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by

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Progressive Collapse-Resisting Capacity of Steel Moment Frames Considering Panel Zone Deformation

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Abstract: Panel zone deformation is known to be a major parameter affecting the nonlinear behaviour of moment resisting steel frames subjected to earthquake loads, especially in weak column-strong beam conditions. In this study, the reinforcing effect of the panel zone on the progressive collapse and the capacity of moment resisting steel frames was investigated with the aid of nonlinear dynamic analysis. Both gravity load-resisting system structures with weak panel zones and lateral load-resisting system structures with strong panel zones were studied. The analysis results show that the panel zone deformation is highly dependent on the location of removed columns. The panel zone deformation turns out to be very significant when a column adjacent to an exterior panel zone is removed. On the other hand, the panel zone effect is not significant if a column adjacent to an interior panel zone is removed. As the number of stories increases, the effect of a weak panel zone on the overall behaviour decreases regardless of the panel zone strength and the location of any column removed.

Key words: steel moment resisting frames, panel zone, progressive collapse, nonlinear dynamic analysis.

1. INTRODUCTION

The term ‘progressive collapse’ is used to describe the effects of a series of local failures due to local damage to structural elements caused by abnormal loads which ultimately results in global collapse of the structure. From a series of accidents it has been observed that, in order to prevent progressive collapse, a structure should have sufficient continuity to offer an alternative path to stability of the structure even if an element of a vertical load-resisting system is removed. To prevent progressive collapse, the National Building Code of Canada (1996) specifies requirements for the design of major elements, the establishment of connection elements, and the ways of providing load transfer paths. Eurocode 1 (2002) provides a design standard for the selection of plan types suitable for preventing progressive collapse, and recommends that buildings should be integrated. In the United States, specific provisions related to progressive collapse are not yet provided in design codes such as the International Building Code (ICC 2006); however the American Concrete Institute (ACI 318 2002) requires structural integrity so that partial damage due to abnormal load does not result in total collapse. The ASCE 7-05 (2005) recommends a design method and load combinations as well as structural integrity. The General Service Administration (GSA) provides a practical design guideline to reduce the collapse potential of federal buildings (GSA 2003), and the Department of Defense (DoD) also presents a guideline for new and existing DoD buildings (DoD 2005). The analysis method recommended in these guidelines is the alternate path (AP) method, in which the following four procedures are specified for progressive collapse analysis: linear elastic static (LS) analysis, linear elastic dynamic (LD) analysis, nonlinear static (NS) analysis, and nonlinear dynamic (ND) analysis. The AP method is executed in the following manner: 1) remove an arbitrary column and

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conduct an analysis; 2) check the limit state of elements; 3) if the limit state is exceeded, remove the element and re-distribute the loads to adjacent elements; and 4) repeat analysis until no element exceeds the limit state.

Much research has been carried out regarding the validity and applicability of the various analysis methods recommended in design guidelines for accurate prediction of progressive collapse (Powell 2005; Marjanishvili and Agnew 2006; Kim and Kim 2008a,b). Khandelwal and El-Tawil (2007) presented a calibrated micromechanical constitutive model for steel to investigate a number of key design variables that influence the formation of catenary action in special steel moment resisting frame sub-assemblies. Kim and An (2008) investigated the effect of catenary action on the progressive collapse potential of steel structures. Khandelwal *et al.* (2008) applied a macro analysis model to investigate the resistance to progressive collapse of seismically designed steel braced frames.

Even though in depth analytical and experimental studies have been carried out to evaluate the progressive collapse potential of steel structures as stated above, the effect of a weak panel zone on the performance of a steel structure subjected to the loss of any load-resisting element has not yet been investigated. When panel zones are reinforced as in the case for special moment resisting frames, the consideration of panel zone deformation may not be important; however in structures located in regions of medium to low seismic activity, where a strong column-weak beam concept is not applied and panel zones are not required to be strengthened, the panel zone deformation may be significant when the structure is subjected to extreme loads large enough to cause progressive collapse. In fact, significant panel zone deformation was observed in our experiment of steel moment connections, where weak column-strong beam combination was subjected to monotonically increasing load (Kim *et al.* 2007).

In this study the progressive collapse of steel moment-resisting frames has been investigated taking into account the deformation of panel zones. The vertical deflections of model structures subjected to the sudden removal of a first storey column were obtained by nonlinear dynamic analysis using the program code OpenSees (Mazzoni *et al.* 2006). The maximum displacements and the angle of rotation of plastic hinges obtained with and without allowing for panel zone strength were compared. The effects of the location of the removed column, the number of storeys, and whether or not seismic load is included were also investigated.

