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Realization of the global yield mechanism of RC frame structures by redesigning the columns using column tree method

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Global failure mechanism, i.e., the strong-column weak-beam mechanism, can provide higher total energy dissipation capacity with less ductility demand on components than other failure modes, and results in a more uniform story drift distribution and higher resistance to earthquake loads at the system level. However, the current code-based elastic design method cannot guarantee the global failure mechanism of frame structures under severe earthquakes. In this paper, a simple, but practical design procedure is proposed to ensure the global failure mechanism of reinforced concrete (RC) frame structures by redesigning the columns using the column tree method (CTM). CTM considers the yield limit state of all beams and column bases. The code-based design is firstly carried out to determine the section information of all beams and base columns. Then, the internal force demands applied on the column tree can be derived. Lastly, the column moments, shear forces and axial forces are determined according to the free-body diagram of CTM to finish the column redesign. Two RC frame structures with 6 and 12 stories are illustrated to verify the design procedure. The analytical results demonstrate the proposed approach can realize the global failure mechanism.

global failure mechanism, column tree method (CTM), reinforced concrete frame structures, failure mode, plastic hinges

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1 Introduction

The earthquake-resistant design of structures in seismic-prone zones is performed to provide the structures with sufficient distribution of strength, stiffness and ductility for the given seismic hazard. Accordingly, the code-conforming methodology usually employs the equivalent static force procedure (ESFP) to determine the configuration of the structures. However, it is widely recognized that structures designed using the ESFP do not take post-yield behaviors into consideration, which results in unpredictable and uncontrollable structural damage. This is evident through the damage observed in recent earthquakes [1–4]. Generally, structures subjected to severe earthquake shaking have many potential failure modes, such as local mechanisms, soft-story mechanisms and global mechanisms [5], as shown in Figure 1. Local failure and soft-story mechanisms generally should be avoided because they typically require large ductility demands on components, which cannot be achieved easily and economically, and can result in collapse of a structure. Global failure mechanisms, i.e., the strong-column weak-beam mechanism, can provide higher total energy dissipation with less ductility demand on components, which results in a more uniform story drift and higher resistance to earthquake loads at the system level.

To achieve the global failure mechanism, many design methods, seismic measures and structural systems have

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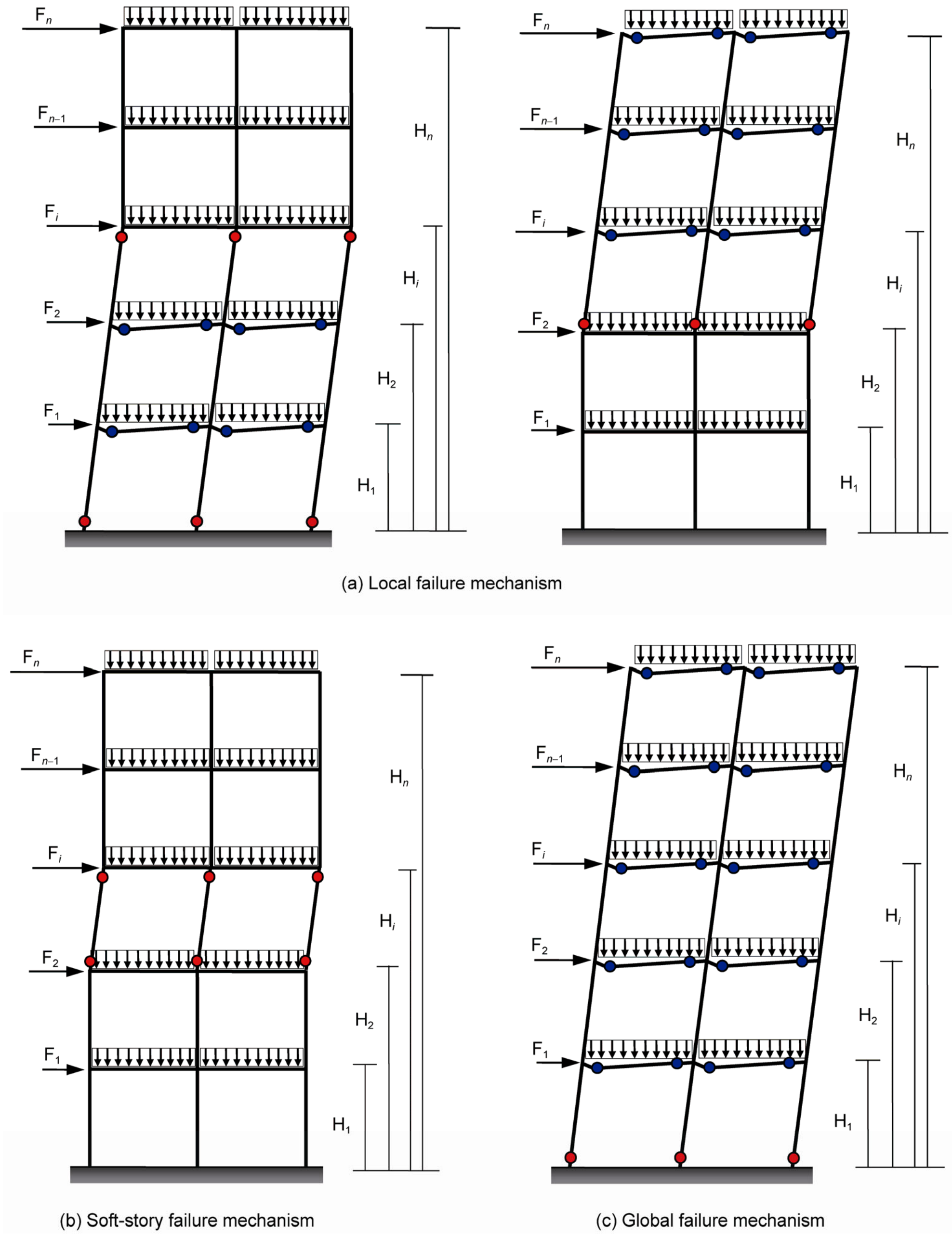

Figure 1 (Color online) Main failure modes for frame structures subjected to severe earthquakes [1].

been developed. Mazzolani and Piluso [5] proposed a plastic design approach to design steel moment-resisting frames to achieve a desirable global failure mode. However, iteration was required to determine column sizes. Qu et al. [6] introduced a retrofitting strategy of adding a stiff wall in an existing 11-story frame system to avoid a soft-story failure mechanism, and the results indicated that this strategy is capable of creating a desirable global failure mechanism. However, no procedure has been proposed to design such systems for new constructions. Goel and Bai et al. [7,8] proposed the performance-based plastic design (PBPD) method to design new structures. This method utilizes the energy balance concept and global plastic mechanism to design structural members that satisfies both strength and drift limits for the seismic load without iterations.

