimensions of beams/(mm × mm)	Reinforcement of beam ends/mm^2 Up	Reinforcement of beam ends/mm^2 Down	Cross section dimensions of columns /(mm × mm)	Cross-sectional area of all the longitudinal reinforcement of each column/mm^2
1	200 × 400	603	402	300 × 300	1 608
2	200 × 400	556	402	300 × 300	1 608
3	200 × 400	462	308	300 × 300	1 608
4	200 × 400	308	308	300 × 300	1 608
5	200 × 400	308	308	300 × 300	1 608

In this example, if the column is required to be stronger, then $\eta_{c\text{-}b}$ should be adjusted to be smaller. If the column is not required to be so strong but is required to achieve the "strong column-weak beam" failure mode, then $\eta_{c\text{-}b}$ can be adjusted to be larger, but smaller than 0.7. If the structure is required to be mixed yield mode and the members of this joint are required to achieve the "strong beam-weak column" failure mode, then $\eta_{c\text{-}b}$ can be adjusted to be larger than 0.7.

4 Conclusions

The RC frame structures in earthquakes seldom achieve the "strong column-weak beam" failure mode. Combined with the analysis of the "strong column-weak beam" concept, the column-beam relative factor was proposed to characterize the relative yield situation of column ends and beam ends. By limiting the column-beam relative factor, RC frame structures could achieve the "strong column-weak beam" failure mode.

Flexible adjustments to expected yield modes of joints can be done and the level of assurance to achieve the yield modes can be analyzed by referring to the limit value of column-beam relative factor to control the overall yield mode of structures.

Some conclusions about the limit value of column-beam relative factor were summarized for corner columns in the bottom story of structures, which are destroyed most seriously in earthquakes.

(1) The limit values of column-beam relative factor should be analyzed under bi-directional ground motion input and with different axial compression ratios of columns.

(2) When the axial compression ratio of columns is the same, the limit values of column-beam relative factor have little change under the same ground motion with different PGAs, while the limit values change greatly under different ground motions adjusted to the same PGA. For engineering application, the strong ground motions corresponding to the smallest value of $\eta_{\text{c-b,lim}}$ and the mean value of $\eta_{\text{c-b,lim}}$ can be acquired by statistical analysis under a large number of ground motions.

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