Bitlis Eren Üniversitesi Fen Bilimleri Dergisi
BİTLİS EREN UNIVERSITY JOURNAL OF SCIENCE
ISSN: 2147-3129/e-ISSN: 2147-3188
VOLUME: 11 NO: 2 PAGE: 694-705 YEAR: 2022
DOI: 10.17798/bitlisfen.1094003

Progressive Collapse Response of Reinforced Concrete Buildings Designed According to Turkish Earthquake Code

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Keywords: Progressive collapse, Nonlinear dynamic analysis, Reinforced concrete, Government buildings occupancy class, Turkish earthquake code.

Abstract
In this study, the progressive collapse response of reinforced concrete buildings designed for the 'government buildings' occupancy class was investigated numerically. For this purpose, two reinforced concrete framed buildings were initially designed according to the Turkish Earthquake Code published in 2018. Later, those buildings' progressive collapse responses were evaluated using the Alternate Path direct design approach defined in the GSA-2016 and UFC 4-023-03 guidelines. Three different column removal scenarios were employed independently by applying the nonlinear dynamic analysis method. Nonlinear fiber hinges were used to simulate the plasticity of the structural load-bearing members. As a result of this study, it is deduced that a limited local collapse disproportioned to the initial failure was observed on the investigated buildings. In addition to the conventional seismic design methods, the buildings designed according to the Turkish Earthquake Code should also be assessed with respect to the explicit design approaches against unforeseeable extreme events to reduce their progressive collapse risk.

1. Introduction

Structures may be expose to unforeseeable events such as accidents, deliberate attacks, misuse, etc., in their service lives. They may lose some of their load-bearing members during such circumstances. For example, a car accident hitting a building or a bomb explosion due to terrorist attacks may result in severe damage and the loss of some structural elements. Because these events are generally not considered in the design process, they might lead to dramatic human casualties and economic losses. Several dramatic events have been experienced in Turkey and worldwide, specifically due to terrorist attacks targeting government buildings due to their significance [1], [2]. For example, the bombing of the Alfred P. Murrah Federal Building in Oklahoma City in 1995 was the deadliest act of terrorism in US history, resulting in 168 casualties and several hundred more injured [3]. Moreover, a terrorist attack that targeted Elazig Police Headquarters in 2016 led to the martyrization of 3 police officers and the injury of 217 people [4]. The buildings exposed to these events were severely damaged as well (Fig. 1).

An initial local failure caused by an extreme event may propagate from structural member to member and eventually might disproportionately collapse a large part of a structure or entirely. This phenomenon is called progressive collapse (PC) of structures [5]. There are quite a few direct design methods to evaluate the progressive collapse resistance of structures. The guidelines released by the US General Service Administration (GSA-2016) [6] and the US Department of Defense (UFC 4-023-03) [7] implement the direct design approaches to evaluate the PC response of new and existing government and military buildings. In these methods, the progressive collapse response of structures is evaluated by simulation of a column loss scenario in the different locations on the structure [8].

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Received: 27.03.2022, Accepted: 12.05.2022

a) Alfred P. Murrah Federal Building [3] b) Elazig Police Headquarters [4]

Figure 1. Examples of the building damage after explosions

On the other hand, conventional design codes such as Eurocode 2 [9], ASCE/SEI 7-16 [5], IBC [10], etc., do not give specific design criteria to reduce the risk of progressive collapse. Instead, they implement indirect design approaches that aim to improve the robustness of structures by providing sufficient strength, ductility, redundancy, etc. Moreover, it is a fact that proper seismic detailing as prescribed in current seismic design codes improves the resistance of structures against PC. However, it should be noted that since progressive collapse is not considered and evaluated explicitly, the PC risk of those buildings will never be abolished. Thus, special attention is still needed to design against extreme events [11]–[13].

In the literature, Tsai and Lin [11] observed the effect of the seismic design on the PC resistance of reinforced concrete (RC) structures subjected to a column failure. That behavior was also investigated by Marchis et al. [12] on the mid-rise and by Sheikh et al. [13] on the RC framed structures. Abdulsalam and Chaudhary [14] evaluated the progressive collapse of RC buildings by defining flexure-axial-shear interaction in plastic hinges. Marchis and Botez [15] researched the effect of the number of stories on the collapse resistance of RC frames.

Turkey also has one of the most modern seismic design codes. It frequently updates its seismic code depending on recent scientific advances due to being in an earthquake-prone zone [2]. However, it also has no special explicit design or evaluation methods against PC of structures built in the country. The last update of the Turkish Earthquake Code was released in 2018 (TEC2018) [16]. Like its contemporaries, it is expected to improve the structural integrity during an extreme event by providing sufficient structural robustness. Because it also does not include any explicit design prescriptions to reduce the risk of progressive collapse, the response of the buildings designed according to TEC2018 [16] should be investigated to observe their performance under any extreme event. The studies in the literature for this objective are very scarce, specifically for RC buildings. Mahad [17] investigated the global collapse response of 2-dimensional RC structures under blast loads. For that reason, in the present study, the progressive collapse response of RC buildings designed according to TEC2018 [16] was investigated numerically. The occupancy class of the buildings was selected as 'government buildings' due to their inherent high risk against extreme events.

