Journal of Structural Engineering & Applied Mechanics (2024) 7(4):299-314
DOI 10.31462/jseam.2024.04299314

RESEARCH ARTICLE

Investigation of progressive collapse resistance of cantilevered structure based on concrete strength

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Article History

Received 2 September 2024
Accepted 20 December 2024

Keywords

Progressive collapse
Cantilevered structures
Reinforced concrete
Applied element method
ELS software

Abstract

Turkey is a region known for its high seismic activity, and unfortunately, many of its existing buildings are not well-prepared to withstand earthquakes. This study delves into the consequences of removing ground floor columns, either due to architectural modifications or damage from earthquakes to the structural integrity of buildings. To investigate this, a model simulating a cantilevered structure was developed, allowing for a detailed analysis of potential collapse mechanisms. The structural model, designed using STA4CAD software, represents a seven-story building with a ground floor height of 4.2 meters and a typical floor height of 3.2 meters. The building has a cantilever length of 1.5 meters, with 5-meter spacing between axes in both the X and Y directions. A uniform slab thickness of 150 mm and the load-bearing system is composed of columns and beams. These designs were then transferred into the Extreme Loading Structures (ELS) software, where further analysis was conducted. In the ELS program, models were created with varying concrete classes of C10, C15, and C20, incorporating specific cross-section dimensions and reinforcements. The primary focus of the analysis was to assess the buildings' vulnerability to progressive collapse, particularly when critical ground-floor columns were removed. The evaluation followed the guidelines set out in the "Design of Buildings to Resist Progressive Collapse" (UFC 4-023-03) code. The findings of this study are significant. Buildings constructed with C10-grade concrete were found to be highly susceptible to collapse when either interior or corner columns were removed. On the other hand, buildings with stronger C15 and C20 concrete demonstrated greater resilience, only suffering damage rather than complete collapse upon the removal of corner columns. These results underscore the importance of both material strength and architectural design in ensuring the seismic safety of buildings in earthquake-prone areas.

1. Introduction

Turkey is in an active seismic zone that frequently experiences intense earthquakes. As a result, it is one of the few countries where earthquakes have led to significant loss of life. In recent years, devastating earthquakes have occurred in Turkey, particularly the February 6, 2023, earthquakes, which resulted in substantial loss of life and property damage. Given that a large portion of the population lives in earthquake-prone areas, the damage and social problems caused by earthquakes occasionally impact the country's agenda

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and economy. This situation highlights the serious earthquake threat facing much of Turkey and underscores the need for continued efforts to minimize losses that may result from future earthquakes.

Unfortunately, a large proportion of Turkish buildings have insufficient earthquake resistance. This inadequacy often leads to loss of life in earthquakes and raises serious concerns about the safety of these structures. Research conducted after earthquakes has shown that, despite differences in seismic regions, structural damage to buildings is often similar. The large number of low and medium-rise buildings that are not earthquake-resistant contributes negatively to the overall seismic risk. Key factors negatively affecting the seismic performance of these buildings include inadequate material strength, short column behavior, slab discontinuities, soft-story irregularities, and cantilevers.

In Turkey, many buildings are constructed with cantilevers on the upper floors above the ground floor. The rationale for this design is to increase usable space on the upper floors by constructing a smaller ground floor area based on floor area ratio regulations. However, in rooms with cantilevers, instead of directly connecting beams from column to column, beams are connected to columns by wrapping around the cantilevers, preventing the formation of a complete frame system. This affects the continuity of the frame due to discontinuous beams and increases deflection problems in slabs. Cantilevers negatively impact the rigidity of the structure and adversely affect its behavior during an earthquake. Nevertheless, cantilevers have continued to be constructed extensively to provide more usable space. The Kahramanmaraş earthquake in Turkey in 2023, which resulted in numerous fatalities and widespread destruction, has clearly demonstrated the consequences of not considering seismic effects during the design phase and the insufficient earthquake resistance of existing buildings. In this context, it is necessary to make the necessary regulations and improve engineering practices to enhance the seismic performance of buildings. Accordingly, many local administrations are working on preventing certain practices, particularly these cantilevers.

Over the years, various methodologies and factors influencing progressive collapse have been investigated to enhance the robustness and safety of building structures. Helmy et al. [1] explored the application of computer-aided tools in assessing the progressive collapse potential of reinforced concrete (RC) structures following the GSA guidelines. Their study highlighted the importance of advanced computational methods in accurately predicting collapse mechanisms and identifying critical structural vulnerabilities. Similarly, Li et al. [2] introduced an improved tie force method, which demonstrated enhanced resistance to progressive collapse in RC frames by optimizing the distribution of tie forces.

The interaction between soil and structure significantly impacts a building's response to progressive collapse. Özgan et al. [3] conducted multiple studies examining the soil-structure interaction (SSI) effect on progressive collapse resistance. In their investigation of steel frames, they found that incorporating SSI using linear static and nonlinear dynamic analyses revealed a notable variation in collapse behavior. Further [4], they evaluated an RC school building and demonstrated that SSI could influence the progression of collapse mechanisms, emphasizing the need for its consideration in structural design and analysis.

Kiakojouri et al. [5] provided a comprehensive review of the state-of-the-art knowledge on progressive collapse in framed building structures. They identified critical research gaps, particularly the need for more robust design frameworks to address the complex interactions of material properties, geometric configurations, and external loads. Similarly, Adam et al. [6] reviewed research trends and practical approaches to enhancing structural robustness, underscoring the importance of adopting integrated design strategies to mitigate progressive collapse risks.

Parametric studies, such as those by Li et al. [7], have provided valuable insights into the sensitivity of RC frames to progressive collapse under various conditions. Their work, which incorporated SSI effects, revealed how variations in parameters such as soil stiffness and structural configuration could influence collapse potential. This underscores the need for a holistic approach to structural analysis that accounts for the dynamic interplay between soil and structural elements.