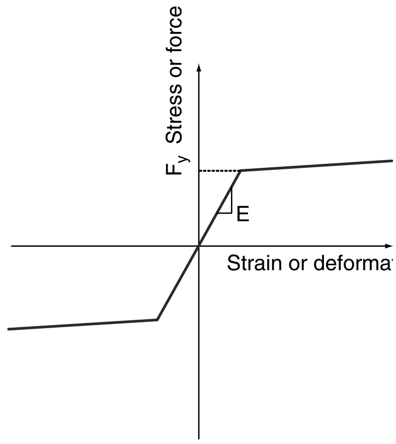


Figure 1. Material modeling for steel members (Steel01)

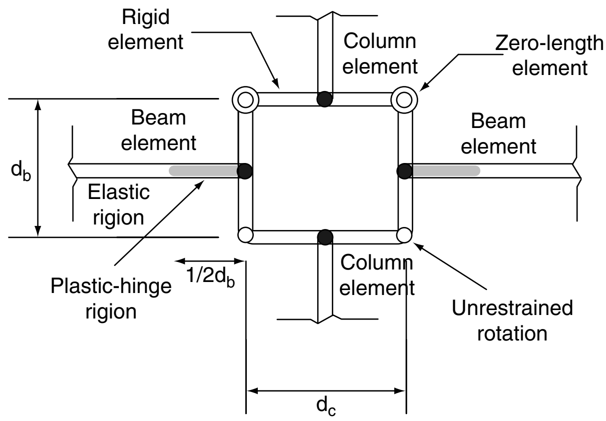


Figure 2. Modeling of beam-column joint with panel zone (Gupta and Krawinkler 1999)

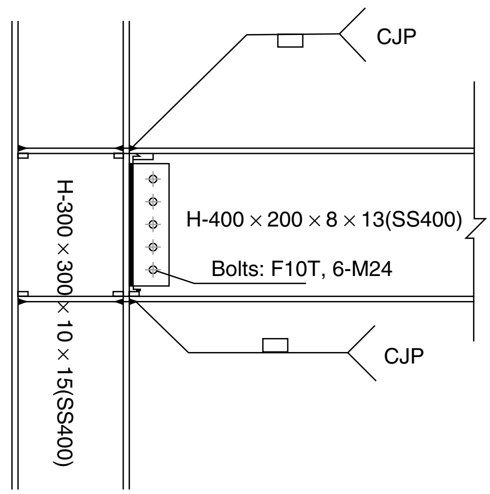


Figure 3. Specimen for monotonic loading test (Kim *et al.* 2007)

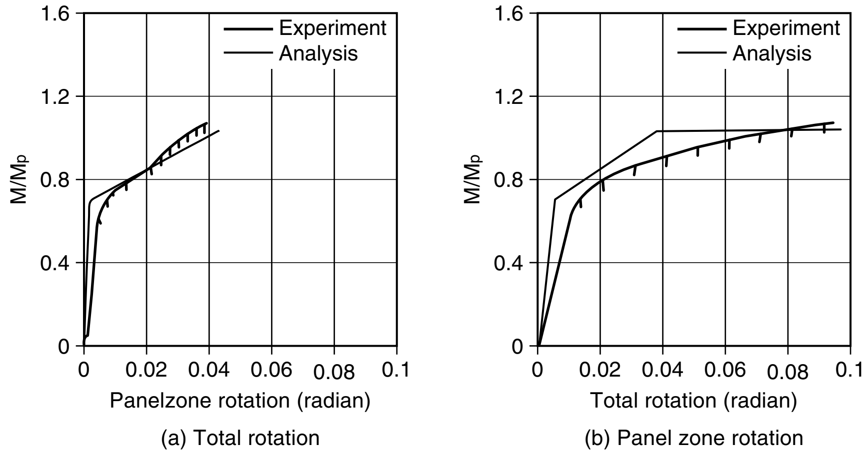


Figure 4. Comparison of moment-rotation relationships of steel joint obtained from analysis model and experiment

