Progressive collapse resistance of two typical high-rise RC frame shear wall structures

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Abstract: Existing research on progressive collapse of building structures mainly focuses on concrete and steel frame structures. To investigate the progressive collapse resistance of high-rise reinforced concrete (RC) frame shear wall structures, two typical 15-story building models are designed with equivalent overall lateral resistance to seismic actions. However the structural layouts in resisting the lateral forces are quite different for the two buildings. Building A is a weak wall-strong frame structure whilst Building B is a strong wall-weak frame system. 3-D finite element models of the two structures are established using fiber beam and multilayer shell elements. The progressive collapse resistances of the frames and the shear walls in both structures are evaluated under various column (shear wall) removal scenarios. Results demonstrate that there is a difference in progressive collapse prevention performance for different structural layouts. The progressive collapse resistance tends to be inadequate for the strong wall-weak frame system. Such a system is subsequently re-designed using the linear static AP method proposed in GSA2003, through which the reliability and efficiency of the method is confirmed. The outcome of this study has provided a reference for progressive collapse prevention designs of typical and representative high-rise RC frame shear wall structures.

Key words: Progressive collapse; High-rise frame shear wall structure; Alternative load path method; Collapse resistance.

Introduction

Progressive collapse is considered as a type of chain reaction failure that follows the initial damage in a relatively small portion of a structure (Ellingwood and Leyendecker 1978). Progressive collapse of building structures, which is a significant threat to public safety, has been studied for

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25 more than 40 years since the collapse of the Ronan Point Apartment in 1968.

26 Existing research studies have investigated the progressive collapse resistance of primarily
27 frame type of structures via experimental and numerical methods. By the experimental means,
28 Sasani et al. (2011a) studied the progressive collapse resistance of an actual 11-story RC frame
29 structure following an initial damage of four adjacent columns and two beam segments. It should be
30 noted that large-scale tests of an entire structural system remain challenging given the limitations of
31 test facilities and costs involved. Thus, structural components or sub-structure systems are
32 frequently tested to study their progressive collapse behaviors under large deformations. Such tests
33 include: a one-third scale four-bay and three-story planner RC frame (Yi et al. 2008), continuous
34 beam sub-assemblages (Su et al. 2009), interior beam-column sub-assemblages (Kai and Li 2011),
35 large-scale beam-column sub-assemblages (Sadek et al. 2011), a two-story steel moment frame
36 (Chen et al. 2011) and a steel-concrete composite frame with rigid beam-to-column connections
37 (Guo et al. 2013).

38 In addition to the experimental studies, numerical simulation techniques have also been
39 extensively employed in collapse analysis because they are effective and suitable for studying the
40 behavior of entire building structures. Parametric studies can also be easily conducted using
41 numerical means, covering static or dynamic, linear or nonlinear analysis. Extensive numerical
42 simulations of progressive collapse of frame structures have been performed, such as steel frames
43 analyzed using the pushdown method (Kim et al. 2009), a 20-story steel frame structure considering
44 different column removal scenarios (Fu 2009), an existing 8-story steel frame simulated using a
45 nonlinear dynamic finite element procedure (Kwasniewski 2010), RC frame wall structures based
46 on a macro model-based approach (Bao and Kunnath 2010), RC structures analyzed using a bar
47 fracture model (Sasani et al. 2011b) and a steel frame building using a 3-D nonlinear model (Li and
48 El-Tawil 2013). Despite these research efforts, existing numerical and theoretical studies primarily
49 focus on the frames itself within a frame shear wall structure. The performance and contribution of
50 the shear walls in the progressive collapse process are much less concerned.

51 In this study, the finite element models of two 15-story RC frame shear wall buildings, which

are designed according to the Chinese routine design codes, are established to evaluate their progressive collapse resistances via the nonlinear dynamic AP method. The structure that cannot effectively resist progressive collapse is re-designed using the linear static AP method specified in GSA guideline (GSA 2003), through which the reliability and efficiency of the existing design method are evaluated.

