SAKARYA ÜNİVERSİTESİ
FEN BİLİMLERİ ENSTİTÜSÜ DERGİSİ

Sakarya University Journal of Science
SAUJS

ISSN 1301-4048 | e-ISSN 2147-835X | Period Bimonthly | Founded: 1997 | Publisher Sakarya University | http://www.saujs.sakarya.edu.tr/

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Recieved: 4.05.2023

Accepted: 26.09.2023

Article Type: Research Article

Volume: 27
Issue: 6
Month: December
Year: 2023
Pages: 1322-1336

How to cite
Munyaradzi GONDOBWE, Aydın DEMİR; (2023), Progressive Collapse Evaluation of a Reinforced Concrete High-rise Building Designed According to Turkish Earthquake Code. Sakarya University Journal of Science, 27(6), 1322-1336, DOI: 10.16984/saufenbilder.1292075
Access link
https://dergipark.org.tr/en/pub/saufenbilder/issue/80994/1292075

Sakarya University Journal of Science 27(6), 1322-1336, 2023

Progressive Collapse Evaluation of a Reinforced Concrete High-rise Building Designed According to Turkish Earthquake Code

Munyaradzi GONDOBWE[1], Aydın DEMİR[*1]

Abstract

In this study, a numerical progressive collapse response evaluation was performed on a 40-story high-rise building designed according to the Turkish Earthquake Code of 2018 (TEC-2018). The alternate path method specified in the General Services Administration of 2016 (GSA-2016) and the Unified Facilities Criteria (UFC 4-023-03) was used for the evaluation. A total of 18 scenarios were investigated for column and shear wall removals. In the cases where hinges were observed, the extent of damage was evaluated using the performance criteria given in the Turkish Seismic Code and the American Society of Civil Engineers (ASCE) 41-17. In this study, the most damage was observed when the corner column was removed at the bottom floor, while the least damage was observed on the shear walls close to the center and core of the building. For all the scenarios evaluated, no collapse was observed. The study deduced that the high-rise building designed according to TEC-2018 showed sufficient resistance to progressive collapse.

Keywords: Progressive collapse, nonlinear dynamic analysis, reinforced concrete, Turkish Earthquake Code

1. INTRODUCTION

The progressive collapse response of buildings became a subject of research after the partial collapse of the Ronan Point apartment building in 1968. The building collapsed due to a minor gas explosion, but the blast caused severe damage to the building. Since then, progressive collapse (PC) has started appearing in building codes and regulations, with one of the earliest being PC in British Standards in 1970.

Progressive collapse can be defined as the spread of failure from member failure to the partial or complete failure of the entire structure. In essence, the loss of the member leads to damage that is significantly greater than the initial damage caused by the failure of the member [1].

Progressive collapse can be caused by abnormal loads from vehicle and aircraft collisions, construction errors, gas explosions, etc. The probability of a building experiencing these abnormal load conditions

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is very low; therefore, it would not make economic sense to design structures to resist each of these events. However, it is more reasonable to design the structure to resist progressive collapse [2].

Starosesek proposed six types of progressive collapse mechanisms [3]. The first type proposed is the pancake-type collapse, where the building collapses in a rigid vertical motion. In this type of collapse, the damage is due to the impact loads of the top floors crushing on the lower floors. The second type is the zipper-type collapse, which is common in cable-stayed bridges and is the result of the failure of one of the cables. The third type is a domino-type mechanism where unbraced members fall onto other unbraced members in a lateral direction, leading to the progression of damage throughout the whole structure.

This is common in scaffold towers, which have a high potential for overturning and are usually arranged in a uniform pattern horizontally. The fourth type is the section-type mechanism, where damage occurs at the section level, leading to the spread of damage across the entire member and eventually to the rest of the building. The fifth type is the instability-type collapse, where there is the failure of small stabilizing elements like braces and stiffeners, which leads to instability of the member and eventual collapse of the building. The sixth type is the mixed-type collapse, where it is not possible to classify the mechanism in any one of the mechanisms mentioned, but rather characteristics of two or more of the mechanisms are observed in the failure mechanism.

It has been shown in the literature that buildings designed to resist seismic loads according to ASCE 41-17 generally have the capacity to resist progressive collapse [2]. This is a general observation, and it does not eliminate the need for progressive collapse assessment, especially for buildings designed according to other seismic codes that are not ASCE 41-17. Since seismic design considers mainly local failure and does not explicitly address the potential for global failure, an additional assessment is needed specifically for progressive collapse mitigation. In line with this objective, two main guidelines have been published to enable engineers to design or evaluate structures for progressive collapse.

These guidelines are the GSA-2016, which was developed by the US General Service Administration, and the UFC 4-023-03, which was developed by the US Department of Defense. These guidelines suggest various methods to approach the PC of buildings, and Marjanishvili and Agnew conducted a study to evaluate the pros and cons of the given methods [4]. The guidelines, however, recommend the alternate path method, which is the most commonly used method in the literature. This method basically ensures that after the removal of a structural element, the building does not collapse, but rather the structural system redistributes the load using a new load path. This was the method used in this study.

Other building codes, including ASCE 7-05, British Standards (BS-8110), and Eurocodes (EN 1991-1-7), also address the issue of progressive collapse and give some recommendations on the progressive collapse response of buildings [5].

When the progressive collapse performances of mid-rise buildings designed in accordance with ASCE 41/17 were examined by removing a single element with the help of a numerical program, it was observed that no damage that would pose a problem occurred in the structure [6-8]. Not many studies have been done on the progressive collapse response of tall buildings. Marchis performed a numerical assessment of the progressive collapse resistance of RC frames with respect to the number of stories. This study and many other studies found that the PC response of buildings improves with the increase in the number of stories [2]. This is mainly because taller buildings have more structural members that can provide an alternative load path. Ren and Lin performed a PC assessment on a typical high-rise building with shear walls. They found that the PC response of buildings with strong walls and weak frame systems was generally inadequate compared to buildings with strong frames and weak wall systems [9].