The capacity design method has been widely adopted in many building codes and provisions (e.g. Chinese Seismic Design Code [9], International Building Code [10], National Building Code of Canada [11] and Euro Code [12]) by amplifying column capacity. For example, in the Chinese Code system [9], except the roof columns and columns of which the axial compression ratio is less than 0.15, the following requirements of bending moment of column ends must be satisfied:

$$\sum M_c = \eta_c \sum M_b, \tag{1a}$$

$$\sum M_c = 1.2 \sum M_{\mathrm{bua}}, \tag{1b}$$

where $\sum M_c$ is the summation of design bending moments of all column ends for a specific beam-column joint, $\sum M_b$ is the summation of design bending moments of all beam ends for the specific beam-column joint, $\sum M_{\mathrm{bua}}$ is the summation of actual bending moments of all beam ends determined from the actual beam reinforcements, and η_c is the moment amplification factor of column ends. For different seismic grades of structures, η_c have different values, and high seismic grade of structures gets the high value of η_c. For the case of seismic grade I, when eq. (1a) cannot be satisfied, the requirement of eq. (1b) must be reached. It should be pointed out that the determination of beam and column moments in the code-based method is based on the elastic analysis of structural system, where the structure is designed to remain almost elastic for a fraction of the total seismic load. Such design philosophy does not account for the post-yield behavior of components, especially the influence of beam yielding on columns. As a result, the bending moment demands of column ends may be underestimated, which makes the column reinforcements insufficient and usually controlled by the minimum reinforcement requirements [13]. In addition, the code-based method can only amplify the bending moments of column ends which are intersected by the specific beams, which is actually a local design method, not a global or system manner [14]. Therefore, it is very difficult to ensure the structures design using the code-based method yielding in the global mechanism.

In the present study, a simple but practical design procedure is proposed to ensure the formation of global yield mechanism of reinforced concrete (RC) frame structures under severe earthquakes. Initially, the structures are designed based on the code-conforming requirements, and the beam and column section configurations are determined. The section properties of all beams and base-columns are assumed to be known quantities based on the code-based design results, and the demands of bending moments and shear forces of beam ends and the base-column moments can be derived. According to the free-body diagram of column tree method (CTM), the column moments, shear forces and axial forces can be derived and the columns will be further redesigned to achieve the global failure mechanism. Two case structures with 6 and 12 stories are used to verify the design procedure. The analytical results demonstrate that the proposed approach can achieve the desired yield mode.

2 Column tree method for frame structures

2.1 Column tree method based on code design results

For the RC frame structures, they will sustain gravity loads and be subjected to lateral seismic forces, under which the global failure mechanism will be developed, as illustrated in Figure 2. In this mechanism, all beams and base columns yield to dissipate the earthquake energy, while the other parts of the columns keep elastic. Therefore, how to design the columns is crucial to ensure the fulfillment of this mechanism.

To achieve the global failure mode using column tree method [15], the free-body diagram of interior columns and exterior columns can be developed, as shown in Figure 3. As can be seen, five types of force demands are applied in the column tree, including the shear forces and bending moments of beam ends, vertical force at beam-column joints, the moment demands of base columns and the applied lateral forces, to keep the equilibrium of column tree. Basically, the proposed method in the present study performs the design of interior columns and exterior columns in the global

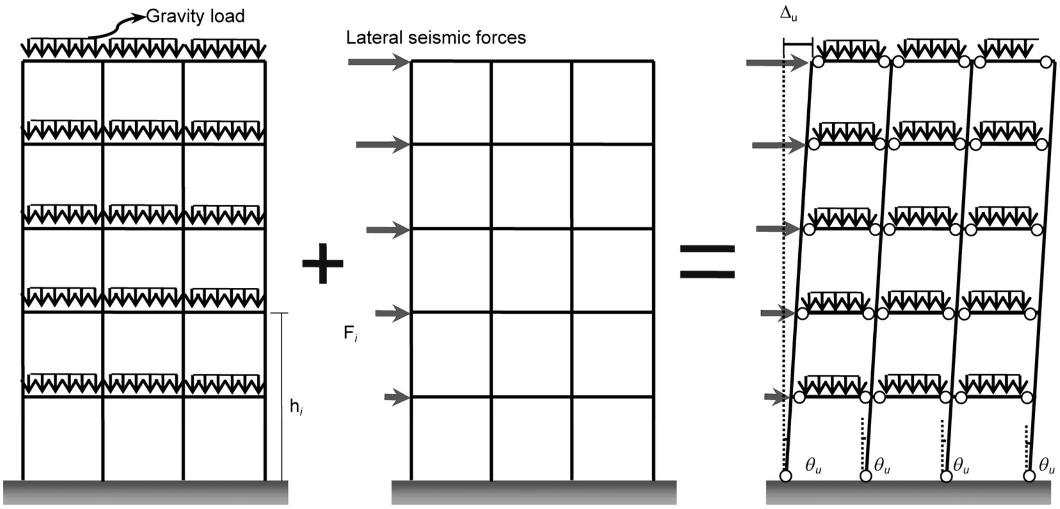

Figure 2 Formation of global failure mechanism of frame structures under gravity load and lateral seismic loads.

