PERFORMANCE-BASED DESIGN OF MID-RISE BUCKLING-RESTRAINED KNEE BRACED TRUSS MOMENT FRAMES

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Abstract

This paper presents an application of a newly developed seismic resistant buckling-restrained knee braced truss moment frame (BRKB-TMF) structural system for mid-rise buildings. The design of this system is based on a design approach called the performance-based plastic design (PBPD) method modified to account for the characteristics of mid-rise frames. The design focuses on eliminating damage concentration, particularly in the first story. At the design state, the plastic strengths of the columns are carefully chosen to enhance the seismic behavior of a mid-rise building and to prevent a soft-story mechanism. The concept of the BRKB-TMF system is first described. The design and analysis of example mid-rise frames are then presented. A parametric study was carried out to find a suitable design approach for mid-rise frames. Nonlinear dynamic analyses were performed to investigate the response of the example frames. The analytical results illustrated that mid-rise BRKB-TMFs can be designed to provide excellent seismic behavior under both design basis earthquake and maximum considered earthquake intensities. The soft-story mechanism of the first story commonly found in mid-rise frames can be completely eliminated. Moreover, concentration of interstory drift in any 1 story can also be reduced.

Keywords: Performance-based plastic design, buckling-restrained brace, mid-rise buildings, soft-story mechanism, truss moment frames

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Introduction

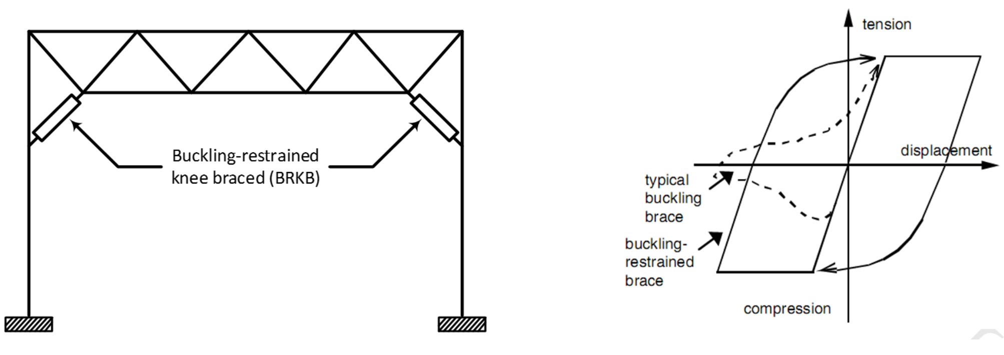
Recently, an innovative earthquake resisting structural system called buckling- restrained knee braced truss moment frames (BRKB-TMFs), as shown in Figure 1, has been introduced (Wongpakdee *et al.*, 2014). This BRKB-TMF system utilizes an open-web truss girder for less initial material costs especially for a long-span structure (Yang *et al.*, 2015). Ductility of the frames is provided by installing buckling restrained braces (BRBs) at strategic locations. BRBs are ideal components to dissipate seismic energy due to their full and stable hysteretic behavior as compared to conventional members (Figure 1(b)) (Clark *et al.*, 1999). For this structural system, the inelastic behavior is only allowed in the BRBs while all other members are designed to remain elastic. A BRKB-TMF system can be designed using a procedure called the performance-based plastic design (PBPD) method (Goel and Chao, 2008). Wongpakdee *et al.* (2014) applied this procedure to design a low-rise, 4-story, BRKB-TMF structure and evaluated the frame behavior using a series of nonlinear analyses. The example structure behaved in an excellent manner with the inelastic behavior and yield mechanism as prescribed in the performance-based design framework.

This study extends the application of BRKB-TMF to mid-rise structures. Further development of this structural system focuses on modifying the PBPD design procedure so that it is compatible with the characteristics of mid-rise frames. In a mid-rise frame, the influence of higher modes and P-delta effects become pronounced placing a significant burden on the columns particularly in the first story. This may result in a concentration of story deformation leading to a soft-story mechanism, a dominant collapse mode for a mid-rise frame. In the case of a BRKB-TMF, the concentration of story deformation leads to the fracture of BRBs which, in turn, triggers the soft-story mechanism.

In this paper, a parametric study is carried out to evaluate the design approach that is suitable for mid-rise frames. The design and analysis of example mid- rise frames are presented. Nonlinear dynamic analyses were performed to investigate the response of the example frames.

