Analysis of Strong Column and Weak Beam Behavior of Steel-concrete Mixed Frames

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SUMMARY:
According to the demand of seismic concept design of buildings, frame structures should have multiple lines of seismic resistance, one of which is the strong column-weak beam. But for RC frame, column plastic mechanism not the beam plastic mechanism was normally found in Wenchuan earthquake. So the problem about strong column-weak beam needs to be investigated in depth. For general mixed frame consisting of concrete-filled steel tubular (CFT) columns and steel-concrete composite beams (CB), research on their aseismic behavior is less and the method to ensure strong column-weak beam behavior is not available in China code. In this paper, pushover method is applied to analyze the damage mechanism of the CB-CFT column frame. The influence of ultimate moment ratio (column to beam) on the structural damage mechanism is studied. According to analysis results, the method by adjusting the elastic inner force for RC frame in China code can not give a guarantee to achieve the strong column-weak beam damage mechanism. Furthermore the design formula to make sure the strong column-weak beam mechanism of CB-CFT column frames is preliminarily put forward, which can offer some valuable references in design of composite frame structures.

Keywords: strong column-weak beam, mixed frames, ultimate moment ratio

1. GENERAL INSTRUCTIONS

A large number of buildings were destroyed in Wenchuan earthquake on May 12th, 2008. But for RC frame, column plastic mechanism not the beam plastic mechanism was normally observed in the earthquake. One of the main reasons was insufficiently considering the cast-in-place floor slabs effect on the strength and stiffness of beams (Wang, 2008). According to the demand of seismic concept design of buildings, frame structures should have multiple lines of seismic resistance, one of which is the strong column-weak beam. It is achieved by improving the bearing capacity at column end near the node for RC frame structures and steel frame structures in Chinese code (GB50011-2010). For general steel-concrete mixed frames, studies on the problem of strong column-weak beam is less and the method to ensure strong column-weak beam behavior is not available in Chinese code. In order to make the mixed frame structure system having good ductility and energy dissipation capacity under earthquake action, a practical method needs to be developed to realize strong column-weak beam behavior of mixed frame structures.

For general mixed frames consisting of concrete-filled steel tubular (CFT) columns and steel-concrete composite beams (CB), the method to achieve strong column-weak beam behavior is not available. Pushover method is applied to study the failure mechanism of the CB-concrete-filled square steel tubular (CFST) column frame in this paper. The influence of ultimate moment ratio (column to beam) on the structural yielding mechanism is discussed.

2. THE DEFINITION OF STRONG COLUMN-WEAK BEAM

Because structural members have no strength reserve in strong earthquake actions, the actual moment at beam and column end is equal to its flexural capacity. 'Strong column-weak beam' means that the actual flexural capacity of beam end M^{a}_{by} and M^{a}_{cy} of column end at the node should meet the following inequality:

$$M^{a}_{cy} > M^{a}_{by} \tag{2.1}$$

Because of the complication of earthquake, the influence of floor slab and the overstrength of yield strength of steel bar, it is very hard to achieve the concept design by accurate calculation. So for RC frame structures, it is achieved by improving moment design value at column end according to different anti-seismic grade and fortification intensity in Chinese code for seismic design of buildings (GB50011-2010). The design value of moment at column end is adjusted according to following Eqn. 2.2 in code for seismic design of buildings:

$$\sum M_{c} = \eta_{c} \sum M_{b} \tag{2.2}$$

The frame structures of anti-seismic grade 1 and in 9 degree seismic precautionary intensity are allowed not to comply with Eqn. 2.2, and but should satisfy the following Eqn.:

$$\sum M_{c} = 1.2 \sum M_{bua} \tag{2.3}$$

Where $\sum M_c$ is the sum of moment design value of the top and bottom column end on the node. $\sum M_b$ is the sum of moment design value of the left and right beam end on the node. The moment design value of the top and bottom column end can be apportioned by elastic analysis. $\sum M_{bua}$ is sum of the actual flexural capacity of beam end on the node. η_c is the column-end moment magnification factor; η_c=1.7, 1.5, 1.3, 1.2 for frames of anti-seismic grade 1, 2, 3, 4 respectively.
The revision of Chinese code for seismic design of buildings in 2010 improved the column-end moment magnification factor and added column-end moment magnification factor for the frame of anti-seismic grade 4. But even if frame structures were designed according to strong column-weak beam, it would be possible to appear column plastic mechanism not the beam plastic mechanism under strong earthquake action. For RC frame structures, the different values of moment magnification factor are proposed in literatures, and generally greater than those given in the code (Ye and Ma, 2010; Han and Li, 2010; Guan and Du, 2009; Dooley and Bracci, 2001).

