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# EFFECTS OF COLUMN CAPACITY ON THE SEISMIC BEHAVIOR OF MID-RISE STRONG-COLUMN-WEAK-BEAM MOMENT FRAMES

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## ABSTRACT

The strong-column-weak-beam (SCWB) design philosophy where plastic deformation is allowed to occur only in the beams while the columns remain essentially elastic is widely adopted in seismic design codes for steel moment frames. Under this yield mechanism, the seismic energy is dissipated primarily by the plastic hinges in the beams and at the column bases. The relative plastic flexural capacities of the beams and column bases can significantly affect the response of a SCWB frame. This paper investigates the inelastic behavior of SCWB frames with different distributions of beam and column plastic strength at different ductility demand levels. 9-story steel frames were designed as SCWB frames with different strength distributions. A series of nonlinear static and dynamic analyses were carried out to investigate the response of the frames. The results indicate that the deformation and the behavior of the frames strongly depend on the relative strength distribution of the column bases and the beams. The results provide a design guideline to select an optimum strength distribution for a SCWB frame.

## INTRODUCTION

Steel moment frames are widely used as the seismic load resisting system. Research studies in the past have shown that strong-column-weak-beam (SCWB) frames where the plastic deformation is allowed to occur only in the beams while the columns remain essentially elastic provide better response and energy dissipation than the weak-column-strong-beam (WCSB) frames (Roeder et al., 1993). The condition of SCWB is generally enforced by requiring moment capacity checks locally at beam-to-column joints and by designing the columns for a load combination considering the overstrength in the system (AISC, 2010a). The distribution of plastic beam moments to the columns is generally not specified. The design is generally done based on the elastic distribution or by using the plastic analysis of a sub-structure assuming inflection points at mid-height of the columns and at the center of the beams. Research in the past has shown that these traditional approaches may not be adequate and may not be able to capture the response of the columns in the inelastic range (Bondy, 1996, Nakashima and Sawaizumi, 2000, Medina and Kwawinkler, 2005).

In a more sophisticated design, a method based on a plastic mechanism analysis can be used (Goel and Chao, 2008). Alternatively, a pushover analysis may also be utilized to obtain the design moments. The analysis was done by pushing the frame assuming elastic columns up to the required ductility level. However, the designer still needs to assign the relative beam and column strength

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distribution prior to the analysis. One of the aspects that has not been appreciated much in the design is how the the relative beam and column strength distribution can be properly assigned. Under this chosen yield mechanism the seismic energy is dissipated primarily by the plastic hinges in the beams and at the column bases. The relative plastic flexural strengths of the beams and the column bases can have significant effects on the response of a SCWB frame.

This paper investigates the inelastic behavior of SCWB frames. The analysis focuses on a parameter called work ratio ($\omega_r$) which is defined as the ratio of the plastic energy dissipated by the columns at the bases to the total energy dissipated in a SCWB mechanism. A 9-story structure was selected and used as an example. The six frames were designed as SCWB frames by using the plastic design method with the $\omega_r$ ranging from 0.15 to 0.4. A series of nonlinear static and dynamic analyses were performed to investigate the deformation and the response of the moment frames. The key results are presented and discussed. Finally, a design guideline to select an optimum strength distribution to achieve appropriate SCWB structural behavior is provided.

## PLASTIC BEHAVIOR OF SCWB FRAMES

An example of a frame with a SCWB plastic mechanism is shown in Fig.1. Assuming that the distribution of the lateral forces remains unchanged and the gravity loads are small, the following equation can be obtained.

