17th World Conference on Earthquake Engineering, 17WCEE
Sendai, Japan - September 13th to 18th 2020

EFFECT OF PERCENTAGE COMBINATION RULES ON BUILDING COLLAPSE RISK

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Abstract

As prescribed by modern design codes and standards, orthogonal seismic effects can be considered by using the percentage combination rule (100%-*p*%), which takes the sum of 100% of the force demands in one direction and *p*% force demands in the other. The rule is typically applied to determine the design forces for individual structural components where, depending on the adopted code or standard, the *p*% value can range from as low as 30% (100-30 rule) to as high as 100% (100-100 rule). With the purpose of quantifying the effect of the adopted *p*% value on system-level performance, specifically structural collapse risk, a reliability-based methodology is proposed. The methodology is applied to an 8-story SCBF building case with corner columns shared by orthogonal braced frames. Three designs are developed for the building case using 100%-30%, 100%-65% and 100%-100% rule, respectively. Nonlinear structural models of the three designs are constructed in *OpenSees*. Response history analyses (RHAs) using bidirectional and unidirectional loading are performed to obtain the "true" orthogonal seismic force demand and the percentage-rule-based force demand in the shared columns, respectively. The RHA results indicate that providing insufficient strength in the shared columns determined by 100%-*p*% rule could have a substantial effect on collapse risk. This study provides a comparison between collapse fragilities for cases with and without considering shared column failure due to the underestimation of design forces based on the chosen *p%* value. Also, based on the proposed methodology, a quantitative relationship between the adopted *p*% value and seismic collapse risk is established.

Keywords: percentage combination rule; seismic risk; orthogonal seismic effect; SCBF; column failure

1. Introduction

Ground motions produced by earthquakes are normally decomposed into three orthogonal components acting on buildings simultaneously, two in horizontal direction and one in vertical direction. It is a common practice to determine the seismic demands by performing an independent seismic analyse in each principle direction of the structure and combine the responses from two horizonal directions accordingly. One example that require the consideration of orthogonal seismic effects, as per ASCE/SEI 7-16 [1], is the columns placed at the intersection of an orthogonal lateral-force resisting system (LFRS). Previous studies have emphasized the need to consider combinatorial seismic effects in orthogonal shared LFRS columns. Hisada et al.[2] was among the first to indicate that the axial force demands in shared columns under bidirectional seismic loadings were almost double to demands under unidirectional loading. Similarly, Bisadi and Head [3] found that the force and displacement demands in bridge columns under bidirectional seismic loadings were larger than demands under unidirectional loadings. And the angle of excitation is a major influential factor. In addition, Elkady and Lignos [4] experimentally validated the biaxially loaded column sustained more severe damage than uniaxially loaded column.

Several combination methods to account for orthogonal seismic effects have been prescribed in modern design codes and comprehensively studied in the past. The most widely accepted methods include percentage rules (100-*p*%), square root of the sum of the squares (SRSS) and three-dimensional complete quadratic combination (CQC3) methods. The percentage rule states that the design demand is to be taken as the sum of 100% response due to seismic loading acting in one direction and some percentage (p%) of the response due to loading in the perpendicular direction. The p% values of 30% and 40% were originally suggested by Rosenbluth and Contreras [5] and Newmark [6], respectively. The 100-30% rule is currently prescribed in seismic design provisions like ASCE/SEI 7-16 [1], NBCC [7], EN 1998-1 [8] while the 100-40% rule is mostly specified in design codes for bridges [9] and nuclear facilities [10]. It is worth noting that the AISC 341-10 [11] suggests 100-30% rule and in the recently released version of AISC 341-16 [12], the 100-100% rule is implied.

The effectiveness of percentage rules has been extensively studied in the past. For example, Reyes-Salazar et al.[13] assessed the 100-30% rule of predicting the force demands in steel frame columns and they concluded that the combined results underestimate the demands derived from bidirectional response history analyses. Similarly, it is found by Heredia-Zavoni and Machicao-Barrionuevo [14] that 100-30% and 100-40% rule could either overestimate or underestimate seismic demands in irregular frames. The accuracy of 100-30% rule is found to be sensitive to ground motion characteristics and structural configurations. In addition, Gao et al. [15] applied 100-30% and 100-40% rules to bridges and the obtained demands are conservative compared to bidirectional seismic analyses. Wang et al. [16] recently reviewed the prior studies regarding the evaluation of percentage rules. A key finding was that the component-level force demand was usually taken as the parameter of interest for evaluating the percentage rules. So far, there has been no effort to investigate the implication of the percentage rules to system-level performance. Therefore, this paper provides a reliability-based approach to building the link between seismic collapse risk and the p% value used in the percentage rule. The probabilistic relationship aims to inform the adequacy of the adopted percentage rule to the building collapse safety under earthquake. The methodology is introduced in the following section. Then, an 8-story special concentrically braced frame (SCBF) building is considered for application. The SCBF columns are designed by force demands from different percentage rules. For each design scheme, incremental dynamic analyses are conducted. The increased collapse risk due to underestimated column force demands are evaluated. And key findings are summarized in the Conclusion section.