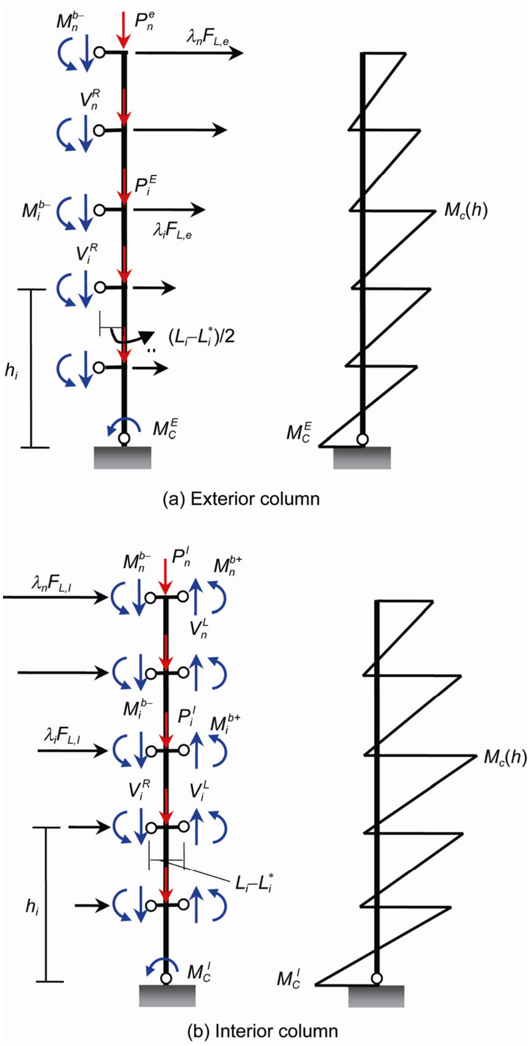

Figure 3 (Color online) Free-body diagram for column tree method.

manner by considering the yielding of all beams and base columns and plastic excursion into strain-hardening condition, based on the premise of the formation of global failure mechanism.

It should be seen that the global failure mechanism requires that the earthquake-induced energy will be totally dissipated by the plastic hinges of all beams and base columns, while the other parts of the columns keep within elastic condition. Although the code-based elastic design approach cannot exactly predict the internal force demands of structures, the design results are obtained based on the specific seismic hazard level that the structure is subjected to, element stiffness distribution and spatial configuration. Therefore, the beam design results can be deemed to be reliable and useful since both the seismic actions and factored gravity load are considered. In other words, it is required that the design of all beams and base columns will be determined through the code-based design method, while the other parts of the columns will be determined by using the column tree method.

The beam bending moments applied on column trees can be calculated based on the section information of beam ends:

$$M_i^{b+} = \xi \cdot M_{y,i}^{b+} = \xi \cdot f_y A_{s,i}^{+}\left(h_{0,i}^{+} - \frac{f_y A_{s,i}^{+}}{2\alpha_1 f_c b_i}\right), \tag{2a}$$

$$M_i^{b-} = \xi \cdot M_{y,i}^{b-} = \xi \cdot f_y A_{s,i}^{-}\left(h_{0,i}^{-} - \frac{f_y A_{s,i}^{-}}{2\alpha_1 f_c b_i}\right), \tag{2b}$$

where M_i^{b+} and M_i^{b-} are the beam positive and negative bending moments of the ith story applied on column trees, respectively; ξ is the over-strength factor; $M_{y,i}^{b+}$ and $M_{y,i}^{b-}$ are the beam positive and negative yielding moments of the ith story, respectively; $A_{s,i}^{+}$ and $A_{s,i}^{-}$ are the beam bottom and top longitudinal reinforcements of the ith story determined by the code-based method, respectively; $h_{0,i}^{+}$ and $h_{0,i}^{-}$ are the beam section effective heights for positive and negative moments of the ith story, respectively; f_y and f_c are the tensile strength for steel reinforcement and compression strength for concrete, respectively; α_1 is the coefficient and b_i is the beam width of the ith story.

The over-strength factor is formed mainly due to the strain-hardening effects for post-yield behavior of beams. In this paper, a unified value for all stories is employed to consider that all beams reach the post-yield limit state simultaneously. Of course, this selection is within the safe margin. It must be seen that different over-strength factors represent the degree of inelastic response that the structures may experience during strong earthquakes. Usually, it is recommended that tall buildings have a high over-strength factor to consider the local higher-mode deformation.

Furthermore, the beam shear forces applied on the column trees can be derived:

$$V_i^L = \left(\left|M_i^{b+}\right| + \left|M_i^{b-}\right|\right) \big/ L_n - V_{Gb}, \tag{3a}$$

$$V_i^R = \left(\left|M_i^{b+}\right| + \left|M_i^{b-}\right|\right) \big/ L_n + V_{Gb}, \tag{3b}$$

where V_i^L and V_i^R are the beam left and right shear forces, respectively; L_n is the beam clear span; V_{Gb} is the shear forces of beam ends for a simply supported beam with the factored gravity load applied on it.

The base column moments M_c^I and M_c^E can be approximately calculated according to the column axial forces and reinforcements. Meanwhile, the vertical forces P_i^E and P_i^L applied at beam-column joints can be determined through the structural vertical configuration and load condition.

The last unknown force demand applied on the column tree is the lateral forces. Since the beam bending moment distribution along the height has been calculated, the lateral force pattern will directly determine the column moment for each story. Currently, the code-conforming lateral force patterns, including the inverted triangular pattern and parabolic pattern are mainly based on the first-mode dynamic solutions of elastic structural system [16]. It is widely recognized that the structures designed by the code lateral force distributions will undergo large inelastic deformation. Due to the solution of lateral force distributions without considering the inelastic behavior, the use of these lateral equivalent forces will not lead to the optimum distribution of structures [17]. As a consequence, some new lateral force pattern have been developed to achieve the uniform story drift to fully exploit the material capacity and maximize the energy-dissipating capacity [16]. In the present study, the lateral force pattern developed by Chao et al. [18] will be adopted, since it can result in a much better prediction of inelastic seismic demands at global as well as at element levels:

$$\lambda_i = \left(\beta_i - \beta_{i+1}\right)\left(\frac{w_i h_i}{\sum_{j=1}^{n} w_j h_j}\right)^{0.75T_e^{-0.2}}, \tag{4}$$

$$\beta_i = \left(\sum_{j=i}^{n} w_j h_j \Big/ w_n h_n\right)^{0.75T_e^{-0.2}}, \tag{5}$$

where λ_i is the ith story lateral force factor, β_i is the ith story shear distribution factor, w_i, w_j and w_n are the story seismic weight for the ith, jth and roof floors, respectively, h_i and h_j are the height from the base to the ith floor and jth floor, respectively, h_n is the height of roof floor from the base, and T_e is the structural fundamental period.