Design Details of Two Frame Shear Wall Building Models

The two frame shear wall building structures are referred to as Building A and Building B. They are designed following the Chinese design codes (i.e., the Code for Design of Concrete Structures (MOHURD 2010a) and the Code for Seismic Design of Buildings (MOHURD 2010b)). Buildings A and B, both of 15 stories, have equivalent story heights - the height of the first story is 4.5 m and that of each remaining stories (the second to the fifteenth stories) is 3.6 m. The total height of each building is 54.9 m. The dead load on each story is 7.0 kN/m^2 , whereas the live load on each story is 2.0 kN/m^2 . Both buildings have the same seismic design intensity of VII (the peak ground acceleration for a 10% exceedance probability in 50 years is $0.10 g$, where g is the acceleration of gravity) and site class of II (soil shear wave velocity $500 \text{ m/s} \geq v_s \geq 250 \text{ m/s}$). Such a seismic design requirement is equivalent to that the design ground acceleration is $0.10g$ for type B ground and a non-collapse requirement specified in Eurocode 8 (CEN 2003). Due to the same seismic design intensity and site class chosen for both buildings, they are regulated to have the same seismic resistance and the same detailing requirement, i.e. the minimum reinforcement ratio and the maximum axial compression ratio of the structural components are identical. This in turn yields an identical overall lateral resistance for both buildings. The difference between the two buildings is that: Building A is a weak wall-strong frame structure whilst Building B is a strong wall-weak frame system, as is shown in their floor plan (symmetrical half) (see Figure 1). That results in Building A having higher reinforcement ratio in its frame beams and columns than that of Building B, which in turn leads to higher redundancy of the frames in Building A in resisting progressive collapse. For Building B, on the other hand, more RC walls are arranged to provide adequate lateral resistance for seismic prevention. As is shown in Figure 1, the shear walls are singly arranged in Building A but in Building B, they are arranged in a common “C” shape.

Figure 1 Floor plan of the building models (unit: m)

Table 1 displays the dimensional and material parameters of the structural members in these two buildings. Although the strength of the longitudinal reinforcing steel in Building A is lower than that in Building B, the reinforcement ratio of Building A is much larger. Therefore, the lateral capacity of the frames in Building A is higher whereas the overall lateral capacities of the two are identical.

Table 1 Parameters of structural elements in Building A and Building B

	Building	Beams	Coupling beams	Columns	Shear walls (thickness)	Floor slabs (thickness)
Cross Section	A	300 mm × 600 mm	400 mm × 700 mm	600 mm × 600 mm	400 mm	120mm
	B	250 mm × 450 mm	300 mm × 700 mm	600 mm × 600 mm	300 mm	120mm
Concrete	A & B	C30 (f_c =20.1MPa)	1 st ~10 th story: C40 (f_c =26.8MPa) 11 th ~15 th story: C35 (f_c =23.4MPa)			
Longitudinal reinforcing steel	A	HRB335 (f_y =335MPa) ^a	HRB400 (f_y =400MPa)			
	B	HRB400 (f_y =400MPa)				
Hoop reinforcing steel	A & B	HPB 300 (f_y =300MPa)				

Note: f_c = compressive strength of concrete; f_y = yield strength of steel

^a As the beam section of Building A is relatively larger, lower strength steel is used to satisfy the minimum requirement of the reinforcement ratio specified in the design code. Otherwise, the beams will be significantly over-strength.

90 **The Finite Element Model**

91 The finite element models of Buildings A and B are established based on the general finite-element
92 program MSC.MARC (MSC 2007), which has an excellent nonlinear computational capacity. The
93 beams, columns and coupling beams are simulated using the fiber beam model developed by the
94 authors, whereas the shear walls are simulated using the multilayer shell model of MSC.MARC. In
95 addition to the brief description below of these two element models, more detailed information can
96 be found in the work of Lu et al. (2013a). In the existing literature (Tsai and Lin 2008, Kokot et al.
97 2012), the slab contribution to the progressive collapse resistance was often neglected which
98 produced conservative outcomes. This study follows the same strategies where the loads on the slab
99 (including its own weight) are assigned to the corresponding beams according to the load
100 distribution relationship.

101 ***Fiber Beam Model***

102 According to the successful work of Lu et al. (2013a), a section of the fiber beam model is divided
103 into 36 concrete fibers and 4 reinforcement fibers (see Figure 2a). Different material properties are
104 considered for steel, concrete cover (unconfined) and core (confined) concrete fibers. The concrete
105 constitutive model proposed by Légeron et al. (2005) is adopted to take into account the
106 confinement effect of the stirrups. The reinforcement fiber can be defined with equivalent
107 reinforcement area. Thus, cutoff of the longitudinal bars in beams can be modeled by altering the
108 reinforcement fiber areas at different sections along the beam. For the finite element models of
109 Buildings A and B, each frame element (beam or column) is subdivided into six fiber beam
110 elements to ensure sufficient accuracy (Lu et al. 2013a).