2. Methodology overview

Percentage rules are used to estimate the component force demands under bidirectional seismic effects. An underestimated design force may result in failures of structural components and then, trigger the building collapse. The reliability-based methodology is presented to quantify the effect of the adopted p% value in the percentage rule on the building collapse risk. Fig.1 provides the flowchart of the methodology. During the reliability-based assessment, the force demand derived from nonlinear response history analyses (NRHAs) with simultaneous application of two horizontal ground motion components is assumed as the "true" demand, denoted as $P_{bi\text{-}dir}$ hereafter. An initial value of p% may first start with 30% and 40% which comply with the conventional practice. The force demand estimated from percentage rule, $P_{100\text{-}p\%}$ is determined using Eq.(1) where $P_{uni\text{-}x}$ and $P_{uni\text{-}z}$ represent force demand under X-unidirectional and Z-unidirectional loading, respectively. Then, the probability of percentage-rule-based demand exceeding the "true" demand is developed with the aid of parameter R_p (Eq.(2)).

$$P_{100-p\%} = \max\{P_{uni-x} + p\% \cdot P_{uni-z}, p\% \cdot P_{uni-x} + P_{uni-z}\} \tag{1}$$

$$R_p = \frac{P_{bi-dir}}{P_{100-p\%}} \tag{2}$$

The value of R_p is the ratio of the "true" demand (i.e. $P_{bi\text{-}dir}$) to the percentage-rule-based demand (i.e. $P_{100\text{-}p\%}$). An R_p value that is greater than 1 indicates the predetermined 100-p% rule fails to estimate the bidirectional seismic effect. If NRHAs are conducted with a set of paired ground motions, the probability P(R_p > 1) is determined based on the fraction of records that the corresponding analyses produce R_p > 1. In the next step, the collapse fragilities with and without considering the collapse mechanism due to percentage-rule-induced component failures are evaluated. The former case, P($C|IM$), is computed using Eq.(3) and the latter is the second term of Eq.(3).

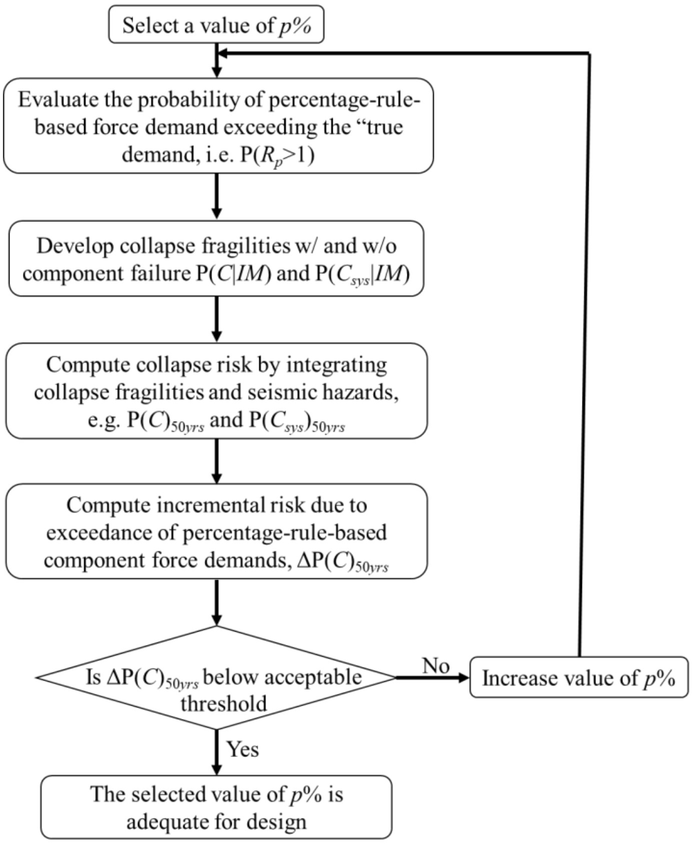


Fig. 1 Flowchart of the reliability-based assessment methodology

$$\mathrm{P}(C|IM) = \mathrm{P}\left(C_{sys}\middle|IM\right) + \mathrm{P}\left(C_{Col}\middle|\overline{C_{sys}}, R_p > 1\right)\mathrm{P}\left(\overline{C_{sys}}, R_p > 1\middle|IM\right) \tag{3}$$

where $\mathrm{P}\left(C_{sys}\middle|IM\right)$ and $\mathrm{P}\left(C_{Col}\middle|\overline{C_{sys}}, R_p > 1\right)\mathrm{P}\left(\overline{C_{sys}}, R_p > 1\middle|IM\right)$ represent the collapse probabilities conditioned on a certain shaking intensity with the consideration of system-level (i.e. dynamic instability or side-sway collapse) failure and component-failure-induced collapse, respectively. And $\mathrm{P}(C|IM)$ is the total collapse probability considering these two failure cases. As for the SCBF buildings, columns are designed as force-controlled components which means the strength of columns should be sufficient enough to maintain elastic under earthquakes. For braced columns shared by orthogonal frames, the combinatorial seismic effects should be considered. If the column strength as determined by 100-p% rule is not adequate, the failure of column component would occur and progressively cause building collapse. To this end, the R_p at shared columns is used to compute P($C|IM$) using Eq.(3).