According to the equilibrium of the column trees, the normalized lateral forces can be derived:

$$F_{L,e} = \frac{\sum_{i=1}^{n}\left|M_i^{b-}\right| + \sum_{i=1}^{n} V_i^L \cdot \left(L_i - L_i^*\right)/2 + M_c^E}{\sum_{i=1}^{n}\lambda_i h_i}, \tag{6}$$

$$F_{L,I} = \frac{\sum_{i=1}^{n}\left(\left|M_i^{b+}\right| + \left|M_i^{b-}\right|\right) + \sum_{i=1}^{n}\left(V_i^L + V_i^R\right)\cdot\left(L_i - L_i^*\right)/2 + M_c^I}{\sum_{i=1}^{n}\lambda_i h_i}, \tag{7}$$

where $F_{L,e}$ and $F_{L,I}$ are the normalized lateral force for exterior and interior column tree, respectively, L_i is the beam span of the ith story and L_i^* is the distance of plastic hinges for the ith story beam. Then, the design lateral forces F_i can be calculated using the lateral force factor λ_i. Furthermore, the internal force demands of columns can be derived:

$$M_c\left(h\right) = \sum_{i=1}^{n}\delta_i M_i + \sum_{i=1}^{n}\delta_i V_i \frac{L_i - L_i^*}{2} - \sum_{i=1}^{n}\delta_i F_i\left(h_i - h\right), \tag{8}$$

$$N_c\left(h\right) = \sum_{i=1}^{n}\delta_i V_i + \sum_{i=1}^{n}\delta_i P_{ci}, \tag{9}$$

$$V_c\left(h\right) = \sum_{i=1}^{n}\delta_i F_i, \tag{10}$$

$$\delta_i = \begin{cases} 1, & h < h_i, \\ 0, & h > h_i, \end{cases} \tag{11}$$

where $M_c(h)$, $N_c(h)$ and $V_c(h)$ are the column moment, axial force and shear force at the height of h, respectively; δ_i is the influence factor of the ith story; M_i is the beam bending moment of the ith story applied on column trees, and $M_i = \left|M_i^{b-}\right|$ for exterior columns and $M_i = \left|M_i^{b-}\right| + \left|M_i^{b+}\right|$ for interior columns; V_i is the beam shear force applied on column trees, and $V_i = V_i^R$ for exterior columns and $V_i = V_i^R + V_i^L$ for interior columns; F_i is the design lateral force of the ith story, and $F_i=F_{i,e}$ for exterior columns and $F_i=F_{i,I}$ for interior columns; P_{ci} is the vertical force applied on beam-column joint of the ith story, and $P_{ci}=P_i^E$ for exterior column and $P_{ci}=P_i^L$ for interior column.

When all the column internal force demands have been figured out, the section reinforcements can be carried out based on the Concrete design code [19].

2.2 Step-by-step design flowchart

Figure 4 shows the design flowchart of the implementation of global failure mechanism using column tree method. As can be seen, the whole design is based on the code-conforming result. Furthermore, no iterations are required, which makes the proposed design method very simple and practical.

3 Design examples

The developed approach is demonstrated and verified by

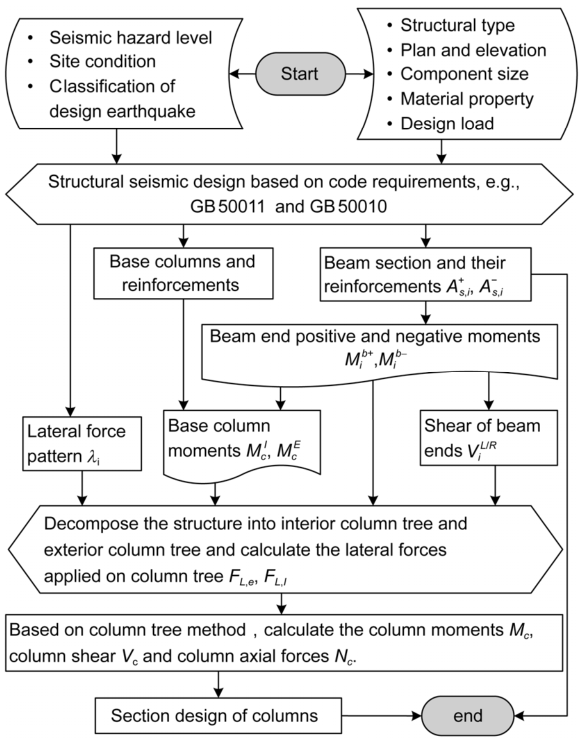

Figure 4 Design flowchart of the implementation of global failure mode using column tree method.

using two case buildings with 6 and 12 stories respectively. Both structures have three bays, and the structural elevation view and member sizes are shown in Figure 5. For the 6-story structure, the first story has a height of 4.1 m and the upper stories are 3.7 m, while all stories have the same height of 3.3 m for the 12-story structure. The side span of 6-story structure is 6.0 m and middle span is 2.5 m, which is a typical representation of school buildings or hospital buildings, while the 12-story structure has the same spans to represent the office buildings. It should be pointed out that the exterior column sizes are smaller than interior column sizes to achieve approximately the same axial force ratio for exterior and interior columns. Steel reinforcement of HRB400 is selected for both structures. Concrete material of C30 is adopted in 6-story structure while C40 is used in 12-story structure. The floor (roof) dead load and live load are taken as 6.0 and 2.0 N/mm^2, respectively for both structures. In addition, the beams of 6-story structure are applied with a uniform vertical load with amplitude of 8 kN/m to consider the infilled walls.