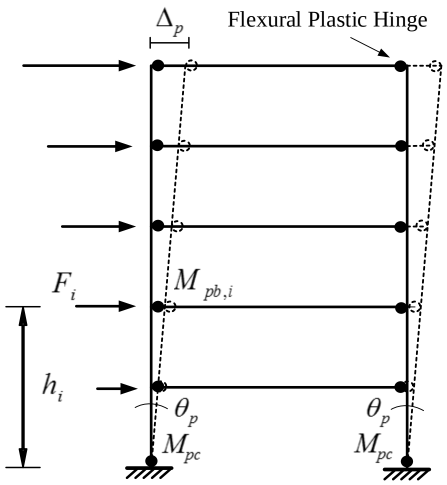


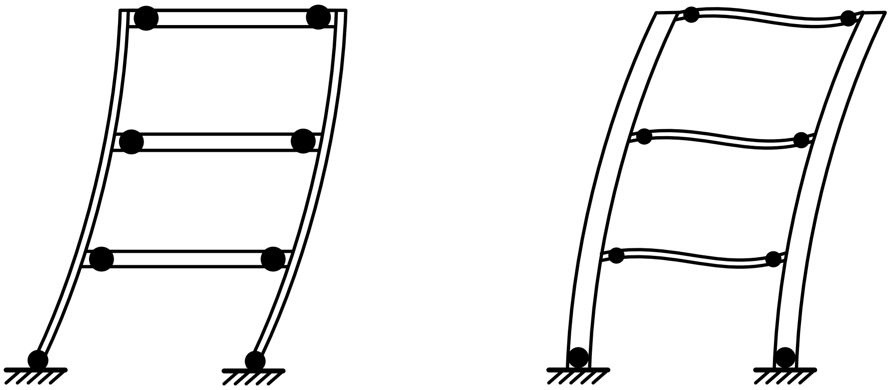

Figure 1. Global plastic mechanism of moment frame according to SCWB philosophy

$$\sum F_i h_i \theta_p = \sum M_{pc} \theta_p + \sum M_{pb,i} \theta_p \quad (1)$$

where $F_i$ is the lateral force at level $i$, $h_i$ is the height from the ground to level $i$, $M_{pc}$ is the plastic moment of the columns at the bases, $M_{pb,i}$ is the plastic moment of beams, and $\theta_p$ is the plastic rotation of the frame.

It can be seen that the overall lateral strength of the frame depends on the combination of the beam and column plastic strengths. Different distributions of beam and column plastic strengths can be assigned to achieve the same lateral frame strength. While the lateral strength remains relatively unchanged, the deformation pattern of the frame may vary significantly depending on the strength distribution of the beams and the column bases. Consider these two extreme cases illustrated in Fig.2. If the beams are substantially large while the columns are small, the relatively stiff beams restrict the bending of the columns in the upper stories. The deformation of the columns in this case tends to concentrate in the lower stories particularly in the first story. The plastic hinges in this case may form at the bases very early in the response and may lead to large plastic rotation demands of the columns.

The collapse of this frame would be most likely governed by the failure of the columns. On the other hand, if the opposite is true, i.e., the beams are relatively small while the columns are significantly large, the axial forces in the columns created as a result of the beam shear would be small leading to a small coupled resisting force. The overturning resistance has to be provided primarily by the bending moment in the columns. The deformation tends to concentrate in the upper stories similar to that of a shear wall. The plastic hinges form quickly in the beams while the columns remain mostly elastic. The failure of the frame in this case would be governed by that of the beams.



(a) Frame with strong beam (b) Frame with strong column

Figure 2. Structural deformed shape of moment frames

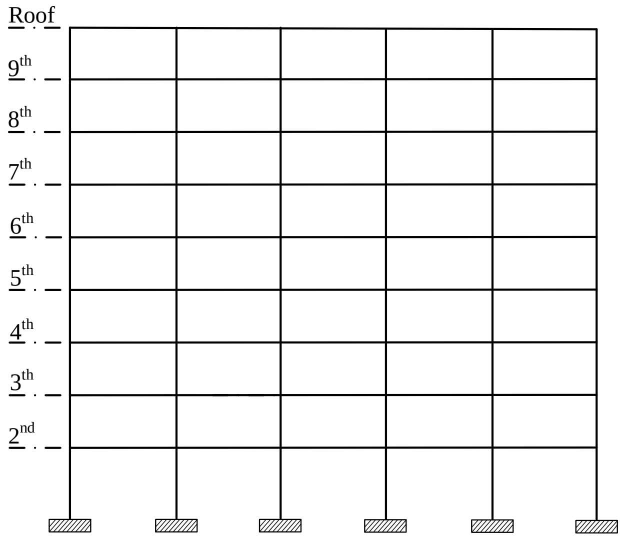
In an actual frame, the distribution of the beam and column strength is somewhere between these two extreme cases. Ideally, the distribution should be provided such that the deformation is uniform along the height of the frame which would minimize damage concentration in the beams or in the columns. In this paper, the distribution of the beam and column strengths is defined by a parameter called work ratio ($\omega_r$) which is the ratio of the energy dissipated by the columns at their bases to the total absorbed energy in a SCWB mechanism.

