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Progressive Collapse Evaluation of Low-Rise Reinforced Concrete Buildings Designed for Different Occupancy Classes

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ABSTRACT

The study aims to investigate the progressive collapse behavior of low-rise reinforced concrete buildings designed for different occupancy classes. For this objective, two low-rise reinforced concrete framed buildings were designed independently according to the Turkish Seismic Code for Buildings by considering the Residential Occupancy Class and Government Buildings Occupancy Class. A nonlinear dynamic analysis method was employed to evaluate the progressive collapse response of the buildings by using the alternate path direct design approach of UFC 4-023-03 and GSA-2016 guidelines. Three-dimensional finite element models were created for the analyses, and fiber hinges were used to represent the nonlinear behavior of the load-bearing members. Three column loss scenarios were implemented independently. The analysis results show that the residual displacement of the residential building is higher than that of the government building in all column removal scenarios. Moreover, the damage conditions of the residential building are commonly worse than the government building for all column loss scenarios. It is deduced from the study that the buildings experienced the most severe local damage, disproportionate to the initial failure, under the inner column loss scenario.

Keywords: Progressive Collapse, Occupancy Class, Nonlinear Dynamic Analysis, Reinforced Concrete, Turkish Seismic Code for Buildings.

INTRODUCTION

Structures are generally designed under design loads of dead, live, snow, wind, earthquake, etc. Some unforeseeable events, e.g., accidents, misuse, and deliberate attacks, are not explicitly considered during the design process. Therefore, they may lead to the structures' progressive collapse (PC). The conventional design codes, such as Eurocode 2 (2004), ASCE/SEI 7-16 (2016), etc., do not include direct design criteria against those extreme events. Instead, they propose prescriptions to improve the general structural integrity and robustness. However, the guidelines of the US General Service Administration (GSA-2016) and the US Department of Defense (UFC 4-023-03) propose alternative direct design and evaluation approaches to increase progressive collapse resistance of the structures.

It is known that seismic detailing requirements of the conventional design codes contribute to progressive collapse resistance of the buildings. However, the type and scale of extreme events are still unpredictable; thus, this issue is still under question today (Tsai and Lin, 2008; Usefi et al., 2015; Chaya and Naveen, 2018; Marchisa and Botez, 2019; Sheikh et al., 2021). Moreover, the occupancy class of the buildings is considered during their design process to make an economical design. For example, government buildings are designed more robust than residential buildings due to their immediate occupancy needs in case of any natural hazard such as earthquake, heavy snowing, strong wind, etc. Additionally, those buildings are more susceptible to deliberate attacks due to their public and governmental importance. Thus, terrorist attacks generally target those kinds of structures.

Like many other countries, Turkey has experienced several dramatic events attacking the government buildings resulting in many casualties and economic losses. An example event is given in Figure 1a to show the structural damage after an explosion (Aljazeera, 2022). While the extreme event exposure risk of government buildings is higher than that of residential buildings, many dramatic events have also happened to residential buildings in Turkey. For example, a recent explosion in 2021 in an apartment building in Ankara (Figure 1b) caused the death of two and the injury of four people (NTV, 2022). Because of that reasons, investigation of the

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progressive collapse response of the buildings designed with different occupancy classes seems vital to reduce the probability of losses due to extreme events.

a) Building damage due to a terrorist attack targeted Elazig Police Headquarters (Aljazeera, 2022)

b) Structural damage due to an explosion happened on an apartment building in Ankara (NTV, 2022)

Figure 1. Example of building damages after explosions

Turkey is located on a very seismically active fault zone. Therefore, it has a very modern seismic design code for building design (TSCB-2018) that is frequently updated depending on current scientific developments. It was updated last in 2018. In the literature, there is an excessive number of studies investigating the seismic performance of the buildings designed according to TSCB-2018 (Caglar, 2015; Ozturk et al., 2017; Buzuki, 2019; Kurkcu, 2019; Zolmaz, 2019).