The two structures are assumed to locate at the region with seismic precautionary intensity of 8 and characteristic period of 0.35 s, and designed using the elastic method based on Chinese Seismic Design Code [9,19]. The code-conforming beam and column section longitudinal reinforcements are shown in Table 1. As can be seen, many column design results are controlled by minimum reinforcement requirement.

Based on the code-conforming design results, the columns of two structures are redesigned by using the proposed CTM, and the resulting column moments are demonstrated in Figure 6, where the over-strength factor ξ is taken as 1.2 for 6-story structure and 1.5 for 12-story structure. Also, the column shear force and axial force can be derived, and based on these results, the new column longitudinal reinforcements can be determined, as illustrated in Table 1. As can be seen, the column tree design is larger than the code-based results, which indicates that the code-based elastic design underestimates the internal force demand of columns. As will be shown later, the increase of column longitudinal reinforcements can result in slight reduction of maximum interstory drift ratio, but more importantly can achieve the global yield mechanism that is emphasized in this study.

OpenSees platform was used to develop the analytical model of the structures [20]. Columns and beams were modeled using *beamWithHinges* elements where the nonlinearity was modeled using the lumped plastic hinges at the ends with the middle region modeled with linear-elastic

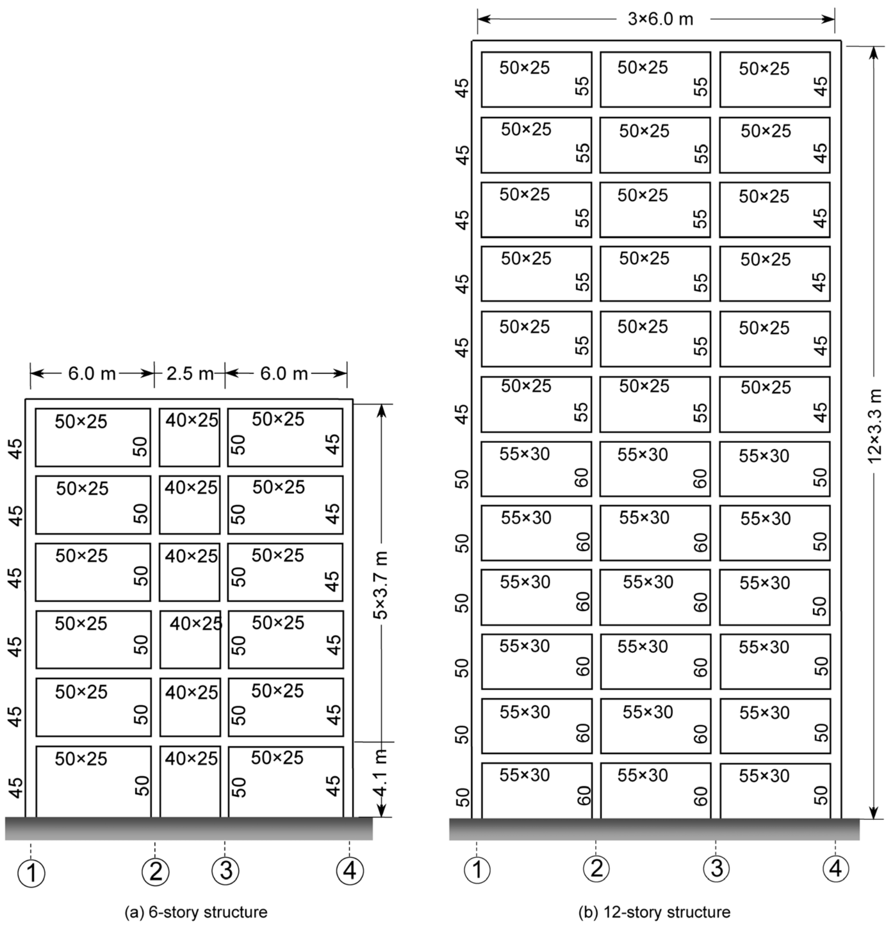

Figure 5 Structural elevation for different building heights and the component sizes (unit: cm).

Table 1 Beam and column longitudinal reinforcements for the code-based design and CTM

Structure	Story	Beam (mm^2)				Code-based column (mm^2)		Column tree method column (mm^2)	
		Side bay		Middle bay		Exterior column	Interior column	Exterior column	Interior column
		Top	Bottom	Top	Bottom	Each side	Each side	Each side	Each side
6 stories	1	1702.0	628.0	1610.0	1231.0	1017.0	1742.0	1140.0	2010.9
	2	1702.0	628.0	1610.0	1231.0	647.0	1272.0	1610.0	2454.0
	3	1610.0	628.0	1272.0	1008.1	647.0	1231.0	1924.0	2454.0
	4	1388.0	628.0	1008.1	728.1	763.0	1140.0	1924.0	2344.0
	5	1140.0	628.0	662.9	452.0	763.0	1008.1	1924.0	2122.0
	6	1017.0	710.0	452.0	452.0	1140.0	1140.0	1256.0	1702.0
12 stories	1	1610.0	804.0	1610.0	804.0	1017.0	1388.0	1074.2	1388.0
	2	1742.0	917.1	1742.0	917.1	917.1	1388.0	1520.0	1388.0
	3	1742.0	917.1	1742.0	917.1	917.1	1388.0	1451.0	1388.0
	4	1702.0	882.5	1702.0	882.5	917.1	1388.0	1451.0	1388.0
	5	1610.0	817.0	1610.0	817.0	917.1	1388.0	1520.0	1388.0
	6	1610.0	817.0	1610.0	817.0	917.1	1388.0	1610.0	1520.0
	7	1520.0	804.0	1520.0	804.0	763.0	1140.0	2122.0	1964.0
	8	1451.0	763.0	1451.0	763.0	763.0	1140.0	2454.0	2454.0
	9	1388.0	710.1	1388.0	710.1	763.0	1140.0	2454.0	2834.1
	10	1140.0	603.0	1140.0	603.0	763.0	1140.0	2344.0	2592.0
	11	1017.0	603.0	1017.0	603.0	763.0	1140.0	1964.0	2454.0
	12	1017.0	603.0	1017.0	603.0	1017.0	1140.0	1451.0	1742.0