$$\omega_r = \frac{\sum M_{pc}}{\sum M_{pc} + \sum \left(M_{pb,i}\right)} \tag{2}$$

When $\omega_r$ value approaches zero, the frame behaves as in Fig.2(a). If it approaches one, the frame behaves as in Fig.2(b). Based on the moment frames used in several past studies (Gupta and Krawinkler, 1999, Shen et al., 2000, Jin and El-Tawil, 2005, Xu et al., 2006, and Choi et al., 2013), typical value of $\omega_r$ for mid-rise moment frames designed to satisfy SCWB condition is in the range of 0.15 but it can be as low as 0.1.

## EXAMPLE STRUCTURE

In this study, a benchmark 9-story moment frame structure shown in Fig.3 was selected as an example. The frame was similar to the one used in an earlier study but without the basement. The descriptions of the building can be found elsewhere (Gupta and Krawinkler, 1999). The example structure was assumed to be located on a soil profile type D. The spectral acceleration for design was 0.33g with an estimated period of 1.9s. Based on this frame geometry, six different moment frames were designed using six different values of $\omega_r$. All the frames were designed to have a similar level of overall lateral strength. The frames were designed using the plastic design method (Goel and Chao, 2008). They were designed with a target maximum story drift ratio of 2% for the design basis earthquake (DBE) hazard level having a 10% probability of exceedance in 50 years. This maximum story drift was specified in order to determine the level of the frame overall lateral strength.




Dimension: center line measurement

| | |
|---|---|
| ground level height | 5.49 m |
| $1^{st}$-$8^{th}$ level heights | 3.96 m |
| bay width | 9.14 m |

Seismic mass (tons):

| | |
|---|---|
| $1^{st}$ level | 504 |
| $2^{nd}$-$8^{th}$ level | 496 |
| $9^{th}$ level | 534 |

Figure 3. Nine-story example moment frame

The values of $\omega_r$ considered in this study included 0.15, 0.2, 0.25, 0.3, 0.35, and 0.4. The lower bound and upper bound values were selected such that the sizes of the columns at the bases would remain within a practical range. Once $\omega_r$ was selected, the plastic strength values of the beams and the column bases were assigned. The columns above the bases which were intended to remain elastic were designed using pushover analysis. The structure was pushed to the 2% roof drift by assuming elastic columns except at the bases where the plastic hinges were allowed to form. The moment and axial force diagrams of the elastic columns were then used to select the member sizes. The column design was carried out in accordance with section H1.3 of the AISC (2010b). Design results (member sizes) are given in Tables 1 and 2 for the beams and the columns, respectively. All the frames satisfied the joint column-to-beam strength ratios as required by the AISC seismic provisions (AISC, 2010a) for special moment frames.