(a) Section mesh

(b) Comparison between numerical simulation and test result

Figure 2 Fiber beam model

111 In order to verify the feasibility and accuracy of the fiber beam model for simulating the
 112 structural collapse, an experimental test of RC assemblies under a column removal scenario (Lew et
 113 al. 2011) is simulated herein. Figure 2b indicates that the experimental load-displacement behavior
 114 can be well predicted by the fiber beam model. Validation of the fiber beam model on both system
 115 and component levels has also been done previously by Lu et al. (2013a) through simulation of two
 116 compressive-flexural column tests and a RC planner frame collapse test.

117 **Multilayer Shell Model**

118 In both buildings, the shear wall is made up of a middle wall and two boundary elements (see
 119 Figure 3a). In the numerical model, the integrated shear wall is modeled by the multilayer shell
 120 elements. The additional reinforcing bars within the boundary elements are simulated by the truss
 121 elements which are embedded in the multilayer shell model at the corresponding nodal locations
 122 (see Figure 3b). The mesh size of the multilayer shell element is 300mm×300mm and that of the
 123 truss element is also 300mm in the Z direction (see Figure 3b). The multilayer shell model is
 124 composed of a number of concrete and steel layers, for which the thickness and material properties
 125 can be defined separately. The reinforcing bars in the shear walls are defined as one or more steel
 126 layers with an equivalent reinforcement ratio. Based on the previous successful studies using the
 127 layered elements (Lu et al. 2011, Lu et al. 2013a), each of the shear walls concerned is divided into

ten concrete layers and four steel layers to simulate the longitudinal and transverse steel bars on both the inner and outer sides of the wall. Note that MSC.MARC defines the steel layers following a bilateral constitutive material law. As such, an orthotropic material model must be adopted for the steel layers. In other words, the material properties are defined in the longitudinal direction of the reinforcing bars while those in the orthogonal direction are defined as zero.

Figure 3 Modeling of the shear wall

The multilayer shell element model (including the embedded truss element) has been validated by Lu et al. (2013a) through simulation of a shear wall and two RC tube specimens. In addition, more successful applications of the above two types of elements in simulating structural collapses can be found in the work of Li et al. (2011) and Lu et al. (2011, 2013a, 2013b).

Nonlinear Dynamic AP Analysis

The nonlinear dynamic AP method is one of the most accurate methods for progressive collapse analysis because it explicitly considers material and geometric nonlinearities, as well as the dynamic effect. Using this method, progressive collapse resistances of these two RC frame shear wall structures can be comprehensively studied, following sudden removal of frame columns or shear walls. For column removals, four typical scenarios (i.e., a corner column, a long edge column,

143 a short edge column and an interior column) on each story (see Figure 1) are assumed to be
144 considered. Note that only one column is removed in each analysis.

145 For shear wall removals in Buildings A and B, different removal approaches are considered
146 according to DoD2010 (DoD 2010). The DoD2010 (DoD 2010) specifies that for each load-bearing
147 wall of length greater than $2H$, a length that is twice the clear story height H can be removed;
148 whereas if the length of the wall is less than $2H$, the entire length of the wall can be removed. Based
149 on these specifications, the entire length of the wall in the Y-direction (i.e., 6 m in Building A which
150 is less than $2H$ ($2H = 7.2$ m)) is removed, as shown in Figure 1a. In Building B, three removal
151 approaches of the inner load-bearing walls are implemented, as detailed in Figure 1b. “Removal
152 approach 1” is similar to that used for Building A, i.e., the entire length of the shear wall in the
153 Y-direction is removed. “Removal approach 2” refers to the removal of the entire length of the wall
154 in the X-direction. For “Removal approach 3”, a length of the wall that is equal to the story height
155 H in both the X- and Y-directions is removed from the inner walls, in which one or both of the
156 interconnecting walls is load-bearing (DoD 2010). In this study, the removal scenarios of the shear
157 walls from the bottom, mid and top stories (i.e., the 1st, 7th and 15th stories) are considered to
158 evaluate the progressive collapse responses of these two structures.

