The 2018 Structures Congress (Structures18)
Songdo Convensia, Incheon, Korea, August 27 - 31, 2018

A robustness measure for seismically designed RC structures under column loss

*Meng-Hao Tsai[1)] and Wen-Bing Zhuang [2)]

[1), 2)] *Department of Civil Engineering, NPUST, Pingtung 912, Taiwan*
[1)] *mhtsai@mail.npust.edu.tw*

ABSTRACT

Analytical relationship between the seismic and progressive collapse resistances of RC building frames was investigated in this study. A resistance ratio was defined and its analytical expression was derived with plastic analysis technique. A robustness measure for progressive collapse of the RC building frames under column loss was derived as the product of the resistance ratio and the seismic coefficient. Analysis results indicated that the span length is the most critical factor for the structural robustness under column loss. The robustness index decreases with increased span length, which implied a higher collapse potential. Also, it was directly proportional to the variation of the seismic coefficient and the numbers of stories. Numerical verification confirmed that the robustness of a seismically designed RC building frame under column loss can be estimated by the proposed analytical formulation.

1. INTRODUCTION

As defined in the commentary of the American Society of Civil Engineers (ASCE) Standard 7 and the Unified Facilities Criteria guidelines UFC 4-023-03 of Department of Defense (DoD), progressive collapse means the collapse of an entire structure or a disproportionately large part of it resulted from the spread of an initial local failure (ASCE 2010, DoD 2013). The importance of progressive collapse resistance of building structures has been recognized since the partial collapse of the Ronan Point apartment building in 1968. Especially after the 9/11 terrorist attacks in 2001, modern structural design codes and guidelines have either directly or indirectly accounted for the prevention of such failure in the regulations. Compared with the periodical natural hazards like earthquake and typhoon, progressive collapse is regarded as a relatively rare event. Hence, except for specially designed protective systems, it is usually impractical to design a structure resistant to general collapse caused by severe abnormal loads. In other word, it may be more feasible to consider the progressive collapse resistance as a derivative from conventional codified structural design

[1)] Professor
[2)] Graduate Student

governed by frequent natural hazards. Several researches have shown that seismically designed structures may have better progressive collapse resistance. Bao *et al*. (2008) investigated the column-loss responses of planar reinforced concrete (RC) frames designed with varied seismic resistances and indicated that progressive collapse resistance may be increased with the seismic resistance. Kim and Choi (2011) conducted tests on two-span gravity-load and lateral-load resisting RC beam-column sub-assemblages. Higher loading resistance was obtained for the lateral-load resisting specimen. Marchis *et al*. (2012) investigated the vulnerability to progressive collapse of mid-rise RC frames designed with varied seismic and ductility demands. They indicated that with same seismic demand, frames designed with a larger ductility capacity may be more vulnerable to progressive collapse. Dinu *et al*. (2015) investigated the effects of joint rigidity and composite slabs on the robustness of an earthquake-resistant steel frame. It was mentioned that earthquake resistance is beneficial to progressive collapse resistance.

Most of the aforementioned studies have a consistent conclusion that the progressive collapse resistances of building frames under column loss can be increased by enhancing their seismic resistances. Since earthquake-resistant design is a must for building structures located in seismically hazardous region, it will be practical if the progressive collapse resistance can be preliminarily estimated from the seismic capacity. Also, there is a lack of quantitative relationship between the column-loss and seismic resistances. Hence, a robustness index was proposed for the progressive collapse potential of seismically designed building frames under column loss in this study. Alternate load-path method (GSA 2013, DoD 2013) was used to evaluate the progressive collapse resistance. Seven RC building frames were designed with different seismic coefficients, span length, and number of stories. Nonlinear static pushdown and pushover analyses were performed to calculate their robustness indices under different column-loss scenarios. Robustness variations with those design parameters were evaluated.

2. DEFINITION OF ROBUSTNESS INDEX

In conventional seismic design of building structures, the design base shear V_b is usually expressed as

$$V_b = C_s W \tag{1}$$

where C_s is the seismic design coefficient and *W* is the total structural weight considered in the design. If a regular building frame with uniform floor weight, constant height, and consistent structural plan in each story is assumed, then the total structural weight can be written as

$$W = w_d [L_B (B+1) + L_D (D+1)] N \tag{2}$$

where w_d is the uniformly distributed gravity beam loads in the seismic design and *N* is the number of stories. L_B and L_D are respectively the longitudinal and transverse plan dimensions. B and D are the number of bays in the longitudinal and transverse direction, respectively.

On the other hand, consider a bottom side-column loss in the longitudinal direction of the building frame. The collapse resistance under the column-loss scenario, P_b, can be expressed as

$$P_b = w_m [L_B (B+1) + L_D (D+1)] N \tag{3}$$

where w_m is the ultimate uniformly distributed beam loads. Along with the previous two expressions, it can be obtained that

$$\frac{w_m}{w_d} = \frac{P_b}{V_b} C_s \tag{4}$$

The above equation indicates that the ultimate uniformly distributed beam loads can be estimated from the product of w_d, P_b / V_b, and C_s. Hence, P_b / V_b is defined as the resistance ratio and w_m / w_d is proposed as the robustness index for the seismically design building frame under column loss. Eq.(4) reveals that the collapse resistance is less than the seismic dead weight of the building frames as the robustness index is smaller than one. Therefore, it may imply a higher progressive collapse potential.