Performance-based Plastic Design of BRKB-TMFs

The PBPD method begins by selecting a preferred yield mechanism and a target drift of the structure (Goel and Chao, 2008). This selection specifies the maximum deformation limit allowed for the system. By determining the seismic energy dissipated by the structural members up to the selected target drift, the



(a) BRKB-TMF (b) Hysteretic comparison

Figure 1. BRKB-TMF system

required base shear strength can be calculated using the energy balance between the input and the dissipated energy (Figure 2). The resulting required base shear of the structure can be calculated using Equations 1 and 2:

$$V_y/W = \left(-\alpha + \sqrt{\alpha^2 + 4\gamma S_a^2}\right)/2 \quad (1)$$

$$\gamma = (2\mu_s - 1)/R_\mu^2 \quad (2)$$

where V_y is the required design base shear, W is the total seismic weight of the structure, α is a dimensionless parameter that depends on the natural period of the structure and the intended plastic drift ratio (Lee and Goel, 2001), S_a is the spectral response acceleration obtained from the code design spectrum, and γ is the energy modification factor. The energy modification factor (γ) depends on the structural ductility factor (μ_s) and the ductility reduction factor (R_μ) and can be calculated as Equation 2. The details are fully described elsewhere (Lee and Goel, 2001).

Once the required base shear has been determined, the sizes of the BRBs and the columns can be calculated based on the yield mechanism of the frame using the work-energy equation:

$$\sum_{i=1}^{n} F_i h_i \theta_p = 2M_{pc}\theta_p + \sum_{i=1}^{n} 2(\beta_i N_{BRB})\delta \quad (3)$$

where F_i is the lateral force corresponding to the design base shear for 1 bay at level i, h_i is the height from the ground floor to level i, θ_p is the design plastic drift of the frame, M_{pc} is the plastic moment of the columns at the bases, β_i is a factor indicating the distribution of the BRB strength along the height of the frame, N_{BRB} is the axial strength of the BRB at the roof level, and δ is the plastic deformation of the BRB which is a function of θ_p.

As can be seen, 2 structural members, the BRBs and column bases, are mainly responsible to dissipate seismic energy. In general, the required amount of plastic moment of column bases (M_{pc}) is first assigned and the BRB strengths are calculated using Equation (3). The size of the columns can be computed by finding the strength of the first story column that is strong enough to prevent the soft-story mechanism (Figure 3). Assuming that plastic hinges form at the top and base of the first story columns, the work equation for this mechanism can be expressed as:

$$(FS)V_1 h_{c1}\theta_p = 4M_{pc}\theta_p \quad (4)$$

$$M_{pc} = \frac{(FS)V_1 h_{c1}}{4} \quad (5)$$

where V_1 is the base shear for the equivalent 1 bay model, h_{c1} is the height of the first story, and FS is the factor of safety for the plastic

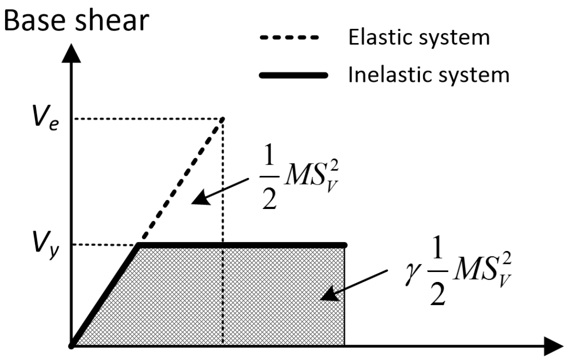


Figure 2. Energy-work balance concept

design (Leelataviwat *et al.*, 2002). For low-rise structures, the *FS* of 1.1 has been used satisfactorily to prevent the soft-story mechanism (Leelataviwat *et al.*, 2002). Once the column bases and BRBs have been designed, the truss members and columns, except at the bases, are