159 According to GSA2003 and DoD2010, the procedures of the nonlinear dynamic AP method
160 involve: (a) starting at a zero load, monotonically increase the acceleration of gravity from 0 to 1g
161 to the entire model (i.e., the column or wall section has not been removed yet) until the gravity load
162 is fully applied; and (b) after the static equilibrium in gravity load and internal forces is reached, the
163 target structural element is rapidly removed and the nonlinear dynamic response of the structure is
164 analyzed until the structure collapses or reaches a new steady state. In this paper, the DoD2010
165 collapse criterion for RC frames is used to identify the structural collapse. In the following analysis,
166 it is found that when the vertical displacement of the joint (at the top of the removed column or
167 shear wall) exceeds the threshold value specified in DoD2010, i.e. 1/5 of the shortest span of the
168 connecting beams, the deformation of the structure continues to develop without converging, which
169 confirms the rationality of the threshold value as the collapse criterion.

170 **Progressive Collapse Resistance of the Two Structures**

171 ***Removal of the Frame Columns in Building A***

172 As summarized in Table 2, no progressive collapse occurs in Building A irrespective of the removal
173 of any column from any story. As a typical example, Figure 4 presents the structural behavior
174 following the removal of the corner column from the 1st-story. To better illustrate the overall
175 deformation of this building, Figure 4a shows its deformed shape with a deformation amplification
176 factor of 20. The vertical displacement-time history responses of the joints at the top of the removed
177 corner columns from selected stories are illustrated in Figure 4b, in which the removal of a column
178 (or a shear wall in the last two sub-sections) on the x^{th} -story is represented by " x^{th} ". Note that the
179 displacement-time history response is presented from 0-1 s only in Figure 4b because the
180 displacements are found to reach a stable free vibration status prior to 1 s. Figure 4b also indicates
181 that amongst the removal scenarios from three different stories, the removal of the 15th-story
182 column results in the minimum displacement. When the column from the lower story is removed,
183 larger axial force in the column is subsequently released, resulting in a larger vertical displacement
184 at the joint (top of the removed column). It can be observed from Figure 4b that the largest vertical
185 displacement of the joint is only 37.8 mm, much smaller as compared to the span of the beam (5 m).
186 According to the work of Li et al. (2011), when the vertical displacement of the joint is small,
187 progressive collapse is predominantly resisted by the bending moment of the beam ends, namely, by
188 the flexural capacity of the beam. For Building A, similar phenomena are also observed when the
189 edge or interior column is removed. This is due to the strong frames of this building, in which the
190 reinforcement in the beam ends exhibits higher redundancy. This in turn leads to higher flexural
191 capacities of the beam ends and much enhanced progressive collapse resistance of the frames within
192 Building A. The above analysis results confirm that Building A can provide an effective alternative
193 load path after any typical frame column is removed; thus, progressive collapse can be prevented in
194 this building.

195

Table 2 Outcome of frame column removal scenarios for Building A and Building B

Building analyzed	Removed column			
	Corner column	Short edge column	Long edge column	Interior column
Building A	No collapse	No collapse	No collapse	No collapse
Building B	Collapse	No collapse	Collapse	Collapse

Figure 4 Removal of the corner column in Building A

198 **Removal of the Frame Columns in Building B**

199 For Building B, Table 2 indicates that progressive collapse does not occur when the short edge
200 column on any story is removed. However, collapse is triggered when the corner, the long edge or
201 the interior column is removed from any story. Following removal of the short edge column, the
202 amount of unbalanced gravity loads resisted by the connecting beams is less than that resisted by
203 the beams in the other areas. This implies that the progressive collapse resistance demand under the
204 short edge column removal scenario is the lowest, which enables the survival of the building after a
205 column removal catastrophe. As a typical example, Figure 5 demonstrates the removal scenario of
206 the long edge column in Building B. The deformed shape due to the 7th-story column removal is
207 presented in Figure 5a, in which progressive collapse is found to occur on and above the 7th-story.
208 Figure 5b shows that the vertical displacement of the joint continues to increase to more than 1 m

209 (i.e., 1/5 of the beam span) without converging, and an irreversible collapse process occurs. Figure
 210 5b also displays the displacement-time history responses after the long edge column is removed
 211 from the 1st and the 15th stories. Similar progressive collapse phenomena are also observed.