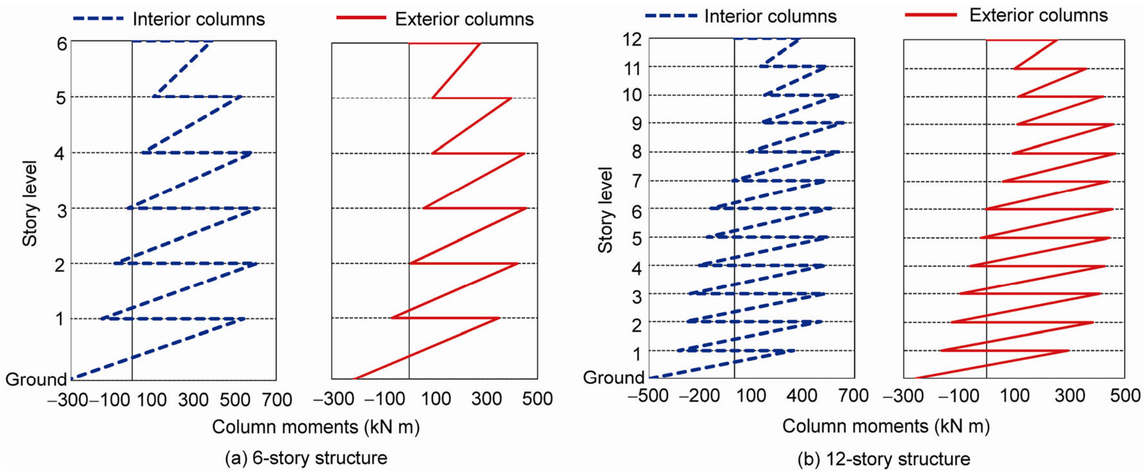

Figure 6 (Color online) Column moments determined by column tree method.

properties. Fiber section model was employed to model the nonlinear behavior of the hinges. Uniaxial material, Concrete01 and Steel02, were used to model the concrete and steel reinforcement, respectively. To account for the component stiffness reduction due to concrete cracking, reduction factors of 0.5 and 0.7 were selected to model the elastic portion of beams and columns, respectively. In addition, P-Delta effects were considered in all nonlinear dynamic analyses. Soil structure interaction (SSI) effects were not considered. Rayleigh damping of 5% was assigned at the first and third mode periods.

To compare and verify the effectiveness of the proposed method, four ground motions were selected, including three frequently-used real ground motions and one synthetic ground motion. The three real ground motions were El Centro 1940 (NS), Northridge 1994 (Sylmar-olive) and Taft 1952 (Kern County), while the synthetic ground motion was generated using the SIMQKE software [21] to match the spectrum acceleration response well with the code-defined spectrum. All the ground motions were scaled to peak ground acceleration (PGA) of 400 cm/s^2 to represent the severe seismic hazard level, and the pseudo-acceleration response spectra are shown in Figure 7.

4 Results and discussion

Interstory drift ratio is frequently used as a good indicator of structural global damage in assessing the seismic performance of frame structures. Figure 8 shows the interstory drift ratio demand of the two structures under severe earthquakes for the code-based method and CTM. For the 6-story structure, the code-based design structure can achieve the maximum interstory drift ratio less than 2%, except the case of Northridge excitation, while the CTM-based structure can meet the requirement under all ground motions. The CTM-based design makes the maximum interstory story drift ratio reducing by 8.6%, 6.2% and 3.2% under El Centro, Northridge and Taft excitations, respectively, but a light increase of maximum story drift ratio is observed for the synthetic ground motion. For the 12-story structure, both the code-based design structure and CTM-based structure can meet the 2% maximum interstory drift ratio requirement, but the difference of the story drifts for the two designs is very small, with the reduction of maximum interstory drift ratio being 1.1%–3.6%. It must be stressed that the main objective of the proposed approach is to ensure the global failure mechanism of structures subjected to strong earthquake shakings. Since the two methods have the same beam designs and the earthquake-induced energy is mainly dissipated by beam plastic hinges, it is not strange that the two methods have nearly the same maximum interstory drift demands.

Figure 9 shows the plastic hinge distribution and their plastic rotation of the 6-story structures subjected to four ground motions. As demonstrated in Figure 9(a), for the code-based structure under 4 ground motions, column plastic hinges except that of the column-base are observed, which confirms that the code-based design cannot ensure the strong-column weak-beam global failure mode. Meanwhile, it can be seen from Figure 9(b) that CTM can achieve the global failure mechanism. In addition, small difference exists in the distribution of beam plastic hinges and beam plastic rotation along the height.

The plastic hinge distribution and plastic rotation of the 12-story structures are demonstrated in Figure 10. Since the columns except the column-base for code-based and CTM-

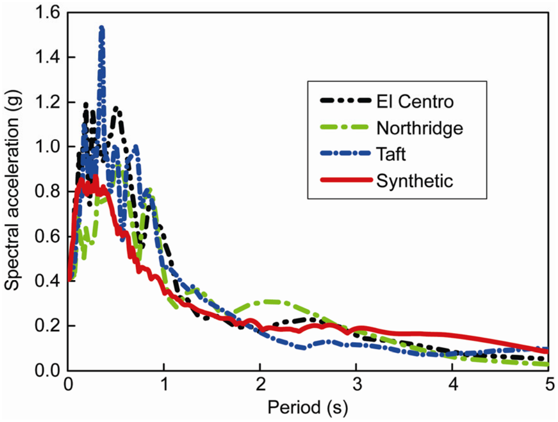

Figure 7 (Color online) Pseudo-acceleration response spectra of the four scaled ground motions (5%-damped).