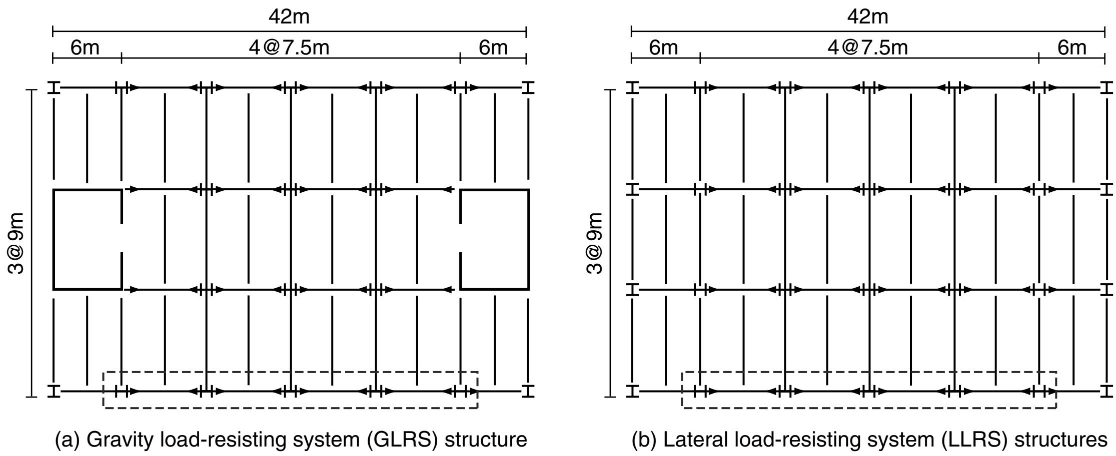


Figure 5. Structural plan of analysis model structures

2. MODELING OF STRUCTURAL ELEMENTS

The columns and beams in the example structures were modeled using the 'Nonlinear Beam-Column' and the 'Beam with Hinges' elements provided by OpenSees, respectively. In addition, the 'Steel01' and 'Hysteretic' material models were used for columns and beams, respectively. Figure 1 shows the bilinear load-displacement relationship of the 'Steel01' material model. The post-yield stiffness was assumed to be 2% of the initial stiffness.

In this study the panel zone model of Gupta and Krawinkler (1999) shown in Figure 2 was used to simulate a panel zone adopting the full dimensions of the panel zone with rigid links, and two bilinear springs were employed to control the deformation of the panel zone. The panel zone springs were modeled by the 'zeroLength' element and the 'Steel01' material model. To check the validity of the panel zone modeling, the predicted panel zone bending moments and the corresponding rotation angles were compared with the results of monotonic loading test on the non-seismic beam-column joints (Kim *et al.* 2007) as shown in Figure 3. The beam and the column are composed of H-400 × 200 × 8 × 13 and H-300 × 300 × 10 × 15 sections, respectively. Figure 4(a) compares the bending moment obtained by the applied load multiplied by the beam length and the rotation angle obtained by dividing the beam-end displacement by the beam length. Figure 4(b) compares the bending moment with the shear deformation of the panel zone. It is shown that, even though the stiffness predicted by the analysis model is slightly larger than the experimental result, the prediction of the overall behavior generally matches well with the experimental results.

3. DESIGN OF ANALYSIS MODEL STRUCTURES

The analysis model structures are 3-, 6-, and 15-storey steel moment-resisting frames the plans of which are shown in Figure 5. Two types of load-resisting systems were considered: a gravity load resisting system (GLRS) designed only for gravity load (Figure 5a) and a lateral load resisting system (LLRS) designed for both gravity and lateral loads (Figure 5b). The design load

Table 1. Member sizes of model structures (mm)

(a) 3-storey

Members	Storey	GLRS	LLRS
Columns	1~3	H 208 × 202 × 10 × 16	H 304 × 301 × 11 × 17
Beams		H 340 × 250 × 9 × 14	H 340 × 250 × 9 × 14

(b) 6-storey

Members	Storey	GLRS	LLRS
Columns	1~3	H 298 × 299 × 9 × 14	H 400 × 400 × 13 × 21
	4~6	H 208 × 202 × 10 × 16	H 208 × 202 × 10 × 16
Beams	1~3	H 340 × 250 × 9 × 14	H 450 × 200 × 9 × 13
	4~6		H 450 × 209 × 9 × 14

(c) 15-storey

Members	Storey	GLRS	LLRS
Columns	1~3	H 394 × 405 × 18 × 18	H 800 × 800 × 65 × 65
	4~6	H 350 ×357 × 19 × 19	H 750 × 750 × 45 × 50
	7~9	H 344 × 348 × 10 × 16	H 650 × 650 × 35 × 35
	10~12	H 298 × 299 × 9 × 14	H 550 × 550 × 25 × 25
	13~15	H 208 × 202 × 10 × 16	H 300 × 305 × 15 × 20
Beams	1~3	H 450 × 200 × 9 × 14	H 500 × 200 × 10 × 16
	4~6	H 450 × 200 × 9 × 14	H 340 × 250 × 9 × 14
	7~9	H 450 × 200 × 9 × 14	H 404 × 201 × 9 × 15
	10~12	H 450 ×200 × 9 × 14	H 404 × 201 × 9 × 15
	13~15	H 496 × 199 × 9 × 14	H 450 × 200 × 9 × 14