Table 1. Beam design results

| Floor level | Work Ratio ($\omega_r$) | | | | | |
|---|---|---|---|---|---|---|
| | 0.15 | 0.20 | 0.25 | 0.30 | 0.35 | 0.40 |
| Roof | W21x57 | W21x57 | W21x57 | W21x50 | W21x50 | W21x44 |
| 9 | W24x76 | W24x76 | W21x73 | W21x73 | W21x73 | W21x62 |
| 8 | W24x94 | W24x94 | W24x84 | W24x84 | W24x76 | W21x73 |
| 7 | W27x94 | W27x94 | W24x94 | W24x94 | W24x84 | W24x76 |
| 6 | W27x102 | W27x102 | W27x94 | W27x94 | W24x94 | W24x84 |
| 5 | W30x108 | W30x108 | W27x102 | W27x102 | W27x94 | W24x94 |
| 4 | W30x108 | W30x108 | W30x108 | W27x102 | W27x94 | W24x94 |
| 3 | W30x108 | W30x108 | W30x108 | W27x102 | W27x102 | W27x94 |
| 2 | W30x116 | W30x116 | W30x108 | W30x108 | W27x102 | W27x94 |

Table 2. Column design results

| Story | WR | | | | | |
|---|---|---|---|---|---|---|
| | 0.15 | | 0.2 | | 0.25 | |
| | Exterior | Interior | Exterior | Interior | Exterior | Interior |
| 8-9 | W24X68 | W24X84 | W24X68 | W24X117 | W24X84 | W24X162 |
| 6-7 | W24X94 | W24X146 | W24X146 | W24X207 | W24X131 | W24X207 |
| 4-5 | W24X146 | W24X229 | W24X229 | W24X279 | W24X162 | W24X229 |
| 2-3 | W24X176 | W24X229 | W24X250 | W24X279 | W24X229 | W24X335 |
| 1 | W24X207 | W24X250 | W24X250 | W24X335 | W24X306 | W36X330 |
| Story | WR | | | | | |
| | 0.3 | | 0.35 | | 0.4 | |
| | Exterior | Interior | Exterior | Interior | Exterior | Interior |
| 8-9 | W24X94 | W24X162 | W24X104 | W24X162 | W24X94 | W24X146 |
| 6-7 | W24X131 | W24X207 | W24X117 | W24X192 | W24X104 | W24X162 |
| 4-5 | W24X162 | W24X229 | W24X162 | W24X279 | W24X176 | W24X279 |
| 2-3 | W24X250 | W30X357 | W24X250 | W36X330 | W24X279 | W30X357 |
| 1 | W24X335 | W36X395 | W24X370 | W36X487 | W36X330 | W36X529 |

2D analytical models of the frames were created. The models included lumped gravity columns for considering the P-Δ effect. Both material and geometric nonlinearity were considered. Beam-column members with lumped plasticity were used. A bilinear moment-rotation relation with 2% strain hardening was applied for all the plastic hinges. For dynamic analysis, the Rayleigh damping with 2% damping in the first and the third modes were used. Seven different ground motions were used. Their response spectra compared with the design spectrum are shown in Fig.4. All the analyses were done using Perform 3D computer software (CSI, 2007).

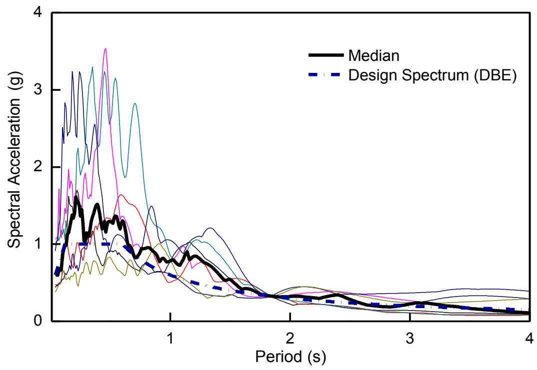



Figure 4. Response spectra of 7 ground motions and the DBE design spectrum

## PERFORMANCE EVALUATION

First mode pushover analysis and nonlinear dynamic analysis were performed to assess the performance of the frames with different $\omega_r$. Fig.5 shows the base shear versus roof drift plots of the six frames. In the elastic range, all the models had approximately the same elastic stiffness and comparable response in all the cases. The differences became visible as the frames entered the inelastic range. The first yielding occured between 0.8-1.0% drift. The structures reached a similar level of lateral load resistance but with different response behavior. After reaching the peak, the lateral strength decreased because of the influence of P-Δ effect. It can be observed that for the frames designed with high $\omega_r$ the response was more stable. The post-peak behavior was affected by the P-Δ to a lesser degree compared to the frames designed with low $\omega_r$. For the frame with the $\omega_r$ of 0.15, there was a rapid decrease in the lateral strength. For frames with low $\omega_r$, the yielding began early in the columns which implied that the collapse behavior of these frames would likely be influenced by the deformation capacity of the columns.