Figure 5 Removal of the long edge column in Building B

212 As mentioned previously, the frames in Building B are weaker than those in Building A,
 213 resulting in a significantly lower collapse resistance than that of Building A. The analysis results
 214 reveal that the chosen high-rise RC frame shear wall structure (Building B), designed to provide
 215 lateral capacity mainly via the shear walls, presents a greater risk of progressive collapse on its
 216 frame component; thus, a special collapse prevention design is required.

217 **Removal of the Shear Wall in Building A**

218 For Building A, the analysis results indicate that progressive collapse does not occur when the shear
 219 wall is removed from any representative story (i.e., 1st, 7th, and 15th stories). Figure 6a illustrates the
 220 deformed shape of Building A after the shear wall in the Y-direction is removed from the 1st-story.
 221 Figure 6b shows the vertical displacement-time history responses of the joints (at the top of the
 222 removed wall) when the shear wall is removed from the x^{th} -story ($x=1, 7$ and 15). It is noted that
 223 when the shear wall on the lower story is removed, the joint displacement is larger and it takes
 224 longer for the structure to reach a stable state. Although a significant displacement is observed, the
 225 structure can reach a new stable state and progressive collapse does not occur. The maximum
 226 displacement of the joint is approximately 118 mm, which is also significantly smaller than the

227 threshold value for progressive collapse (i.e., 1/5 of the beam span, 1000 mm).

Figure 6 Removal of the shear wall in Building A

228 **Removal of the Shear Wall in Building B**

229 For Building B, the shear wall removal is considered under three different removal approaches. The
230 analysis results reveal that regardless of which removal approach is selected, the structure can
231 adequately redistribute the unbalanced gravity loads to prevent progressive collapse after the shear
232 wall is removed from any representative story. Figure 7a shows the deformed shape after part of the
233 interconnecting shear wall is removed from the 1st-story (Removal approach 3 in Figure 1b). The
234 vertical displacement-time history responses of the joints (at the top of the removed wall), as
235 illustrated in Figure 7b, indicate that the structural deformation is very small (approximately 0.52
236 mm).

(a) After the 1st-story shear wall is removed
(Removal approach 3) ($t=1.00$ s / unit: mm)

(b) Vertical displacement-time history after the removal of the
1st-story shear wall

Figure 7 Removal of the 1st-story shear wall in Building B

Although the design redundancy of the frame (mainly influenced by the reinforcement ratio and the member size) in Building B is much lower than that in Building A, the vertical displacement of Building B following the removal of the x^{th} -story shear wall is much smaller. This is because the redundancy of the shear walls (mainly influenced by the number and arrangement of the shear walls while the reinforcement ratio has less effect) in Building B is larger as its shear walls are interconnected to form a “C” shape (Figure 1b). Thus the deflection response of Building B is significantly reduced, resulting in better performance than Building A, in which the shear walls are arranged in only one direction. For Building A, in spite of the lower redundancy of the shear walls, the frame connected with the removed shear wall has higher reinforcement ratio (in order to meet seismic resistant requirements) which can serve as an alternative load path to resist progressive collapse. In summary, in RC frame shear wall structures designed to satisfy the seismic requirements, the shear walls are robust enough to resist progressive collapse.

Despite the fact that both Buildings A and B can resist progressive collapse after the removal of the shear wall, their alternative load paths are rather different. Figure 8 displays the reaction forces F_Z at the column supports after the removal of the 1st-story shear wall in the Y-direction for Building A (Figure 8a) and Building B (Removal approach 1, Figure 8b). In Building A, the redistribution of the internal forces mainly relies on the frame columns adjacent to the removed

shear wall. On the other hand, the internal force redistribution in Building B relies primarily on the remaining interconnecting shear walls. Hence, Building B provides higher progressive collapse resistance under wall removal scenarios than Building A, because its “C” shaped walls have much higher redundant capacity than that of the frames in Building A.