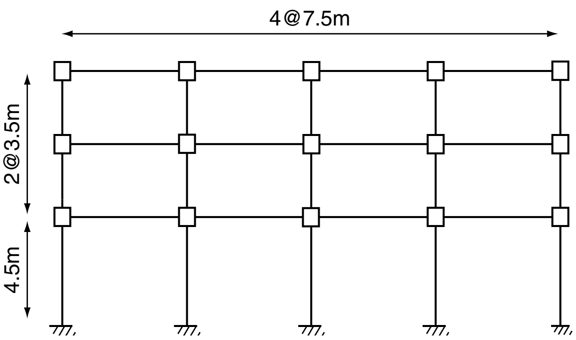


Figure 6. Side-view of 3-storey model structure

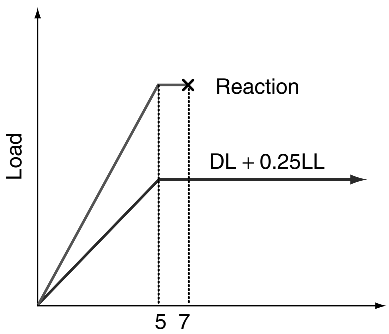


Figure 8. Application of vertical load for dynamic analysis

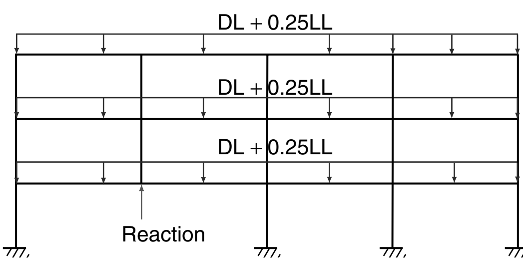


Figure 7. Gravity loads for progressive collapse analysis

was based on the Korean Building Code (KBC 2005) and member design followed AISC LRFD (AISC 2000). The design seismic load corresponds to $S_{DS}=0.36g$ and $S_{D1}= 0.15g$ in IBC (ICC 2006) format. The two-dimensional frames indicated by the dotted rectangular box in Figure 5 were analyzed separately for progressive collapse. The dimensions of the selected members are shown in Table 1. The structural steel properties used for columns and beams were SM490 ($F_y=324$ MPa) and SS400 ($F_y=235$ MPa), respectively. Figure 6 shows the side view of the 3-storey analysis model structure.

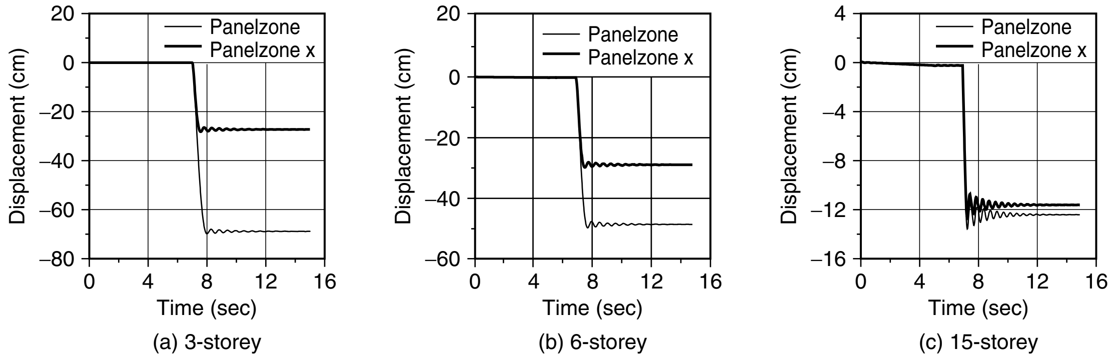


Figure 9. Nonlinear dynamic analysis results of the GLRS model structures with the second column removed

4. ANALYSIS RESULTS

The effects of panel zones were investigated by analyzing model structures both with and without the flexibility of panel zones taken into account and comparing the differences in the vertical deflection, the rotation of beams, and the panel zone deformation. Nonlinear dynamic analyses of the model structures were then performed after removing a column located in the middle of the frame as well as the second column from the corner. The load combination recommended in the GSA guidelines was used for the analysis.

4.1. Nonlinear Dynamic Analysis Procedure for Progressive Collapse

Nonlinear analysis procedures generally provide a more sophisticated analysis than linear procedures in their ability to characterize the performance of a structure. However, advances in computer hardware and general-purpose analysis software packages have now made it possible to employ nonlinear assessment techniques on large and complex structures, including the dynamic time history nonlinear response of high-rise structures containing thousands of members and connections.

Progressive collapse is generally initiated by the sudden loss of one, or many, structural members. Once a structural member (usually a column in the first storey) is suddenly removed, the stiffness matrix of the system also needs to be suddenly changed. This may cause difficulty in the analytical modelling process. To avoid this problem, all member forces were first obtained from the full structural model subjected to the applied load. The structure was then re-modeled with the appropriate column removed and its member forces applied to the structure as dummy forces to maintain equilibrium (Figures 7 and 8). The preliminary analysis results showed that the structure became stable after 5 seconds. The member force was suddenly removed after 7 seconds to initiate progressive collapse. In this way the progressive collapse analysis started from the moment that the structure was already deformed by the applied load, which reflected the loading situation quite realistically.