The details for developing collapse fragilities in terms of $\mathrm{P}\left(C_{sys}\middle|IM\right)$ and $\mathrm{P}(C|IM)$ are given in Section 4.3. Afterwards, the building collapse risk can be obtained by integrating the fragility results and the seismic hazard curve. Analogous to $\mathrm{P}\left(C_{sys}\middle|IM\right)$ and $\mathrm{P}(C|IM)$, the collapse risk with and without considering column failure can be derived. The difference between these two is the risk increment due to the column failure caused by the inadequate design force demand following the predetermined 100-p% rule. If the estimated risk increment is beyond the acceptable threshold, the p% value shall be increased and repeat the analysis process. If not, the p% value can be directly used in design.

3. SCBF building design and numerical modeling

The methodology is applied to an 8-story SCBF building. The plan and elevation view are shown in Fig.2. The plan is 30 m × 50 m and the typical story height is 4 m. The structural design is conducted as per ASCE/SEI 7-16 [1], AISC 341-16 [12] and AISC 360-16 [17]. The building is assumed to be located at a high seismic-prone site in Los Angeles with Site Class D. The mapped spectral parameters are S_S = 2.17g and S_1 = 0.79g. Risk Category II and Seismic Design Category D are considered in the seismic design of the building. The response modification factor, R, overstrength factor, Ω_0, deflection amplification factor, C_D, and importance factor, I, are taken to be 6, 2, 5 and 1, respectively. Braces adopt rectangular hollow sections (HSS) fabricated with ASTM A500 grade C steel. Beams and columns use wide-flange sections conforming to ASTM A992 steel. The nominal yield stress of F_y = 345 Mpa is assigned to all structural components. Response spectrum analysis is used to compute the seismic demands. The seismic design is conducted and examined with the help of SAP2000 [18]. The design results regarding the brace and beam sections are indicated in Fig.2b.

Noted that the braced frames are located on the perimeter of the building and corner columns are shared by frames in the two principal directions (Fig.2a). Therefore, the design of corner columns has to account for the orthogonal seismic effects. As aforementioned, AISC 341-10 suggests 100-30% rule by referring to ASCE/SEI 7-10 whereas the AISC 341-16 implies using 100-100% rule for SCBF columns placed at the intersection of orthogonal braced frames. For the selected 8-story SCBF building, a total of three sets of columns are designed based on 100-30%, 100-65%, and 100-100% percentage rule. The reliability-based method presented in Section 2 is performed on each design scheme to build the relationship between the p% value and the collapse risk. With the purpose of isolating the effect of the considered p% value, the columns are designed to achieve a consistent safety margin in terms of demand to capacity ratio (DCR). The sectional dimensions of columns based on 100-30%, 100-65%, and 100-100% percentage rules are summarized in Table 1 and the corresponding DCR values are shown in Fig.3. It can be seen that the DCR results of the three designs are nearly the same.

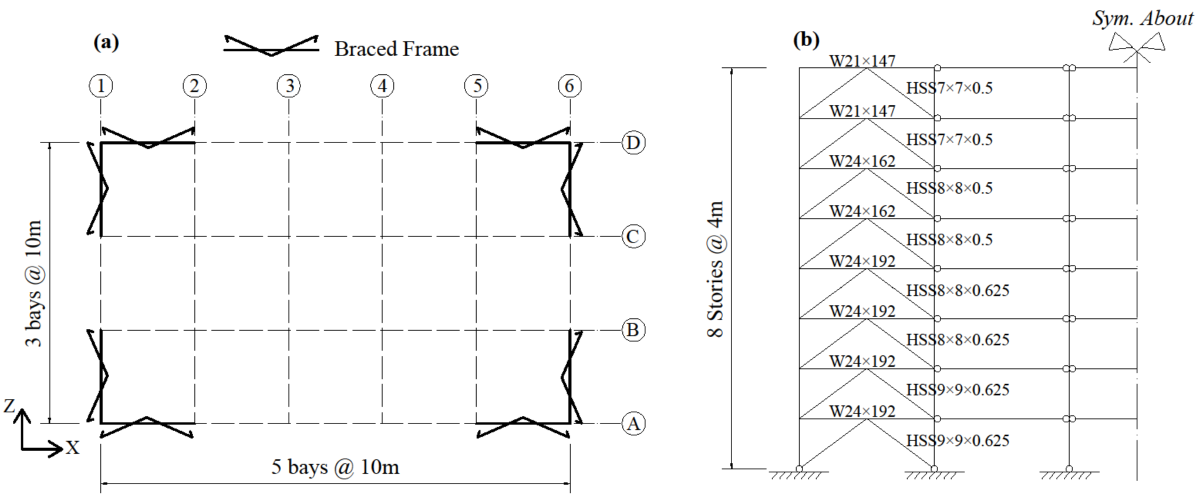


Fig. 2. The 8-story SCBF building views : (a) floor plan and (b) elevation

Table 1. The column sections based on different percentage rules

Story	100/30%-design	100/65%-design	100/100%-design
1-2	W14×342	W14×426	W14×500
3-4	W14×257	W14×342	W14×398
5-6	W14×193	W14×233	W14×283
7-8	W14×132	W14×159	W14×193