This study explores how removing ground floor columns for architectural purposes or their loss due to earthquakes affects the progressive collapse behavior of buildings with cantilevers. For this purpose, the aim was to create a building model that can provide a better understanding of collapse mechanisms and stages. For this objective, a model was developed using the Extreme Loading for Structures (ELS) software with the Applied Element Method (AEM). The resistance of buildings to progressive collapse was checked using the three-dimensional nonlinear dynamic Alternate Path Method in accordance with UFC guidelines. The analysis results were evaluated according to the UFC code to determine the necessary measures to enhance structural strength and improve the seismic safety of existing buildings.

2. Conceptual theory

Frame discontinuity is a widespread issue in our country, mainly because users often prioritize architectural convenience and the creation of more spacious areas in their buildings. However, the common use of enclosed projection systems leads to breaks in the continuity of the structural frame. Observations from past earthquakes, such as the example shown in Fig. 1, have revealed that cantilevers can negatively impact the seismic performance of buildings [8-10].

Many studies conducted by researchers have observed that cantilevers lead to an increase in weight, which in turn increases the building's period and displacement demands. This negatively impacts the seismic behavior of structures. Although cantilevers are generally preferred to provide more usable space, they affect risks to structural integrity and safety. Therefore, to enhance the seismic performance and safety of buildings, it is crucial to carefully assess the effects of cantilevers and develop appropriate engineering solutions [11,12].

3. Progressive collapse mechanism

Progressive collapse is a process that begins with the failure of one or more vertical load-bearing elements in a structure, leading to local or general collapse of the building. As shown in Fig. 2, vertical load-bearing elements like columns or shear walls can become dysfunctional due to an explosion or another abnormal loading event. In such cases, the loads carried by the failed vertical elements are redistributed to adjacent elements. If these neighboring elements lack the capacity to bear the additional loads, progressive collapse occurs.

Progressive collapse is a chain reaction where local damage causes the failure of adjacent structural components, eventually leading to the collapse of a significant portion or even the entirety of the structure. In other words, it is a process where local damage caused by rare and extreme events triggers a collapse sequence that results in the partial or total failure of the building [13].

The concept of progressive collapse first gained attention on May 16, 1968, when a gas explosion on the 18^{th} floor of the 22-story Ronan Point apartment building in London caused the building to collapse down to the lower floors. This incident highlighted the dangers of progressive collapse and made it a significant topic of research in the fields of engineering and building safety. Later events, such as the 1995 terrorist bombing of the Alfred P. Murrah Federal Building in Oklahoma and the attacks on the World Trade Center on September 11, 2001, further underscored the potential dangers of progressive collapse. These incidents led to increased research and the incorporation of progressive collapse concepts into building codes in many countries [15-17].

Fig. 1. Damage to Cantilevers due to the 2003 Bingöl Earthquake

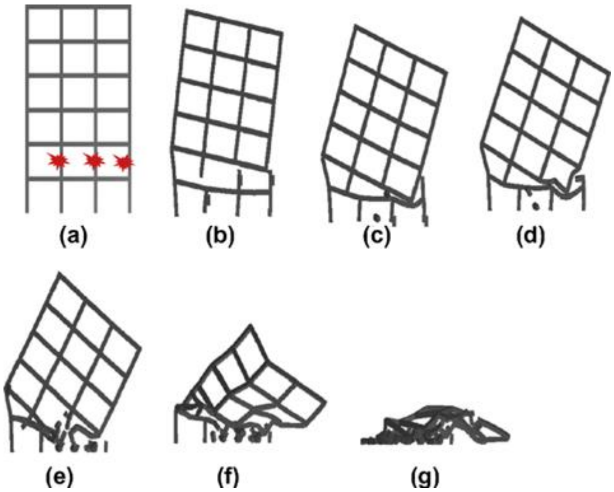

Fig. 2. The collapse process of a frame building demolished by a controlled explosion, considering impact effects: (a) initial explosion, (b) collapse, (c) impact between elements, (d) deformation in elements due to impact, (e) element separation, (f) progressive collapse [14]

3.1. Alternate load path

In progressive collapse studies, the column removal method is a commonly used approach to evaluate the response of structures to extreme events. Given the difficulty in predicting the probability and severity of such extreme events, designing buildings to handle these situations with traditional methods is neither economical nor practical. Therefore, current progressive collapse prevention design concepts include both direct and indirect design methods. One direct design method, the Alternate Load Path (ALP) method, assesses whether buildings can provide effective alternative load transfer paths under ideal column removal scenarios. This threat-independent design method is considered the most reliable approach for evaluating the progressive collapse resistance of building structures. [18]

In a building structure, the most severe local damage occurs when one or more vertical load-bearing components (such as columns or shear walls) fail. This leads to a chain reaction of damage that can result in the complete collapse of the entire building or a significant portion of it. To prevent progressive collapse, as shown in Fig. 3, it is essential to have alternative load paths available to transfer the load supported by the damaged column to adjacent elements. If effective alternative paths are not present, progressive collapse becomes inevitable unless additional design measures are taken.