3. ANALYTICAL FORMULATION

3.*1 Planar Sub-assemblage Model*

The analytical relationship between the seismic shear strength and collapse resistance under column loss is investigated with plastic analysis technique. At first, consider an N-story and two-bay frame with uniform floor weight, story height, and equal bay length. Assume an inversely triangular, linearly distributed lateral seismic loading pattern along the building height for the frame, as shown in Fig. 1(a). Then, the equivalent lateral load on the i-th floor, F_i, can be expressed as

$$F_i = \frac{2i}{N(N+1)} V_b \tag{5}$$

where V_b is the design base shear. With the assumption of inflection points at the mid-height of columns, a beam-column sub-assemblage can be extracted from the i-th floor of the frame, as shown in Fig. 1(b). The (i+1)-th story shear is thus expressed as

$$V_{i+1} = \frac{(N+i+1)(N-i)}{N(N+1)} V_b \tag{6}$$

When the elastic strain energy is neglected, the equal work and energy principle gives

$$V_{i+1} h \theta_u + F_i \frac{1}{2} h \theta_u = \sum_{j=1}^{2} (M_{p,j}^{+} + M_{p,j}^{-}) \theta_u \tag{7}$$

in which strong-column-and-weak-beam mechanism is implied. $M_{p,j}^{+}$ and $M_{p,j}^{-}$ are respectively the positive and negative plastic flexural strength at the j-th beam ends. θ_u is the story drift ratio under the lateral load. From the above equations, the shear resistance of the i-th story V_i can then be written as

$$V_i = F_i + V_{i+1} = \frac{(N+i)(N-i+1)}{(N^2 - i^2) + N} \frac{1}{h} \sum_{j=1}^{2} (M_{p,j}^{+} + M_{p,j}^{-}) \tag{8}$$

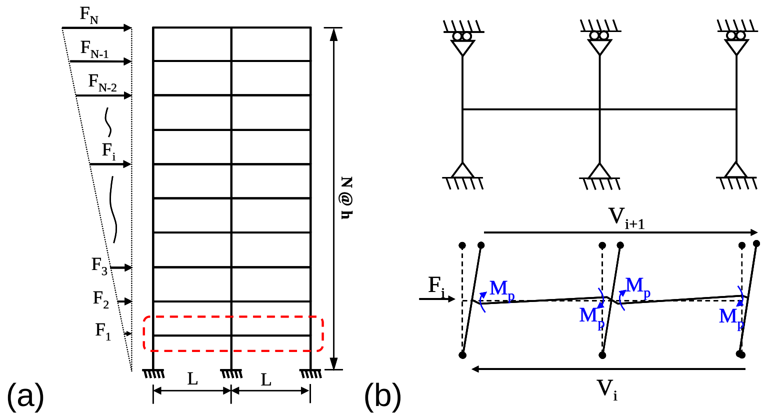


Fig. 1 (a). Assumed lateral loading pattern. (b). Sub-assemblage model under lateral load.

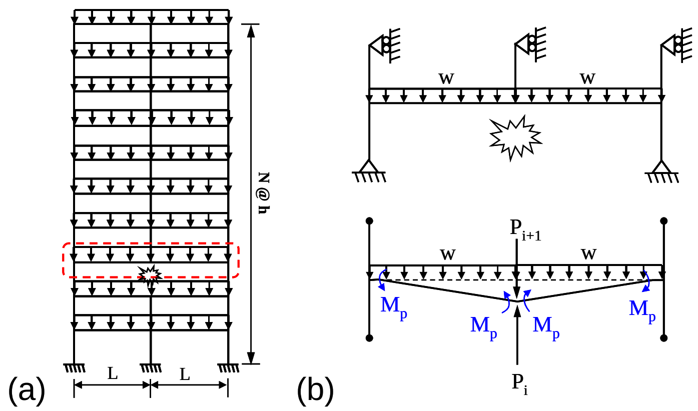


Fig. 2 (a). Column-loss loading pattern. (b). Sub-assemblage model under column-loss.

Similarly, as the middle column of the i-th story is lost, as shown in Figs. 2(a) and 2(b), the uniformly distributed loading for the plastic collapse mechanism of the two-span sub-assemblage model can be written as

$$w_m = \sum_{j=1}^{2} (M_{p,j}^{+} + M_{p,j}^{-}) / L^2 \tag{9}$$

Considering all the uniformly distributed beam loads above the column-loss level, the maximum sustained loading is written as

$$P_i = 2w_m L(N - i + 1) = 2(N - i + 1) \sum_{j=1}^{2} (M_{p,j}^{+} + M_{p,j}^{-}) / L \tag{10}$$

If the frame members have the same flexural strength under the lateral seismic loading and column-loss conditions, the resistance ratio P_i / V_i can be obtained from dividing Eq.(10) by Eq.(8) and expressed as

$$\frac{P_i}{V_i} = \frac{2h}{L} \frac{(N^2 - i^2) + N}{(N + i)} \tag{11}$$

3.*2 Extension to 3D Frame Model*

For the application to a whole building frame, the plastic hinges at the bases of bottom columns cannot be neglected. Based on the strong-column and weak-beam assumption, the plastic strain energy of an N-story building frame under the inversely triangular, linearly distributed lateral seismic load in the longitudinal direction can be estimated as

$$E_s = \sum_{i=1}^{N} \sum_{j=1}^{D+1} \sum_{k=1}^{B} [M^{+}_{px,jk} + M^{-}_{px,jk}]_i \theta_x + \sum_{j=1}^{D+1} \sum_{k=1}^{B} M^{c}_{px,jk} \theta_x \qquad (12)$$

where $M^{+}_{px,jk}$ and $M^{-}_{px,jk}$ are respectively the positive and negative plastic moment strength at the k-th beam ends of the j-th longitudinal bay. θ_x is the longitudinal story drift ratio under the lateral load. $M^{c}_{px,jk}$ is the plastic moment strength of the k-th bottom column of the j-th longitudinal bay. Then, with the assumption of a constant story height *h*, the external work done by the equivalent lateral load is written as

$$W_s = \sum_{i=1}^{N} F_i i h \theta_u = \frac{h(2N+1)}{3} V_b \theta_x \qquad (13)$$