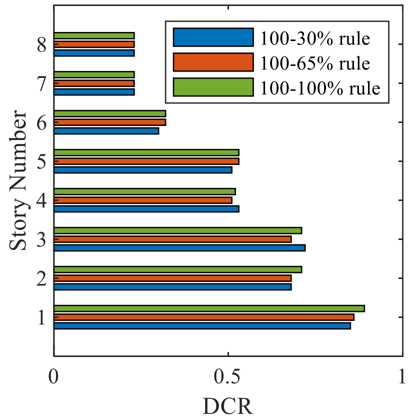


Fig. 3 DCRs of column design based on different percentage rules

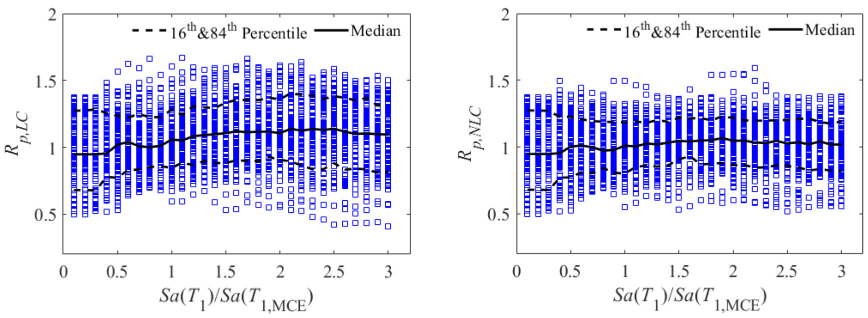
The numerical model of the SCBF building is built with the aid of Open System for Earthquake Engineering Simulation (*OpenSees*) [19]. The braced frames are explicitly modeled and the gravity frames are considered using a leaning column placed at the center of mass. The fiber sectional element is used for braces and the buckling behavior is simulated by assigning the brace element with an initial imperfection. Every brace element is divided into eight segments and each segment uses five integration points. Also, wide-flange columns and beams employ fiber discretization sections to simulate nonlinear behaviors. The Steel02 material available in *OpenSees* is adopted for fiber elements. The nominal yield strength, F_y = 345 Mpa, is assigned to beams and columns and expected yield strength, R_yF_y = 483 Mpa, is specified for braces, as per AISC 341-16. The strain-hardening ratio b is taken as 0.1%. The parameters related to kinematic hardening response are considered as : R_0 = 20, cR_1 = 0.925, and cR_2 = 0.25 and parameters for considering isotropic strain hardening response are : a_1 = 0.4. a_2 = 10, a_3 = 0.4, and a_4 = 10. Moreover, the fatigue material model together with Steel02 material is specified for brace elements to simulate low-cycle fracture failures. The corresponding model parameters are determined according to empirical equations provided by Karamanci and Lignos [20]. A 3% Rayleigh damping in correspond to the first and third modes is assumed for RHAs.

In order to evaluate the collapse risk with and without considering SCBF column failure, two models using different elements for columns are built. The first one (designated as NLC model hereafter) uses force-based element with fiber discretization section specified for beams, braces and columns. This model is developed to account for the effect of SCBF column failure on building collapse capacity. An initial camber is considered to simulate column buckling behavior as suggested by Imanpour et al [21]. For the second one (designated as LC model hereafter), columns are modeled using elastic elements while the beams and braces use the same nonlinear elements with NCL model. Such model ensures columns remain intact and the column-failure-induced collapse is excluded.

4. Application of the Reliability-based Methodology

4.1 Probability distribution of R_p values

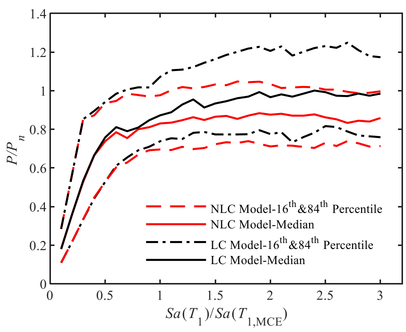
Incremental dynamic analyses (IDA) are performed on the building model to evaluate the probability distribution of the R_p values. The spectral acceleration corresponding to the first mode period, $Sa(T_1)$, is taken as the intensity measure (*IM*). An intensity range of 10% to 300% of the spectral acceleration at the maximum considered earthquake (MCE), $Sa(T_{1,\mathrm{MCE}})$, with 10% $Sa(T_{1,\mathrm{MCE}})$ increment is considered in IDA. The 22 pairs of ground motions provided in FEMA P695 guidelines [22] are used as input. The R_p values obtained from LC model and NLC model for all corner columns (shared by orthogonal frames) at the 1st story of the building along the considered intensity range are shown in Fig.4a and Fig.4b, respectively. In the figure, each blue rectangle represents an R_p value observed in an individual column under a pair of ground motion. Under each intensity, there are 88 (22 pairs of ground motions times 4 shared columns) R_p values. In addition, the 50th (median), 16th and 84th percentile values are plotted using solid and dash lines, respectively. The results of LC model (Fig.4a) show that the R_p values experience a slight increase from 0.95 at the lowest intensity to 1.16 at 260%$Sa(T_{1,\text{ MCE}})$. Fig.4b shows that the median values of R_p values from NLC model fluctuate around 1.0 which implies that there is a 50% probability that the 100-30% rule is adequate to estimate the axial force demands of columns. The difference between the two dash lines is used to quantify the dispersion of R_p distribution. It is noted that at intensities of 200% - 300% $Sa(T_{1,\text{ MCE}})$ the dispersion for LC model is around 0.52 while the value is approximately 0.35 for NLC model.