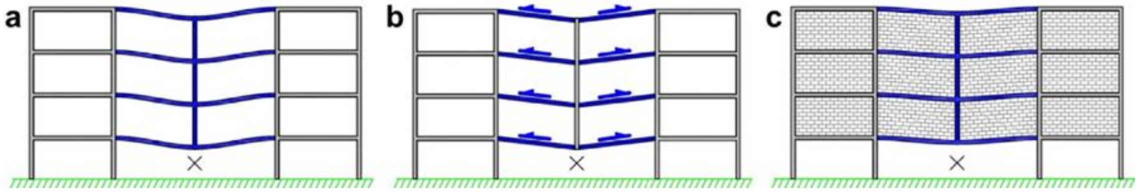

Fig. 3. Alternative load paths: (a) Vierendeel effect; (b) Catenary effect; and (c) Contribution of non-structural elements [19]

For framed buildings, five resistance mechanisms can provide alternative load paths and minimize the risk of progressive collapse:

a) **Bending of Beams at the Location of the Failed Column:** This mechanism is often ineffective because it requires beams to be oversized and is rarely utilized.
b) **Vierendeel Behavior of the Frame at the Failed Column:** This refers to the ability of the frame to redistribute loads through a Vierendeel mechanism when a column fails.
c) **Catenary Effect of Lateral Beams When a Column Fails:** This mechanism is effective when the horizontal displacement of neighboring columns is minimal, allowing the lateral beams to act in a catenary action to support the structure.
d) **Catenary/Membrane Behavior of Beams/Slabs Bridging the Damaged Column with Large Rotations and Displacements:** This mechanism involves the beams and slabs taking on catenary or membrane-like behavior to bridge the gap created by the failed column.
e) **Contribution of Non-Structural Elements:** Elements such as exterior walls and partitions can provide additional support and load paths in the event of structural failure.

In a building floor plan, columns placed in different locations are characterized by varying loadings and constraints. Consequently, the structural system's response to the loss of these columns will naturally differ. The Alternate Load Path Method specified in UFC-4–023-03 requires buildings to be designed to withstand different column removal scenarios based on specific risk categories. These locations include corner columns, edge columns, and interior columns.

Corner columns are more vulnerable structural components compared to edge and interior columns. Protecting corner columns from explosions or impacts is generally more challenging compared to interior columns. Additionally, the removal of corner columns often results in less development of secondary resistance mechanisms due to the reduced horizontal constraint provided by surrounding elements. As a result, the probability of an initial failure involving the loss of corner columns is typically higher [20].

Therefore, this study analyzes scenarios specified in the UFC, including the removal of exterior corner columns ECC and, additionally, scenarios involving the removal of interior corner columns ICC due to architectural concerns.

4. Methodology

4.1. Modeling properties

A model representing cantilever structures, with a ground floor height of 4.2m and a typical floor height of 3.2m, has been designed with a total of 7 floors using the STA4CAD program. As shown in Fig. 4, the cantilever length is 1.5m, and the axis spacings in the X and Y directions are 5m. Beams with the same reinforcement and cross-section are named similarly. The load-bearing system consists of columns and beams. The slab thickness is taken as 150mm for all floors. The cross-sectional details for the columns and beams are provided in Table 1.