A study was done on the progressive collapse robustness of high-rise buildings [10]. The alternative path method was implemented using nonlinear time history analysis on a model designed using Chinese building codes. A parametric study was done investigating factors like section size, inter-module connection, and bracing systems. The study found that bracing systems can enhance structural robustness by sharing redistributed loads and restraining structural members. This study also found that corner members are more critical because there are fewer adjacent members to share the redistributed load [10]. Another study explored a new system to design super-tall seismic-progressive collapse-resilient buildings. A 48-story building with a total height of 192 meters, designed according to Chinese building codes, was used. The system proposed was based on the common "frame-truss-core tube" system. The proposed system mainly contained a vibration reduction substructure, seismic-progressive collapse resilient composite frames, a composite braced tube with self-centering energy dissipation braces, and truss systems with outriggers and belt trusses. The proposed system effectively controlled response against column loss and earthquakes [11].

Gamaniouk also performed a parametric study on structures with different structural systems and story heights that were designed according to ASCE 41-17. The results also showed that as the number of stories increased, fewer hinges formed on the members, and the progressive collapse response also improved [6].

Many studies have been conducted on PC worldwide. However, only a few studies have been done for buildings designed according to TEC-2018. Demir performed a progressive collapse evaluation for low-rise buildings designed for different occupancy classes. The study showed that government buildings have better performance than residential buildings since they are generally designed to be more robust [12].

There are a few studies in the literature about the progressive collapse robustness of buildings designed according to Turkish earthquake code of 2018 (TEC-2018). Demir evaluated the progressive collapse response for buildings designed according to the Turkish earthquake code. The study suggested that there is a need for collapse response evaluation for buildings designed according to the Turkish earthquake code [13]. All the studies made by Demir [12-14] were performed on low- and mid-rise buildings. To the best knowledge of the authors, no studies have been made on high-rise buildings designed according to the TEC-2018.

In this study, a 40-story high-rise building designed according to TEC-2018 was numerically modeled using a finite element model in SAP 2000. The nonlinear dynamic alternate path method specified in the GSA-2016 was used to evaluate the progressive collapse response of the building. By performing 18 case studies in the finite element program, a preliminary study on the progressive collapse behavior of high-rise buildings designed according to the requirements of the Turkish earthquake code is added to the literature. Based on the results obtained from the analyses, a preliminary assessment of the adequacy of the provisions of the Turkish earthquake code in mitigating the risk of progressive collapse in high-rise buildings is made, and recommendations and discussions are presented accordingly.

2. PROPERTIES OF THE DESIGNED BUILDING

A symmetric 40-story existing building was designed according to TEC-2018 and the Requirements for Design and Construction of Reinforced Concrete Structures by the Turkish Standards Institute (TS-500) [15]. SAP2000, finite element software specialized for structural design, was used for the design of the structure.

The story height is 3.20 meters for the normal floors and 3 meters for the roof floor. This makes the total height of the building 131 m. Plan and elevation views of the building are given in Figures 1-2, respectively.

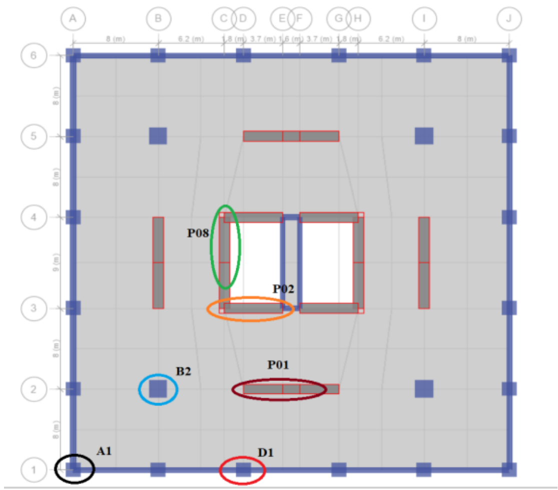

Figure 1 Plan view of the building

The building's occupancy class was 'residential'. The building was assumed to be located on ZC soil class. The loads imposed on the building were calculated according to the Design Loads for Buildings published by the Turkish Standards Institute in 1997 (TS-498) [16]. The design parameters used for the design and assigned loads are summarized in Tables 1 and 2, respectively. Since the study was done on an existing building, expected strengths were used during material definitions. C50 concrete was defined as 65 MPa of compressive strength (modulus of elasticity is 37000 MPa), while B420C rebar was defined as 504 MPa of yield stress.

The structural system for this building was a flat plate system with perimeter beams. All the columns have square sections. The column sections were optimized along the height of the building. This was done by dividing the tower into five groups, each with eight floors. The column sections and arrangements are the same for all floors in the same group.

Figure 2 Elevation view of the building

Table 1 Design parameters for the building

Parameter	Value
Soil class	ZC
Soil shear velocity ($V_{s,30}$)	500 m/s
Building usage class	3
Building height class	1
Seismic design class	1
Ductility level	High
Ground Motion Level	DD-2
Short period ($\mathbf{S_{DS}}$)	1.466
1s period ($\mathbf{S_{D1}}$)	0.492
Response modification (R)	8
Overstrength coefficient (D)	2.5

The column sections are reduced in size for each group along the height of the building. All perimeter beams have the same section (50x70cm) but different reinforcement according to the design. Table 2 shows the loads that were assigned to the model. Cross-sectional dimensions of structural members are given in Table 3. The other building details, such as the sections' reinforcing configuration, etc., are provided in the reference of [17]. Figure 3 shows the group labels for the model.

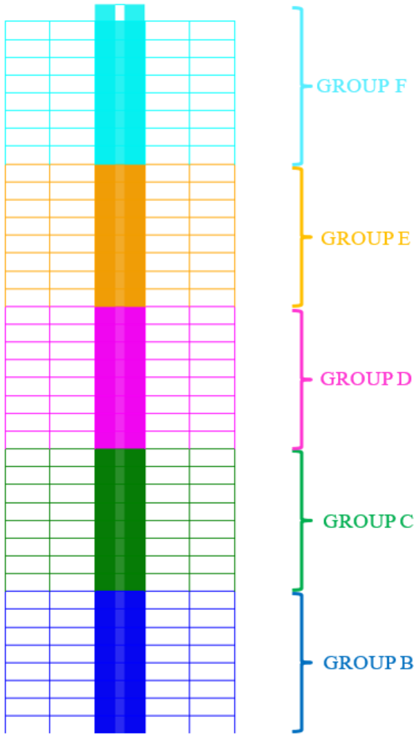

Figure 3 Group assignment along the height of the building

For the Progressive Collapse analysis, GSA-2016 and UFC 4-023-03 standards were used. The GSA-2016 and UFC 4-023-03 standards outline design guidelines to reduce the likelihood of PC for both new and existing governmental and military buildings that can be vulnerable to localized structural damage due to unpredictable extreme events. By offering a strong and well-balanced structural framework, they hope to stop the original damage from spreading. All three-story or higher government buildings must adhere to these guidelines. Without expressly taking the first event's cause into account, the guidelines employ a threat-independent approach. In UFC 4-023-03 [18], the three direct design approaches are alternate path, enhanced local resistance, and tie forces. The Alternate Path (AP) design technique is the only one utilized by GSA-2016 [19]. Both standards use the AP technique, and vertical structural components are removed conceptually and separately at precise plan and elevation positions. It is also necessary for the building to be able to bridge over the deleted portion.