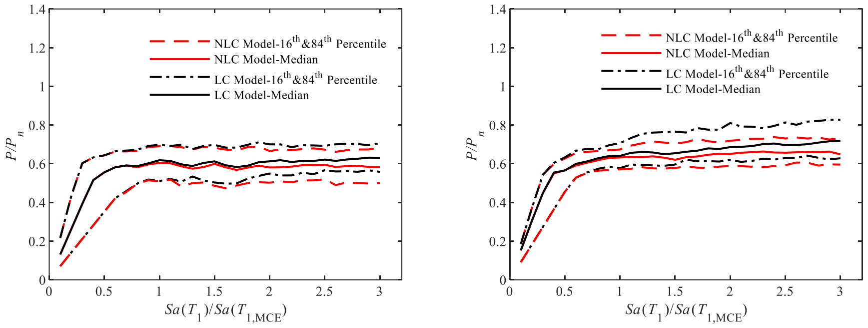
(a) LC Model (b) NLC Model

Fig. 4 Distribution of R_p values for shared columns at the 1st story of the building

The findings made from Fig.4 can be explained by Fig.5, which show the median, 16th and 84th percentile values of the axial force demand normalized by the nominal axial strength, P/P_n, in all shared columns at the 1st story of the 100-30%-rule-designed building. The Figs.5a, 5b and 5c compare the results derived from NLC and LC model under bidirectional loading, X-unidirectional loading, Z-unidirectional loading, respectively. It can be seen that the red and black lines in Figs.5b and 5c are close which implies columns remain basically elastic under unidirectional loadings. The bi-linear pattern is due to the yielding of adjoining braces. In contrast, under bidirectional loading, the black and red lines show a significant departure starting from 100% $S_a(T_{1,\text{MCE}})$ intensity. The 84th percentile values of P/P_n from LC model are beyond 1.0 while results from NLC model saturate to 1.0 when the intensity is beyond 100% $Sa(T_{1,\text{MCE}})$. Such difference is because the columns in NLC model undergo inelastic response. The narrowed gap between the lines of the median and 84th percentile P/P_n values for the NLC model is also an indicator of inelastic column response at higher intensities. The inelastic response of the shared columns under bidirectional loading leads to the reduced variations in $R_{p,NLC}$ at high seismic intensities (Fig. 4). The results from the LC model could reflect the exceedance of the percentage-rule-based force demands and hence the $R_{p,LC}$ is used in the following analyses.



(a) Bidirectional loading



(b) X-unidirectional loading (c) Z-unidirectional loading

Fig. 5 Effect of shaking intensity on P/P_n values of shared columns

4.2 Collapse mechanism induced by exceedance of percentage-rule-based force demands

For the considered SCBF building, there are four columns at each story shared by two orthogonal frames. Under a single pair of ground motions at 100% $Sa(T_{1,\mathrm{MCE}})$ intensity, each shared column would produce a unique R_p. Two examples are shown in Fig.6 where the $R_{p,LC}$ values are obtained by using the 100-30% rule (i.e. p = 30 in Eq.(1)). The corresponding P/P_n value for each column is also indicated in the figure. As seen from the figure, the $R_{p,LC}$ values show great variation among the four corner columns. For example, the minimum and maximum of $R_{p,LC}$ values as shown in Fig.6a are 0.73 and 1.44, respectively, with an average value of 1.08. For the results shown in Fig.6b, the $R_{p,LC}$ value varies from 0.79 to 1.21 with an average of 1.01. Such findings are consistent with prior studies such as Reyes-Salazar et al.[13] and Hernández and López [23] who concluded that the 100-30% rule could overestimate and underestimate force demands of shared structural components. The purpose of the proposed reliability-based method is to link the accuracy of the combination rules and seismic collapse risk. Therefore, the column with the largest P/P_n value is assumed to be the one most likely trigger building collapse and such column is denotated as "critical" column. When assessing the probability of collapse due to column failure using Eq.(3), the P(R_p > 1) is computed based on this critical column.