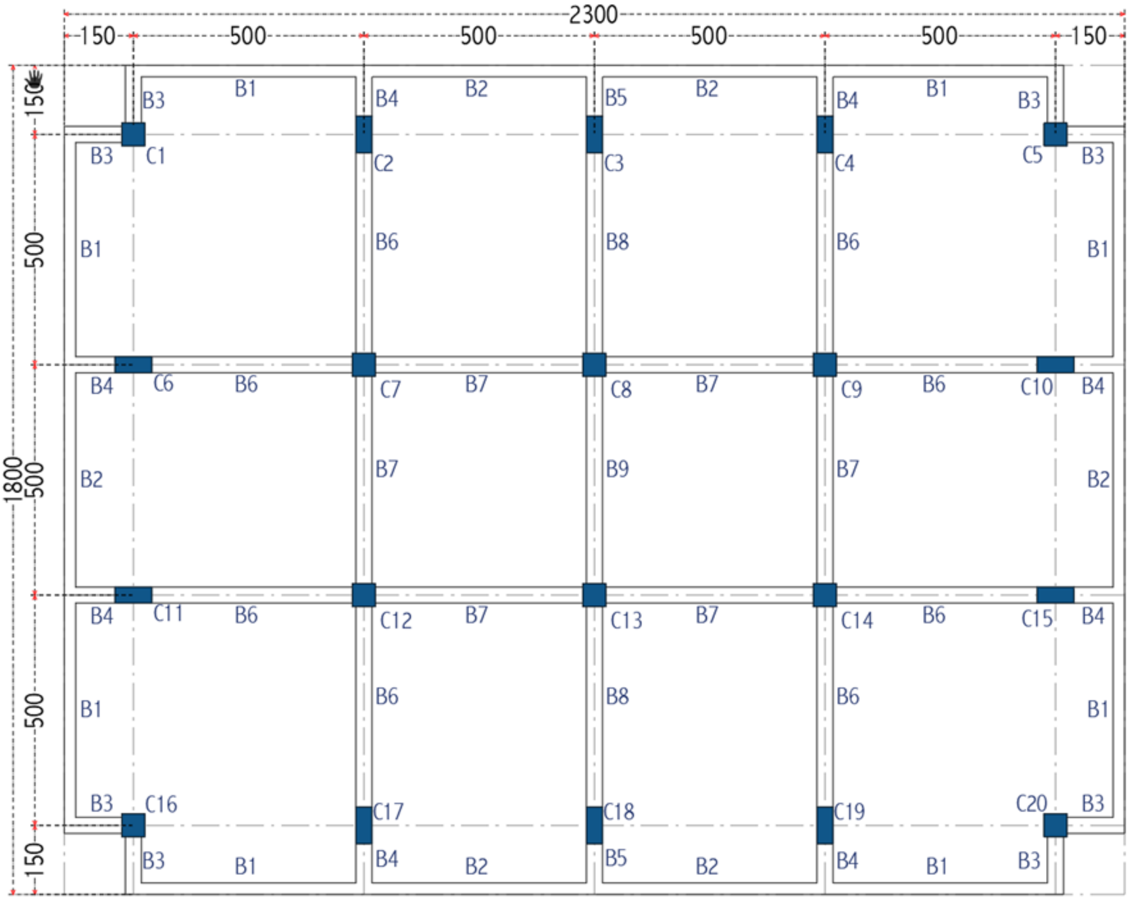

Fig. 4. Plan of the model

Table 1. Column and beam section dimensions and reinforcement

Member	Ground story		2nd to 4th story		5th to 7th story	
RC element	Section (mm)	Range of reinforcement details	Section (mm)	Range of reinforcement details	Section (mm)	Range of reinforcement details
Corner and Interior Columns	500×500	Long. bars: 20ϕ14 mm Ties: 3ϕ8 mm/150 mm	450×450	Long. bars: 16ϕ14 mm Ties: 3ϕ8 mm/150 mm	400×400	Long. bars: 14ϕ14 mm Ties: 3ϕ8 mm/150 mm
Side Columns	350×800	Long. bars: 20ϕ14 mm Ties: 3ϕ8 mm/150 mm	350×800	Long. bars: 20ϕ14 mm Ties: 3ϕ8 mm/150mm	350×800	Long. bars: 20ϕ14 mm Ties: 3ϕ8 mm/150 mm

Table 1. Continued

Member	Ground story		2nd to 4th story		5th to 7th story	
External Beam	250×500	Top bars from: 2ϕ14 mm to 4ϕ14 mm Bottom bars from: 3ϕ14 mm to 4ϕ14 mm End stirrups: ϕ8 mm/100 mm Midspan stirrups: ϕ8 mm/200 mm	250×500	Top bars from: 2ϕ14 mm to 4ϕ14 mm Bottom bars from: 3ϕ14 mm to 4ϕ14 mm End stirrups: ϕ8 mm/100 mm Midspan stirrups: ϕ8 mm/200 mm	250×500	Top bars from: 2ϕ14 mm to 4ϕ14 mm Bottom bars from: 3ϕ14 mm to 4ϕ14 mm End stirrups: ϕ8 mm/100 mm Midspan stirrups: ϕ8 mm/200 mm
Internal Beam	350×500	Top bars from: 3ϕ16 mm to 6ϕ16 mm Bottom bars from: 3ϕ6 mm to 4ϕ16 mm End stirrups: ϕ8 mm/100 mm Midspan stirrups: ϕ8 mm/200 mm	350×500	Top bars from: 3ϕ16 mm to 6ϕ16 mm Bottom bars from: 3ϕ16 mm to 4ϕ16 mm End stirrups: ϕ8 mm/100 mm Midspan stirrups: ϕ8 mm/200 mm	350×500	Top bars from: 3ϕ16 mm to 6ϕ16 mm Bottom bars from: 3ϕ16 mm to 4ϕ16 mm End stirrups ϕ8 mm/100 mm Midspan stirrups: ϕ8 mm/200 mm
Slab thickness	150	Bottom bars both direction: ϕ8 mm/160 mm Top bars both direction: ϕ8 mm/110 mm	150	Bottom bars both direction: ϕ8 mm/160 mm Top bars both direction: ϕ8 mm/110 mm	150	Bottom bars both direction: ϕ8 mm/160 mm Top bars both direction: ϕ8 mm/110 mm

The seismic parameters used in the study are determined based on the location of Nilüfer district in Bursa Province, Turkey, and the Earthquake Hazard Map prepared by the Ministry of Public Works and Settlement [20] The loads affecting the slabs and beams are shown in Table 2. The concrete class is selected as C20/25 (characteristic cylinder compressive strength of 20 MPa), and the reinforcement class is S420 (characteristic yield strength of 420 MPa). According to TS500, the modulus of elasticity for C20/25 concrete is 28,000 MPa. This value has been used to simulate the mechanical behavior of concrete in the models.

Table 2. Earthquake parameters and load information

Number of Floors of the Building	:	Ground Floor + 6 Floors
Height of the Floor	:	4.2 m (Ground floor), 3.2 m (floor)
Total Length of the Building in X Direction	:	23.00 m
Total Length of the Building in Y Direction	:	18.00 m
Earthquake Zone	:	Region 1
Earthquake Zone Coefficient (Co)	:	0,1
Live Load Participation Coefficient (n)	:	0,3
Building Importance Coefficient (I)	:	1
Earthquake Structure Behavior Coefficient (K)	:	1
Spectrum Characteristic Periods	:	TA=0.15 s, TB= 0.40 s
Concrete Class Used in Existing Structure	:	C20/25
Steel Class Used in Existing Structure	:	S420
Slab Type	:	Solid slab
Floor Thickness	:	120 mm
Floor Loads: Dead Load / Live Load	:	0.512 t/m^2 - 0.2 t/m^2
Beam Loads: External Beams / Internal Beams	:	0.85 t/m - 0.66 t/m

5. Modeling analysis by ELS

Structural designs determined using the STA4CAD program were modeled in the Extreme Loading Structures (ELS) program for progressive collapse analysis (Fig. 5). Initially, the sections and reinforcements were introduced into the program, and then analyses were performed separately for concrete strengths C10, C15, and C20. The results were evaluated according to the Unified Facilities Criteria (UFC) code titled "Design of Buildings to Resist Progressive Collapse" (UFC 4-023-03). The UFC provides three different analysis methods based on building categories: the Alternate Load Path Method, the Improved Local Resistance Method, and the Connection Force Method (UFC).