3. STRUCTURE MODEL

The two-span and three-story composite beam-CFST column plane frame is shown in Fig. 3.1. And the composite beams are designed by full shear connection. The length of beam span is L, and the height of the story is H. Accordingly, a lateral distributed loading with inverted triangle pattern based on first mode is adopted to perform the pushover analysis.

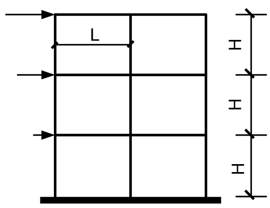


Figure 3.1. Sketch of elevation and load pattern

4. THE INFLUENCE OF ULTIMATE MOMENT RATIO（STRENGTH RATIO）

When conducting ultimate moment ratio analysis, the story height H of models is 3.6m; length of the section side of concrete-filled steel square tubular (CFST) column is 400mm; the section of composite beam is HN400×200; the width and thickness of RC flange is 1400mm and 100mm respectively.

The ultimate moment in the negative region of composite beams is far less than that in the positive region. And the negative region of beam-end would suffer damage earlier under earthquake action. Moreover ultimate moment of CFST column is related to the axial compression ratio. For comparison's purpose, ratio of ultimate moment M_{cua} of CFST column under pure bending to that M'_{bua} of composite beam in the negative region, denoted by β_c, is used as ultimate moment ratio, shown in Eqn. 4.1.

$$\beta_c = M_{\mathrm{cua}} / M'_{\mathrm{bua}} \tag{4.1}$$

Different ultimate moment ratios are achieved by changing material strength, span and quantity of reinforcement in concrete flange plate of composite beam and material strength and wall thickness of steel tubular of CFST column. And the line stiffness ratio of beam to column is kept a constant 0.5. Pushover analysis is conducted when β_c= 0.8, 1.0, 1.2, 1.6, 2.0 and 2.2 respectively. The standard value of dead load is 3.5kN/m, and the standard value of live load is 2.8 kN/m on the beams. The structure is applied one time dead load and live load before pushover analysis. And in the course of analysis, P-Δ effect is considered.

Capacity curves are showed in Fig. 4.1. for different ultimate moment ratio β_c. With the change of ultimate moment ratio from 2.2 to 0.8, the ductility of structures becomes poor. When β_c＞1.2, ductility coefficient reduces slowly with the reduction of ultimate moment ratio within a certain range. When β_c changes from 1.6 to 1.2, ductility coefficient rapidly decreases.

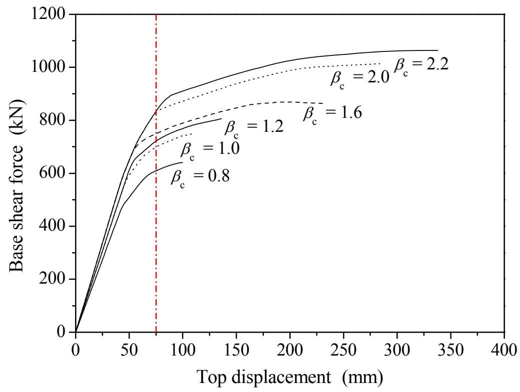
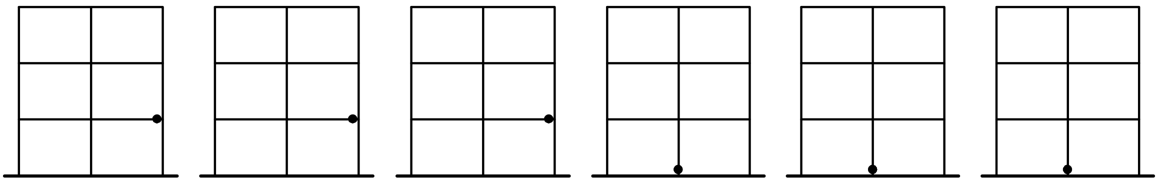


Figure 4.1. Comparisons of capacity curves for different β_c

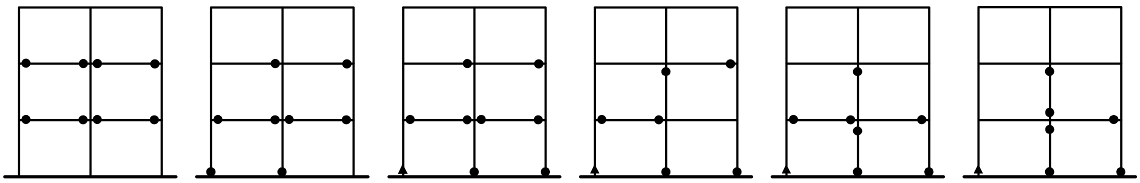
The position of the first plastic hinge, the damage state when the structural top displacement is 75mm (corresponding to the dotted line in Fig. 4.1.) and the ultimate failure mode of structures for different ultimate moment ratio are showed in Fig.4.2. to Fig.4.4.