From $W_s = E_s$, the base shear V_b can be determined and the i-th story shear can be expressed as

$$V_i = \frac{(N+i)(N-i+1)}{N(N+1)} \frac{3E_s}{h(2N+1)\theta_x} \qquad (14)$$

Assume a constant span length of L_x and L_y in the longitudinal and transverse direction, respectively. For the N-story building frame subjected to side-column loss at the i-th story, the external work done by the uniformly distributed beam load w_m is represented by

$$W_i = (N-i+1) \frac{w_m \theta_x L_x}{2} (2L_x + L_y) \qquad (15)$$

Similarly, the corresponding plastic strain energy is expressed as

$$E_i = \sum_{n=i}^{N} [\sum_{k=0}^{1} (M^{+}_{px,1(j-k)} + M^{-}_{px,1(j-k)})\theta_x + (M^{+}_{py,j1} + M^{-}_{py,j1}) L_x \theta_x / L_y]_n \qquad (16)$$

The maximum sustained loading of the i-th story under the side-column loss is

$$P_i = \frac{2E_i}{L_x \theta_x (2L_x + L_y)} (L_x B(D+1) + L_y D(B+1))(N-i+1) \qquad (17)$$

Hence, the resistance ratio of the 3D frame under the i-th story side-column loss is expressed as

$$\frac{P_i}{V_i} = \frac{2N(N+1)(2N+1)}{3(N+i)} \frac{h}{L_x} \frac{E_i}{E_s} \frac{(L_x B(D+1) + L_y D(B+1))}{(2L_x + L_y)} \qquad (18)$$

From the above derived result, it is observed that the resistance ratio and robustness increase with the numbers of stories, while they may decrease with increased span length. For the special case of bottom column loss with identical flexural strength, the above expression is further reduced to

$$\frac{P_1}{V_1} = \frac{4N}{3}\frac{h}{L}\frac{2BD + D + B}{B(D+1)} \tag{19}$$

4. APPLICATION TO SEISMICALLY DESIGNED BUILDING FRAMES

4.1 *Seismic design of the prototype RC building frames*

Seven RC moment-resisting building frames were designed based on the Seismic Design Specifications and Commentary for Buildings of Taiwan (MOI 2005). Compressive strength of 27.5 MPa (280 kgf/cm^2) and tensile yield strength of 412 MPa (4200 kgf/cm^2) were adopted for the concrete and reinforcement in the seismic design. Three different numbers of stories, namely five, ten, and fifteen were considered and a constant story height of 3 m was assumed for the building frames. As shown in Fig. 3, a consistent four bay-by-three bay plan layout and three different span lengths, 4, 6, and 10 m, were used to consider the span length factor. The design dead load (DL) was composed of the structural self weight, a uniform slab loading of 3.92 kN/m^2 (400 kgf/m^2) and the weight of 24 cm-thick exterior non-structural brick walls. The service live load (LL) is 1.73 kN/m^2 (300 kgf/m^2). Considering all possible combinations of different site conditions and spectral acceleration coefficients in the design code, three seismic coefficients, C_s = 0.10, 0.15, and 0.22 were respectively determined for typical low, medium, and high seismic regions. Load combinations of (1.2DL+1.6LL) and (1.2DL+1.0LL+1.0EQ) were considered for the structural design. For the sake of simplicity, same reinforcement was used in both the longitudinal and transverse directions and same section dimensions were used in each building frame. The column sections were determined according to the strong column-and-weak beam mechanism (Ghahremannejad and Park 2016). Table 1 shows the longitudinal and transverse fundamental periods of the building frames. For convenience, they were designated by their number of stories, span lengths, and seismic coefficients. For example, the designation of 10S06R10 stands for the ten-story building frame with span length of 6 m and seismic coefficient of 0.10. It is observed from the table that the ten- and fifteen-story building frames were a little stiffer than conventional design. This was caused by the constant section depth used in the structural design of each building frame. Even though, this did not affect the numerical demonstration of the robustness evaluation.