Furthermore, several studies in the literature have investigated the seismic response of RC buildings designed according to TEC2018 [16]. Firstly, Nemutlu and Sari [18] designed two RC buildings, including different structural load-bearing systems, with respect to TEC2018 [16], ASCE/SEI 7-16 [5], and IBC [10]. Later, they evaluated the analysis results comparatively to determine the differences and causes between TEC2018 [16] and those international design codes. They determined that the base shears calculated from TEC2018 [16] are significantly higher than those obtained by international design codes. Aksoylu and Arslan [19] evaluated the empirical period calculation method of TEC2018 for RC buildings. They compared the results with the Rayleigh period formula on 2-5 story RC frame and shear wall dwellings and school buildings under different soil conditions. As a result of the study, it was suggested that the empirical formula proposed for building height classes (BHC) 6-7-8 should be re-evaluated since it yields to different shear forces for buildings having the given BHC. Similarly, Aksoylu and Arslan [20] investigated different seismic load calculation techniques defined in the 2007 and 2018 updates of the Turkish Earthquake Code for RC buildings. For this purpose, linear elastic analyses were conducted on 3-4-5-story frame and shear wall RC buildings. It was concluded that TEC2018 increases story displacements and natural periods, and decreases base shear forces due to the new cracked section stiffness implementation.

Moreover, Isik and Demirkiran [21] observed the performance of RC structures under different ground motion levels. Four different earthquake motion levels were considered for the buildings located in four different cities under various earthquake hazards with varying probabilities of exceedance. It was deduced that the target top displacements expected from the structures were significantly changed. Lastly, the pre and post-seismic performance of RC buildings was investigated by Isik et al. [22] and Nemutlu et al. [23], respectively. The former authors performed an earthquake hazard analysis of the Eastern Anatolian Region of Turkey using a performance-based seismic evaluation approach. The latter authors investigated the structural damage that occurred on RC and masonry structures after the Elazig-Sivrice earthquake (Jan. 24, 2020). Several structural deficiencies, such as short columns, low concrete strength, strong beam-weak column mechanism, non-ductile detailing, etc., were determined on the buildings.

This study investigated numerically the progressive collapse response of RC buildings designed for the 'government buildings' occupancy class. For this purpose, two RC framed buildings were initially designed according to TEC2018 [16] by using ProtaStructures [24], which is a specialized structural engineering finite element (FE) software for the design of RC structures. Later, the progressive collapse response of those buildings was evaluated using one of the direct design approaches defined in the GSA-2016 [6] and UFC 4-023-03 [7] guidelines. Three different column removal scenarios were employed independently by applying the Nonlinear Dynamic Analysis Method of the Alternate Path direct design approach. Nonlinear fiber hinges were used to simulate the plasticity of the structural load-bearing members. Moreover, progressive collapse analyses of the buildings were conducted using SAP2000 [25], a commercial FE software for static and dynamic analysis of structures. As a result, it is deduced that a limited local collapse disproportioned to the initial failure was observed on the investigated buildings.

2. Properties of the Designed Buildings

A prototype symmetric building plan was created for the analyses, as depicted in Fig. 2. Two different building heights were selected as 10.5 m (3-story) and 24.5 m (7-story). The elevation view of the 7-story building is illustrated in Fig. 2 as well. The 'government buildings' occupancy class was selected for the buildings. They were assumed to be located in a medium soil condition. The dead, live, and snow loads on the buildings were calculated according to TS 498 [26]. The parameters used for the design of buildings according to TEC2018 [16] are reported in Table 1. Moreover, the horizontal elastic design spectrum used in the seismic analyses is shown in Fig. 3.

The compressive strength of concrete is 30 MPa (C30), and the tensile strength of reinforcing steel is 420 MPa (B420C). The columns have a square