4.2. Panel Zone Effect in the Gravity Load-Resisting Structures

To evaluate the panel zone effect, nonlinear dynamic progressive collapse analyses were performed by suddenly removing the middle column and the second column from the corner. Figures 9 and 10 compare the vertical deflections for the 3-, 6-, and 15-storey GLRS model structures with and without considering the panel zone effect. The yield displacements, the maximum displacements, and the ductility demands are shown in Table 2. It can be observed that in all cases the deflections generally increased when panel zone flexibility was allowed for. It can also be noticed that the difference in vertical deflections caused by removal of the second and the middle columns increased significantly when the panel zone effect was taken into account. The panel zone effect caused deflection to increase 147% at the second column, but only 47% at the middle column. This implies that the effect of panel zone flexibility depends significantly on the location of the removed column. This is because panel zone deformation itself depends on the location of removed columns. Figures 11 and 12 depict the plastic rotations of beams and panel zones in the 3-storey model structure with the second and the middle columns removed, respectively. It can be observed in Figure 11(a) that the plastic rotations at the left-hand-ends of the left-hand-side beams are 0.03 radian when the panel zone effect was not allowed for, whereas these rotations are zero when the panel zone effect was considered (Figure 11b). Figure 15(a) shows that, in this case, the panel zone plastic rotation is 0.1 radian, which implies that all plastic rotations are affected by panel zone deformation. When the middle column was removed, however, the panel zone deformation turned out to be 0.008 radian as shown in Figure 15(b), which is quite small compared with the case when the second column

Table 2. Ductility demand of GLRS structures subjected to sudden column loss

Storey	Removed column	Without panel zone effect: Yield displacement (mm)	Without panel zone effect: Maximum displacement (mm)	Without panel zone effect: Ductility demand	With panel zone effect: Yield displacement (mm)	With panel zone effect: Maximum displacement (mm)	With panel zone effect: Ductility demand
3	Corner	–103	–1718	16.7	–93	–3123	33.6
	Second	–71	–282	4.0	–54	–699	12.9
	Middle	–67	–256	3.8	–76	–379	5.0
6	Corner	–101	–1503	14.9	–93	–	–
	Second	–73	–298	4.1	–57	–497	8.7
	Middle	–67	–230	3.4	–74	–253	3.4
15	Corner	–78	–739	9.5	–74	–	–
	Second	–57	–127	2.2	–55	–137	2.5
	Middle	–61	–123	2.0	–56	–135	2.4

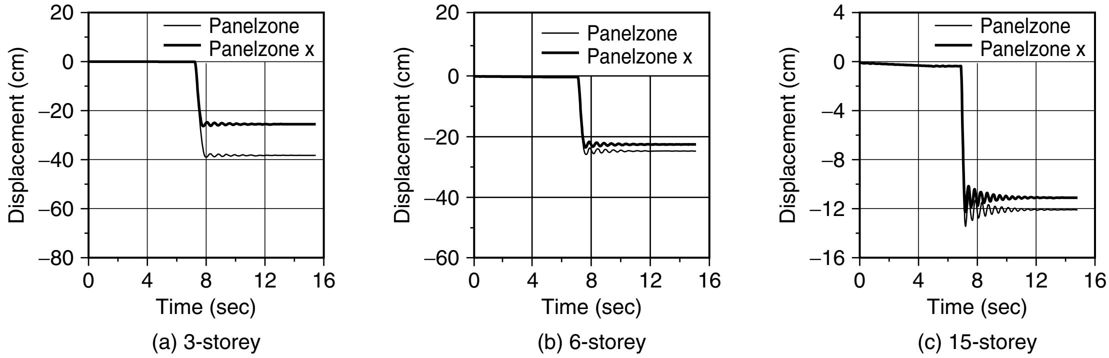


Figure 10. Nonlinear dynamic analysis results of the GLRS model structures with the middle column removed

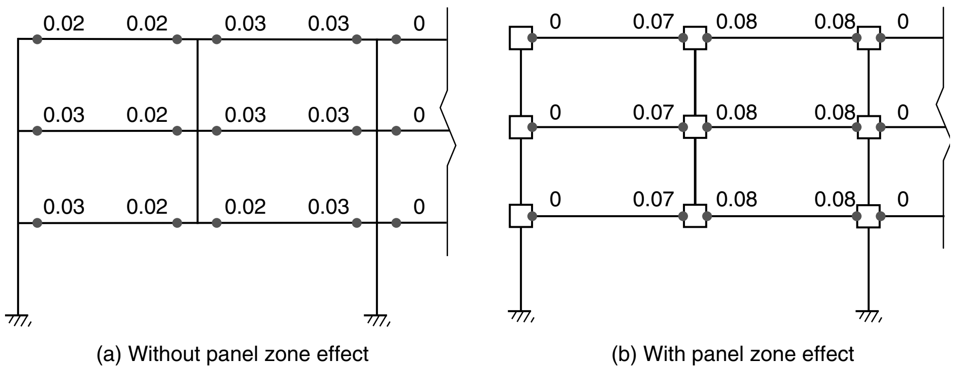


Figure 11. Maximum plastic rotations of beams in 3-storey GLRS structure caused by removal of the second column