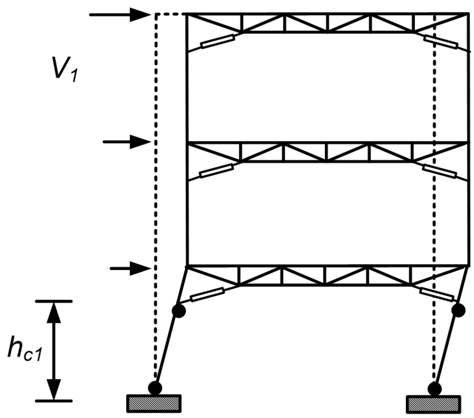


Figure 3. Soft-story mechanism of moment fram

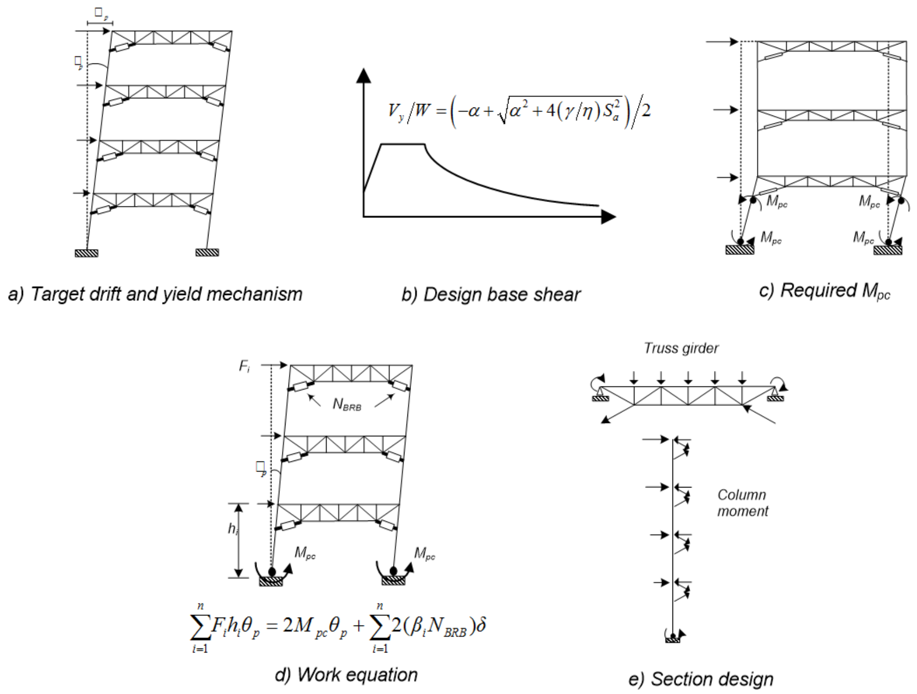


Figure 4. Design procedure of BRKB-TMF building

designed using plastic analysis and capacity design to ensure that all of the members remain elastic. The simplified design procedure of the BRKB-TMF structure are summarized as shown Figure 4.

PBPD Method for Mid-Rise BRKB-TMFs and Parametric Study

As mentioned earlier, in a mid-rise frame, the influence of higher modes and P-delta effects become pronounced placing a significant burden on the columns particularly in the first story. To avoid the soft-story collapse, a safety margin (FS in Equation 5) larger than 1.1 may be needed. An FS as high as 1.5 has been reported (Bayat, 2010).

To find a suitable FS for the plastic design for mid-rise BRKB-TMFs, a parametric study was carried out by varying the FS value as 1.1, 1.5, and 2.0. A 9-story structure similar to the SAC building (Gupta and Krawinkler, 1999) was selected as an example structure of a mid-rise building for this parametric evaluation. The building plan and elevation view of the building are shown in Figure 5. The typical bay width and story height were 9.14 and 3.96 m. except for the first story where the height was 5.49 m. The perimeter moment resisting frames were designed as the BRKB-TMFs.

The example building was designed for the maximum considered earthquake (MCE) with 2% probability of exceedance in 50 years and the design basis earthquake (DBE) defined to be 2/3 of the MCE intensity. The intensities were specified using the design spectrum in American Society of Civil Engineers (2010). The design spectral acceleration value for the MCE level was 0.49g at the fundamental period of 1.83 s. The target drift was selected to be 2% at the DBE level. Yield drift was assumed to be 0.75%. The frames were designed using the design base shear (from Equations 1 and 2) of 6214 kN at the DBE level with FS values of 1.1, 1.5, and 2.0. All the structural members were designed using the specifications in American Institute of Steel Construction, Inc. (2010a). The required strengths of the BRBs, truss member sizes, and the column sections are listed in Tables 1, 2, and 3, respectively.