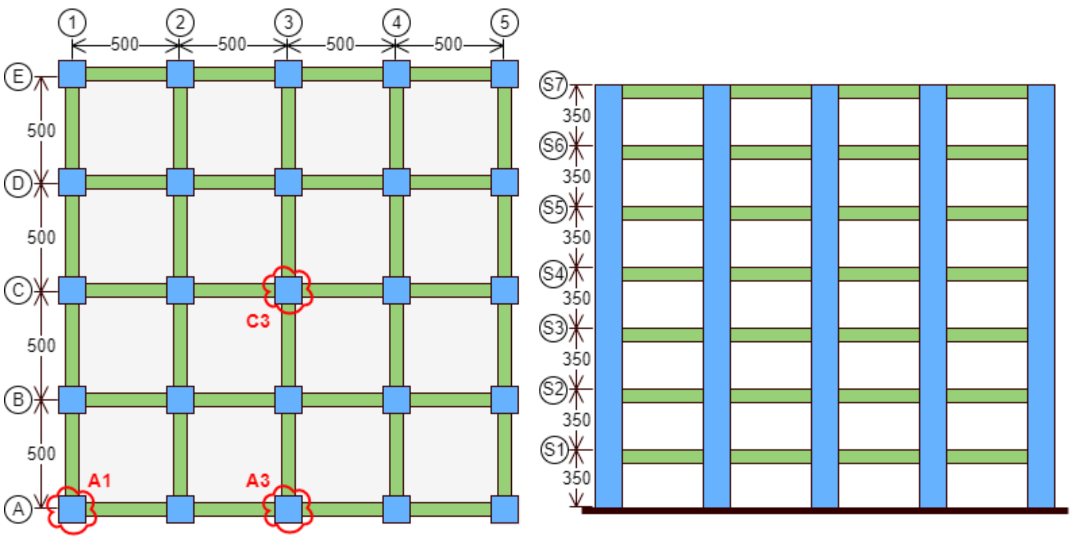


Figure 2. Plan (left) and elevation (right) views of the buildings (dimensions in cm)

section (*A*: section dimension), while the beams have a rectangular section area (*B*: section width and *H*: section height). The geometrical and reinforcing details of the members were kept constant and are reported in Tables 2 and 3. In the tables, ϕ shows the reinforcement diameter in mm, and *s* represents stirrup spacing. Moreover, typical drawings of columns and beams are illustrated in Fig. 4. The buildings were designed according to the requirements of both TEC2018 [16] and TS 500 [27]. A structural engineering finite element software, ProtaStructures [24], specialized in designing RC structures, was used to design the buildings.

Table 1. Design parameters of the buildings

Parameter		Value
Soil class	:	ZC
Soil shear velocity (m/s)	:	500
Building usage class	:	1
Building height class (for 3 & 7-Story)	:	8 & 6
Seismic design class	:	3a
Ductility level	:	High
Earthquake Ground Motion Level	:	DD-2
Design spectral response acceleration parameter at short periods (S_{DS})	:	0.413
Design spectral response acceleration parameter at the 1.0 s period (S_{D1})	:	0.114
Response modification coefficient (R)	:	8
Overstrength coefficient (D)	:	3

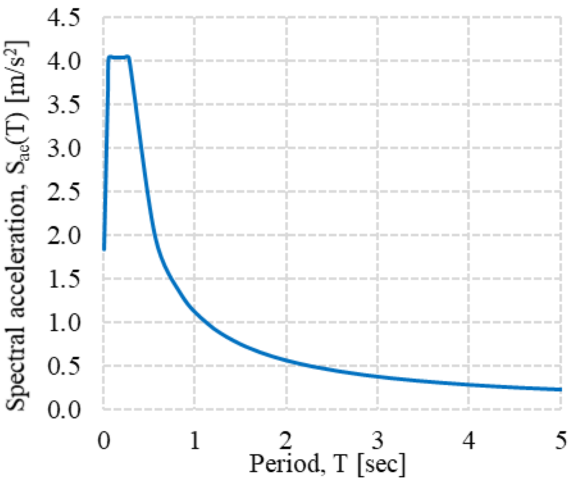


Figure 3. The horizontal elastic design spectrum

On the other hand, GSA-2016 [6] and UFC 4-023-03 [7] standards specify design recommendations to decrease the potential of PC for new and existing governmental structures that may be exposed to localized structural damage due to unforeseeable extreme events. They aim to limit the propagation of the initial damage by providing a robust and balanced structural system. All three or more-story governmental buildings must comply with these guidelines. The standards apply a threat-independent approach without explicitly considering the cause of the initial event. Three direct design approaches are employed in UFC 4-023-03 [7]: Alternate Path, Enhanced Local Resistance, and Tie Forces. However, GSA-2016 [6] uses only the Alternate Path (AP) design approach. The AP approach is common to both guidelines, and vertical structural members are notionally and independently removed at specific plan and elevation locations. It also requires that the building can bridge over that removed element. Moreover, three analysis methods are employed in the AP approach: Linear Static (LS), Nonlinear Static (NS), and Nonlinear Dynamic (ND). While there are some geometric irregularity limitations in LS and NS methods, ND can be used for buildings with irregularity [6], [7].

Table 2. Geometrical and reinforcing details of the column sections

Designed Building	*A* [cm]	Longitudinal Bars	Stirrups & Ties	# of Ties	*s* [cm]
3-story	35	8ϕ16	ϕ8	1	8
7-story	50	8ϕ20	ϕ8	1	8

Table 3. Geometrical and reinforcing details of the beam sections

Designed Building	*B* [cm]	*H* [cm]	Top bars	Bottom Bars	Stirrups	*s* [cm]
3-story	25	40	4ϕ18	3ϕ16	ϕ8	10
7-story	25	50	5ϕ14	3ϕ14	ϕ8	11