On the other hand, the Turkish seismic code for buildings does not include any direct design criteria against extreme events similar to its contemporaries. The studies researching the PC resistance of the buildings designed according to TSCB-2018 are also very limited in the literature. While there are few studies on steel structures (Sehirali, 2011; Erguclu, 2013), the studies conducted on reinforced concrete (RC) buildings are very scarce. The global collapse response of 2-dimensional RC structures under blast loads was researched by Mahad (2021) in the literature. In a recent study, Demir (2022) observed the progressive collapse response of RC buildings designed according to the last two updates of the Turkish seismic code for buildings in 2007 (TSC-2007) and 2018. It was deduced that the buildings designed according to TSCB-2018 are more robust than the TSC-2007 against progressive collapse.

Consequently, the present study aims to investigate the progressive collapse behavior of low-rise RC buildings designed for two different occupancy classes. For this aim, two low-rise RC framed buildings were initially designed according to TSCB-2018 by considering the residential and government buildings occupancy classes. A nonlinear dynamic analysis method was employed to evaluate the PC response of the buildings by using the alternate path direct design approach of UFC 4-023-03 and GSA-2016 guidelines. A three-dimensional finite element (FE) model was created for the analyses. Fiber hinges were used to represent the nonlinear behavior of the load-bearing members. Three column loss scenarios were implemented independently.

MATERIALS AND METHODS

A three-story prototype reinforced concrete building with a 3.5 m story height was created for the analyses. The building has a symmetric structural plan (Figure 2) with square columns and rectangular beams. The building was designed twice for the residential and government buildings occupancy classes according to TSCB-2018 and TS-500 with a high ductility design class. The design loads were determined according to TS-498 as well. While the superimposed load on the slabs was assumed to be 2 kN/m^2, the live load was considered 2 kN/m^2 and 5 kN/m^2 for the residential and government buildings occupancy classes, respectively. A distributed surface load of 1.5 kN/m^2 was applied to the slabs for the weight of infill walls since their exact locations were unknown. The snow load was defined only on the top floor as 0.75 kN/m^2

The buildings are assumed as located on ZC soil class with a medium soil condition (shear velocity: 500 m/s). While the compressive strength of concrete was selected as 30 MPa (C30), the tensile strength of reinforcing steel was taken as 420 MPa (S420). The design of the buildings was employed using a finite element design software, ProtaStructures (2021). The geometrical and reinforcing details of the sections are given in Table 1. They were also kept constant for all stories.

Table 1. Geometrical and reinforcing details of the sections

Type of designed building	Section type	B [cm]	H [cm]	Top bars	Web bars	Bottom bars	Stirrups & ties	# of ties	s [cm]
Residential	Column	30	30	2ϕ18	N/A	2ϕ18	ϕ8	N/A	5
	Beam	25	40	5ϕ14	N/A	3ϕ12	ϕ8	N/A	10
Government	Column	35	35	3ϕ16	2ϕ16	3ϕ16	ϕ8	1	8
	Beam	25	40	4ϕ18	N/A	3ϕ16	ϕ8	N/A	10

*B: section width, H: section height, ϕ: diameter of reinforcement in mm, s: stirrup spacing

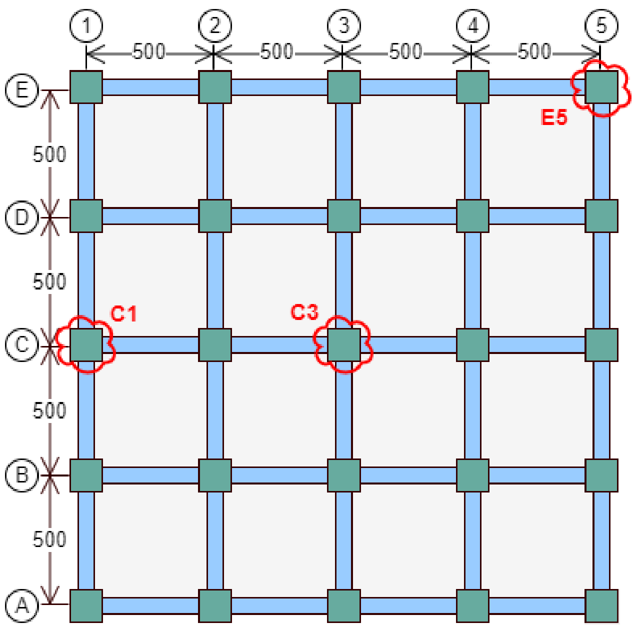


Figure 2. Plan view of the buildings (Dimensions in cm)