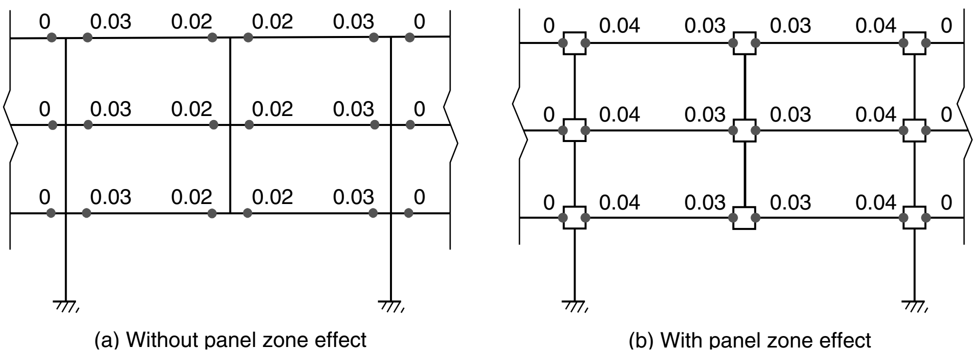


Figure 12. Maximum plastic rotations of beams in 3-storey GLRS structure caused by removal of the middle column

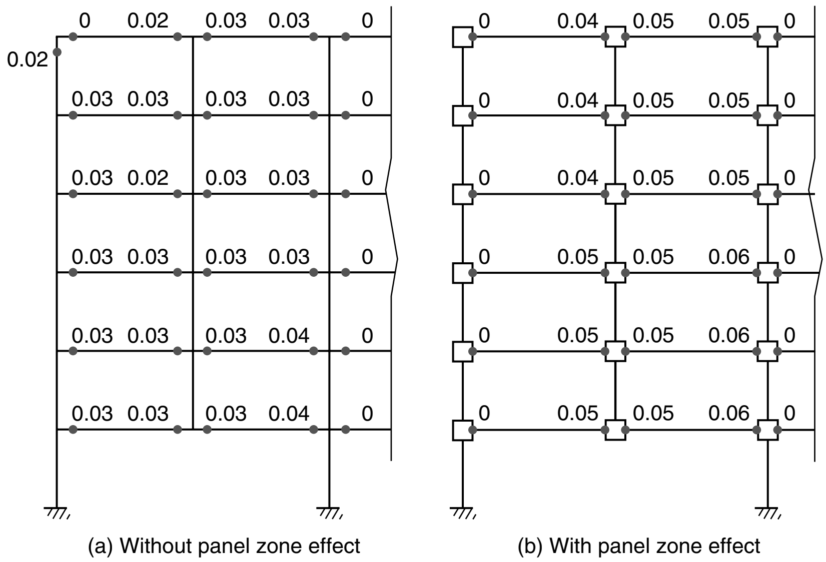


Figure 13. Maximum plastic rotations of beams in 6-storey GLRS structure caused by removal of the second column

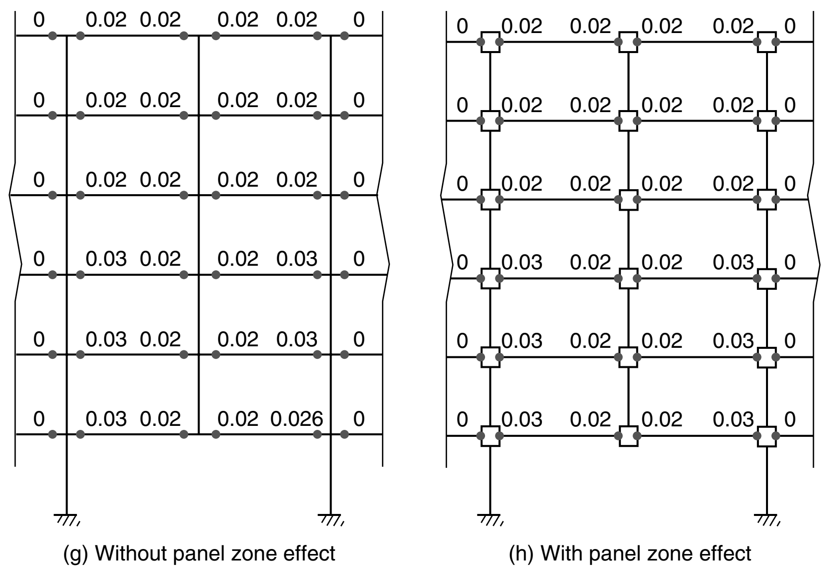


Figure 14. Maximum plastic rotations of beams in 6-storey GLRS structure caused by removal of the middle column