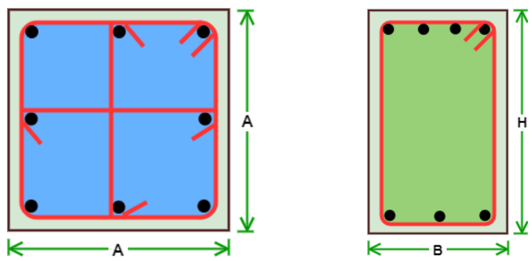


Figure 4. Typical column (left) and beam (right) sections

Additionally, primary and secondary components must be determined in the building before the analysis. The acceptance criteria for the primary members are calculated according to the given prescriptions in the guidelines, which generally refer to the acceptance criteria defined in ASCE/SEI 41-13 [28]. Afterward, locations of the removed load-bearing elements are determined as described in the guidelines: external columns at the corner of the building, near the middle of the short side and the long side, and some internal columns. Moreover, the columns at locations where the building plan geometry changes significantly must be removed. Ultimately, the structural components of the building must not exceed the calculated acceptance criteria. If the analysis predicts these acceptance criteria are not satisfied, the building does not meet the progressive collapse requirements and must be re-designed or retrofitted to eliminate the nonconforming elements [6], [7]. The general outline of the procedure to evaluate the PC response of structures prescribed in the GSA-2016 [6] and UFC 4-023-03 [7] guidelines is summarized in Fig. 5. Further details can be found in the relevant standards.

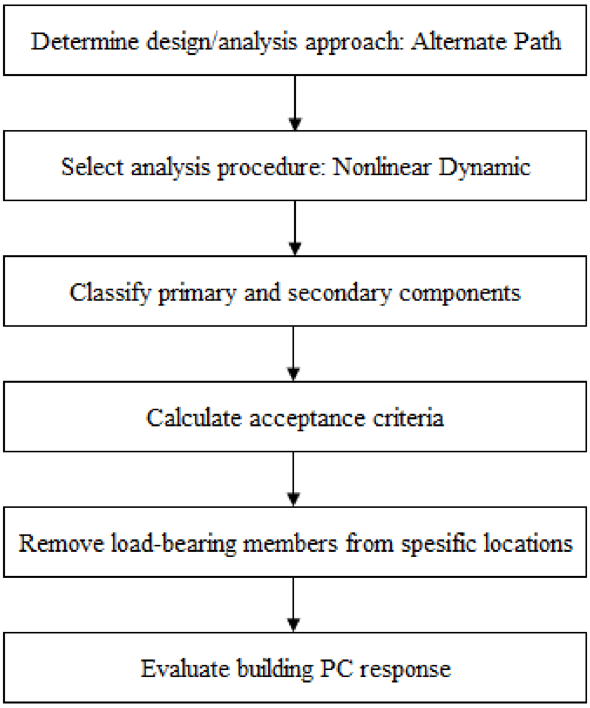


Figure 5. Analysis procedure to evaluate progressive collapse response of structures

3. Numerical Modeling of the Buildings for Progressive Collapse Analysis

The nonlinear dynamic analysis method of the Alternate Path direct design approach according to both GSA-2016 [6] and UFC 4-023-03 [7] guidelines was used for the progressive collapse evaluation of the designed buildings. A three-dimensional finite element (FE) model was created (Fig. 6) in SAP2000 [25], which is a static and dynamic FE analysis software for structures. Beams and columns were modeled using frame elements. Instead of modeling the slabs, a rigid diaphragm was assigned at every story level. The loads on the slabs were distributed and assigned to surrounding beam elements.

There are two main modeling approaches to simulate the post-yield inelastic behavior of the structural load-bearing members: concentrated (lamped) and distributed plasticity models (Fig. 7). It is assumed in the lamped plasticity model (Figs. 7a and 7b) that the deformation beyond the elastic limit occurs only in discrete locations, and the remaining part of the member stays elastic. Inelastic behavior is obtained by integrating plastic strain and curvature occurring in a predefined hinge length. Nevertheless, in a distributed plasticity model, a member's cross-section is discretized into a series of representative axial fibers extending throughout the element or along with a finite length hinge zone (fiber hinge) (Figs. 7c, 7d, and 7e). A stress-strain relationship needs to be defined for each fiber. Ultimately, axial force-deformation and biaxial moment-rotation relationships are obtained by integrating the behavior over the section and multiplying by the hinge length [29]-[31]. On the other hand, it is a fact that RC frame structures can resist the disproportionate collapse by developing two critical load-resisting mechanisms on their members: Vierendeel (arching) Action and Catenary Action. While fiber elements can successfully capture those mechanisms in addition to the flexural behavior of frame members, those features are neglected in the conventional concentrated plastic hinge approach [12], [29], [32]. Because of that reason, in this study, the nonlinear behavior of the structural load-bearing members was simulated using nonlinear fiber hinges, which are one of the methods of distributed plasticity approach (Fig. 7(c)).