Figure 8 Reaction force F_z at the column supports (unit:1000kN)

Note: n_1 ; (n_2); $n_3\%$. n_1 : F_z before the removal of the shear wall; n_2 : F_z after the removal of the shear wall; n_3 : percentage difference in F_z

Progressive Collapse Prevention Design of the Frames in Building B

The above nonlinear dynamic AP analysis results indicate that the collapse resistance of the frames in Building B is inadequate. As such, Building B is re-designed against progressive collapse through the linear static AP method specified in *Section 4.1.2* and *Section 4.1.3* of GSA2003. The step-by-step design procedure is as follows: (1) from the top floor to the bottom floor, remove a frame column from the location being considered (four respective locations on each floor, as is shown in Figure 1b) and conduct a linear-static analysis of the remaining structure. Only one column is removed in each analysis. The static loads applied to the model follow the values specified in GSA2003 (i.e. Load = 2(DL + 0.25LL), where DL is the dead load and LL is the live load.). From the static analysis, the internal forces (e.g. the bending moments of the frame elements) Q_{UD} can be obtained. (2) Conservatively set the DCR values as 1.5 (for atypical structural configurations in GSA2003) for the frame elements. Then the expected ultimate Q_{CE} of the frame

elements can be obtained by Q_{UD}/DCR . (3) If the ultimate loading capacity of the frames is lower than Q_{CE} , then the frame element is re-designed (by increasing the amount of reinforcement) according to Q_{CE} and the increased steel percentages are calculated. The analysis indicates that the reinforcement ratio in frame columns is not necessary to be increased. For the mid-span sections of beams, their ultimate bending capacities are usually larger than Q_{CE} and only a few sections need to be re-designed by increasing the reinforcement ratio. However, the ultimate bending capacities at the beam ends are much smaller than Q_{CE} and therefore the reinforcement ratio at these locations must be considerably increased. This is in accordance with the research findings of Li et al. (2011) in that progressive collapse resistance is predominantly provided by the bending moment of the beam ends under small deformations. Table 3 summarizes the reinforcement amount in the frame beams before and after performing the progressive collapse prevention design. It can be seen that the reinforcement amount has an 8.70% increase in the re-designed structure.

Table 3 Comparison of the amount of longitudinal reinforcement in beams using different design methods for Building B

Story	Original	Linear static AP method			Nonlinear dynamic AP method		
	Steel reinforcement (t)	Steel reinforcement (t)	Increase* (t)	Percentage increase	Steel reinforcement (t)	Increase* (t)	Percentage increase
15	2.415	2.632	0.217	9.00 %	2.478	0.063	2.61 %
14	3.001	3.187	0.186	6.17 %	3.086	0.085	2.83 %
13	3.001	3.169	0.168	5.57 %	3.086	0.085	2.83 %
12	3.004	3.189	0.185	6.13 %	3.092	0.088	2.93 %
11	3.004	3.200	0.196	6.52 %	3.092	0.088	2.93 %
10	3.121	3.326	0.205	6.59 %	3.215	0.094	3.01 %
9	3.121	3.333	0.212	6.82 %	3.215	0.094	3.01 %
8	3.121	3.344	0.223	7.17 %	3.215	0.094	3.01 %
7	3.121	3.357	0.236	7.59 %	3.215	0.094	3.01 %
6	2.714	3.023	0.309	11.37 %	2.751	0.037	1.36 %
5	3.031	3.298	0.267	8.79 %	3.107	0.076	2.51 %
4	2.721	3.032	0.311	11.43 %	2.840	0.119	4.37 %
3	2.721	3.049	0.328	12.05 %	2.840	0.119	4.37 %
2	2.721	3.068	0.347	12.76 %	2.840	0.119	4.37 %
1	2.598	2.983	0.385	14.81 %	2.718	0.120	4.62 %
Total	43.415	47.190	3.775	8.70 %	44.790	1.375	3.17 %

Note: t = tonne.

^a Compared with the original structure.

For comparison, the nonlinear dynamic AP method is also used to re-design Building B through which the minimum demand of reinforcement to resist progressive collapse is obtained. The design procedure follows that: (1) from the top floor to the bottom floor, conduct a dynamic analysis for each column removal scenario as explained in the section “*Nonlinear Dynamic AP Analysis*”. Again, only one column is removed in each analysis. It should be noted that the loads applied to the model are equal to $DL+0.25LL$. (2) If collapse occurs after a column is removed, then slightly increase the reinforcement ratio in the frame beams connected to or exactly above the removed column. Note that, in this step, to ensure minimum reinforcement demand is met, the reinforcement ratio should be gradually increased. (3) Re-run Step 1 until no collapse occurs. During the nonlinear dynamic design process, the initial static analysis results should be referred to, in particular those beam sections having much lower bearing capacity than Q_{CE} . In addition, increasing the reinforcement ratio at the beam ends is more effective to prevent collapse than increasing it at the mid-span sections. Table 3 also lists the minimum demand of reinforcement (with a 3.17% increase) in resisting progressive collapse which is calculated via the nonlinear dynamic AP method. It is evident that the linear static AP method is slightly conservative as compared to the minimum reinforcement demand required by the nonlinear dynamic AP method.