Table 2 Assigned loads

Loads		
Self-weight of concrete	:	25.0 kN/m^3
Ceiling plaster + Floor finishings + Installments	:	2.0 kN/m^2
Infill walls for residential floors	:	1.0 kN/m^2
Residential floor live load	:	2.0 kN/m^2
Snow load	:	1.0 kN/m^2

Furthermore, the AP technique employs three analytical methods: linear static (LS), nonlinear static (NS), and nonlinear dynamic (ND). While the LS and NS approaches have certain geometric irregularity constraints, ND may be employed for buildings with irregularity. Prior to doing the study, the primary and secondary components of the building must be identified. The acceptance requirements for primary members are determined following the guidelines' prescribed prescriptions, which typically adhere to the acceptance criteria described in ASCE/SEI 41-13 [20].

Following that, the placements of the removed load-bearing elements are identified as indicated in the guidelines: external columns at the building's corner, around the center of the short and long sides, and certain interior columns. Furthermore, columns where the building plan geometry changes considerably must be deleted. Finally, the building's structural components must not exceed the computed acceptability standards. If the study forecasts that these acceptance criteria will not be met, the building will not fulfill the progressive collapse standards and must be redesigned or altered to remove the nonconforming parts. More information can be found in the applicable standards.

Table 3 Cross-sectional dimensions of structural members

	Group B	Group C	Group D	Group E	Group F
Perimeter Columns	C140X140	C120X120	C100X100	C80X80	C70X70
Interior Columns	C170X170	C150X150	C130X130	C110X110	C90X90
Perimeter Beams			B50X70		
Secondary Beams			B40X60		
Coupling Beams			B50X80		
Structural Walls	100cm	90cm	80cm	70cm	60cm
Tower Slab			30cm		
Core Wall Slab			15cm		

3. NUMERICAL MODELING OF THE BUILDINGS FOR PROGRESSIVE COLLAPSE ANALYSIS

The nonlinear dynamic analysis technique of the Alternate Path direct design methodology according to both UFC 4-023-03 [18] and GSA-2016 [19] criteria was employed for the progressive collapse evaluation of the designed building. SAP2000, a static and dynamic FE analysis program for structures, was used to generate a three-dimensional finite element (FE) model (Figure 4). Frame components were used to model the beams and columns. The columns and beams were modeled in the section designer menu in SAP2000. Slabs were modeled as shell-thin elements. Shear walls, on the other hand, were modeled as layered shell elements so as to more accurately represent the reinforcement. The loads were applied directly to the slabs as uniform area loads.

To predict the post-yield inelastic behavior of structural load-bearing components, two modeling techniques are used: concentrated (lumped) and distributed plasticity models (Figure 5). The lumped plasticity model assumes that deformation above the elastic limit happens only in discrete areas, with the remaining part of the member remaining elastic. Integrating plastic strain and curvature occurring in a predetermined hinge length will give the inelastic behavior [22].

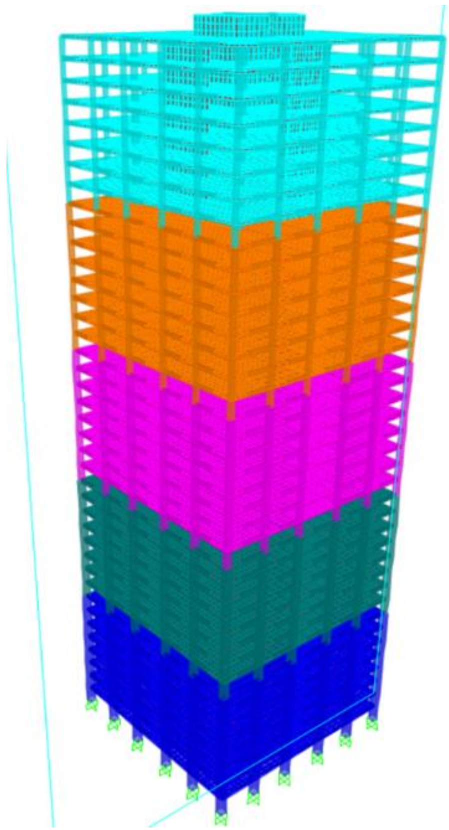

Figure 4 3D finite element model of the building

Nonetheless, in a distributed plasticity model, the cross-section of a member is discretized into a sequence of representative axial fibers that extend throughout the element or together with a limited length hinge zone (fiber hinge) (Figures 5c, 5d, and 5e). Each fiber must have its own stress-strain relationship. Finally, the axial force-deformation and biaxial moment-rotation correlations are derived by integrating the behavior throughout the section and multiplying by the hinge length [21, 23].

RC frame structures may withstand excessive collapse by creating two crucial load-resisting processes on their members: Vierendeel (arching) Action and Catenary Action. While fiber components can successfully capture such mechanisms in addition to the flexural behavior of frame members, the usual concentrated plastic hinge technique ignores those aspects [21]. As a result, the nonlinear behavior of the structural load-bearing components was simulated in this study utilizing nonlinear fiber hinges (Figure 5.c), one of the approaches of the distributed plasticity approach.