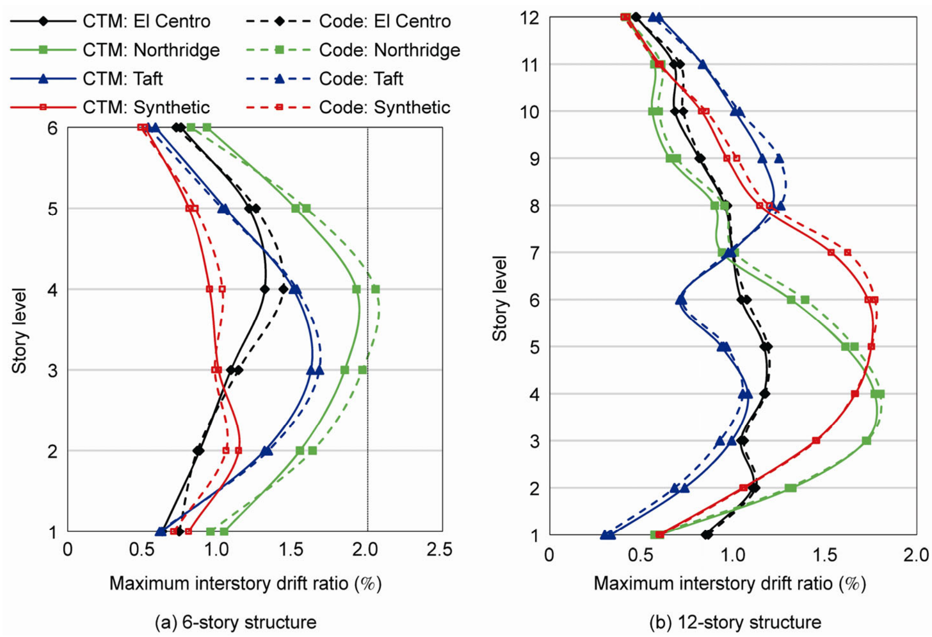

Figure 8 (Color online) Comparison of maximum interstory drift ratio between the code-based method and column tree method.

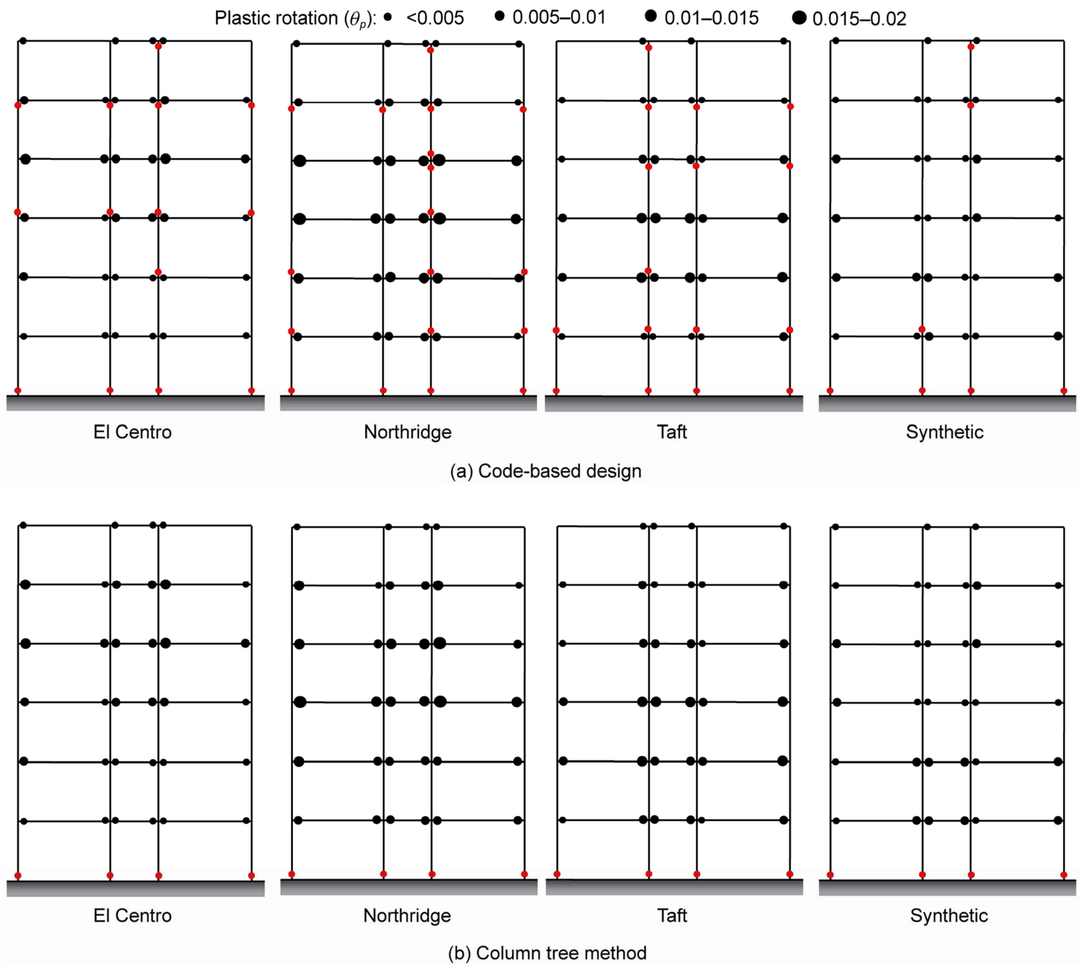

Figure 9 (Color online) Distribution of plastic hinges and the plastic rotation of 6-story structure.