Beyond the $\omega_r$ of 0.25, the columns were relatively large and the yielding did not occur until later in the response. The lateral strength for these frames appeared to be somewhat lower than those of the frames with low $\omega_r$ in the beginning but gradually increased until reaching comparable values later on in the response. For these frames, the positive post-yield stiffness was the result of the columns that still remained largely elastic. In addition, the lateral resistance gradually increased after the beams yielded due to strain hardening in the plastic hinges. The frames did not reach their peak strengths until the drifts were approximately 2%. The deformation was primarily concentrated in the beams and the collapse behavior in these cases would be dictated by the failure of the beams. The $\omega_r$ value of 0.25 appeared to be the threshold between the two types of frame plastic behavior discussed earlier.

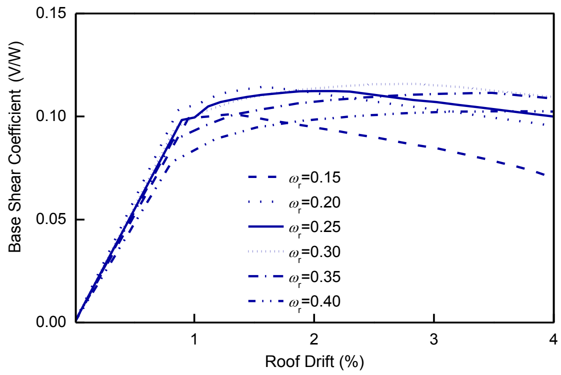


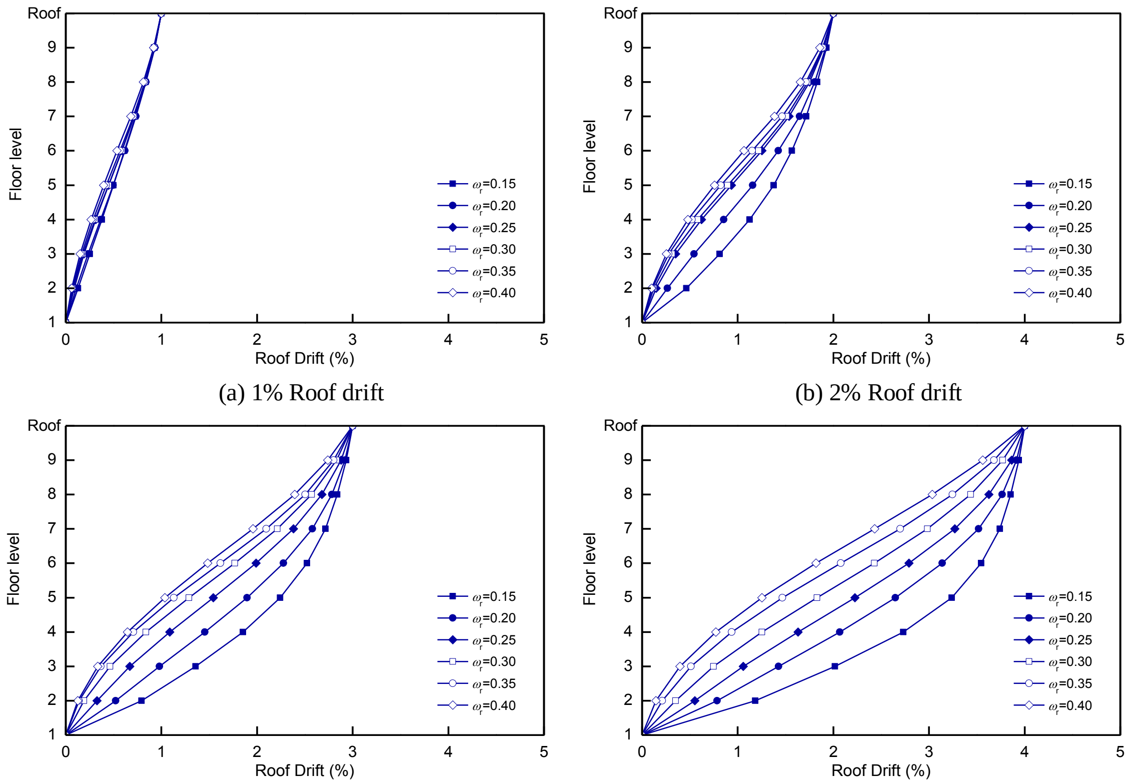

Figure 5. Base shear versus roof drift plot from pushover analysis

The deformed shapes of these frames at different roof drifts are shown in Fig.6. For the roof drift of 1%, the deformation pattern appeared to be similar regardless of the value of $\omega_r$. As the roof drift increased, the deformation pattern began to change. For frames with $\omega_r$ of 0.15-0.20, the deformed shape resembled the shape illustrated in Fig.2(b) indicating a weak column behavior. On the other hand the frames with $\omega_r$ more than 0.25 showed a characteristic of strong column behavior. It should be stressed that all these frames satisfied the joint column-to-beam strength requirements and yet the behavior were drastically different.

The influence of column strength on the response is better viewed in terms of a drift concentration factor (DCF) (MacRae et al., 2004). This normalized interstory drift ratio is defined by the interstory drift at a given story level divided by the roof drift (the roof displacement divided by the total building height)

$$DCF = \frac{\theta_i}{\theta_R} \tag{3}$$

where $\theta_i$ is the interstory drift at level $i$ and $\theta_R$ is the roof drift.