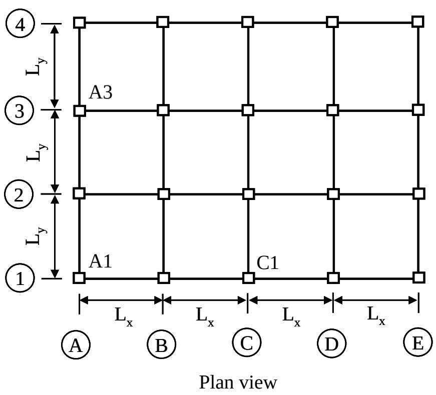


Fig. 3 Plan view of the example building frames

Table 1 Fundamental periods, yield base shears, and seismic coefficients of the building frames

Building	Transverse			Longitudinal		
	Period (s)	Vy (kN)	Cs	Period (s)	Vy (kN)	Cs
05S04R15	0.536	1781	0.140	0.523	1878	0.148
05S06R15	0.590	3107	0.127	0.576	3263	0.134
05S10R15	0.441	11980	0.164	0.430	12663	0.173
10S06R10	0.988	4954	0.090	0.958	5299	0.096
10S06R15	0.812	9254	0.147	0.786	9941	0.158
10S06R22	0.747	16504	0.245	0.723	17469	0.260
15S06R15	1.094	17122	0.158	1.045	18165	0.167

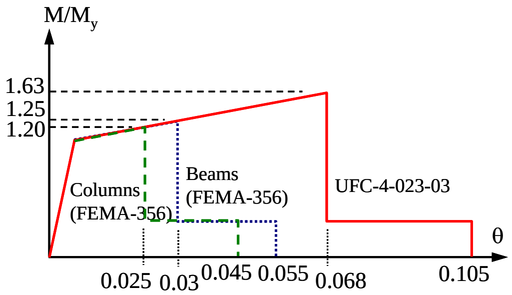


Fig. 4 Flexural hinge properties used in the analysis

4.2 *Seismic resistance*

Nonlinear static (NS) pushover analyses under the story-wise distributed lateral seismic load were conducted to obtain their horizontal seismic resistances. Plastic hinges were assigned to the member ends. The hinge properties suggested in FEMA-356 (FEMA 2000) were adopted, as shown in Fig. 4. A 5% post-yield stiffness ratio and a yield rotation of 0.005 rad were assumed in the hinge models. The longitudinal (X) and transverse (Y) pushover curves of the five-story frames with different span length, the ten-story frames with different seismic coefficients, and the 6 m-span frames with different story numbers are shown in Figs. 5(a), 5(b), and 5(c), respectively. In the figures, the ordinate is the base shear normalized by the design structural weight (1.0DL+0.25LL) and the abscissa is the global drift ratio determined from dividing the roof displacement by the building height. It is seen that both the longitudinal and transverse directions have similar normalized pushover curves. The yield base shear, Vy, which was determined as the base shear corresponding to the first appearance of structural hinges, of the seven building frames and the corresponding seismic coefficient (Cs) are summarized in Table 1. The frames with 10 m span length present apparently larger normalized seismic resistance than the others in Fig. 5(a). This is because that the bottom reinforcement at the beam ends were determined by the minimum reinforcement requirement (ACI 2011), which regulates that positive moment strength at joint faces shall be not less than one-half the negative moment strength provided at that face of the joint. Fig. 5(c) reveals that similar normalized pushover curves were obtained for the 6 m-span building frames designed with a same seismic coefficient but different number of stories. Table 1 indicates that resulting seismic coefficients generally agree with the original design values.

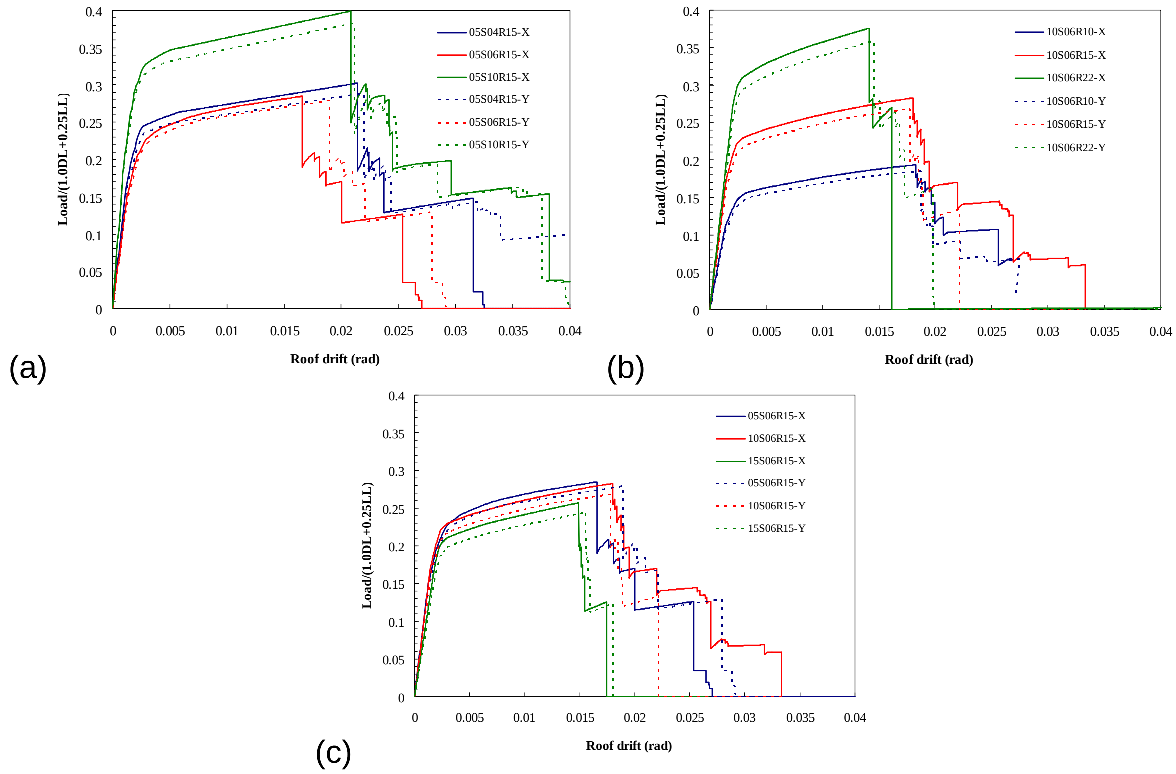


Fig. 5 Normalized non-linear static pushover curves. (a) Five-story building frames. (b) Ten-story building frames. (c) Building frames with 6-m span and seismic coefficient 0.15.