To verify the reliability of the linear static AP method, the re-designed Building B is analyzed again using the nonlinear dynamic AP method. Results show that the re-designed structure can adequately resist progressive collapse following removal of any frame column from any story. This suggests that the progressive collapse resistance of Building B can be significantly enhanced after a collapse prevention design is implemented. Results also demonstrate that the linear static AP method specified in GSA2003 is reliable and efficient for collapse prevention designs of typical and representative high-rise RC frame shear wall structures.

Conclusion

Two typical 15-story RC frame shear wall building models, with equivalent overall lateral resistance to seismic actions, are established in this study to evaluate their progressive collapse resistances. However the structural layouts in resisting the lateral forces are quite different for the

314 two buildings: One is a weak wall-strong frame structure and the other is a strong wall-weak frame
315 system. 3-D finite element models are constructed based on general finite element program
316 MSC.MARC. The modeling techniques are described in some detail with validations. The
317 progressive collapse resistances of the two building models are evaluated via the nonlinear dynamic
318 AP method under various column (shear wall) removal scenarios. The analysis results demonstrate
319 that: if the RC frame shear wall building is designed to resist the horizontal seismic actions mainly
320 through the shear walls, the collapse resistance of the frames tends to be inadequate and thus, a
321 special collapse prevention design is required. On the other hand, if the frames serve as the main
322 lateral-resistant components, then the overall collapse resistance can be enhanced by the seismic
323 design and in turn a progressive collapse can be adequately prevented. Further, this study suggests
324 that the shear walls should be interconnected in the floor plane, for example in a “C” or a tube shape,
325 provided that the functional requirements can be met. This is because interconnected shear walls
326 can provide adequate alternative load path and thus possess superior collapse resistance. Moreover,
327 detailed progressive collapse prevention design procedure is given for frames with inadequate
328 collapse resistance. The design and analysis outcomes also confirm that the linear static AP method
329 specified in GSA2003 is reliable and efficient for progressive collapse prevention designs of typical
330 and representative high-rise RC frame shear wall structures.

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336 **References**

- 337 Bao, Y., and Kunnath, S. K. (2010). “Simplified progressive collapse simulation of RC frame-wall structures.”
338 *Eng. Struct.*, 32(10), 3153-3162.
- 339 Chen, J., Huang, X., Ma, R., and He, M. (2011). “Experimental study on the progressive collapse resistance of a
340 two-story steel moment frame.” *J. Perform. Constr. Facil.*, 26(5), 567-575.

341 Department of Defense (DoD). (2010). *Unified facilities criteria (UFC): design of structures to resist progressive*
342 *collapse*, Washington (DC).

343 Ellingwood, B. R., and Leyendecker, E. V. (1978). "Approaches for design against progressive collapse." *J. Struct.*
344 *Div.*, 104(3), 413-423.

345 European Committee for Standardization (CEN). (2003). *Eurocode 8: Design of structures for earthquake*
346 *resistance. Part 1: General rules, seismic actions and rules for buildings*, Brussels.

347 Fu, F. (2009). "Progressive collapse analysis of high-rise building with 3-D finite element modeling method." *J.*
348 *Constr. Steel Res.*, 65(6), 1269-1278.

349 General Service Administration (GSA). (2003). *Progressive collapse analysis and design guidelines for new*
350 *federal office buildings and major modernization projects*, GSA, Washington, DC.

351 Guo, L. H., Gao, S., Fu, F., and Wang, Y. Y. (2013). "Experimental study and numerical analysis of progressive
352 collapse resistance of composite frames." *J. Constr. Steel Res.*, 89, 236-251.

353 Kai, Q., and Li, B. (2011). "Experimental and analytical assessment on RC interior beam-column subassemblages
354 for progressive collapse." *J. Perform. Constr. Facil.*, 26(5), 576-589.