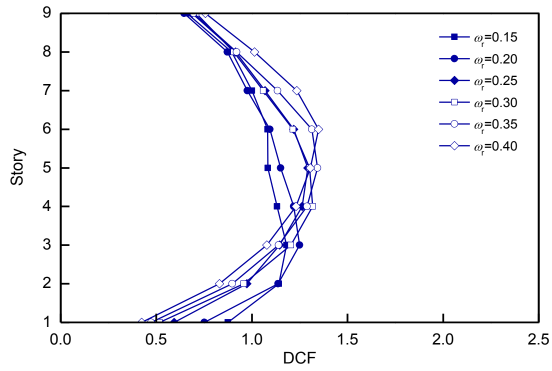



(a) 1% Roof drift (b) 2% Roof drift

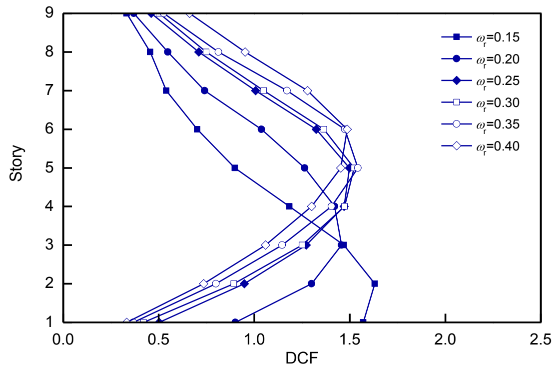
(c) 3% Roof drift (d) 4% Roof drift

Figure 6. Deformed shapes from pushover analysis at different roof drifts

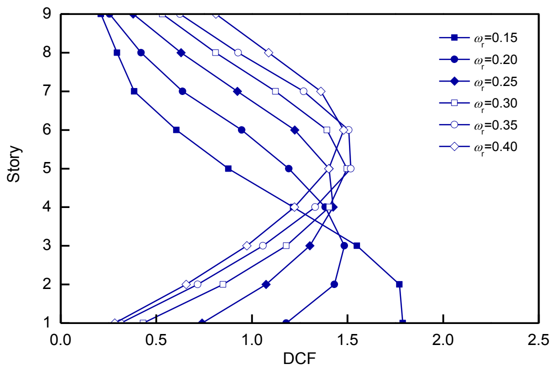
The plots of *DCF* values at different response levels are shown in Fig.7. At 1% roof drift, there appeared to be no appreciable differences in the response. At 2% roof drift, the frames with low $\omega_r$ began to show a concentration of story deformation in the lower stories. At this level, the frames with high $\omega_r$ showed relatively small differences in their response. At 3% drift, there were large differences in the response among the frames. For frames with small $\omega_r$, the deformation primarily concentrated in the lower stories, whereas for frames with large $\omega_r$, the concentration could be found mostly in the upper stories. Ideally, it is desirable to have a uniform distribution of story drift along the height of the frame. The analysis results strongly suggest that local joint column-to-beam strength requirement alone, as specified in some design codes, may not be sufficient to ensure a desirable strong-column-weak-beam behavior if the frame is expected to deform into the inelastic range. The ratio of the base column strength relative to the overall beam strength, as indicated by the $\omega_r$ factor, is also an important parameter to consider in the design. For a common design target drift of approximately 2%, $\omega_r$ value of less than 0.25 will result in frames with columns that are relatively too light and may lead to damage concentration in the lower stories. For a larger target drift, the $\omega_r$ must be properly assigned depending on the value of the target drift.