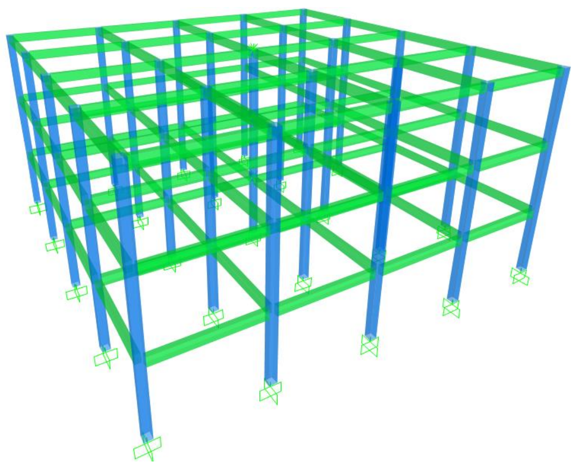


Figure 6. FE model of the 3-story building

The sections were first discretized with an optimum fiber layout to implement fiber sections in SAP2000. The program automatically assigned the fibers to the center of every reinforcement, and confined and unconfined concrete which were

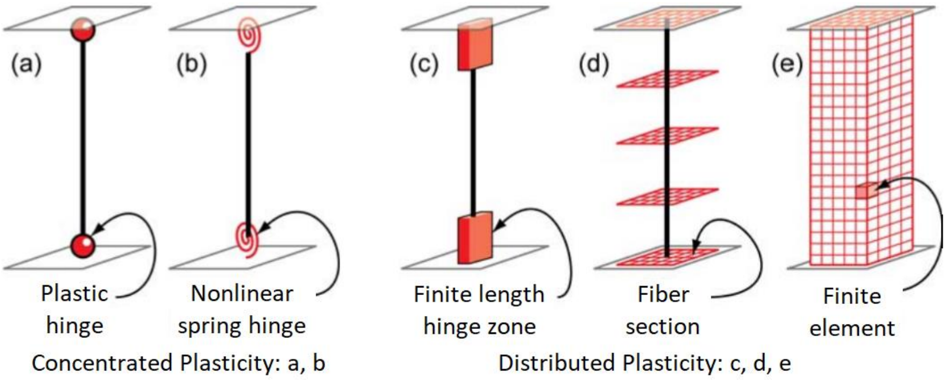


Figure 7. Nonlinear beam-column element models [29]

meshed into several square or rectangular areas. Later, the material properties of both concrete and reinforcing bars were assigned to the relevant fibers. Finally, the fiber hinges were defined at both ends of the beams and columns in a finite length hinge zone as half the section depth [16]. Moreover, the constitutive material models of both concrete and reinforcing steel were created following the material models given in TEC2018 [16].

A nonlinear static analysis case was defined first for the gravity loads combined with Eq. 1 as given in UFC 4-023-03 [7]. DL represents Dead Load, LL stands for Live Load, and S is Snow Load in the equation. That load case was used to obtain the forces present at equilibrium in each removed column. It was also considered the starting condition for the column removal analysis case. Later, the column member was omitted in the structural model, and the equivalent column loadings were applied to the node above the removed column end as a new load case. This model case simulates the condition in which the removed column exists on the building. Afterward, a new load pattern was created to include the equivalent removed column loadings with the opposite signs to simulate the column removal procedure. As depicted in Fig. 8, a ramp function was defined to apply this load pattern incrementally to the same joint above the removed column to abolish the equivalent column loadings by a nonlinear dynamic time-history load case [6], [7].

$$1.2DL + 0.5LL + 0.2S \qquad (1)$$

Moreover, the ramp function's column removal duration was considered less than one-tenth of the building's fundamental response period, as suggested by UFC 4-023-03 [7] (Fig. 8). Therefore, it was taken as 0.10 s for the 3-story and 0.15 s for the 7-story building since the fundamental period of the buildings was obtained as 1.04 s and 1.56 s, respectively. The column removal was conducted on the 0.5^{th} second after equilibrium was reached for the gravity analysis. The total duration of the removal was set to 3 seconds to see the residual vertical displacement after damping the oscillation. The Direct Integration solution algorithm was used for the nonlinear dynamic time-history load case. A Rayleigh damping with 5 % was defined depending on the buildings' fundamental first and second periods. Newmark time integration method was employed for the analysis, and its Gamma and Beta coefficients were taken as 0.50 and 0.25, respectively. Lastly, P-Delta and Large Displacements options were activated in the program to consider the geometric nonlinearity of the members and catenary behavior on the surrounding beams due to column removal [30], [33].

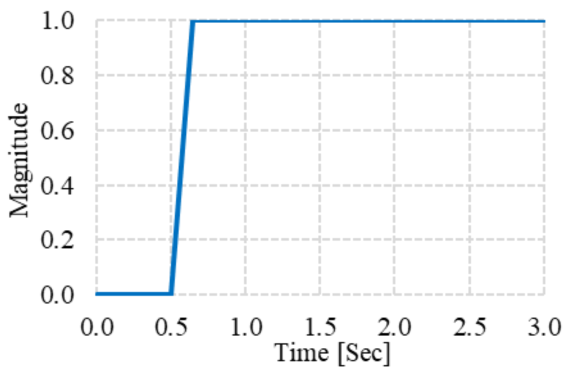


Figure 8. The defined ramp function for the 7-story building to simulate column removal

Three column removal scenarios on the first story of the buildings were implemented as suggested by GSA-2016 [6] and UFC 4-023-03 [7]. The simulations of one exterior corner column (A1), one column in the middle of an outermost axis (A3), and one column at the center of the building (C3) were conducted independently to investigate the progressive collapse response of the structures. The locations of the removed columns are shown in Fig. 2.