355 Kim, T., Kim, J., and Park, J. (2009). "Investigation of progressive collapse-resisting capability of steel moment
356 frames using push-down analysis." *J. Perform. Constr. Facil.*, 23(5), 327-335.

357 Kokot, S., Anthoine, A., Negro, P., and Solomos, G. (2012). "Static and dynamic analysis of a reinforced concrete
358 flat slab frame building for progressive collapse." *Eng. Struct.*, 40, 205-217.

359 Kwasniewski, L. (2010). "Nonlinear dynamic simulations of progressive collapse for a multistory building." *Eng.*
360 *Struct.*, 32(5), 1223-1235.

361 Légeron, F., Paultre, P., and Mazars, J. (2005). "Damage mechanics modeling of nonlinear seismic behavior of
362 concrete structures." *J. Struct. Eng.*, 131(6), 946-954.

363 Lew, H. S., Bao, Y., Sadek, F., Main, J. A., Pujol, S., and Sozen, M. A. (2011). "An experimental and
364 computational study of reinforced concrete assemblies under a column removal scenario." *NIST Technical*
365 *Note 1704*, National Institute of Standards and Technology (NIST), Gaithersburg, MD.

366 Li, H., and El-Tawil, S. (2013). "Three-dimensional effects and collapse resistance mechanisms in steel frame
367 buildings." *J. Struct. Eng.*, 10.1061/(ASCE) ST.1943-541X.0000839.

368 Li, Y., Lu, X. Z., Guan, H. and Ye, L. P. (2011). "An improved tie force method for progressive collapse resistance
369 design of reinforced concrete frame structures." *Eng. Struct.*, 33(10), 2931-2942.

370 Lu, X., Lu, X. Z., Guan, H., and Ye, L. P. (2013a). "Collapse simulation of reinforced concrete high-rise building
371 induced by extreme earthquakes." *Earthq. Eng. Struct. Dyn.*, 42(5), 705-723.

372 Lu, X. Z., Lu, X., Guan, H., Zhang, W. K., and Ye, L. P. (2013b). "Earthquake-induced collapse simulation of a
373 super-tall mega-braced frame-core tube building." *J. Constr. Steel Res.*, 82, 59-71.

374 Lu, X., Lu, X. Z., Zhang, W. K., and Ye, L. P. (2011). "Collapse simulation of a super high-rise building subjected
375 to extremely strong earthquakes." *Science China Technological Sciences*, 54(10), 2549-2560.

376 Ministry of Housing and Urban-Rural Development of the People's Republic of China (MOHURD). (2010a).
377 *Code for design of concrete structures. (GB50010-2010)*, Beijing.

378 Ministry of Housing and Urban-Rural Development of the People's Republic of China (MOHURD). (2010b).
379 *Code for seismic design of buildings. (GB50011-2010)*, Beijing.

380 MSC. MARC (2007). "*Element library.*" *User's Manual*, vol. B, MSC. Software Corporation.

381 Sadek, F., Main, J. A., Lew, H. S., and Bao, Y. (2011). "Testing and analysis of steel and concrete beam-column
382 assemblies under a column removal scenario." *J. Struct. Eng.*, 137(9), 881-892.

383 Sasani, M., Kazemi, A., Sagioglu, S., and Forest, S. (2011a). "Progressive collapse resistance of an actual
384 11-story structure subjected to severe initial damage." *J. Struct. Eng.*, 137(9), 893-902.

385 Sasani, M., Werner, A., and Kazemi, A. (2011b). "Bar fracture modeling in progressive collapse analysis of
386 reinforced concrete structures." *Eng. Struct.*, 33(2), 401-409.

387 Su, Y., Tian, Y., and Song, X. (2009). "Progressive collapse resistance of axially-restrained frame beams." *ACI*
388 *Struct. J.*, 106(5), 600-607.

389 Tsai, M. H., and Lin, B. H. (2008). "Investigation of progressive collapse resistance and inelastic response for an
390 earthquake-resistant RC building subjected to column failure." *Eng. Struct.*, 30(12), 3619-3628.

391 Yi, W. J., He, Q. F., Xiao, Y., and Kunnath, S. K. (2008). "Experimental study on progressive collapse-resistant
392 behavior of reinforced concrete frame structures." *ACI Struct. J.*, 105(4), 433-4