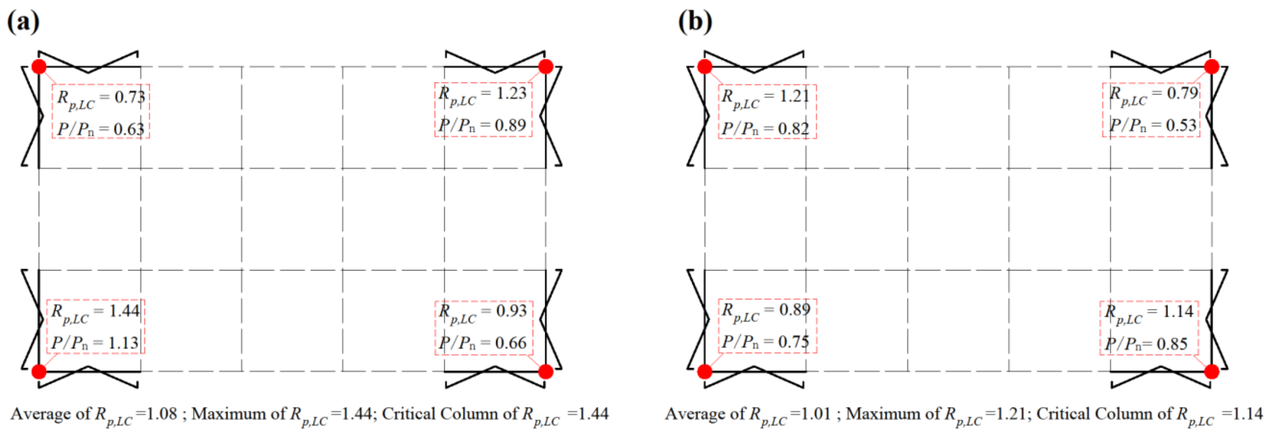


Fig. 6 R_p values observed from the shared columns at 1st story (each subfigure (a) and (b) represents the results for a single ground motion pair)

For the two examples presented in Fig.6, the $R_{p,LC}$ value in the critical columns is 1.44 and 1.14, respectively. For the former case, the corresponding P/P_n in the critical column is larger than 1.0 which implies inelastic response and compression failure may occur. For the latter, although P/P_n value is below 1.0 the $R_{p,LC}$ = 1.14 implies the underestimated axial force at the critical column facilities axial-flexure interaction failure. These two failure mechanisms for columns, specifically global compression buckling and axial-flexure interaction failure, could trigger the building structural collapse. In general, building collapse is assumed to happen when dynamic instability is observed or the drift demand for any column exceeds 10% [24]. The two types of failure mechanisms inferred from Fig.6 are verified by the RHA results from NLC model as shown in Fig.7. The blue counters are the story drift ratio history under bidirectional ground motions and the red dash lines exhibit the deformed shape of the critical column at the peak drift ratio. Recall that in Fig.6a the critical column has a $P_{bi\text{-}dir}/P_n$ value greater than 1.0, the corresponding deformed shape of the column as shown in Fig.7a is a buckled mode as the out-of-plane displacement at the middle is nearly 1.8 times the value at the top. For the other case (Fig.6b) where the critical column has a $P_{bi\text{-}dir}/P_n$ value smaller than 1.0, a typical axial-flexure interaction failure mechanism occurred on the column is observed as the X-directional drift exceeds 10% with minimal out-of-plane displacement along the height. Therefore, a critical column with $R_{p,LC} > 1$ would sustain either compression or axial-flexure interaction failure and afterwards trigger building collapse, thereby validating its use in the risk analysis.

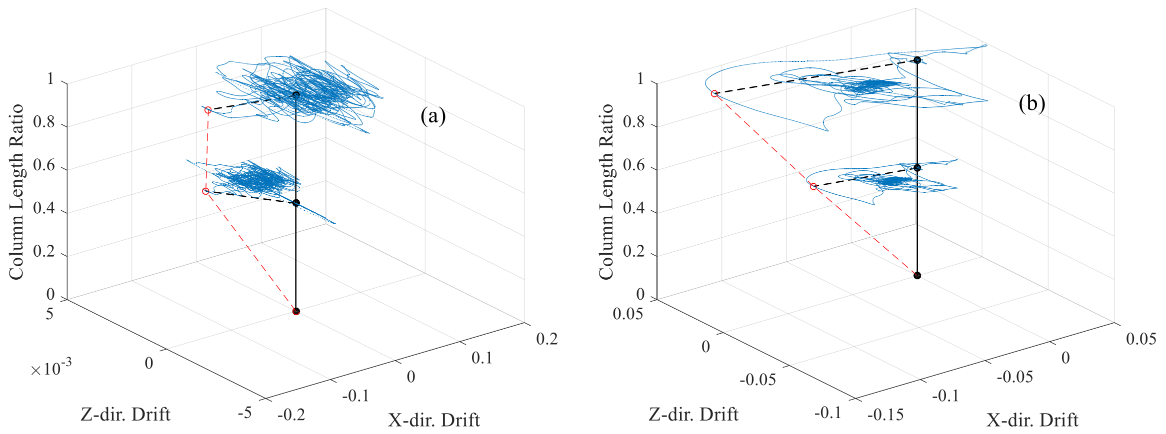


Fig. 7 Building collapse by: (a) column compression buckling failure; (b) column axial-flexural interaction failure

4.3 Effect of *p*% value used in percentage rule on collapse vulnerability and risk

Fig.8 presents the procedure for developing collapse fragilities with and without considering column failure caused by the inadequacy of percentage rules. The IDA is first conducted on the LC model of the SCBF building. Since the LC model does not consider column failure, if building collapse is observed from an RHA case, it is attributed to dynamic instability and the corresponding results are used to compute $P(C_{sys}|IM)$. For the non-collapse cases where the $R_{p,LC}$ value of the critical column is beyond 1.0, the NLC model is used to re-run the same analysis case. If collapse occurs using the NLC model, it is assumed that the column failure is the cause. Consequently, the collapse fragility $P(C|IM)$ is determined by the combination of $P\left(C_{Col}\middle|\overline{C_{sys}}, R_p > 1\right)P(\overline{C_{sys}}, R_p > 1|IM)$ and $P(C_{sys}|IM)$. The collapse margin ratio (CMR) [22] defined by Eq.(4) is used to describe the influence of percentage-rule-induced column failures on collapse fragilities.

$$CMR = \frac{\hat{S}a(T_{1,Collape})}{Sa(T_{1,MCE})} \tag{4}$$

where $\hat{S}a(T_{1,Collape})$ is the median collapse intensity.