The acceptance criteria for the damage states of the RC members were determined according to TEC2018 [16] depending on the strain measured both in concrete and longitudinal reinforcement. Three different damage limits are prescribed in the code: Limited Damage (LD), Moderate Damage (MD), and Severe Damage (SD). The Severe Damage strain limit for concrete is defined in TEC2018 [16] as given in the following equation (Eq. 2):

$$0.0035 + 0.04\sqrt{\omega_{we}} \leq 0.018 \tag{2}$$

where, ω_{we} is the mechanical reinforcement ratio of effective confinement bars, and it can be calculated as follows (Eq. 3):

$$\omega_{we} = \alpha_{se}\rho_{sh,min}\frac{f_{ywe}}{f_{ce}} \tag{3}$$

where, α_{se} represents the efficiency coefficient of confinement reinforcement and $\rho_{sh,min}$ shows the minimum of the volumetric confinement reinforcement ratio of the section in both lateral directions. Moreover, f_{ywe} and f_{ce} are the expected tensile yield strength of lateral reinforcement, and compressive strength of concrete, respectively. The Severe Damage strain limit for reinforcing steel is defined in TEC2018 [16] as $0.4\varepsilon_{su}$, where ε_{su} is the ultimate tensile strain of reinforcement that is 0.08 for the B420C reinforcing steel. Moreover, the Moderate Damage strain limits for both concrete and reinforcing steel can be calculated by taking 75 % of the Severe Damage strain limits. Lastly, the Limited Damage strain limit is constant for both concrete and reinforcing steel as 0.0025 and 0.0075, respectively. The calculated damage state limits of the members were defined in the sections' material model and reported in Table 4.

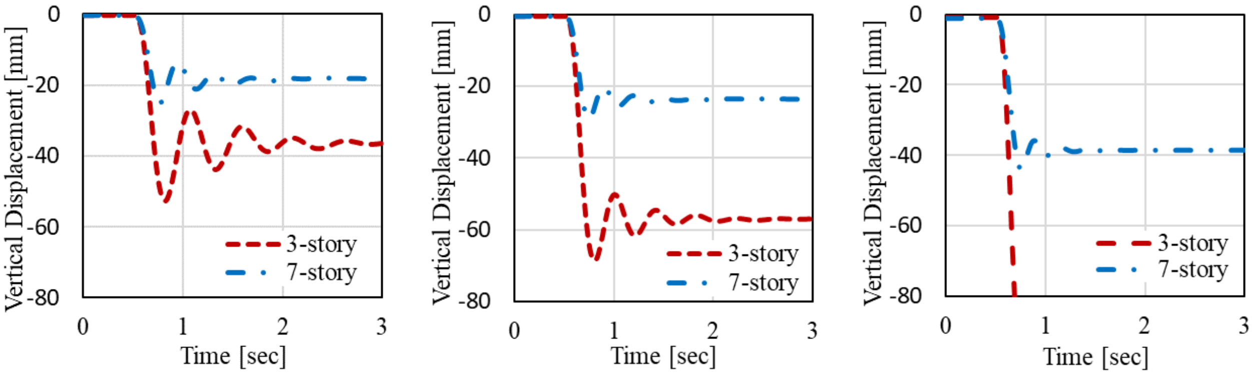
4. Results and Discussion

The vertical displacement time-history results of the nodes above the removed columns are depicted in Fig. 9 for different column loss scenarios. The residual vertical displacement (u_r) result of those nodes is reported in Table 5. The residual displacements experienced due to the corner column (A1) loss scenario were obtained as -36.5 mm and -18.3 mm for the 3-story and 7-story buildings, respectively. It was determined that u_r is higher for the 3-story building than for the 7-story structure (Fig. 9a). Similar behavior was observed for the middle edge column removal (A3), and u_r was obtained as -57.0 mm and -23.6 mm in this scenario (Fig. 9b). It is a fact that more beams participate in load transfer from the removed column to surrounding members through Vierendeel and Catenary Actions while the number of the stories of the buildings increases. Moreover, the most severe damage case was experienced due to removing the middle column on the inner axis (C3). The residual vertical displacement increased dramatically as the number of floors decreased. The residual displacement of the 7-story building was obtained as -38.6 mm while u_r of the node above the removed columns did not stop at any point. As a result, the members above and surrounding the removed column collapsed (Fig. 9c).

The damage response of the load-bearing members was assessed according to the acceptance criteria calculated in the previous section. The fiber hinge occurrence schemes of the members are depicted in Figs. 10 and 11. Since the buildings are symmetric in their plans and the results are the same for the perpendicular axis, only elevation views of the axis on which the removed column exists are shown. The beams bridging over the removed column had limited and moderate damage at their ends upon removing the corner column A1 from the 3-story building (Fig. 10a). No damage occurred to the surrounding columns. When column A3 was removed from the building, most of the beams in the vicinity of the removed column experienced moderate damage at their hinge locations. Moreover, limited damage occurred on the surrounding top story columns' upper ends (Fig. 10b). The removal of the inner axis column C3 led to a dramatic increase in the vertical displacement of the surrounding members. They failed due to the exceedance of their load-carrying