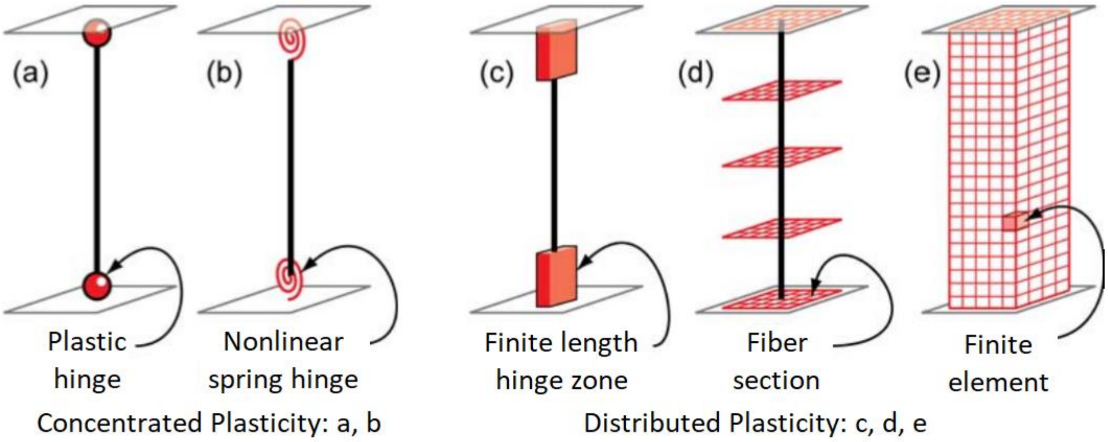

Figure 5 Beam-column nonlinear modeling approaches [21]

To incorporate fiber sections in SAP2000, the sections were initially discretized using an ideal fiber arrangement. The program automatically allocated fibers to the center of each reinforcement, as well as confined and unconfined concrete that was meshed into many square or rectangular sections. The appropriate fibers were afterward assigned the material characteristics of both concrete and reinforcing bars. Finally, in a finite-length hinge zone, the fiber hinges were specified as half the section depth at both ends of the beams and columns. Furthermore, concrete and reinforcing steel constitutive material models were built using the material models provided in TEC-2018 [24].

First, a nonlinear static analysis scenario for gravity loads was created using Eq. 1 from UFC 4-023-03 [18]. In the equation, DL refers to dead load, LL stands for live load, and S represents snow load. That load example was utilized to calculate the forces existing in each deleted column at equilibrium. It was also taken into account as the starting condition for the column removal analysis instance.

$$1.2DL + 0.5LL + 0.2S \tag{1}$$

Column removal was executed in SAP2000 following the procedure prescribed by Demir [13]. The column was removed on the 0.5th second after the gravity analysis achieved equilibrium. The total time of the removal was set to 3 seconds. For the nonlinear dynamic time-history load situation, the direct integration solution approach was applied. Depending on the first and second periods of the building, a Rayleigh damping of 2.5% was established. For the study, the Newmark time integration approach was used, using gamma and beta factors of 0.50 and 0.25, respectively. Finally, the P-delta and large displacement options in the software were enabled to account for the geometric nonlinearity of the members and catenary behavior on the surrounding beams due to column removal.

According to UFC 4-023-03, for shear wall removal, if the wall length is less than 2H, the entire shear wall can be removed. If the wall length is greater than 2H, then the shear wall length to be removed should be 2H. H is the story height [18]. For shear wall removal, the section of the wall to be removed was assigned to a group. This group was removed at 0.5 s after the model had achieved equilibrium following the nonlinear static case.

Both column and wall removal were performed in this study. Three different columns were removed at three different heights: ground floor, middle floor, and top floor, as prescribed by the GSA-2016. Three shear walls were also removed at the three building heights. In total, 18 removal scenarios were investigated.

Acceptance criteria were calculated according to both TEC-2018 and ASCE-41, depending on the plastic rotations observed at the hinges. The three performance levels prescribed are immediate occupancy (IO), life safety (LS), and collapse prevention (CP).

4. RESULTS AND DISCUSSION

The maximum vertical displacements and the residual vertical displacements of the nodes above the removed members for the 18 scenarios are summarized in Table 4. Figures 6-7 show a visual representation of the model before and after column removal. The removal of the edge column from the ground floor was used as an example. The shear wall removal scenario model illustration is shown in Figures 8–9. Moreover, the following naming convention was used: CR—Column Removal, WR—Wall Removal, GF—Ground Floor, MF—Middle Floor, and TF—Top Floor.

Of the 18 damage scenarios, only two could be considered significant in terms of overall damage. Therefore, the fiber hinge plastic damage results of these two damage scenarios are shown in Figure 10-11. Since the damage remains at an elastic level starting from the 8th floor and going up, fiber hinge damage results are shown up to the 11th floor.

The vertical displacement time history plots for the column and shear wall removal scenarios are shown in Figures 12–25.

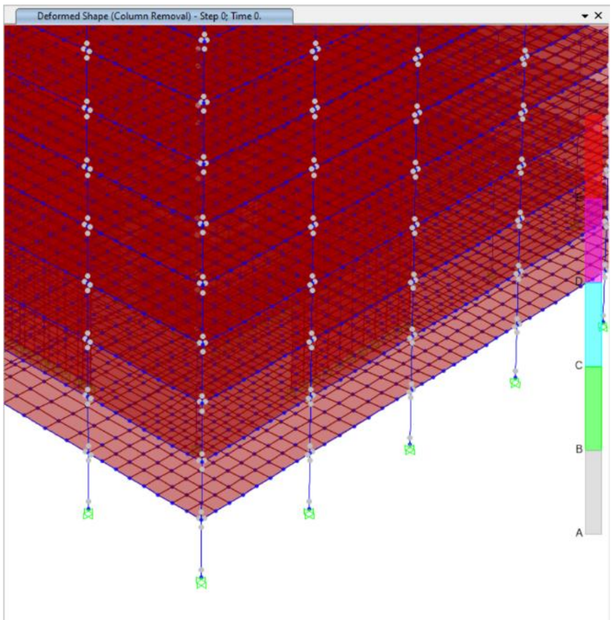

Figure 6 3D illustration of column removal scenario before removal for scenario CR-GF-Edge

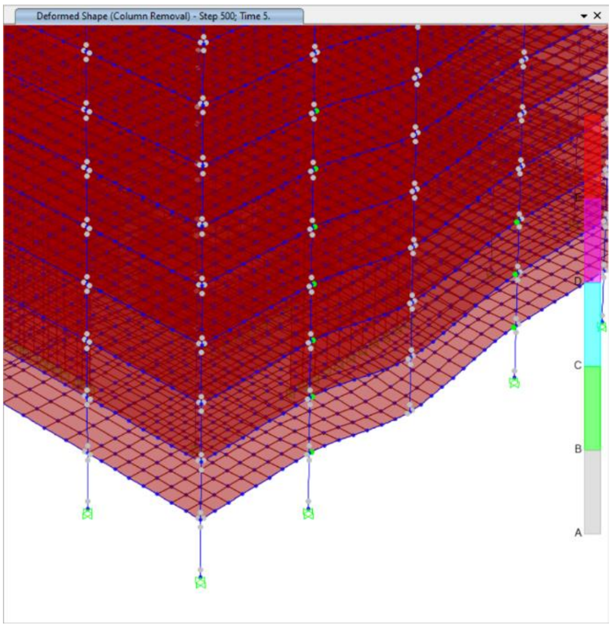

Figure 7 3D illustration of column removal scenario after removal for scenario CR-GF-Edge