(a) 2.2 (L=4.5m) (b) 2.0 (L=4.5m) (c) 1.6 (L=4.5m) (d) 1.2 (L=5.0m) (e) 1.0 (L=5.3m) (f) 0.8 (L=6.15m)

●：yielding

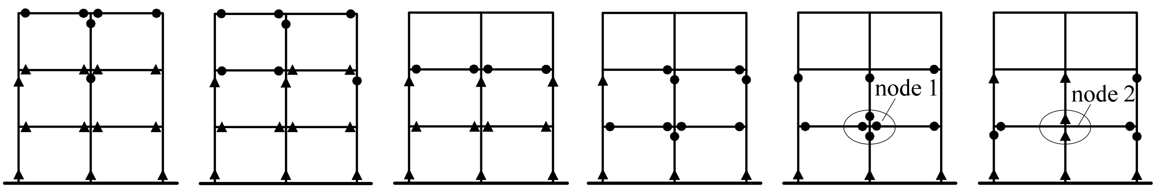
Figure 4.2. Location of the first plastic hinge for different β_c



(a) 2.2 (L=4.5m) (b) 2.0 (L=4.5m) (c) 1.6 (L=4.5m) (d) 1.2 (L=5.0m) (e) 1.0 (L=5.3m) (f) 0.8 (L=6.15m)

●：yielding ▲：ultimate

Figure 4.3. Comparison of destruction state under top displacement 75mm for different β_c



(a) 2.2 (L=4.5m) (b) 2.0 (L=4.5m) (c) 1.6 (L=4.5m) (d) 1.2 (L=5.0m) (e) 1.0 (L=5.3m) (f) 0.8 (L=6.15m)

●：yielding ▲：ultimate

Figure 4.4. Comparison of ultimate failure modes for different β_c

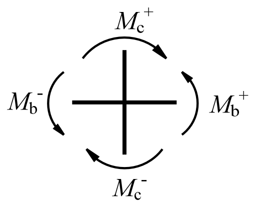
In contrast Fig.4.1. with Fig. 4.2.~Fig. 4.4., the ductility of structures decreases, columns suffer damage more and more seriously and beams suffer damage more and more lightly with the change of β_c from 2.2 to 0.8. For $\beta_c \geq 1.6$, the plastic hinge firstly appears in the beam, then the structure forms beam-hinge failure mechanism and finally the structure forms mechanism and reaches ultimate state with the deformation increasing and columns end failed in Fig. 4.2.~Fig. 4.4. For $\beta_c \geq 2$, plastic hinges at beam ends all develop well. For $\beta_c = 1.6$, the bottom two storeys of the frame forms local failure mechanism and the structural ductility is not as good as entire failure mechanism. For $\beta_c = 1.2$, the plastic hinge firstly appears at the bottom of column, then at the beam end, and finally at the top of column. The frame is the mixed failure mechanism. For $\beta_c = 1$, the plastic hinge firstly appears at the bottom of column too, and but then at the beam end and the top of column almost at the same time. For $\beta_c = 0.8$, the frame forms the column hinge mechanism. Corresponding to Fig. 4.1., the ductility of structures is very poor for $\beta_c = 1$, 0.8.

β_c is adopted the ratio of ultimate moment M_{cua} of pure bending columns to the ultimate moment M'_{bua} in the negative region of composite beams. Because actual flexural capacity (ultimate bearing capacity) of composite beams in positive moment region is larger than that in the negative moment region, it is possible that the actual flexural capacity M_{cy}^{a} in column ends is smaller than that M_{by}^{a} in beam ends at the interior node of rigid connections, as shown node 1 in Fig.4.4. (e).

From the above analysis, ultimate moment ratio (column to beam) $\beta_c \geq 1$ can not ensure that the actual flexural capacity M^{a}_{by} of beam ends and M^{a}_{cy} of column ends at the node meets inequality 2.1; in the other words, it can not ensure the structure achieves strong column-weak beam yield mechanism. In order to obtain the strong column-weak beam yield mechanism, β_c should take the value bigger than 1. β_c=1.2 is the boundary value of the two yield mechanisms of the examples in this paper.