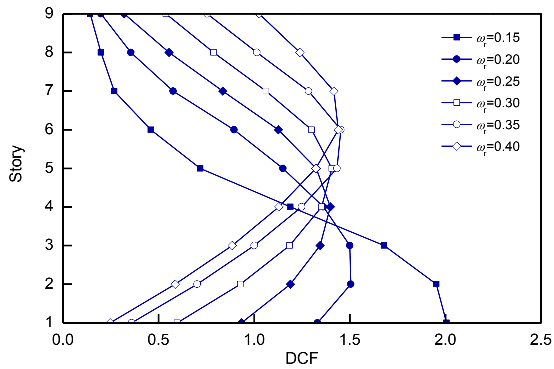

(a) Roof drift 1%




(b) Roof drift 2%



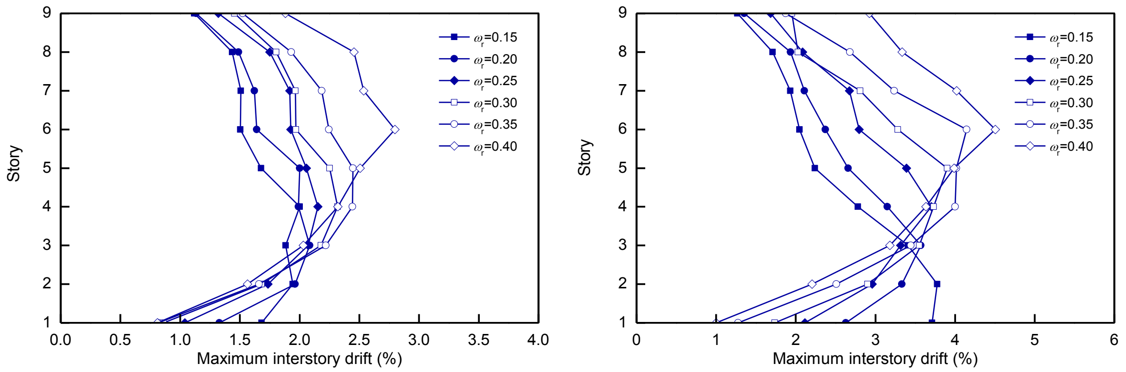

(c) Roof drift 3%




(d) Roof drift 4%

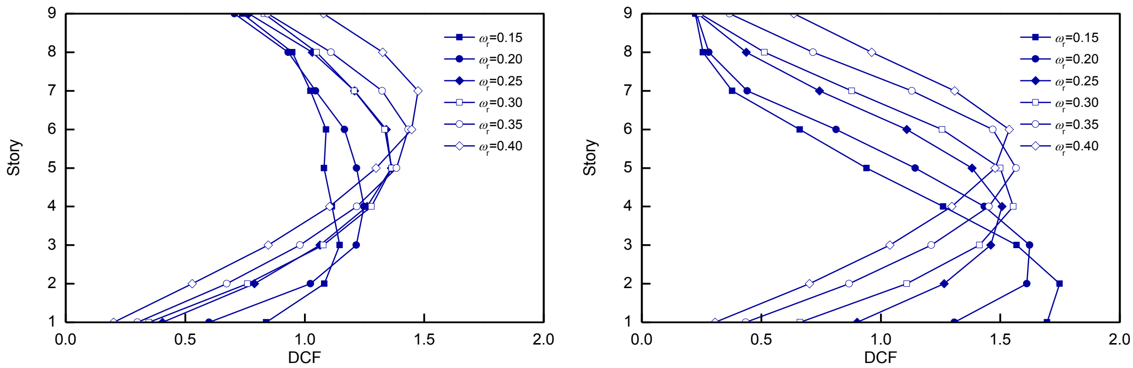
Figure 7. Drift concentration factors from pushover analysis at different roof drifts

Fig.8 shows the key results from nonlinear dynamic analyses at the design basis earthquake (DBE) and maximum considered earthquake (MCE) intensities. The figure shows median maximum interstory drifts of the frames under seven ground motions used in this study. The average roof drifts are in the vicinity of 1.5% and 2.5% for the DBE and MCE levels respectively. The *DCF*s for these two earthquake intensity levels are given in Fig.9. The overall behavior correlates well with the pushover results. In terms of story drift concentration, for low level of deformation, all the frames responded roughly in a similar manner. The effect of $\omega_r$ began to intensify as the deformation became larger. For the MCE level, the frames with low $\omega_r$ showed a concentration of deformation in the lower stories. The opposite was true for the frames with high $\omega_r$. These results again highlighted the importance of the ratio of the column base strength to the overall beam strength in SCWB frames.




(a) DBE (b) MCE

Figure 8. Maximum interstory drifts from time-history analysis at DBE and MCE levels




(a) DBE (b) MCE

Figure 9. Drift concentration distribution from time-history analysis at DBE and MCE levels

## CONCLUSIONS

Frames with different values of $\omega_r$, defined as the ratio of the energy dissipated by the columns at the bases to the total absorbed energy in a SCWB mechanism, were designed and analyzed. Nonlinear static and dynamic analyses were performed to investigate the effects of column strength on the response of SCWB moment frames. The main findings are as follows.

1. The relative plastic flexural strengths of the beams and the column bases can significantly affect the response of SCWB frames particularly in the inelastic range of deformation.
2. The local joint column-to-beam strength ratio may be an important factor for SCWB design, but it may not be sufficient to depend on the check of this ratio alone. To ensure a desirable behavior, the ratio of the base column strength relative to the overall beam strength, as indicated by the $\omega_r$ factor, must also be considered in the design.
3. For the frames with low $\omega_r$, the response was sensitive to the P- $\Delta$ effect. The response was characterized by the weak column behavior and the deformation tended to concentrated in the lower stories. For the frames with high $\omega_r$, the response was strongly influenced by the columns that remained mostly elastic. The deformation would concentrate in the upper stories. The value of $\omega_r$ must be carefully chosen to ensure a desirable SCWB behavior. The $\omega_r$ factor and column design must be carried out based on the level of expected deformation.
4. Based on the moment frames used in this study, for the level of drifts expected in a common design case (approximately 2%), the $\omega_r$ factor of 0.25 appears to be a suitable value. For a larger deformation level, no specific value of $\omega_r$ can be assigned. The selection of $\omega_r$ value depends on the level of deformation.

## ACKNOWLEDGEMENT

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