4.3 *Column-loss resistance*

Three different column-loss scenarios, which are the middle column of the longitudinal peripheral frame C1, the penultimate column of the transverse peripheral frame A3, and the corner column A1, as indicated in Fig. 3, were considered for the seven building frames. Since uniformly distributed gravity loading was used in the seismic design, uniform loading pattern was adopted in the NS pushdown analysis. Different from the cyclic loading demands under earthquake excitations, the structural members can exhibit larger plastic rotation capacity under the column-loss scenarios (DoD 2013). Hence, the plastic rotation capacity suggested in the UFC 4-023-03 guidelines was adopted in the NS pushdown analysis. Since the dynamic effect under column loss was neglected in the NS pushdown analysis, the work-energy principle was used to construct the corresponding pseudo-static response for the dynamic collapse resistance (Tsai 2010, Tsai and You 2012). Figs. 6(a), 6(b), and 6(c) show the pseudo-static response curves under C1, A3, and A1 column-loss scenarios, respectively. The ordinate in the figures is the total imposed gravitational loading normalized by the design structural weight (1.0DL+0.25LL). The abscissa is the chord rotation, which is defined as the displacement of the column-removed joint divided by the span length.

The dynamic collapse resistances of the seven building frames under the column loss were then determined as the peak pseudo-static responses. It is seen that each building frame has approximate normalized collapse resistances under the three different column-loss scenarios. This is due to the regular and uniform structural design for each building. Meanwhile, reducing the span length, increasing the seismic design force, and increasing the number of stories are all beneficial to the collapse resistance. From the relative difference in the collapse resistance of the seven building frames, it is observed that reducing the span length is less efficient than the other two approaches.

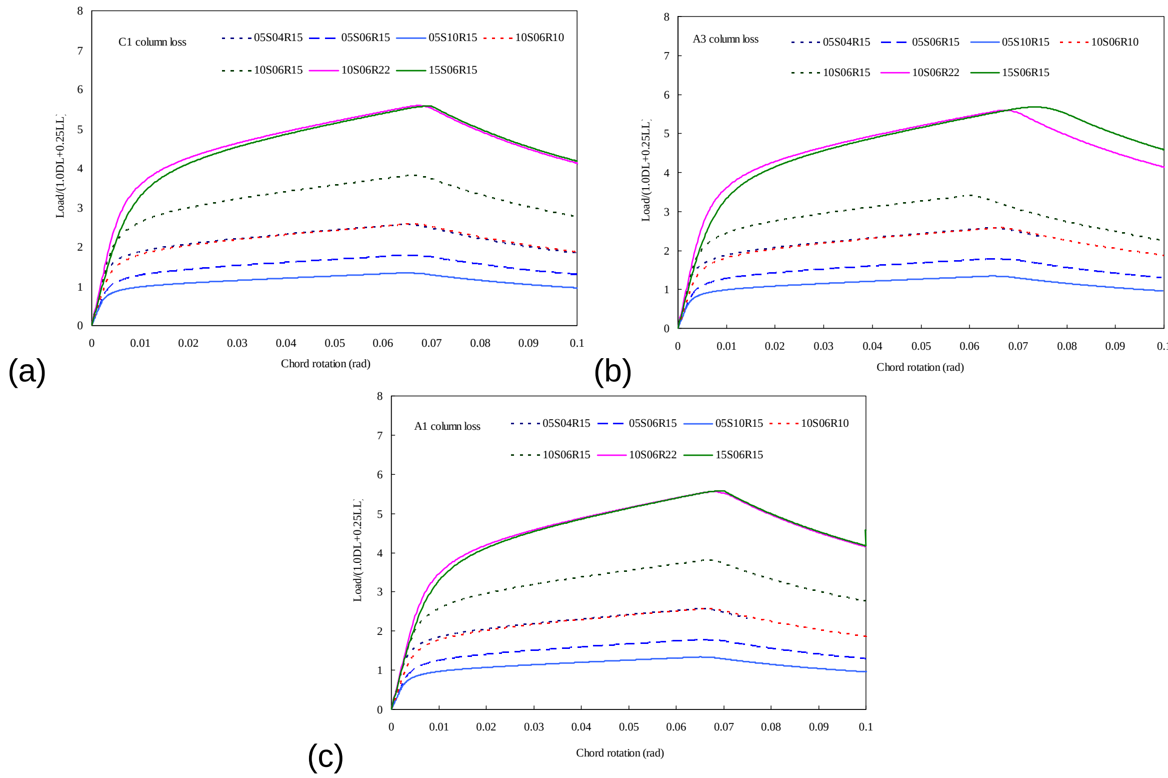


Fig. 6 Normalized pseudo-static pushdown curves. (a) C1-column loss. (b) A3-column loss. (c) A1-column loss.