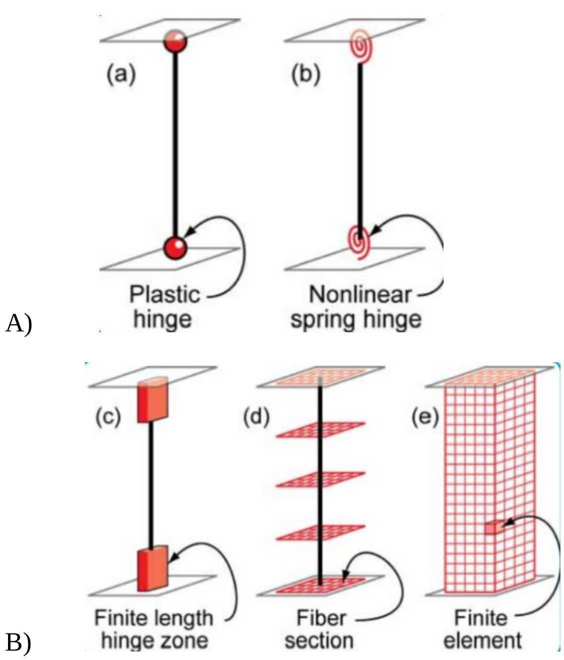


Figure 3. Idealized nonlinear element models (Deierlein et al. 2010). A) Concentrated plasticity models; B) Distributed plasticity models

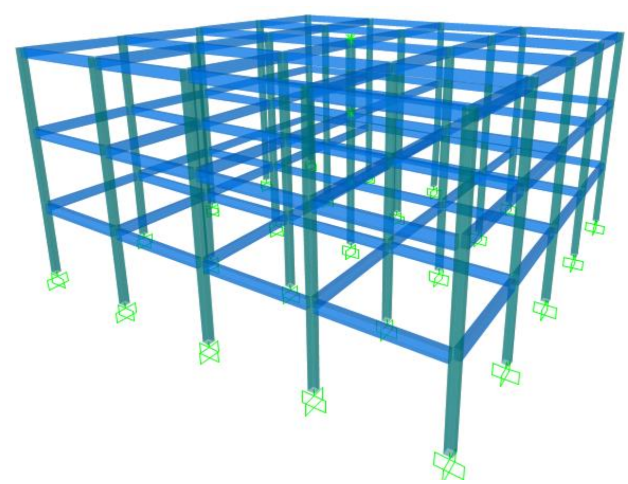


Figure 4. FE model of the buildings

A three-dimensional finite element model was created to investigate PC response of the buildings using SAP2000 (V23), which is a static and dynamic FE analysis software. The modeling and analysis were conducted according to the requirements given in GSA-2016 and UFC 4-023-03 guidelines. The nonlinear behavior of the members could be simulated by ether concentrated and distributed plasticity approaches (Figure 3). The conventional plastic hinge analogy of the concentrated plasticity models assumes that the nonlinearity is lumped in a predetermined region and is defined as the section's moment - curvature response. The moment - axial force interaction of the sections is considered in the model. However, the nonlinear behavior of the members in the distributed plasticity models is simulated by fibers extending along the length of the members. The constitutive stress-strain material models of the members are used to represent plasticity. In the present study, the fiber hinges (Figure 3c), one of the distributed plasticity approaches, were utilized to simulate the nonlinear behavior of the load-bearing members. The slabs were not included in the model, but a rigid diaphragm was defined for every story level. The finite element model of the buildings is depicted in Figure 4.