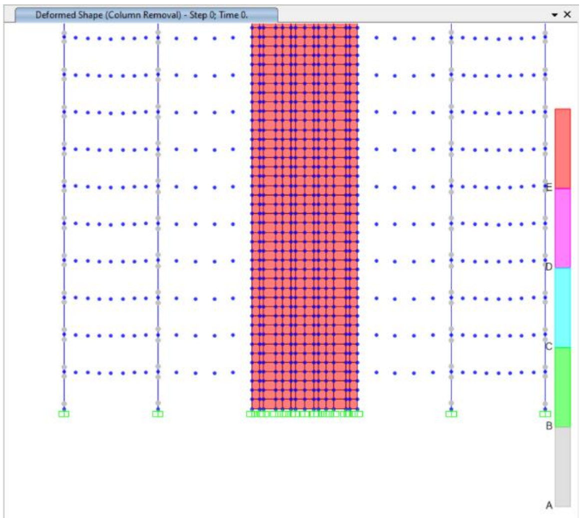

Figure 8 Illustration of wall removal scenario before removal

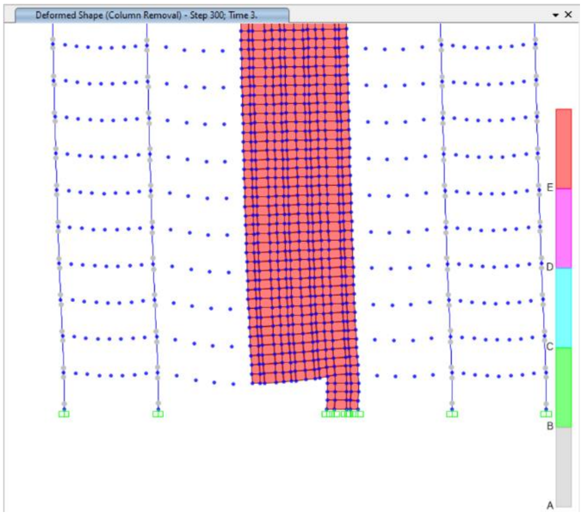

Figure 9 Illustration of wall removal scenario after removal

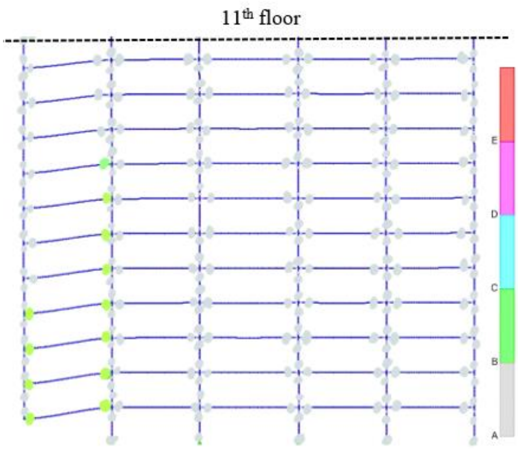

Figure 10 Fiber hinge damage results for removal for CR-GR-Corner scenario*
*Gray: no damage, green: limited damage, cyan: moderate damage, pink: severe damage, red: failure

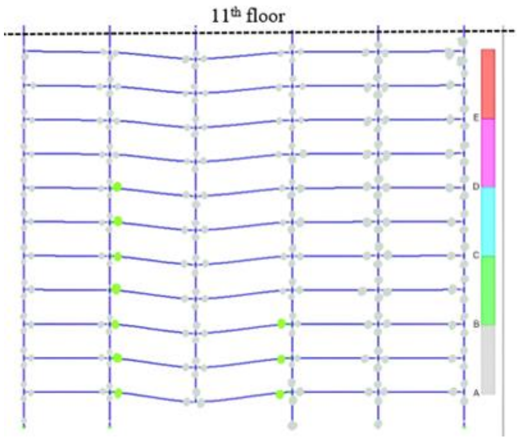

Figure 11 Fiber hinge damage results for removal for CR-GR-Edge scenario*
*Gray: no damage, green: limited damage, cyan: moderate damage, pink: severe damage, red: failure

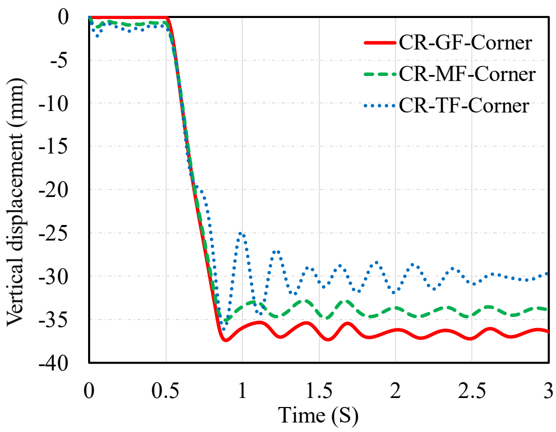

Figure 12 Vertical node displacements for removal of corner column (A1) for ground, middle and top floor

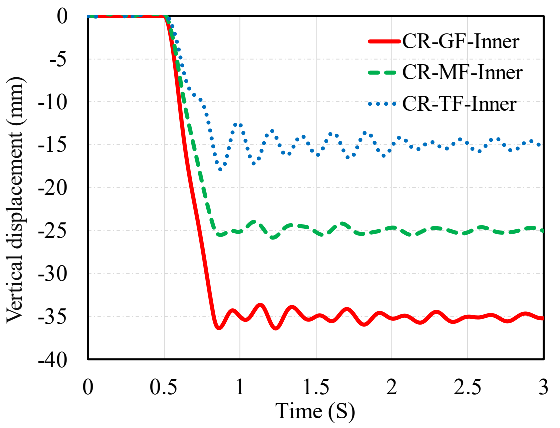

Figure 13 Vertical node displacements for removal of inner column (B2) for ground, middle and top floor