4.4 *Evaluation of the analytical and numerical robustness*

The numerical resistance ratios of the seven building frames were estimated as the ratios of the peak pseudo-static responses in Figs. 6 to the peak pushover resistances in Figs. 5. After multiplied by the corresponding seismic design coefficients in Table 1, the numerical robustness could be obtained. The average pushover resistance and seismic design coefficient of the longitudinal and transverse directions were used in the calculation of the robustness index for the corner column loss scenario. Figs. 7(a), 7(b), and 7(c) show the effect of span variation on the robustness under the longitudinal, transverse, and corner column loss conditions, respectively, for the five-story building frames designed with a seismic coefficient of 0.15. The numerical results and analytical predictions with the planar sub-assemblage and with the 3D frame models are compared in the figures. A robustness index less than 1.0 means that the building structure may fail to sustain its design gravity loadings without damage under the column-loss scenario. This could happen for the five-story building frames as the span length is over 6 m. As expected, increasing the span length has an adverse effect on the robustness. Nevertheless, the rate of robustness reduction decreases in the medium-to-long span (from 6 m to 10 m) range. Also, it is observed that apparently larger robustness indices than the numerical results were obtained with the analytical sub-assemblage model. This is because that the contribution of the plastic hinges at the base of the bottom columns was disregarded in the sub-assemblage formulation. Hence, the building frame would have high progressive collapse potential under column loss if a robustness value less than 1.0 is predicted by the sub-assemblage model.

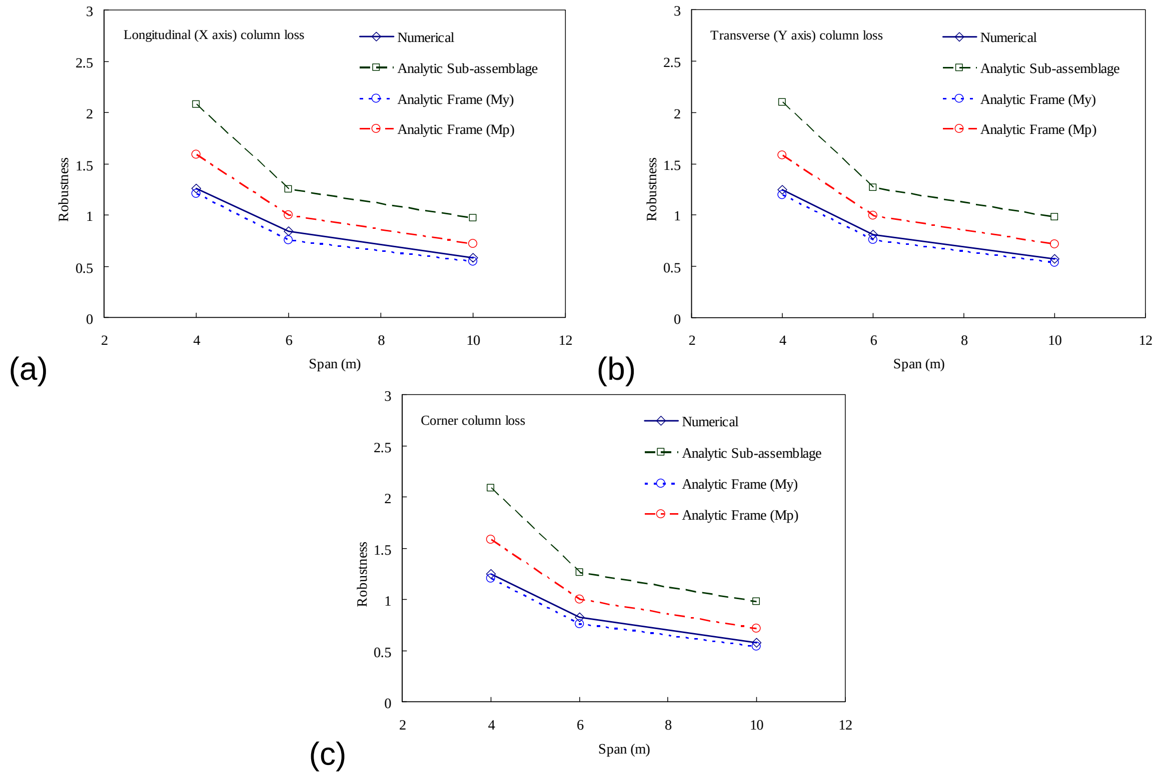
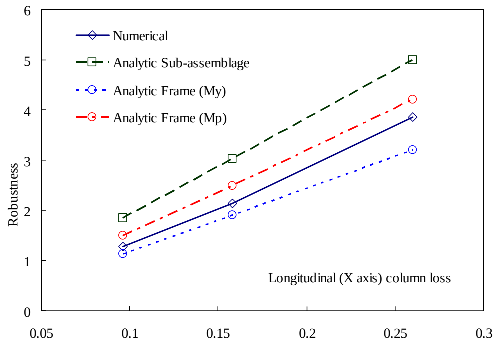


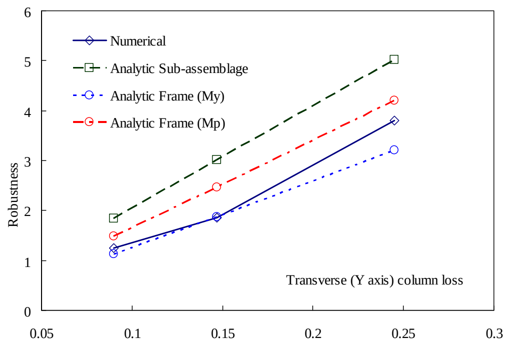
Fig. 7 Effect of span length on the robustness. (a) C1-column loss. (b) A3-column loss. (c) A1-column loss.