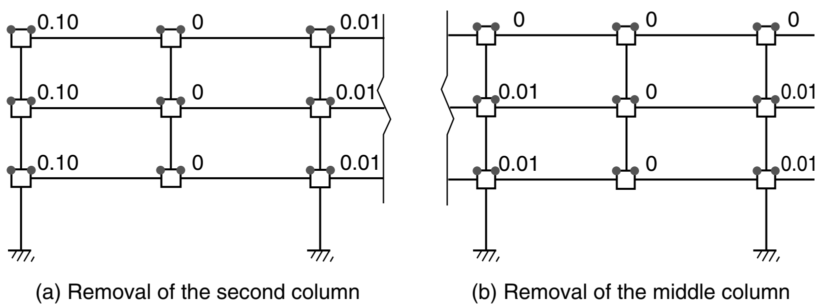


Figure 15. Maximum plastic rotations of panel zones of 3-storey GLRS structure

Table 3. Ductility demand of LLRS structures subjected to sudden column loss

Storey	Removed column	Without panel zone effect: Yield displacement (mm)	Without panel zone effect: Maximum displacement (mm)	Without panel zone effect: Ductility demand	With panel zone effect: Yield displacement (mm)	With panel zone effect: Maximum displacement (mm)	With panel zone effect: Ductility demand
3	Corner	–103	–1718	16.7	–93	–3123	33.6
	Second	–71	–282	4.0	–54	–699	12.9
	Middle	–67	–256	3.8	–76	–379	5.0
6	Corner	–101	–1503	14.9	–93	–	–
	Second	–73	–298	4.1	–57	–497	8.7
	Middle	–67	–230	3.4	–74	–253	3.4
15	Corner	–78	–739	9.5	–74	–	–
	Second	–57	–127	2.2	–55	–137	2.5
	Middle	–61	–123	2.0	–56	–135	2.4

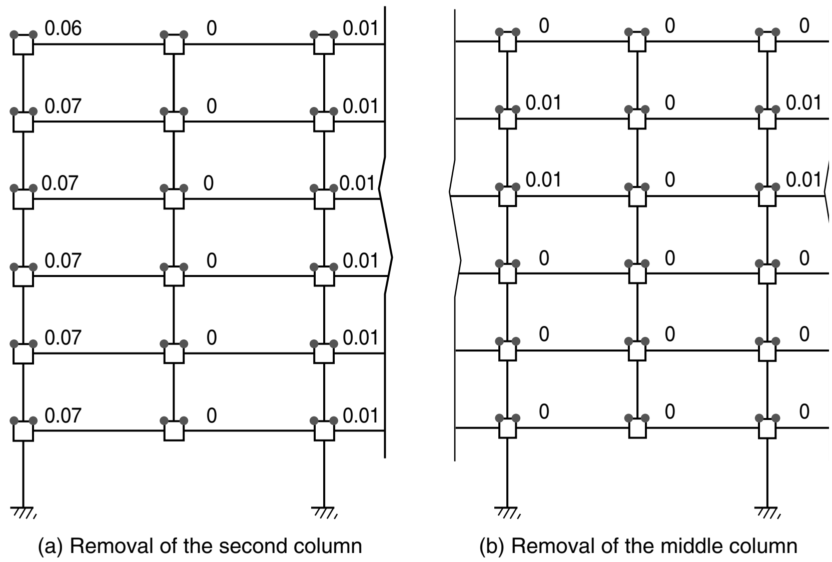


Figure 16. Maximum plastic rotations of panel zones of 6-storey GLRS structure

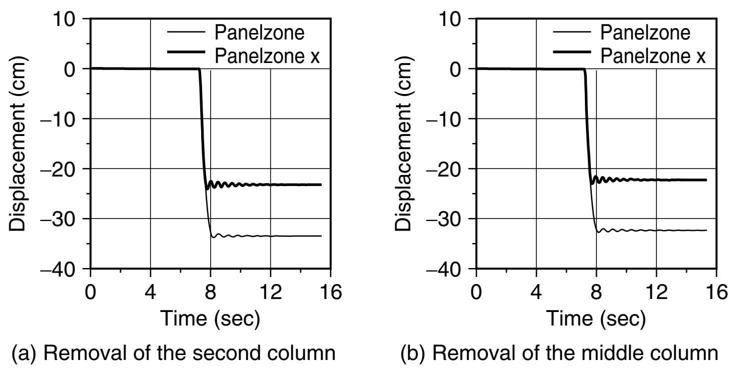
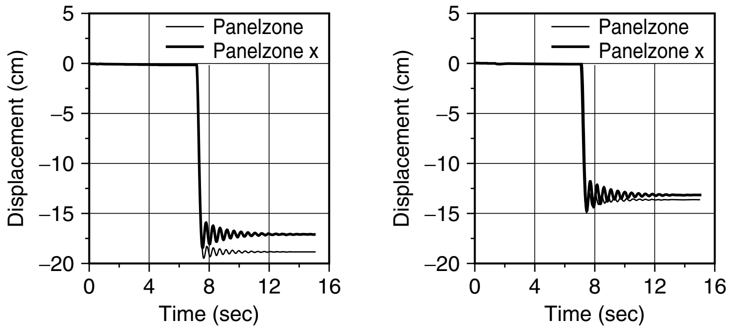
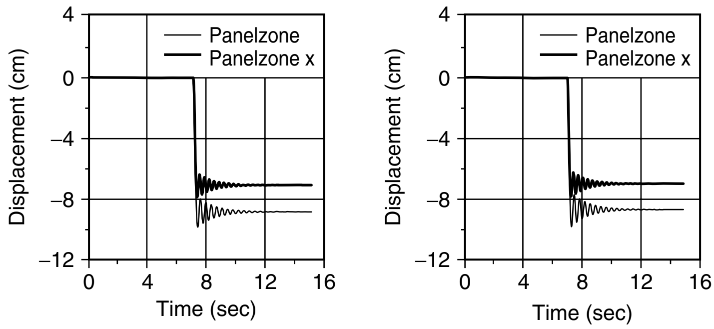