A nonlinear static analysis case was defined first for the gravity loads with a load combination of 1.2 Dead Load + 0.5 Live Load + 0.2 Snow Load as prescribed in the guidelines of GSA-2016 and UFC 4-023-03. Following the gravity analysis, a nonlinear dynamic time-history load case was defined to simulate column removal cases with a created ramp function (Sagiroglu, 2012; CSI Knowledge Base, 2022). The used ramp function is illustrated in Figure 5. The column removal was initiated at 0.5^{th} second, and the total duration was considered 3 seconds. Moreover, a column from the corner edge of the building (E5), a column on the middle of the side axis (C1), and lastly, another column at the interior of the building (C3) were removed independently.

The columns were removed from the first story of the buildings. The acceptance criteria representing the damage status of the members were calculated according to TSCB-2018 in terms of material strain since the inelastic behavior of the members was simulated using fiber hinges. The damage regions of the sections prescribed in TSCB-2018 are illustrated in Figure 6. The calculated concrete strain limits of the sections are reported in Table 2. The reinforcement strain limit depends on the class and properties of the reinforcing steel, and it is constant for all members having the same reinforcing bars. Because the same steel material was used for the buildings, the reinforcement strain limits were obtained as 0.0075, 0.0240, and 0.0320 for Limited, Moderate, and Severe Damage, respectively. Moreover, the fundamental time periods of the buildings were determined as 0.58 s and 0.51 s for residential and government buildings occupancy classes, respectively.

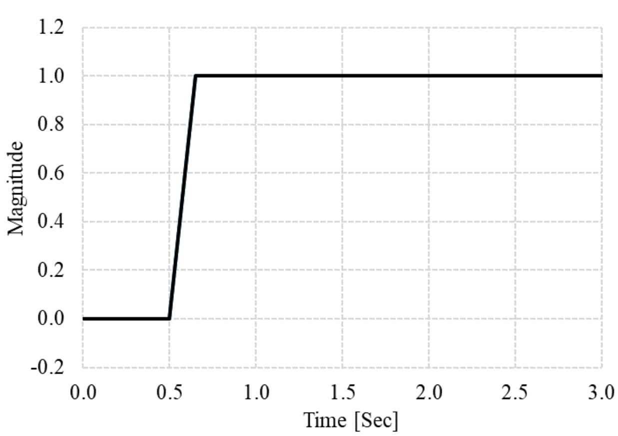


Figure 5. The ramp function used to simulate column removal

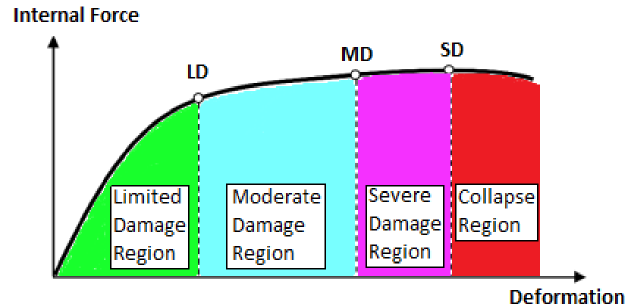


Figure 6. The ramp function used to simulate column removal

Table 2. Acceptance criteria for the structural members (concrete strain)

Type of Analyzed Building	Section Type	Limited Damage	Moderate Damage	Severe Damage
Residential	Column	-0.0025	-0.0084	-0.0112
	Beam	-0.0025	-0.0051	-0.0068
Government	Column	-0.0025	-0.0089	-0.0119
	Beam	-0.0025	-0.0052	-0.0069

RESULTS AND DISCUSSION

The vertical displacement time-history results of the nodes above the removed columns are illustrated in Figure 7 for all column removal scenarios. The residual vertical displacement results (u_r) of those nodes are reported in Table 3 as well. The residual displacement of the residential building is higher than that of the government building in all column removal scenarios. Because the seismic design forces of the buildings designed for government buildings occupancy class are considered higher than the residential structures in TSCB-2018, their members have bigger section dimensions and reinforcement ratios. That leads to having inherently higher member capacities.

As a result, the government buildings perform a higher progressive collapse resistance against extreme events than the residential structures. For the E5 and C1 column removal scenarios, beyond specific displacement values, vertical displacement stopped. However, the vertical displacement did not stop for the inner column (C3) removal. Consequently, all beams bridging over the removed column failed.