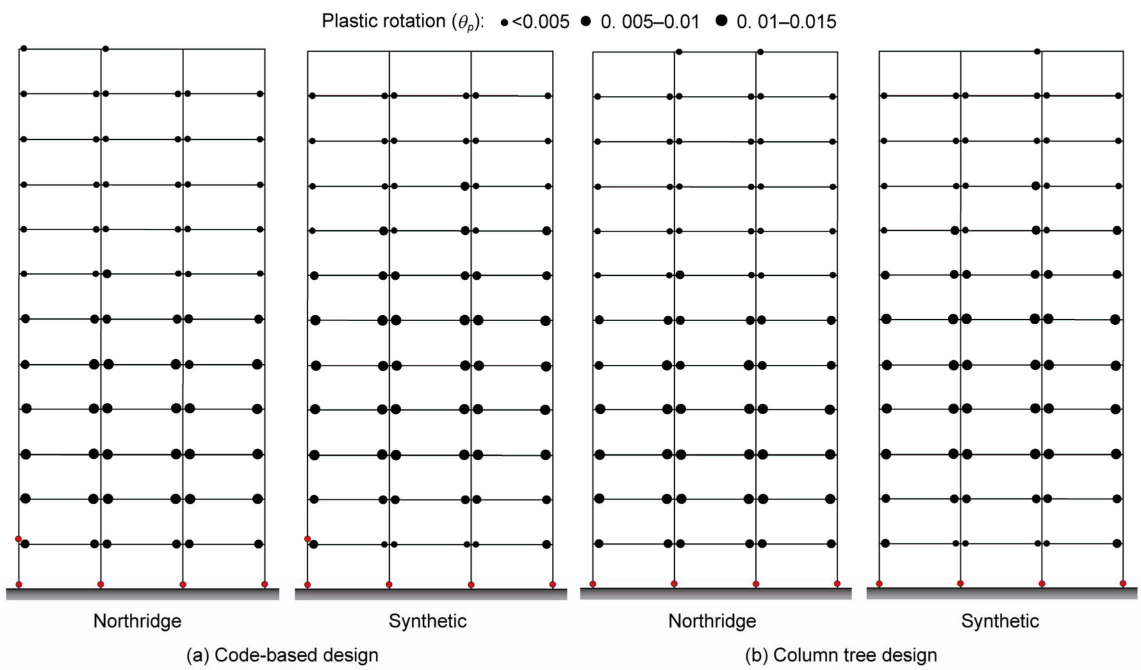

Figure 10 (Color online) Distribution of plastic hinges and the plastic rotation of 12-story structure.

based structures keep within elastic range under El Centro and Taft excitations, only the results for Northridge and synthetic excitations are illustrated. As can be seen, similar results can be observed with the 6-story structures. Plastic hinges of the columns bottom of the 2nd story for the code-based design are observed, while the column tree method can achieve the formation of global failure mechanism. These results indicate that CTM can avoid the yielding of columns to form the lateral collapse mechanism. Meanwhile, the proposed approach presented a potential attraction of building reparability characteristic from an economic perspective since repairing of the damaged columns is relatively difficult and costly.

Figure 11 shows the column moment demands for two CTM-based structures under 4 ground motions and their comparison with the design moment of column tree method. As illustrated in the Figure, the base column moments are larger than the design moments for some excitations. This means the yielding of base columns has occurred, which is the basic requirement of global yield mechanism. For other columns of both structures, the column moment demands are smaller than the design moment demands for CTM, which guarantees the columns within elastic range. These analytical results verify the effectiveness of the proposed method.

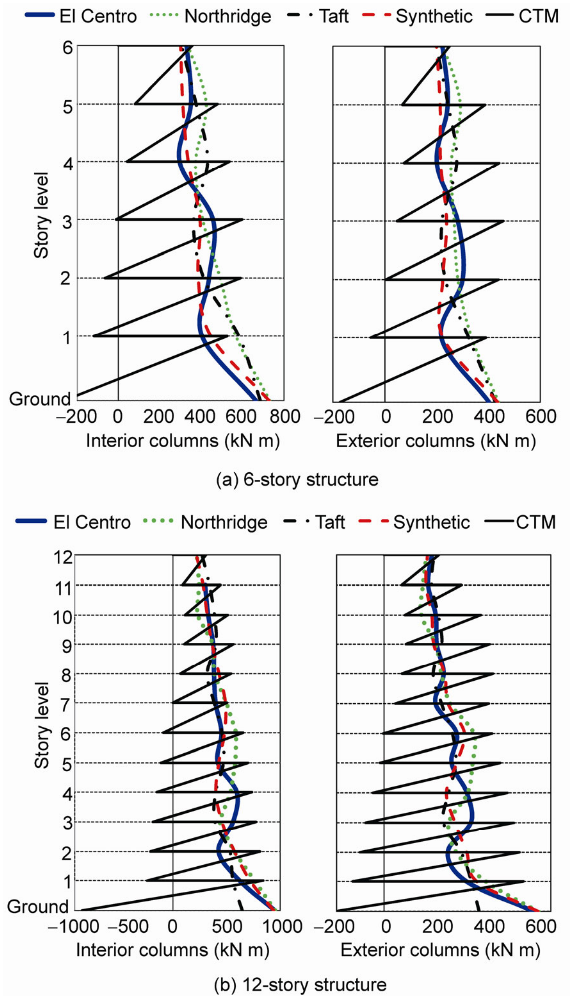

Figure 11 (Color online) Column moment demands of the CTM-based structures subjected to severe earthquakes.

5 Summary and conclusions

Structures have many potential seismic failure modes when subjected to severe earthquake shakings, and the global failure mechanism is the desired mode since it can provide higher total energy dissipation, more uniform story drift and higher resistance to earthquake loads. Traditional code-based seismic design follows the elastic analysis and equivalent static force procedure to derive the member internal force demands and capacity design method is used to amplify the column moment demands to achieve the global yield mechanism. Such design philosophy does not take the post-yield mechanism into account and does not consider the column moment demands in a global or system manner, so it is very difficult to ensure the structures designed using the code-based method yielding in the global mechanism. This paper proposes a simple but practical design procedure to achieve the global failure mechanism of RC frame structures by redesigning the columns using column tree method, and some conclusions can be drawn.

(1) The design of all beams and base columns must be predetermined, and using the code-based design results can represent element stiffness distribution, spatial configuration and the specific seismic hazard level that the structure is subjected to.

(2) The column tree method derives the column internal forced demands using a global manner to consider the yield limit state and strain-hardening of all beams and column bases. Furthermore, no iterations are required for the design procedure to fulfill the desired failure mode.

(3) The analytical results of the case study of 6- and 12-story structures illustrates that the proposed procedure can meet the story drift requirements and achieve the global yield mechanism for severe earthquake hazard, which indicates that the present approach can provide an effective way to the seismic failure mode control of frame structures.

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