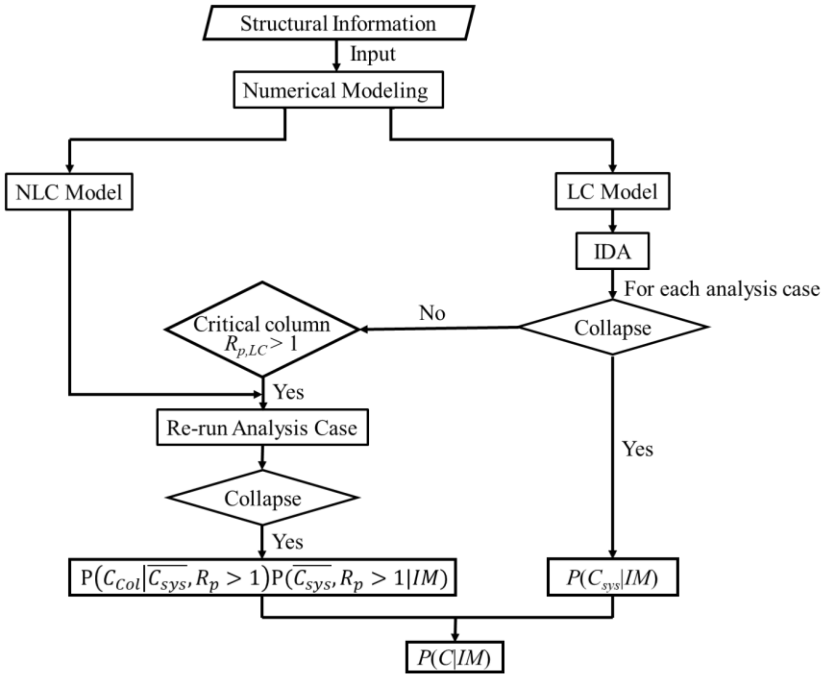
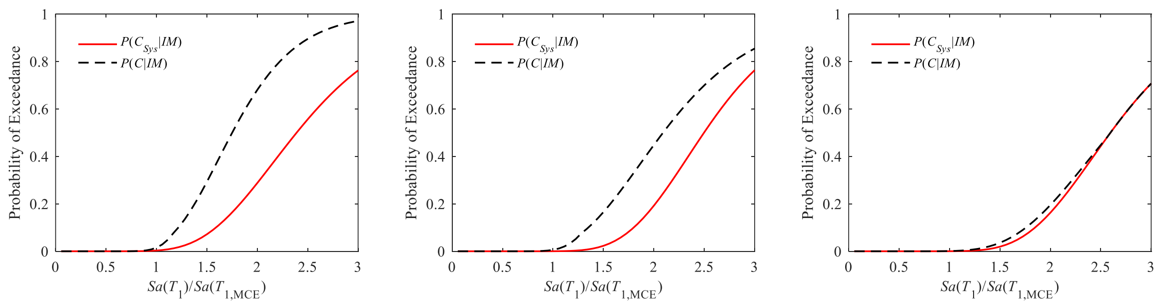


Fig. 8 Procedure for developing collapse fragilities with and without considering column failure

Fig.9 compares the collapse fragilities with and without column failure for the considered SCBF building. Each subfigure corresponds to the results using 100-30%, 100-65% and 100-100% rule for column design. The horizontal axis of Fig.9 is the shaking intensity in terms of $Sa(T_1)$ normalized by $Sa(T_{1,\mathrm{MCE}})$. The median system-level collapse capacity (i.e. corresponding to $\mathrm{P}(C_{sys}|IM)$) is actually the *CMR* value of the LC model. Remind that the collapse fragility of $\mathrm{P}(C|IM)$ is not directly obtained from the NLC model (Fig.8). The median collapse capacity corresponding to $\mathrm{P}(C|IM)$ represents the *CMR* of the model that considers both system-level and percentage-rule-based column failures. As such, a large difference between $\mathrm{P}(C_{sys}|IM)$ and $\mathrm{P}(C|IM)$ means the adopted 100-*p*% rule has a significant influence on the building collapse performance.

It can be seen from Fig.9 that as the adopted *p*% value increases, the two fragility curves of $P(C|IM)$ and $P(C_{sys}|IM)$ converge. The black dash curve for $P(C|IM)$ diverges from the red curve for $P(C_{sys}|IM)$ starting at approximately 100% $Sa(T_{1,\mathrm{MCE}})$ for both the 100-30% and 100-65%-rule-based design. The *CMR* of $P(C_{sys}|IM)$ is 1.37 and 1.19 times that of $P(C|IM)$ for the 100-30% and 100-65%-rule-based design, respectively. As for the 100-100%-rule-based design (Fig.9c), the results of $P(C|IM)$ and $P(C_{sys}|IM)$ have a slight difference over the *IM* range from 130 and 240% $Sa(T_{1,\mathrm{MCE}})$ and the *CMR* values are essentially the same.