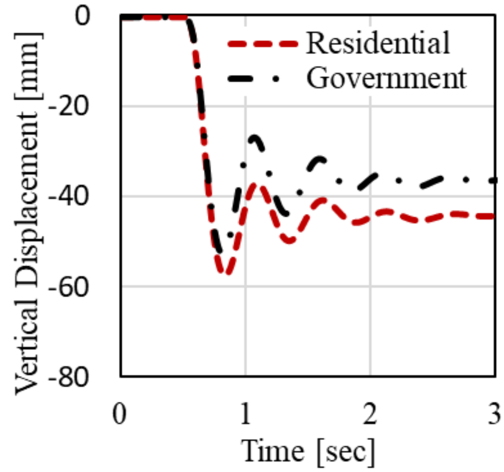
Table 3. Result of the residual vertical displacement value of the node above the removed column

Removed Column	u_r [mm]	
	Residential	**Government**
E5	-44.4	-36.5
C1	-67.2	-57.0
C3	collapse	collapse

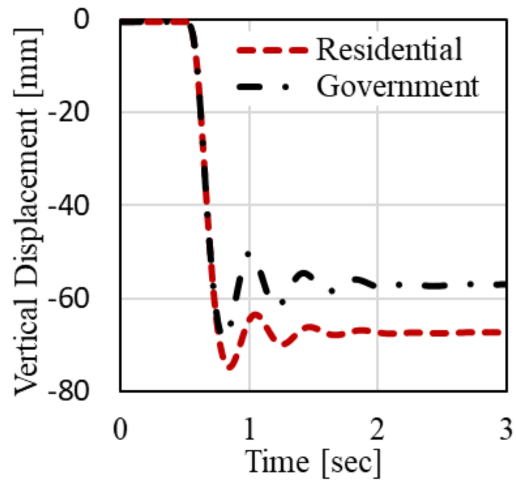
The damage conditions of the fiber hinges are illustrated in Figures 8 and 9 for the residential and government building, respectively. The damage conditions of the residential building are commonly worse than the government building for all column loss scenarios. The limited and moderate damage scatter was observed on both structures subjected to corner (E5) and side (C1) column removal cases. Severe damage occurred on the beams bridging over the removed column in the case of the interior column (C3) removal scenario for both buildings. As a result, a local collapse was experienced among the members surrounding the removed column. However, no total failure happened to the buildings.

Because the buildings were designed according to a seismic design code (TSCB-2018), the code's seismic design and detailing requirements contributed significantly to the progressive collapse resistance of the investigated buildings. If they were not designed according to TSCB-2018, the damage could be worse for corner and side column loss scenarios. Consequently, the progressive collapse risk of the structures designed with prior engineering knowledge or receiving no engineering services is very high. Therefore, the progressive collapse resistance of those buildings should be investigated carefully to reduce the possibility of casualties, injuries, economic losses, etc.

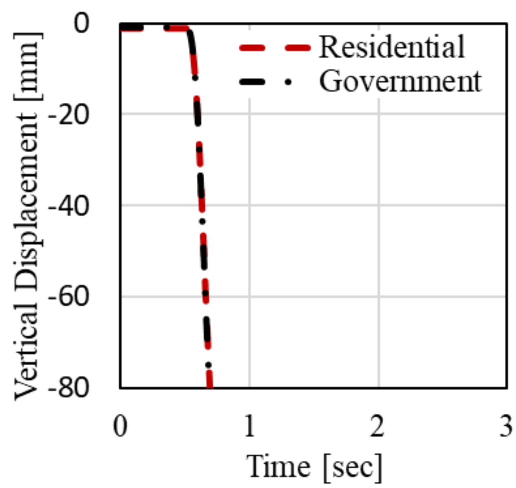
Ultimately, a simplified design approach was utilized to design the observed buildings in the study by generalizing some load-bearing members' sections throughout the buildings. It is also common in practice because the generalization of the members reduces the labor and formwork costs, and construction errors stemming from the complicated structural design drawings. On the other hand, that is expected to result in some degree of over-design of the buildings. The over-design generally accumulates on the corner, side axes, and top floors since the members in those locations typically have less load tributary area than the interior members.