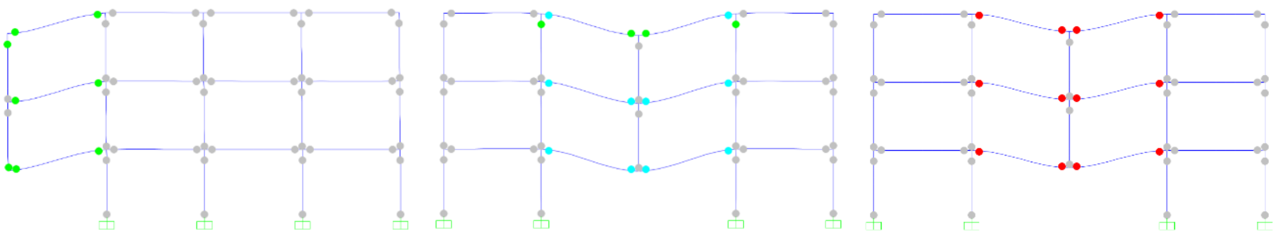
Table 4. Acceptance criteria for the structural members

Analyzed Building	Section Type	Concrete Strain			Reinforcement Strain		
		LD	MD	SD	LD	MD	SD
3-story	Column	-0.0025	-0.0089	-0.0119	0.0075	0.0240	0.0320
	Beam	-0.0025	-0.0052	-0.0069	0.0075	0.0240	0.0320
7-story	Column	-0.0025	-0.0080	-0.0106	0.0075	0.0240	0.0320
	Beam	-0.0025	-0.0041	-0.0055	0.0075	0.0240	0.0320



a) Removal of column A1 b) Removal of column A3 c) Removal of column C3

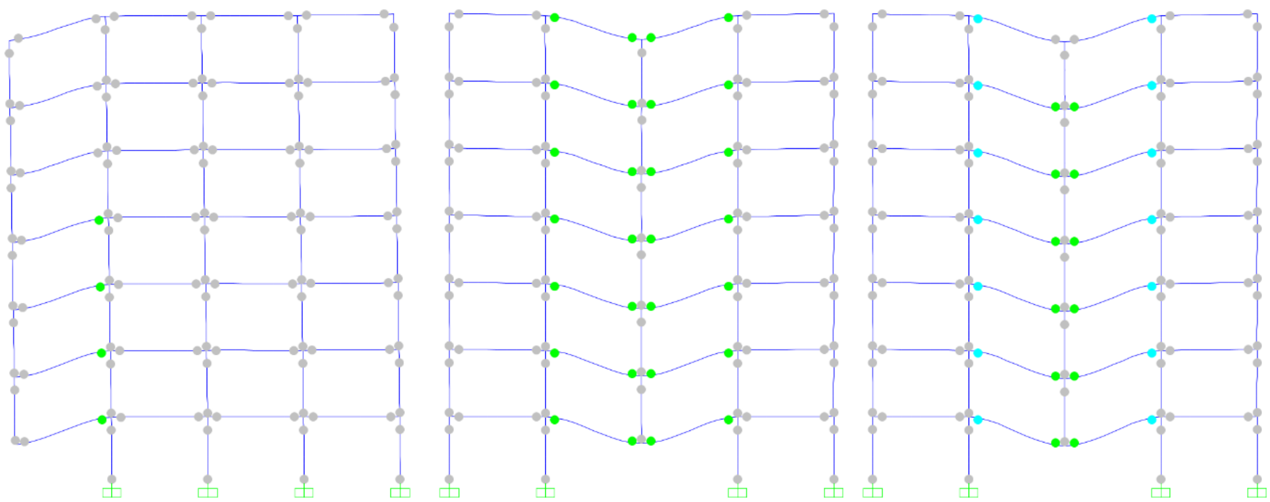
Figure 9. The vertical displacement time-history of the node above the removed column



a) Removal of column A1 b) Removal of column A3 c) Removal of column C3

Figure 10. Fiber hinge damage results of the 3-story building*

*Gray: no damage, green: limited damage, cyan: moderate damage, pink: severe damage (collapse prevention), red: failure (collapse).



a) Removal of column A1 b) Removal of column A3 c) Removal of column C3

Figure 11. Fiber hinge damage results of the 7-story building*

*Gray: no damage, green: limited damage, cyan: moderate damage, pink: severe damage (collapse prevention), red: failure (collapse).

capacities. Ultimately, a local collapse happened on the 3-story building (Fig. 10c).

After removing the corner column A1 from the 7-story building, no significant damage was observed on the members other than limited damage that occurred only on the lower-story beam-ends joining the surrounding columns (Fig. 11a). Limited damage was experienced at both ends of beams bridging over the removed column upon removing the middle column of the outer edge axis (A3) (Fig. 11b). Lastly, when the inner axis column C3 was removed from the building, the surrounding beams experienced limited damage at their ends above the removed column and moderate damage at their other ends connecting to the surrounding columns (Fig. 11c). Any local collapse was not observed on the 7-story building under all column removal scenarios.