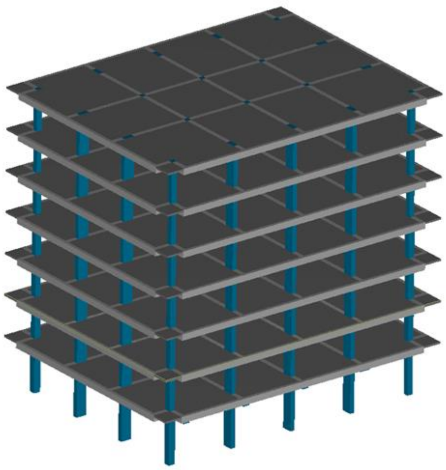

Fig. 5. ELS model view

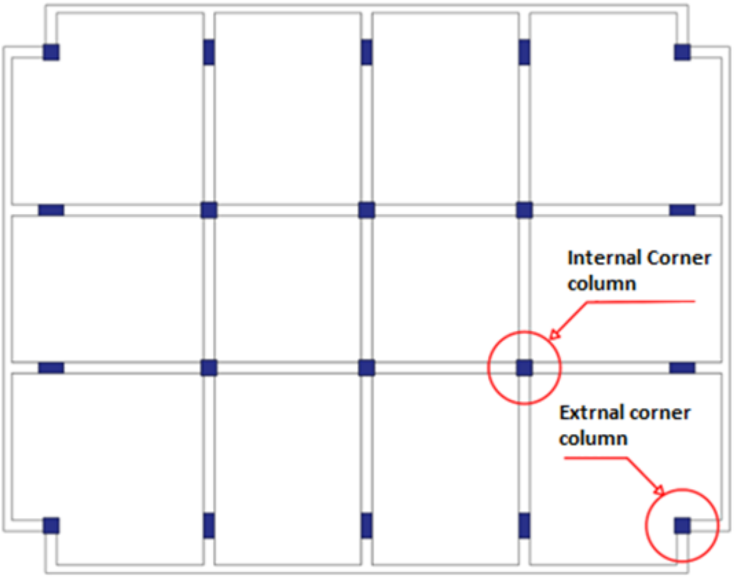

Fig. 6. Column removal scenarios

In this study, since the risk category of the models was determined as Category II according to the UFC, the Nonlinear Dynamic Procedure of the Alternate Load Path Method was used to evaluate the resistance to progressive collapse. The analyses were performed under the 1.2D+0.5L load combination and included scenarios of removing both interior and exterior corner columns on the ground floor, as shown in Fig. 6. The results were then evaluated.

5.1. Models with concrete strength C10

5.1.1. Removal of exterior corner column (C10-ECC)

In the scenario where the exterior corner column ECC is removed from the ground floor, the model with C10 concrete strength shows that the load carried by the removed column was transferred to the neighboring elements. However, these elements were unable to bear the additional load and lost their load-carrying capacity. This process continued until the structure completely collapsed, resulting in a progressive collapse. The collapse steps are shown in Fig. 7.

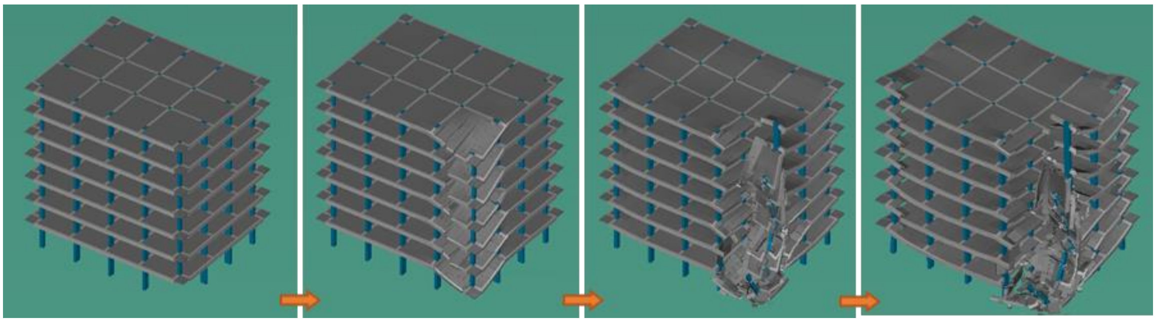

Fig. 7. Collapse steps of the C10-ECC scenario by ELS software

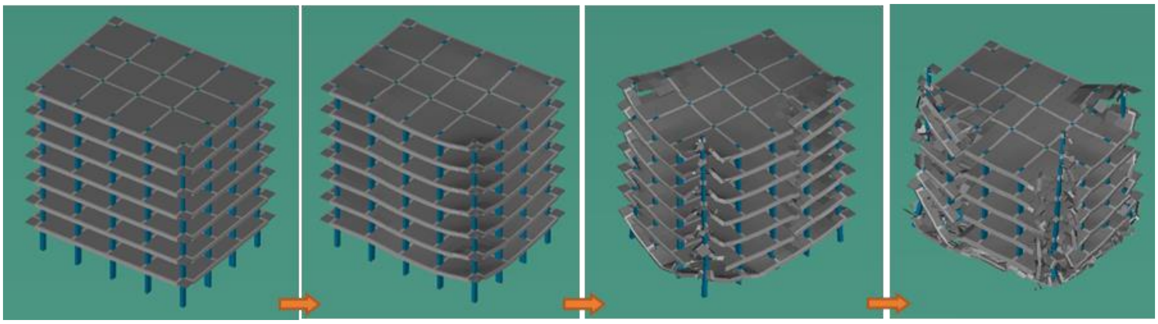

Fig. 8. Collapse steps of the C10-ICC model by ELS software

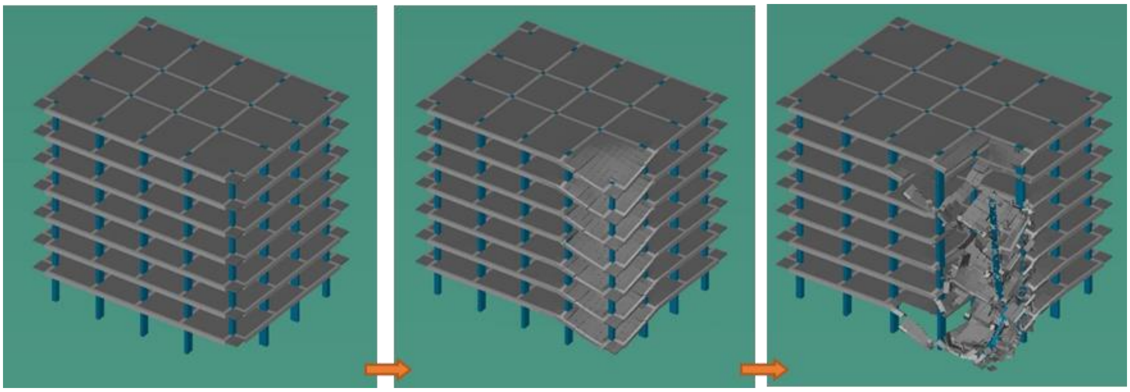

Fig. 9. Collapse steps of the C15-ECC scenario by ELS software

5.1.2. Removal of interior corner column (C10-ICC)

In the model with C10 concrete strength, the removal of the interior corner column ICC resulted in a collapse, similar to the scenario of removing the exterior corner column. The analysis results, as shown in Fig. 8, indicate that the structure ultimately collapsed.