a) Removal of column E5

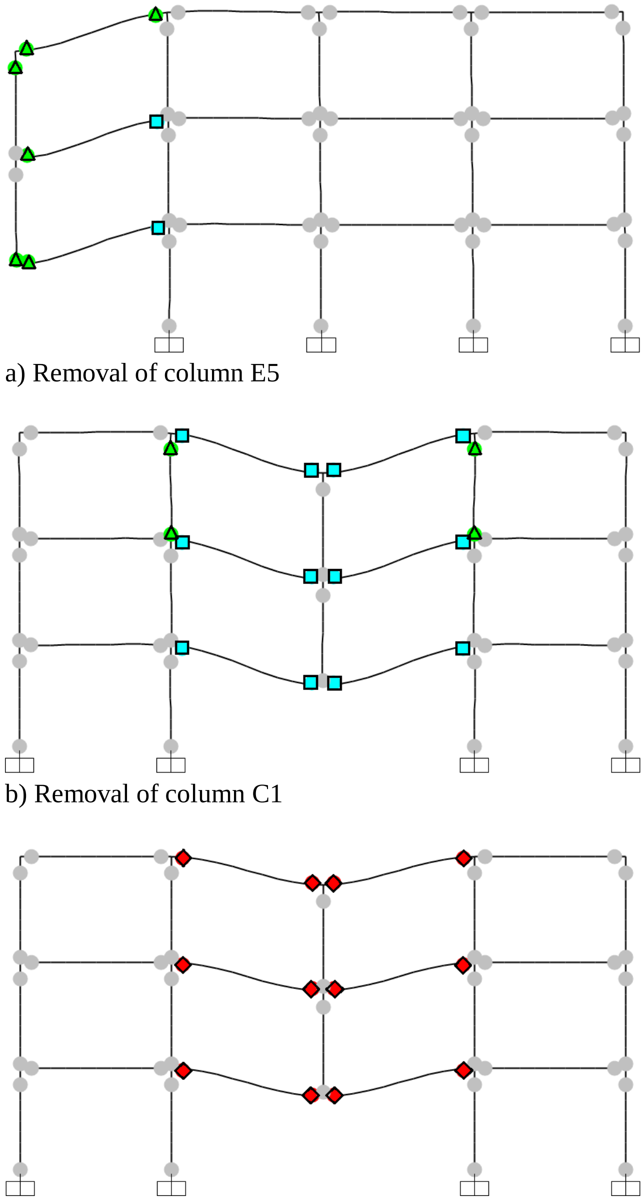


b) Removal of column C1



c) Removal of column C3

Figure 7. Vertical displacement time-history of the node above the removed column