Nonlinear Analysis Results

2D frame models were created using the Perform 3D computer software (Computers and Structures, Inc., 2007). The analytical models included lumped gravity columns for

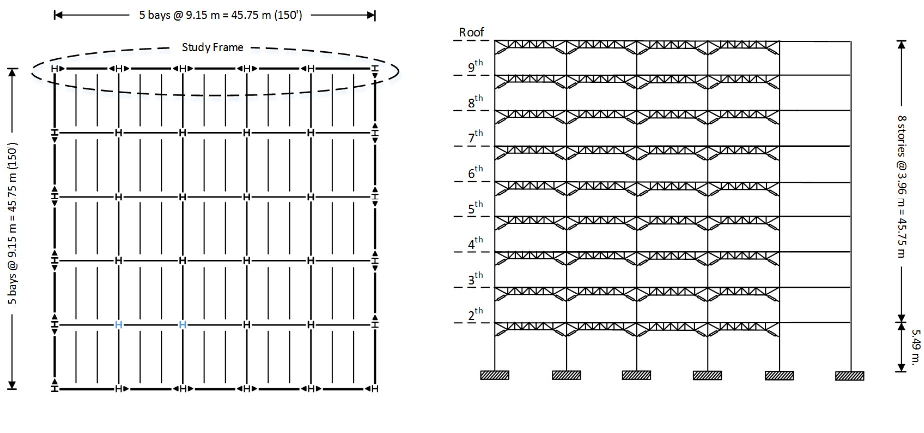


Figure 5. Example building

consideration of the P-Δ effect. Material and geometric nonlinearity were considered. For the dynamic analysis, Rayleigh damping with 2% damping in the first and third modes was used. The analysis was performed with a far-field record set (44 ground motions), as specified in FEMA P695 (Federal Emergency Management Agency, 2009) scaled to be matched with the design spectral acceleration at the fundamental period of the structure.

Table 1. Required strength of BRBs (kN)

Floor level	Factor of safety (*FS*)		
	1.1	**1.0**	**2.0**
Roof	374	356	334
9	556	529	498
8	698	658	623
7	805	765	721
6	894	845	796
5	961	912	859
4	1010	961	903
3	1045	992	934
2	1068	1014	952

Nonlinear Static Analysis Results

Pushover analysis results of the frames designed with the FS of 1.1, 1.5, and 2.0 are shown in Figure 6. The elastic stiffness of these 3 frames are almost the same. For all the frames, the BRBs started to yield at a similar roof drift of 0.5%. In the inelastic range, the frames reached similar maximum base shear coefficient values of 0.096, 0.092, and 0.086 for the design cases with the FS of 1.1, 1.5, and 2.0, respectively. After the yielding of the BRBs, the chord members started to plastify at the roof drift of approximately 1.25%. After that, the overall resistance of the frames gradually decreased until sudden strength degradation occurred due to the BRB fracture. For the frames with the FS of 1.1 and 1.5, plastic hinges of the column bases occurred at roof drifts of 1.0 and 1.75%, respectively. For the frame with the FS of 2.0, plastic hinges formed at a roof drift greater than the target drift of 2%.

Table 2. Truss members

Floor level	Factor of safety (*FS*)					
	1.1		**1.5**		**2.0**	
	Chord	**Diagonal**	**Chord**	**Diagonal**	**Chord**	**Diagonal**
Roof	2C150×15.6	2C150×15.6	2C150×15.6	2C150×15.6	2C150×15.6	2C150×15.6
9	2MC100×20.5	2MC150×17.9	2MC150×17.9	2MC150×17.9	2MC150×17.9	2MC150×17.9
8	2MC150×24.3	2MC150×17.9	2MC100×20.5	2MC150×17.9	2MC100×20.5	2MC150×17.9
7	2MC150×26.8	2MC150×17.9	2MC150×24.3	2MC150×17.9	2MC150×24.3	2MC150×17.9
6	2MC150×26.8	2MC100×20.5	2MC150×26.8	2MC100×20.5	2MC150×24.3	2MC150×17.9
5	2MC180×28.4	2MC100×20.5	2MC180×28.4	2MC100×20.5	2MC150×26.8	2MC100×20.5
4	2MC200×31.8	2MC150×24.3	2MC180×28.4	2MC100×20.5	2MC150×26.8	2MC100×20.5
3	2MC200×31.8	2MC150×24.3	2MC180×28.4	2MC100×20.5	2MC180×28.4	2MC100×20.5
2	2MC200×31.8	2MC150×24.3	2MC200×31.8	2MC150×24.3	2MC180×28.4	2MC100×20.5