Figure 17. Nonlinear dynamic analysis results of 3-storey LLRS



(a) Removal of the second column (b) Removal of the middle column

Figure 18. Nonlinear dynamic analysis results of 6-storey LLRS



(a) Removal of the second column (b) Removal of the middle column

Figure 19. Nonlinear dynamic analysis results of 15-storey LLRS

was removed. This is due to the fact that the beam, continuous at the other side of the panel zone, prevents large deformations of the panel zone. On the other hand, when the second column was removed the plastic deformation was concentrated on this relatively weak panel zone since no restraint exists on the opposite side.

Table 2 and Figure 9 show that the panel zone stiffness effects decrease as the number of storeys increases. This is due to the increase in redundancy as the number of storeys and consequently the number of elements resisting progressive collapse increase. Especially in the 15-storey structure the difference in vertical deflections is almost negligible regardless of whether the second or the middle column has been removed. The plastic rotations of beams in the 6-storey GLRS structure, caused by the removal of the second column with and without the panel zone effect being allowed for, shown in Figure 13 were much reduced compared with those in the 3-storey structure. When the middle column was removed almost no difference was observed whether or not panel zone flexibility was taken into account (Figure 14). Based on the limited analysis results obtained in this study, therefore, it can be concluded that in mid- to high-rise structures the panel zone flexibility effect can be neglected in conducting progressive collapse analyses.

The likelihood of progressive collapse can be determined by the ultimate ductility demand regulated in the GSA guidelines ($\mu = 20$). The ductility demands of the model structures are shown in Table 3, where it can be observed that the ductility demands also increase significantly when the panel zone effect is included. Even though the maximum ductility criterion of 20 is somewhat arbitrary and has not yet been fully proved experimentally, the determination of progressive collapse may depend on whether panel zone flexibility effects are considered or not, especially if a corner column collapses.

4.3. Panel Zone Effect in the Lateral Load-Resisting Structures

Figures 17 to 19 show time-histories of the vertical deflections of LLRS structures caused by the abrupt removal of a first storey column. Table 3 summarizes the deflections at yield, the maximum deflections, and the maximum ductility demands. It can be seen, compared with the results of the GLRS structures, that the vertical deflections generally decreased. This is to be expected from the larger member sizes under the application of lateral load. As in the case of the GLRS structures, the effect of panel zone flexibility decreased as the number of storey increased. It can also be observed that the specific location of a removed column did not significantly affect the maximum deflection. Table 3 shows that ductility demands were less affected by the panel zone effect in the LLRS structures. This is due to the fact that the column size and consequently the panel zone strength had been increased as a result of the increased design load.

5. CONCLUSIONS

In this study, the effect of panel zone flexibility on the capacity of steel moment frames to resist progressive collapse was investigated using nonlinear dynamic analysis. Both gravity load-resisting system structures with weak panel zones and lateral load-resisting system structures with stronger panel zones were studied.

The analysis results shows that the vertical deflection of structures designed only for gravity loads generally increases and so does the potential for progressive collapse when panel zone flexibility is considered. The panel zone deformation turned out to be more significant when one of the columns adjacent to an exterior panel zone was removed. On the other hand, the panel zone effect was less significant when columns adjacent to interior panel zones were removed. As the number of storeys increased, the effect of a weak panel zone on the overall behaviour decreased, regardless of the location of any column removed. It was also found that panel zone deformation had little effect on the overall behaviour of structures designed to withstand seismic loads. Therefore, it can be concluded that the capacity to resist progressive collapse for frames generally decreases if panel zone flexibility effects are taken into account. The effect is most significant at exterior bays of low-rise structures designed only for gravity loads. It should also be pointed out that in some cases panel zone deformations can be excessive even before the limit state for beam ductility demand is reached. The limit states for panel zone deformations, therefore, need to be specified to ensure safety of the structure against progressive collapse.

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