The buildings experienced different damage states on their load-bearing members after losing one of their columns on their first floors. The damage was localized specifically on the members directly interacting with the removed column. The building plans showing the experienced damage zones are depicted in Fig. 12. The most severe damage occurred in the inner column loss scenario. The most likely reasons for that behavior are: i) the loads needed to be transferred to the surrounding structural members become high on the beams bridging over the removed column since the inner axis columns have a larger tributary loading area, ii) gravity loads become conservative for the outer axis columns due to having lower tributary loading areas since the same geometry and reinforcing details were considered for all inner and outer axis' columns. Nonetheless, if a more precise design were to be performed for the outer axis columns, a probable decrease in column geometry and reinforcing detail would lead to severe damage and failure on the members on those axes as well. A local collapse was observed only in the 3-story building in the case of inner column loss. No general collapse happened on any building for any column removal scenario.

Table 5. Result of the residual vertical displacement value of the node above the removed column

Analyzed Building	Removed Column	u_r [mm]
3-story	A1	-36.5
	A3	-57.0
	C3	collapse
7-story	A1	-18.3
	A3	-23.6
	C3	-38.6

Consequently, it can be deduced that the progressive collapse risk of buildings designed according to TEC2018 [16] with 'government buildings' occupancy class is high specifically due to an inner column loss. An unforeseeable initial failure may propagate to the surrounding members and lead to a disproportionate local collapse. Moreover, since there was no irregularity in the investigated buildings, it should be noted that the existence of any irregularity on a structure might further reduce its progressive collapse resistance. Therefore, in addition to the conventional seismic design methods, government buildings should also be designed according to the explicit design approaches against extreme events to reduce their PC risk. Moreover, the progressive collapse response of existing government buildings should also be evaluated accordingly.

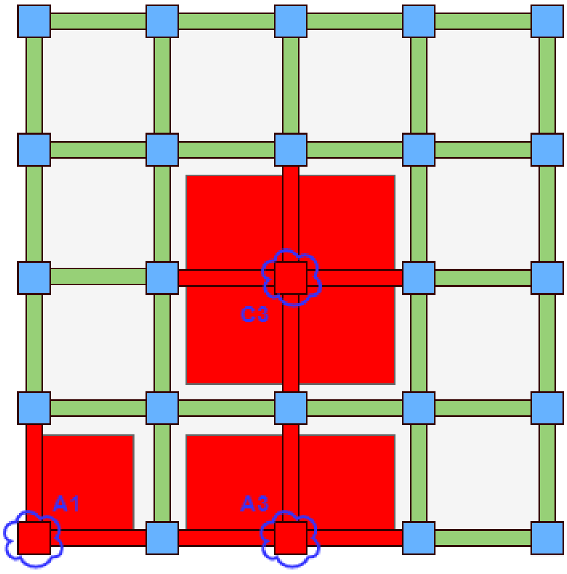


Figure 12. The experienced damage and collapse regions after a column loss

5. Conclusion and Suggestions

The objective of this study is to investigate the progressive collapse response of RC frame buildings designed according to TEC2018 [16] for the 'government buildings' occupancy class. For this purpose, one of the direct design approaches of GSA-2016 [6] and UFC 4-023-03 [7] guidelines was implemented by using the Nonlinear Dynamic Analysis Method of the Alternate Path design approach. Nonlinear fiber hinges were used to simulate the plasticity of the structural load-bearing members. The following conclusions were deduced:

- The residual vertical displacement of the node above the removed column decreases as the number of stories of the buildings increases under all column loss scenarios.
- The damage to the structural members after a column removal was localized on the members surrounding the removed column. The damage was not distributed to other members of the building. Thus, a total collapse was not observed.
- The inner column loss scenario leads to the most severe damage case for the buildings. Moreover, the damage experienced on the buildings increases while the story number decreases for all column loss scenarios.

- In addition to the conventional seismic design procedures, the structures designed for the 'government buildings' occupancy class require additional direct design considerations to improve their progressive collapse resistance against extreme events.

The structural design of the buildings was simplified in the study by generalizing their member sections similar to the common practice to reduce labor and formwork costs, and construction errors. Since corner and side columns connect to fewer load-bearing members to redistribute the loads after a column removal, their PC response could be as severe as the interior column. Therefore, a more precise design could be done for the building to more accurately observe the response of structures subjected to a corner and side column loss scenario. Moreover, the existing buildings designed and constructed before the current TEC2018 [16] may have a higher progressive collapse risk since they were built using prior engineering knowledge. Therefore, special attention should be paid to those buildings' PC resistance under likely dramatic extreme events to prevent casualties and economic losses. For this purpose, new studies can be done on the buildings with different structural systems, loads, soil properties, etc., to be able to assess in a more generalized manner the progressive collapse risk of 'government buildings' occupancy class structures designed to TEC2018 [16] or previous updates of it.

Acknowledgments

The author was supported by the 2219-International Postdoctoral Research Fellowship Program of The Scientific and Technological Research Council of Turkey (TUBITAK). The author also thanks Prof. Dr. Halil Sezen for his exceptional support and guidance.

Statement of Research and Publication Ethics

The study is complied with research and publication ethics

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