The analytical results obtained with the 3D frame model are also shown in Figs. 7. The analytical predictions by using the ultimate and yield flexural strength of the hinge properties are respectively denoted as M_p and M_y in the figures. It is seen that with the yield flexural strength, the analytical predictions are conservatively approximated to the numerical results. However, the analytical predictions with the ultimate flexural strength are moderately larger than the numerical results. This difference comes from the implicit assumption in the analytical formulation that all the plastic hinges can develop their ultimate flexural strength simultaneously. It may overestimate the pushdown resistance of the multi-story building frame under column loss.

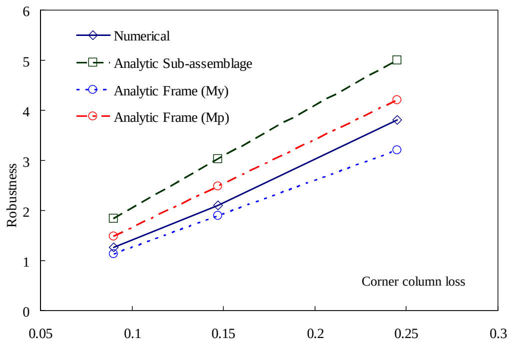
Figs. 8(a), 8(b), and 8(c) show the variation of robustness with the seismic coefficient under the longitudinal, transverse, and corner column loss conditions, respectively, for the ten-story building frames designed with 6-m span. It is seen that the robustness index is approximately linear with the seismic coefficient. As implied in Eq.(4) and Eq.(11), since the analytical resistance ratio is independent of the flexural strength, the robustness index is therefore proportional to the seismic coefficient. This evidence confirms that increasing the seismic design force can benefit the structural robustness under column loss. Meanwhile, similar to the five-story building frames, the analytical predictions of the 3D frame model with yield flexural strength are conservatively approximate to the numerical results. The 6-m span, ten-story building frame can have a robustness index larger than 1.0 as the seismic design coefficient is larger than 0.1.



(a)



(b)



(c)

Fig. 8 Effect of seismic coefficient on the robustness. (a) C1-column loss. (b) A3-column loss. (c) A1-column loss.

Figs. 9(a), 9(b), and 9(c) present the effect of the number of stories on the robustness under the longitudinal, transverse, and corner column loss conditions, respectively. The building frames have a constant span length of 6 m and a seismic design coefficient 0.15. It is seen that the robustness increases with the number of stories. This variation is consistent with the study results by Kim *et al*. (2009) and Ghahremannejad and Park (2016). With a fixed seismic coefficient, both the lateral base shear and progressive collapse resistances increase with the numbers of stories. According to the definition of the robustness index in Eq.(4), the analysis results reveal that the resistance increase in the latter is larger than that in the former. Similar to the variation with the seismic

coefficient, the robustness index also has an approximately linear relation with the number of stories.
From the above evaluation, it is realized that reducing the span length, increasing the seismic design force, and increasing the number of stories can help to enhance the structural robustness under column loss. In practical engineering, higher building frames are usually designed with lower seismic forces because of increased natural periods. Hence, either reducing the span length or increasing the seismic design force is feasible for high-rise buildings. Also, the 3D frame model with yield flexural strength can provide reasonably conservative estimation for the robustness index. Although the sub-assemblage model moderately overestimates the robustness index, it can be used in the preliminary evaluation stage. If the sub-assemblage model gives a robustness index larger than 1.0, then the 3D frame model should be used to confirm the safety under column loss. On the contrary, progressive collapse can be probable if a robustness index less than 1.0 is obtained by the sub-assemblage model.