Table 3. Interior and exterior column sizes

Stories	Factor of safety (*FS*)					
	1.1		**1.5**		**2.0**	
	Exterior	**Interior**	**Exterior**	**Interior**	**Exterior**	**Interior**
8-9	W460×97	W610×125	W460×97	W460×128	W610×113	W610×140
6-7	W610×125	W610×155	W610×140	W610×155	W610×155	W610×195
4-5	W610×217	W610×241	W610×241	W610×262	W610×217	W610×217
2-3	W610×241	W610×262	W610×285	W610×307	W610×307	W610×341
1	W610×285	W610×341	W610×307	W610×415	W610×372	W610×498

Figure 7 shows the values of the drift concentration factor (DCF) defined as the ratio of the interstory drift to the roof drift (MacRae *et al.*, 2004) at different roof drift values. At a small roof drift of 1.0%, the deformations of all frames were uniformly distributed and the values of the DCF in each story were quite similar regardless of the value of the FS used in the design. At 2% roof drift, the differences became visible in that the drift of the frame designed with the FS of 1.1 concentrated in the lower stories. At 3% roof drift, the response of each frame was significantly different from each other. For the frame designed with the FS of 1.1, the story drifts were significantly large in the first story which indicated a possible soft-story collapse. The maximum DCF values for the frames with the FS of 1.5 and 2.0 were similar but they were less than that of the previous case and were located in higher stories.

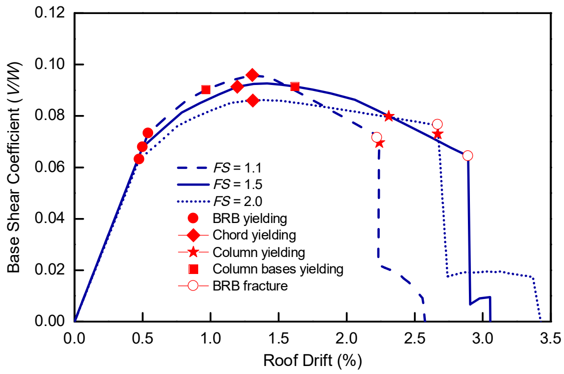
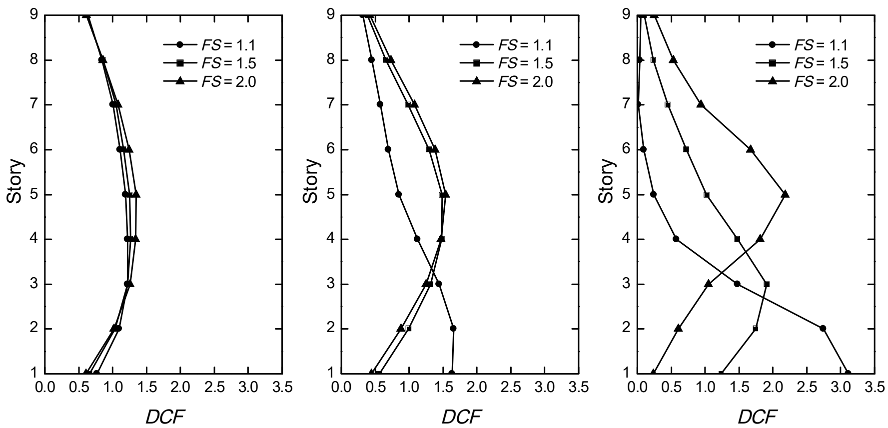


Figure 6. Results of pushover analysis in terms of base shear coefficient versus roof drift