5.2. Models with concrete strength C15

5.2.1. Removal of exterior corner column (C15-ECC)

In the model with C15 concrete strength, the removal of the exterior corner column (ECC) from the ground floor led to a localized collapse. This collapse was confined to the area surrounding the removed column, as illustrated in the plan view in Fig. 6. The removed columns and the columns directly above it, spanning different stories, suffered significant damage. Additionally, the beams and slab connected to the removed column and the affected stories above were damaged. However, the beams not directly connected to the ECC column remained intact. The collapse steps of the model are shown in Fig. 9.

5.2.2. Removal of interior corner column (C15-ICC)

In the model with C15 concrete strength, the removal of the interior corner column did not result in a collapse. However, the final status of the structure concerning progressive collapse will be evaluated based on the limit states outlined in the UFC regulations. Fig. 10 shows the displacements and total rotation values resulting from the column removal graphically. The maximum displacement was observed to be 72.9mm at the point immediately above the removed column, while the maximum rotation observed in the slab was 0.033 rad.

Fig. 11 shows verification of the model against the requirements specified in Table 4.1 of the UFC Code. The analysis checks whether the model meets the requirements outlined in Table 4.1 of the UFC code. According to the beam cross-section and reinforcement properties, the allowable plastic rotation for beams is 0.0063 rad, as specified in Table 4.1 UFC code. The risk of progressive collapse has been assessed based on the limit values marked in the table.

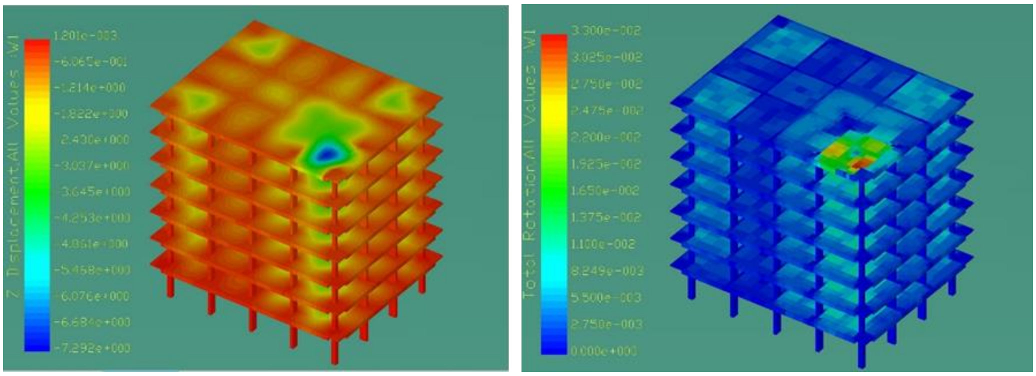

Fig. 10. Displacements (cm) and rotation (rad) values resulting from column removal in the C15-ICC by ELS software

Table 4-1. Nonlinear Modeling Parameters and Acceptance Criteria for Reinforced Concrete Beams (Replacement for Table 10-7 in ASCE 41)

Conditions			Modeling Parameters[1]			Acceptance Criteria[1,2]		
						Plastic Rotations Angle, radians		
			Plastic Rotations Angle, radians		Residual Strength Ratio		Component Type	
							Primary	Secondary
			a	b	c			
i. Beams controlled by flexure[3]								
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[4]	$\frac{V}{b_w d \sqrt{f'_c}}$						
≤ 0.0	C	≤ 3	0.063	0.10	0.2		0.063	0.10
≤ 0.0	C	≥ 6	0.05	0.08	0.2		0.05	0.08
≥ 0.5	C	≤ 3	0.05	0.06	0.2		0.05	0.06
≥ 0.5	C	≥ 6	0.038	0.04	0.2		0.038	0.04
≤ 0.0	NC	≤ 3	0.05	0.06	0.2		0.05	0.06
≤ 0.0	NC	≥ 6	0.025	0.03	0.2		0.025	0.03
≥ 0.5	NC	≤ 3	0.025	0.03	0.2		0.025	0.03
≥ 0.5	NC	≥ 6	0.013	0.02	0.2		0.013	0.02
ii. Beams controlled by shear[3]								
Stirrup spacing ≤ d /2			0.0030	0.02	0.2		0.002	0.01
Stirrup spacing > d /2			0.0030	0.01	0.2		0.002	0.005
iii. Beams controlled by inadequate development or splicing along the span[3]								
Stirrup spacing ≤ d /2			0.0030	0.02	0.0		0.002	0.01
Stirrup spacing > d /2			0.0030	0.01	0.0		0.002	0.005
iv. Beams controlled by inadequate embedment into beam-column joint[3]								
			0.015	0.03	0.2		0.01	0.02

1. Linear interpolation between values listed in the table shall be permitted. See Section 3-2.4 for definition of primary and secondary components and Figure 3-7 for definition of nonlinear modeling parameters *a*, *b*, and *c*
2. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength, in accordance with Section 7.4.3.2 of ASCE 41.
3. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
4. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ *d*/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.