For the steel-concrete composite node, the method by adjusting the elastic inner force for RC frame (Eqn. 2.2) is used to verify whether it can meet the requirements of strong column-weak beam. Take node 2 in Fig. 4.4. (f) as an example, the change of internal forces of beam-column joint of the composite frame is showed in Table 4.1. with the developing of plastic hinge.

Table 4.1. The Change Of Internal Force And Actual Bearing Capacity Of Node 2

Moment Sketch	Frame State	M_c^+	M_c^-	M_b^+	M_b^-
	before appearance of column hinge	281.93	249.19	243.95	287.17
	appearance of hinges of two columns	300.12	298.57	276.35	322.33
	maximum bearing capacity of structure	400.10	401.20	443.33	357.97
	actual bearing capacity	405.72	410.15	685.10	495.73

Firstly, according to Eqn. 4.2, take the moment at beam ends as the sum of elastic design moments before the appearance of column plastic hinge, namely $\sum M_b = 531.12$ kN. For η_c=1.5 (for the frame of anti-seismic grade 2), $\sum M_c = 1.5\sum M_b = 796.68$ kN. It is obvious that the adjusted moment design value at column ends is smaller than the sum of the moment at column ends (400.10+401.20=801.30 kN) of the maximum bearing capacity of the structure. Therefore, if the columns are designed by the adjusted internal force, it can not give a guarantee to achieve the strong column-weak beam yield mechanism for different anti-seismic grade frames.

If the internal force of columns is adjusted according to Eqn. 4.3, namely $\sum M_c = 1.2\sum M_{bua} = 1416.88$ kN, far more than the sum of the moment of column ends at the maximum bearing capacity, it can ensure to achieve strong column-weak beam yield mechanism. But it is too conservative and would cause unnecessary expenses for the low anti-seismic grade frame structures or the structures in low-intensity area. For $M_c = 1.2M'_{bua} = 594.88$ kN, the sum of the adjusted moment design value at column ends is $\sum M_c = 2M_c = 1189.76$ kN, more than the sum of ultimate moment at beam ends $\sum M_{bua}$ =1180.83 kN, which can ensure to achieve strong column-weak beam yield mechanism for the frame in this paper.

From the above analysis, the method that the ultimate moment ratio β_c and the actual ultimate bearing capacity at beam ends are used to adjust the design internal forces of columns is preliminarily suggested. And the suggested design method can be expressed as

$$M_c \ge \max(\beta_c M'_{bua}, 0.5\sum M_{bua}) \qquad (4.2)$$

Where M_c is the design moment of columns, and $\sum M_{bua}$ is the sum of ultimate bearing capacity in the positive moment region and negative moment region of composite beam. And β_c is suggested to be 1.2.

The method of Eqn. 4.2 is used to verify the composite frames of β_c = 2.2、2.0、1.6、1.2、1.0. When β_c is not less than 1.6, Eqn. 4.2 can’t be satisfied, structures all form strong column-weak beam yield mechanism and have good ductility. When β_c = 1.0, it is obvious that Eqn. 4.2 can not be satisfied. When β_c = 1.2, M_c = $1.2M'_{bua} < 0.5\sum M_{bua}$; Eqn. 4.2 can’t be satisfied and the frame is the mixed failure mechanism.

Eqn. 2.2 given in Chinese code is design formula for internal forces that is the design internal forces of columns is adjusted according to the calculated internal forces of beams. And it neither achieves the goal of strong column-weak beam nor clearly explains the seismic demand of columns for steel-concrete composite frame structures. In aseismic design, to achieve the goal of strong column-weak beam, it should ensure that the flexural capacity of columns (ultimate bearing capacity) is more than the flexural capacity of beams (ultimate bearing capacity) and not the design moment of beams (the calculation of elastic response) at nodes. Eqn. 2.3 comes from actual bearing capacity of beams, which can guarantee to realize strong column-weak beam mechanism for composite frames. But it may be too conservative and uneconomical for all the frames.

5. CONCLUSIONS

The problem about strong column-weak beam of CB-CFST column composite frames is analysed in the paper. The influence of the ultimate moment ratio (column to beam) on structural failure mechanism is discussed. According to analysis results, the method by adjusting the elastic inner force for RC frame can not give a guarantee to achieve the strong column-weak beam yield mechanism. A design method to achieve strong column-weak beam for composite frames is preliminarily advised, which is applicable within certain axial compression ratio, and helpful for the other types of composite frames. But as the behavior of composite member is largely different, further experimental research and theoretical analysis is needed for composite frames with different section compositions.

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