(a) (b) (c)

Fig. 9 Collapse fragilities for the S8 building case w/ and w/o considering column failure: (a) 100/30%-rule-based design; (b) 100/65%-rule-based design; (c) 100/100%-rule-based design

Then, the adequacy of the 100-*p*% rule is evaluated in terms of the increased extent in the collapse risk of the structure. Eq.(5) is used to compute the mean frequency of collapse by integrating the fragility curve with the site-specific hazard curve. The hazard curve is obtained from the United States Geological Survey [25] which can be seen in Fig.10. The probability of collapse over a 50-year service life can be computed using Eq.(6).

$$\lambda_C = \int_0^{\infty} P(Collapse|IM = x) \cdot \left|\frac{d\lambda(IM)}{d(x)}\right| \cdot d(x) \tag{5}$$

$$\mathrm{P}_C(T = 50 \text{ years}) = 1 - \exp\left(-\lambda_C \cdot 50\right) \tag{6}$$

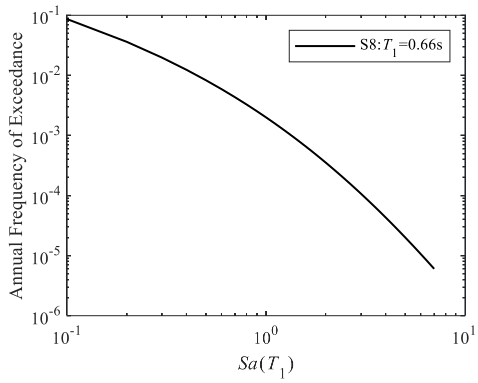


Fig. 10 Seismic hazard curve for the considered SCBF building

Table 2 summarizes the 50-year collapse probability for the 8-story SCBF building with (P_C) and without ($\mathrm{P}_{C,sys}$) considering column failure. It is observed that the P_C value (considering column failure) decreases as the *p*% value gets larger. The P_C value for the building design using the 100-100% rule is reduced by 70% compared to the risk results from the design based on the 100-30% rule. This is because the design using 100-100% rule employed larger column sizes and consequently reduces the likelihood of column-failure-induced collapse. The parameter of δ is used to measure the normalized difference between P_C and $\mathrm{P}_{C,sys}$ and it quantifies the influence of the *p*% value used in the percentage rule on building collapse risk. A greater δ value implies that column failure due to insufficient strength determined by the adopted 100-*p*% rule has a stronger influence on collapse risk. As shown in Table 2, the δ value decreases as *p*% increases. Also, as the value of *p*% increases from 30% to 65%, the δ value decreases by nearly 13%. The benefit when increasing *p*% from 65% to 100% is more significant than the case when changing *p*% from 30% to 65%, and specifically, δ is reduced by a factor of 6.6 when *p*% is increased from 65% to 100%.

Table 2 Probability of collapse in 50 years

Percentage rule used in design	$\mathrm{P}_{C,sys}$	P_C	$\delta = \frac{\mathrm{P}_C - \mathrm{P}_{C,sys}}{\mathrm{P}_{C,sys}}$
100-30% rule	0.25%	0.60%	142.6%
100-65% rule	0.17%	0.39%	124.4%
100-100% rule	0.16%	0.18%	14.9%

5. Conclusion

This study proposed a reliability-based method to evaluate the effectiveness of the percentage rule (100-*p*%) which is commonly used to consider orthogonal seismic effects. The effect of the adopted *p*% value on the increased building collapse risk due to insufficient column design based on percentage rule is assessed. An 8-story SCBF building is selected for the application. The 100-30%, 100-65%, and 100-100% rule were used to design the SCBF building columns (i.e. a total of three designs). For each designed building, two numerical models were built with the aid of *OpenSees*. One model uses nonlinear elements to model columns to consider the possibility of their failures. The other one uses linear elastic elements to model columns to exclude the building collapse due to column failures. With the proposed method, the parameter of R_p is considered to measure the adequacy of percentage rules in terms of force demands at shared columns. The obtained probability distribution of R_p showed that there was an approximately 50% probability that the 100-30% rule would underestimate the SCBF column force demands over a range of intensity levels.

The proposed approach helped to quantify the influence of the adopted *p*% value in the percentage rule on the building collapse fragility. Besides, by incorporating site-specific hazard curves, the effect on collapse probabilities over 50 years was assessed. The results showed that a higher *p*% value would reduce the collapse probability caused by the inadequate percentage-rule-based design force demands at columns. Specifically, column failure due to underestimated orthogonal seismic demands by 100-30% rule increased the 50-year probability of collapse by 143%. The increased risk is reduced by 15% if 100-100% rule is used. Also, the relative benefit derived from increasing the *p*% value from 65% to 100% was found to be much greater than the case when *p*% value was increased from 35% to 65%. The developed method aims to inform engineers and stakeholders to determine an appropriate *p*% value based on their performance-based design objective and risk preferences.

Acknowledgements

The first author would like to acknowledge China Scholarship Council, Power Construction Corporation of China through project KJ-2016-095, the National Natural Science Foundation of China (U1710111 & 51878426).

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