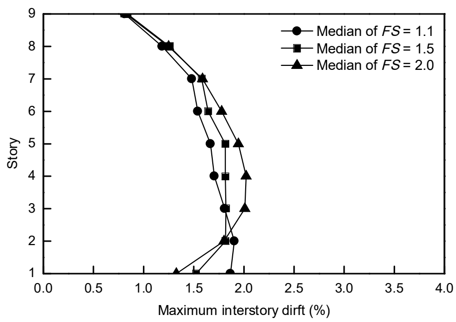


(a) Roof drift of 1% (b) Roof drift of 2% (c) Roof drift of 3%

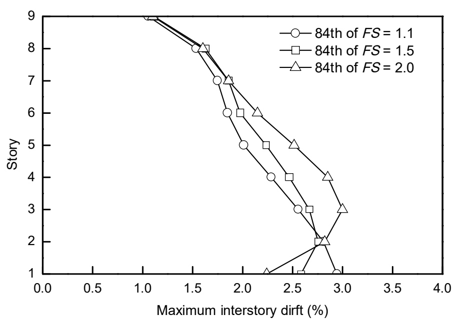
Figure 7. Drift concentration factor at different roof drifts

Nonlinear Dynamic Analysis

Time-history analyses were performed using 44 far-field records recommended by FEMA P695 (Federal Emergency Management Agency, 2009). At the DBE intensity level, the median value of roof drift is in the vicinity of 1.1%. According to Figure 7, there is no significant difference of the structural response at roof drift of 1%. Therefore, the analytical results in this part focus only at the MCE level. In addition, the structural response at the 84th percentile of the MCE intensity is illustrated for investigating the behavior of the structure at large deformation (Gupta and Krawinkler, 2000). The median and 84th percentile values for the maximum drifts at the MCE level are shown in Figure 8. The maximum roof drifts for each frame were approximately the same which were around 1.5 and 2.0%, at the median and 84th percentile, respectively. Figure 9 shows the DCF results from a representative selected ground motion which had the response in terms of maximum interstory drifts under this ground motion followed closely with those of the median and the 84th percentile maximum interstory drift results at the MCE level. In terms of the median value, the DCF in each story for all frames was still less than 1.5. However, it showed a tendency similar to the results from pushover analysis in that the response was concentrated in the first story for the case of

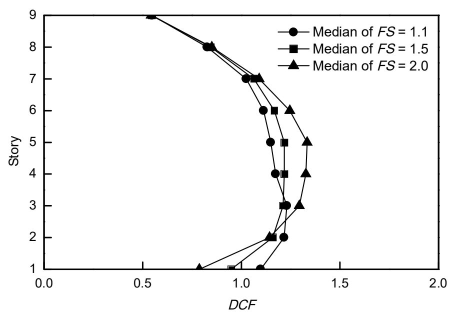


(a) Median

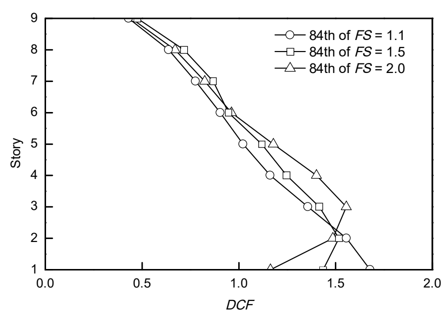


(b) 84th percentile

Figure 8. Median maximum interstory drifts from time-history analysis at the MCE intensity



(a) Median



(b) 84th percentile

Figure 9. Drift concentration factors of a selected ground motion at the MCE intensity

the FS of 1.1. This can be seen clearly when considering the response at the 84th percentile (Figure 9(b)).

Conclusions

This paper presents the performance-based plastic design (PBPD) procedure of mid-rise buckling-restrained knee braced truss moment frames (BRKB-TMFs). The parametric study was conducted to determine an appropriate design approach for mid-rise frames. The example frames were designed and were analyzed by nonlinear static and dynamic analyses to investigate the response. Conclusions regarding the results of the parametric study are as follows:

1. The PBPD procedure can be used effectively to design mid-rise BRKB-TMFs. With a minor modification, the PBPD design procedure developed earlier for low-rise frames can be applied for mid-rise frames. The example mid-rise frames used in this study showed excellent seismic response with all the inelastic response confined to the designated yielding members as envisioned.
2. The influence of higher modes and P-delta effects in mid-rise frames increases the deformation of the columns particularly in the first story. To avoid the soft-story failure, the frame should be designed with an increased safety margin against a soft story plastic mechanism. It was found that columns with a capacity 1.5 times larger than the value computed by assuming a soft-story plastic mechanism in the first story appear to be adequate for the level of drifts expected in a common design case (approximately 2%).

It should be noted that the suggested FS value was based on the example structure investigated in this study. Further investigations of the FS value for a wider range of structures are still needed.

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