Fig. 11. UFC Table 4.1

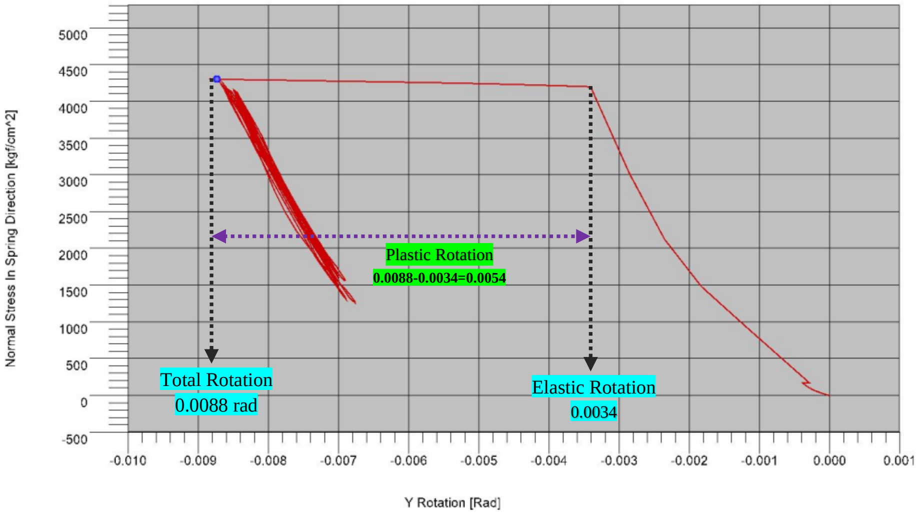

Fig. 12. Determination of beam plastic rotation values for the C15-ICC scenario

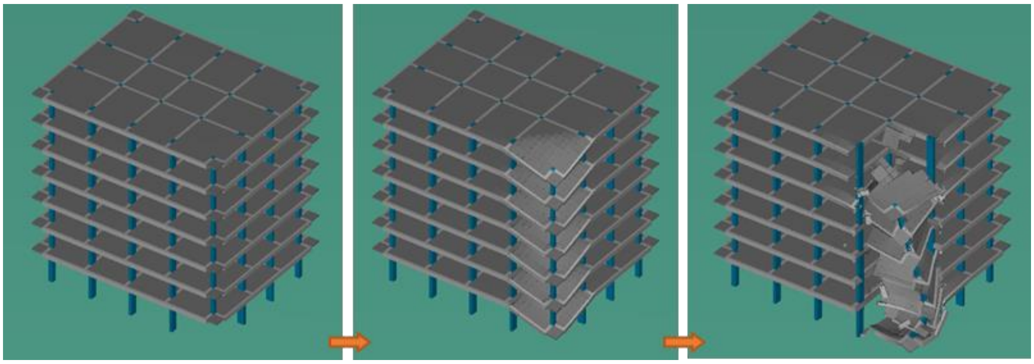

Fig. 13. Collapse steps of C20-ECC scenario

The analysis revealed that the maximum plastic rotation in the beams was 0.0054 rad, as shown in Fig. 12, which is less than 0.063. Since this value is smaller than the limit conditions specified in Fig. 12, the model with C15 concrete strength was determined to be resistant to progressive collapse according to the UFC code when the interior corner column was removed.

To determine the amount of plastic rotation in the beams, the total rotation of the beam element at the point of maximum rotation of 0.0088 rad (as shown in Fig. 12) was calculated by subtracting the elastic rotation of 0.0034 rad from it, resulting in the plastic rotation, which was 0.0054 rad as shown in Fig. 12.

5.3. Concrete strength C20 models

5.3.1. Scenario of removing the exterior corner column (C20-ECC)

The removal of the exterior corner column from the ground floor, collapse was observed in the C20 concrete strength model. However, as shown on Fig. 13, the collapse was confined to the area where the column was removed. The vertical supports adjacent to the removed column did not sustain damage, but the beams connected to the column suffered damage due to the loads transferred from the column, which exceeded their carrying capacity.

5.3.2. Interior corner column removal scenario (C20-ICC)

In the model with C20 concrete strength, the removal of the interior corner column did not result in collapse. However, the final assessment of the building's susceptibility to progressive collapse will be determined based on whether it meets the limit conditions specified in the UFC regulations. Fig. 14 shows the displacements and total rotations resulting from the column removal. The maximum displacement was 41.7mm at the point just above the removed column, while the maximum total rotation in the slab was observed to be 0.0139 rad.

As shown in Fig. 15, the plastic rotation value in the C20-ICC model was calculated to be 0.0032 rad, where total rotation was 0.0068 rad and elastic rotation was 0.0036 rad. Since this value is smaller than the limit conditions specified in Fig. 11 (UFC Table 4.1), the model with C20 concrete strength was determined to be resistant to progressive collapse according to the UFC code following the removal of the interior corner column.

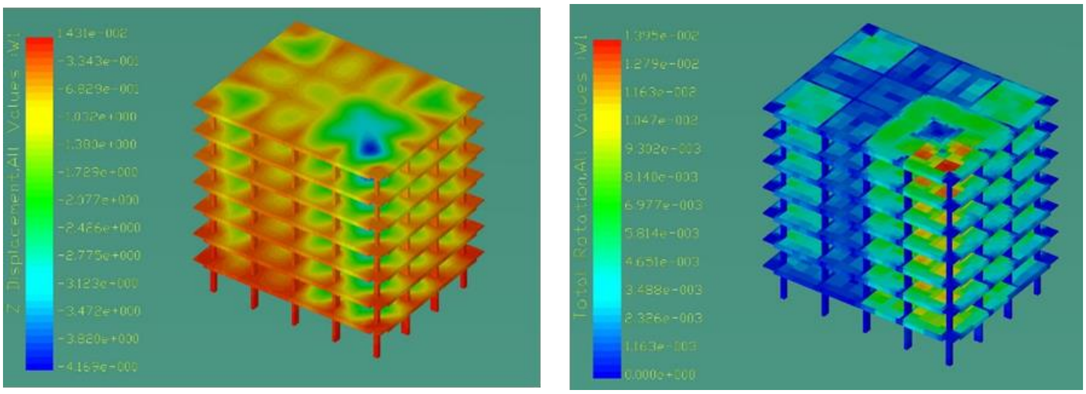

Fig. 14. Displacement (cm) and rotation (rad) values resulting from column removal in the C20-ICC scenario by ELS software