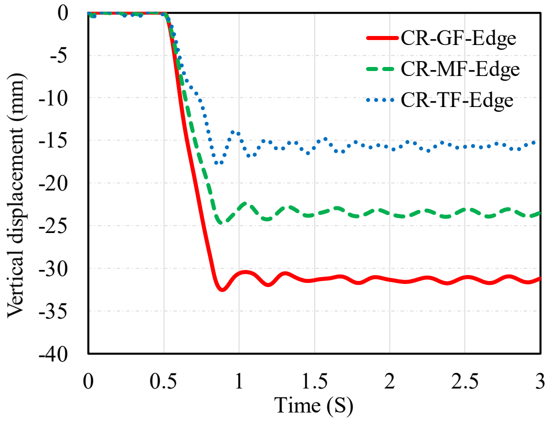

Figure 14 Vertical node displacements for removal of edge column (D1) for ground, middle and top floor

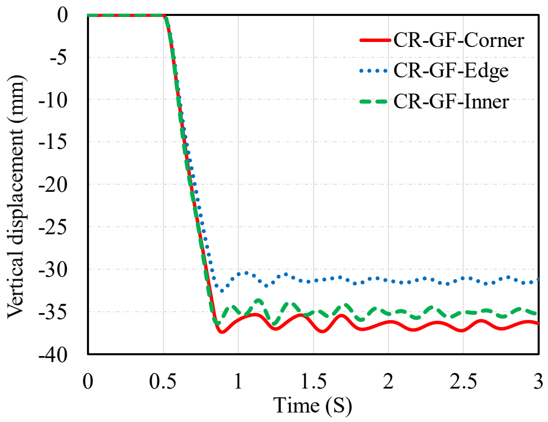

Figure 15 Vertical node displacements for removal of corner (A1), edge (D1) and inner column (B2) on ground floor

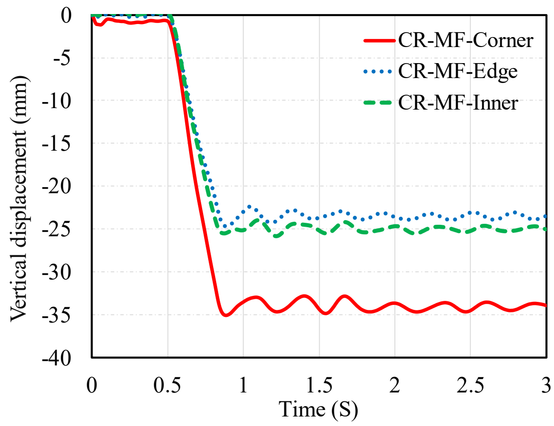

Figure 16 Vertical node displacements for removal of corner (A1), edge (D1) and inner column (B2) on middle floor

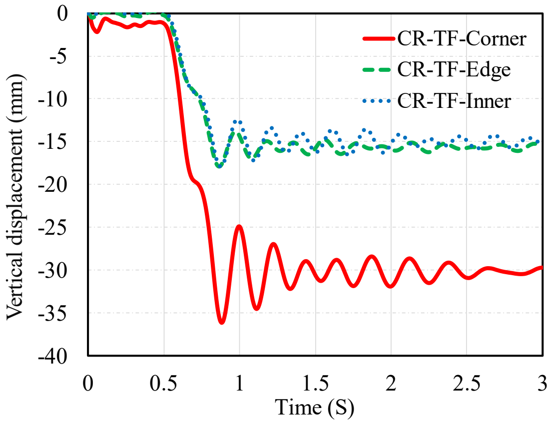

Figure 17 Vertical node displacements for removal of corner (A1), edge (D1) and inner column (B2) on top floor

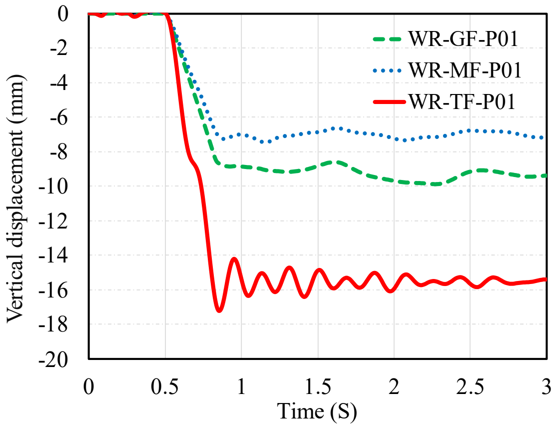

Figure 18 Vertical node displacements for removal of P01 wall for ground, middle and top floor

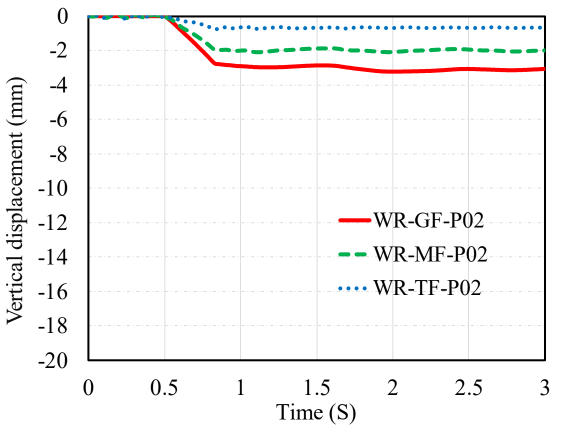

Figure 19 Vertical node displacements for removal of P02 wall for ground, middle and top floor

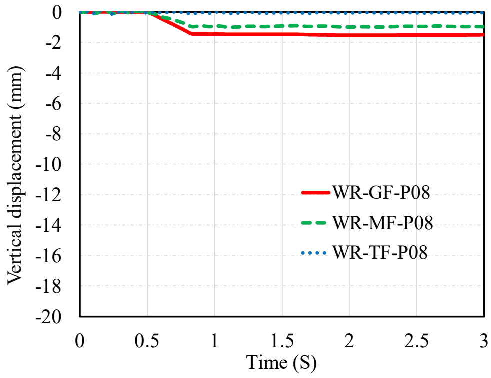

Figure 20 Vertical node displacements for removal of P08 wall for ground, middle and top floor