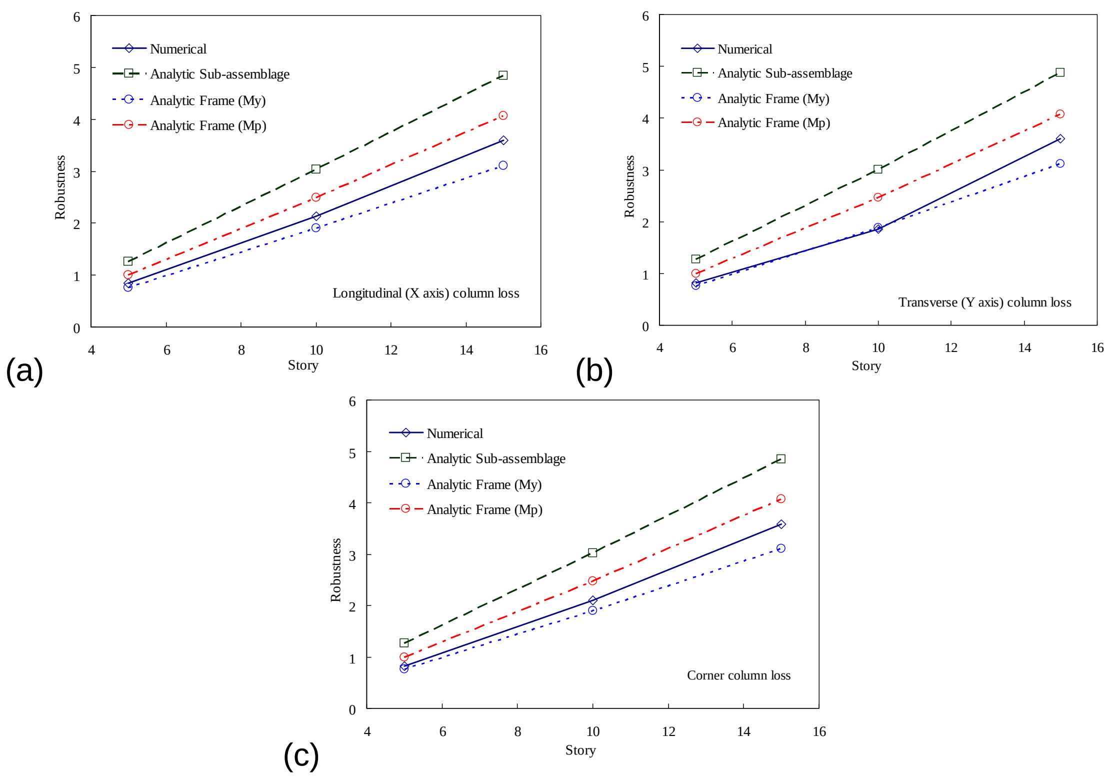


Fig. 9 Effect of numbers of stories on the robustness. (a) C1-column loss. (b) A3-column loss. (c) A1-column loss.

5. CONCLUSIONS

A systematic investigation has been conducted in this study for the progressive collapse resistance of seismically designed RC building frames. A robustness index is proposed as the ratio of the ultimate distributed gravity loading under column loss to the effective structural weight in seismic design. It can be expressed as the product of the seismic coefficient and a resistance ratio defined as the collapse resistance divided by the seismic shear resistance. An analytical expression was derived for the resistance ratio of a beam-column sub-assemblage model by using the plastic analysis technique. The analytical formulation was extended to prototype three-dimensional (3D) building frames. Seven regular seismically designed RC building frames were used to investigate the influence of span length, seismic coefficient, and number of stories on the robustness of the structural frames under three column loss scenarios. For the five-story building frames designed with a constant seismic coefficient 0.15, the robustness index sharply decreases to less than 1.0 as the span length increases from 4 m to 6 m. This implies high progressive collapse potential under the postulated column loss scenarios as the gravity loading of seismic design is imposed on the building frame. On the other hand, the robustness index increases approximately linearly with the seismic coefficient. It is also approximately directly proportional to the number of stories for the building frames designed with constant seismic coefficients and span lengths. Also, the numerical results are consistent with the analytical predictions calculated by using the yield flexural strength in the 3D frame model. However, the planar sub-assemblage analytical model may result in more overestimation of the robustness index although it is much simpler than the 3D frame model. Therefore, if the sub-assemblage analytical model predicts a robustness index less than 1.0, then high progressive collapse potential can be determined for the building frame under column loss. If not, the 3D analytical frame model should be used for further detailed evaluation. As the 3D analytical frame model predicts a robustness index less than 1.0, more rigorous analysis with the consideration of catenary action and slab contribution should be taken to examine the structural safety under column loss.

REFERENCES

ASCE, *Minimum Design Loads for Buildings and Other Structures*, Structural Engineering Institute, American Society of Civil Engineering (ASCE), Reston, Virginia, 2010.

Bao, Y., Kunnath, S.K., El-Tawil, S., and Lew, H.S. (2008), "Macromodel-Based Simulation of Progressive Collapse: RC Frame Structures," *Journal of Structural Engineering*, *ASCE*, **134**(7), 1079-1091.

Dinu, F., Dubina, D., and Marginean, I. (2015), "Improving the structural robustness of multi-story steel-frame buildings," *Structure and Infrastructure Engineering*, **11**(8), 1028-1041.

DoD, Department of Defense, 2013. *Unified facilities criteria: design of buildings to resist progressive collapse*, UFC 4-023-03. Washington D.C., USA.

FEMA 356 (2000), *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, Federal Emergency Management Agency, Report No. FEMA 356, Washington, D.C.

Ghahremannejad, M. and Park, Y. (2016). "Impact on the number of floors of a reinforced concrete building subjected to sudden column removal," *Engineering Structures*, **111**, 11-23.

GSA, General Service Administration (2013), *Alternate Path Analysis & Design Guidelines for Progressive Collapse Resistance*, Washington D.C., USA.

Kim, J. and Choi, H. (2015). "Monotonic Loading Tests of RC Beam-Column Subassemblage Strengthened to Prevent Progressive Collapse," *International Journal of Concrete Structures and Materials*, **9**(4), 401-413.

Kim, T., Kim, J., and Park, J. (2009)," Investigation of progressive collapse-resisting capability of steel moment frames using push-down analysis", *Journal of Performance of Constructed Facilities*, *ASCE*, **23**(5), 327-335.

Marchis, A., Botez, M., and Ioani, A.M. (2012). "Vulnerability to progressive collapse of seismically designed reinforced concrete framed structures in Romania," *15th World Conference on Earthquake Engineering* (15WCEE), September 24-28, 2012, Lisbon, Portugal.

MOI, Seismic Design Specifications and Commentary for Buildings of Taiwan, Construction and Planning Agency, Ministry of the Interior (MOI), Taipei, Taiwan, 2005.

Tsai, M.H. (2010), "An analytical methodology for the dynamic amplification factor in progressive collapse evaluation of building structures", *Mechanical Research Communication*, **37**, 61-66.

Tsai, M.H. and You, Z.K. (2012), "Experimental evaluation of inelastic dynamic amplification factors for progressive collapse analysis under sudden support loss", *Mechanical Research Communication*, **40**, 56-62.