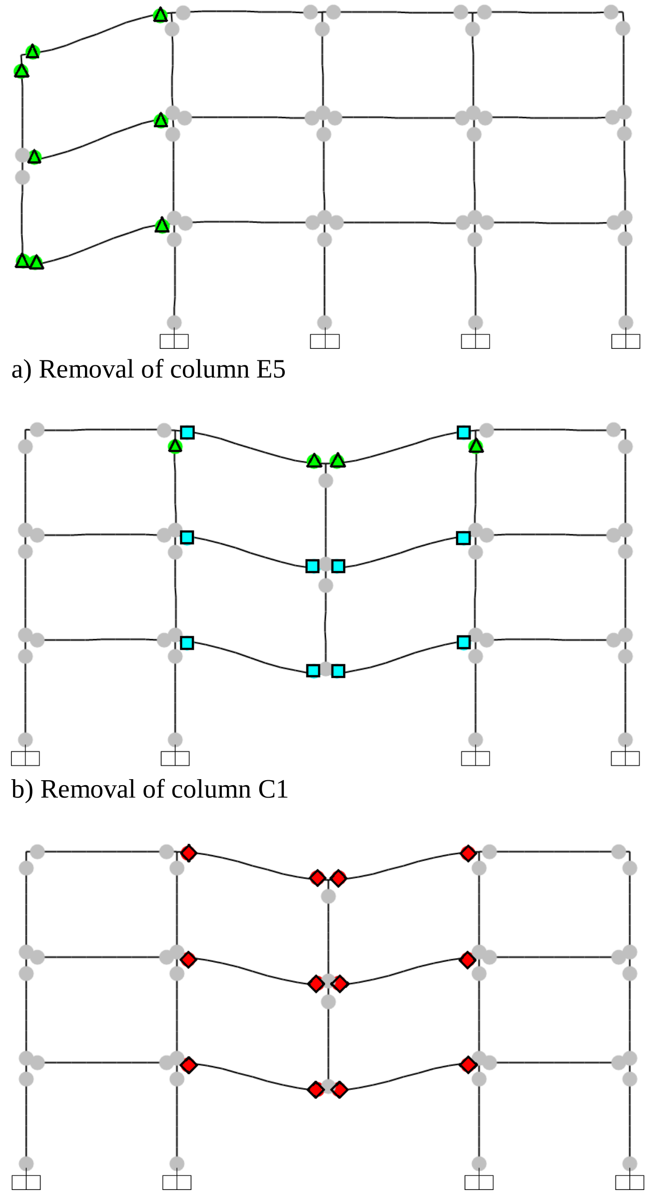
a) Removal of column E5

b) Removal of column C1

c) Removal of column C3

Figure 8. Fiber hinge damage results in the residential building. *Gray-filled circle: no damage, green-filled triangle: limited damage, cyan-filled square: moderate damage, pink-filled star: severe damage (collapse prevention), red-filled diamond: failure (collapse).

As a result, that over-design is expected to contribute significantly to the progressive collapse resistance of those members. The present study deduced that the PC response of the buildings subjected to a corner and side column loss scenario is better than the interior column. However, suppose a more precise design had been performed for the buildings, specifically for the members on the edge axes. In that case, the resulting response of those members could also be as severe as the interior column removal case since they are connected to a smaller number of members to redistribute the loads in case of a sudden column loss.



a) Removal of column E5

b) Removal of column C1

c) Removal of column C3

Figure 9. Fiber hinge damage results in the government building. *Gray-filled circle: no damage, green-filled triangle: limited damage, cyan-filled square: moderate damage, pink-filled star: severe damage (collapse prevention), red-filled diamond: failure (collapse).

CONCLUSIONS

In the study, two low-rise RC framed buildings were designed according to TSCB-2018 by considering the residential and government buildings occupancy classes. A nonlinear dynamic analysis method was employed to evaluate the PC response of the buildings by using the alternate path direct design approach of UFC 4-023-03 and GSA-2016 guidelines. A three-dimensional finite element model was created for the analyses. The fiber hinges were used to represent the nonlinear behavior of the load-bearing members. Moreover, three different column loss scenarios were implemented independently. The following conclusions have been derived from the study:

- The residual vertical displacement values of the residential building are higher than those of the government building in all column removal scenarios.
- The damage conditions of the members of the residential building are commonly worse than the government building for all column loss scenarios.
- Severe damage occurred on the beams bridging over the removed column in the case of the inner column removal (C3) for both classes of the buildings.
- Local damage disproportionate to the initial damage is observed during an inner column loss scenario. Nonetheless, an entire collapse does not occur on the buildings.
- The resistance of the low-rice RC framed buildings is not adequate for some column removal scenarios. Therefore, buildings with a higher probability of exposure to extreme events should also be designed and evaluated against their progressive collapse risks. That will help reduce the number of casualties and economic losses in any unforeseeable extreme events.

The existing buildings designed according to prior engineering knowledge or received no engineering service might have a higher progressive collapse risk than those constructed depending on the most up-to-date engineering knowledge. Therefore, the progressive collapse evaluation of those buildings should be investigated comprehensively. Moreover, new studies can be done to get more generalized results for the progressive collapse resistance of the structures designed according to TSCB-2018. For this purpose, the scope of the present study should be extended for the buildings having a different plan, height, story number, shape, etc. A more precise design could also be made to design the buildings. The study could be implemented for the steel and masonry structures as well.

DECLARATIONS

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Conflict of interest

The author hereby confirms that there is no conflict of interest whatsoever with any third party.

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