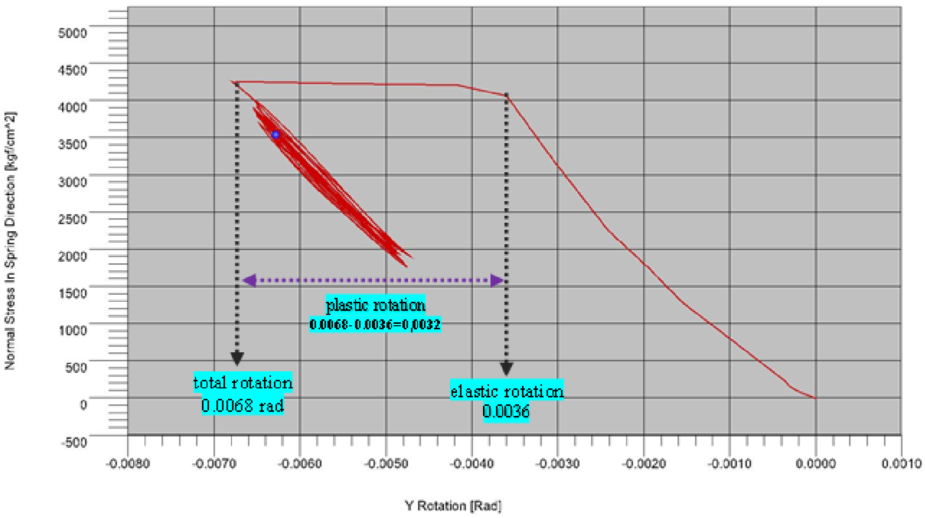

Fig. 15. Determining the plastic rotation value of the beam in the C20-ICC model

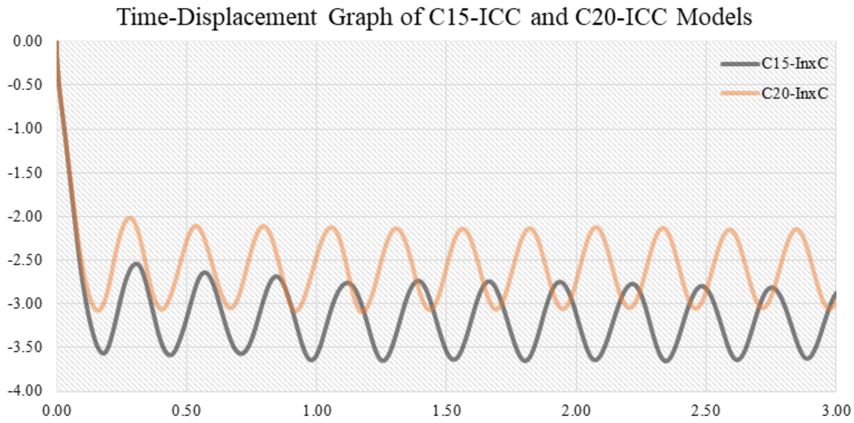

Fig. 16. Displacement vs. Time graphs for C15-ICC and C20-ICC models

Fig. 16 presents the comparison of the displacement changes over time in models with C15 and C20 concrete. Where the maximum displacement for the model with C15 was 3.65cm and for the model with C20, the maximum displacement was 3.08cm.

6. Conclusions

In this study, the progressive collapse resistance of models with cantilevers designed according to the 2018 Turkish Earthquake Code was investigated according to UFC standards, and the effects of changes in concrete strengths on progressive collapse behavior were examined. The main conclusions from this study are as follows:

- In models with C10 concrete strength, the structure completely collapsed in both the external and internal corner column removal scenarios.
- In models with C15 concrete strength, the structure partially collapsed in the external corner column removal scenario, while the internal corner column removal scenario showed that the structure was resistant to progressive collapse.
- In models with C20 concrete strength, similar to the C15 concrete strength model, the structure partially collapsed in the external corner column removal scenario, whereas it demonstrated resistance to progressive collapse in the internal corner column removal scenario.
- Among the models that did not collapse, a comparison between those with C15 and C20 concrete strengths revealed that displacements and rotations in the structure decreased as concrete strength increased, as shown in Figs. 10, 13, and 14. Specifically, the model with C15 exhibited a maximum displacement of 3.65 cm and a plastic rotation of 0.0088 rad. In contrast, the model with C20 showed a maximum displacement of 3.08 cm and a plastic rotation of 0.0036 rad.
- The findings of this study have important implications for engineering practice. To mitigate collapse risks, engineers should prioritize the use of higher concrete strengths in structures with cantilevers, particularly in scenarios where external corner columns are removed.

By implementing these strategies, engineers can improve the structural integrity of buildings and reduce the likelihood of progressive collapse, ultimately enhancing safety in seismic-prone and high-risk areas.

Acknowledgements

We gratefully acknowledge the support provided by the Bursa Uludağ University's BAP project number FGA-2022-714. Additionally, we express our deep appreciation to Senior Structural Engineer Ayman ElFouly at Applied Science International, LLC (ASI) for his valuable contributions

Conflict of interests

The author(s) declared no potential conflicts of interest with respect to the research, authorship, and/or publication of this article.

Funding

This research received no external funding.

Data availability statement

Data generated during the current study are available from the corresponding author upon reasonable request.

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