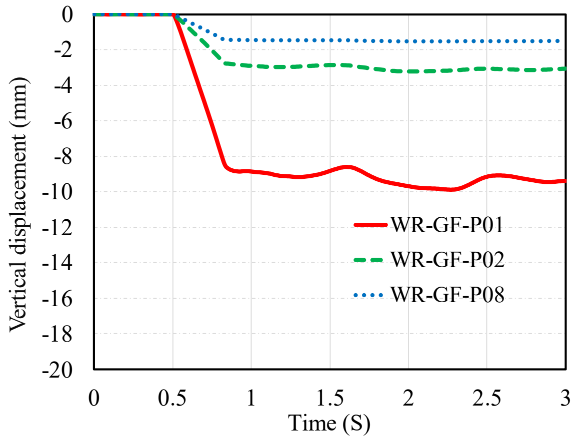

Figure 21 Vertical node displacements for wall (P01, P02, P08) Removal on ground floor

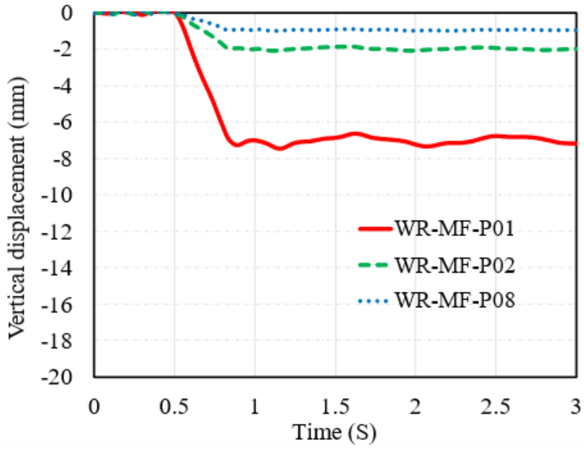

Figure 22 Vertical node displacements for wall (P01, P02, P08) Removal on middle floor

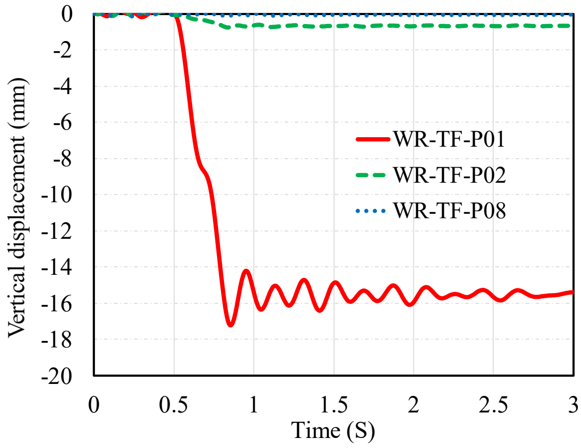

Figure 23 Vertical node displacements for wall (P01, P02, P08) Removal on top floor

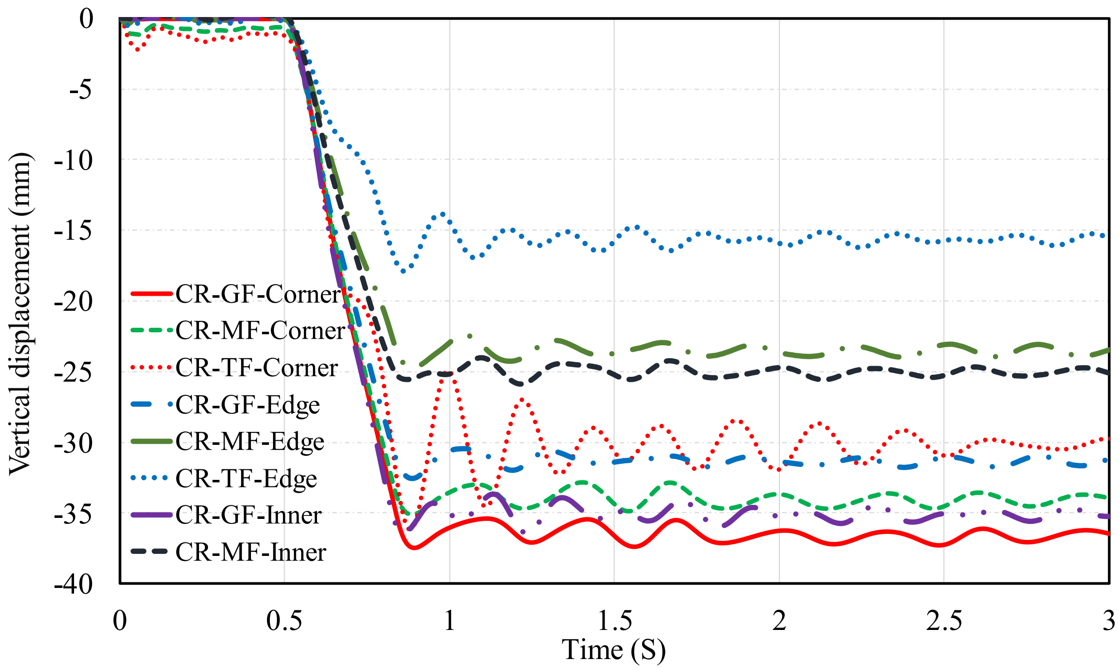

Figure 24 Vertical node displacement for the column removal scenarios

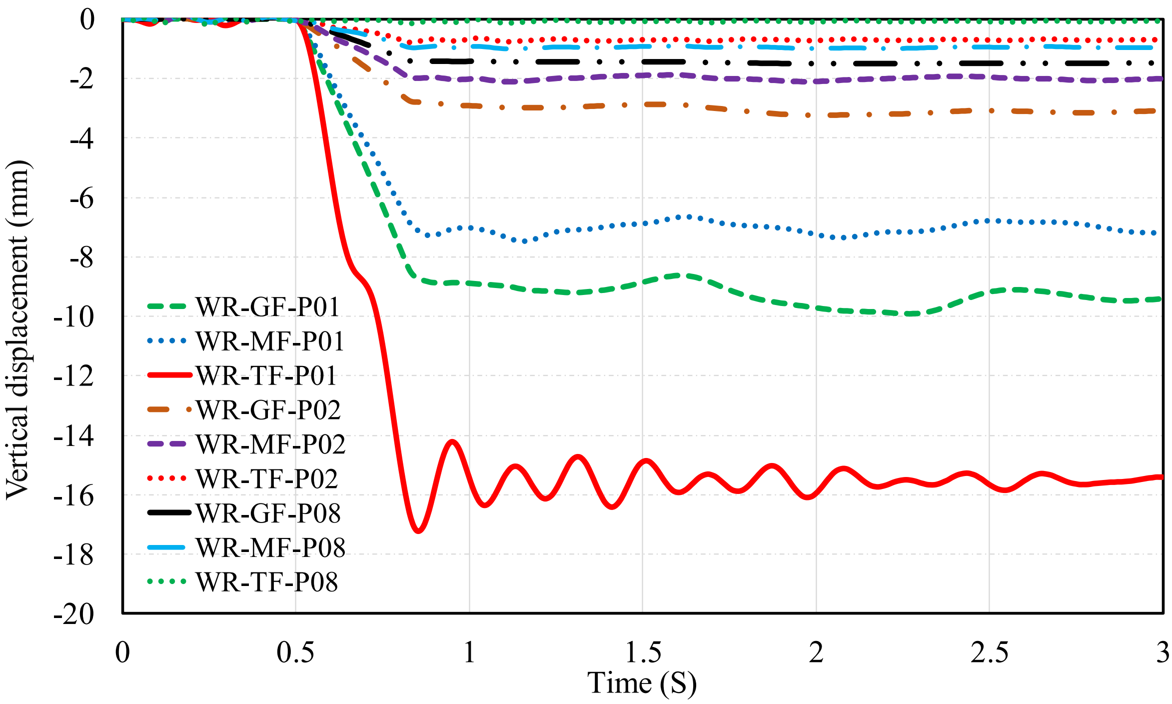

Figure 25 Vertical node displacements for all the shear wall removal scenarios

Table 4 Results of the maximum and residual displacement of the node above the removed member

Scenario	Maximum Displacement (mm)	Residual Displacement (mm)	Damage State
CR-GF-Corner	-37.47144	-36.5	Minimum Damage
CR-GF-Edge	-32.5236	-31.3	Minimum Damage
CR-GF-Inner	-36.38	-35	No Damage
CR-MF-Corner	-35.1	-34	No Damage
CR-MF-Edge	-24.73	-23.5	No Damage
CR-MF-Inner	-25.85	-25	No Damage
CR-TF-Corner	-36.18	-30	No Damage
CR-TF-Edge	-17.89	-15.7	No Damage
CR-TF-Inner	-17.9	-15	No Damage
WR-GF-P01	-9.9117	-9	No Damage
WR-GF-P02	-3.2166	-3	No Damage
WR-GF-P08	-1.5073	-1.5	No Damage
WR-MF-P01	-7.47	-6.5	No Damage
WR-MF-P02	-2.11	-2	No Damage
WR-MF-P08	-1.01	-0.95	No Damage
WR-TF-P01	-17.21	-15.5	No Damage
WR-TF-P02	-0.77	-0.65	No Damage
WR-TF-P08	-0.17	-0.08	No Damage

Hinges were formed for only 2 out of the 18 scenarios: the removal of the corner column (A1) at the ground floor and the edge column (D1) at the ground floor. The maximum displacement and residual displacement for the node above the corner column were -37.47mm and -36.5mm, respectively, and for the edge column, they were -32.52mm and -31.3mm, respectively.

For shear wall removal scenarios, the biggest displacement was observed when wall P01 was removed at the ground floor, with a maximum displacement of -9.9117 mm and a residual displacement of -9 mm. The least

displacement was observed when wall P08 was removed at the top floor, with a maximum displacement of -0.17 mm and a residual displacement of -0.08 mm.
The damage was assessed by comparing the plastic rotations at the hinges formed to the plastic hinge rotation values calculated according to TEC-2018 and ASCE-41, which give the rotation limits for the three performance levels IO, LS, and CP.

The greatest damage was observed on a perimeter beam in the CR-GF-Corner scenario. The plastic rotation on the beam was 0.02808 rads, which falls under the collapse prevention performance, which has a limit of 0.03446 rads for the given section, according to the TEC-2018. The beam also falls under the collapse prevention performance level according to ASCE 41-17, which has a rotation limit of 0.04606 rads for the given section.

The slightest damage was observed when the shear wall P08 was on the top floor. The maximum plastic rotation on the column near the wall was 0.000012 rads, which falls under the life safety performance level, with a rotation limit of 0.02756 rads according to the TEC-2018 and 0.02961 rads according to the ASCE 41-17.

The most extreme case observed in both cases was collapse prevention. No hinge experienced any rotation above the collapse prevention limit. No hinges were formed for the rest of the column removal scenarios or all the wall removal scenarios. No hinges were formed on the columns for all 18 scenarios.

The low levels of damage observed are consistent with the data found in the literature. As the number of stories increases, the progressive collapse response of the building improves. Another possible reason for the low damage levels could be the overdesign of the structure. For example, in the two cases where hinges were formed, the shear demand capacity ratios were also checked. The maximum demand observed was 32% of the shear capacity. Since the sections are generalized for each group, and the beam cross section is the same on all floors, the building might be overdesigned.

Since a flat plate system was used for this building, it is possible that damage could have occurred on the slabs. Estimating damage on slabs is a different issue that was not addressed in this study.

5. CONCLUSION AND SUGGESTIONS

In this study, the progressive collapse response of a tall building designed according to the Turkish earthquake code was investigated. A typical 40-story building was used. Using SAP 2000, the alternate path method prescribed in the UFC was implemented using the nonlinear dynamic procedure. A total of 18 scenarios were investigated for column and shear wall removal. After which, the displacement time histories of the node just above the removed member were plotted. In cases where hinges were formed, the plastic rotations at those hinges were compared to the rotation limits from both the ASCE 41 and the TEC-2018.

The building used for this study was a tall building designed according to TEC-2018, and it showed an acceptable progressive collapse response. The results show that the stresses that occur in the structural elements by the removal of columns and shear walls remain at a limited level in the structural elements and do not cause any distress to the structural integrity. However, it might be misleading to conclude that all tall buildings designed according to TEC-2018 have a good progressive collapse response. The results in this study apply only to the building considered. The same study has to be done for different tall buildings with different structural systems before a definitive conclusion can be reached. In addition to numerical studies, experimental studies are also necessary before it can be established without a shadow of a doubt that tall buildings designed according to TEC-2018 have a good progressive collapse response.

Funding
The author has not received any financial support for the research, authorship, or publication of this study.

Authors' Contribution
The authors contributed equally to the study.

The Declaration of Conflict of Interest/ Common Interest
No conflict of interest or common interest has been declared by the authors.

The Declaration of Ethics Committee Approval
This study does not require ethics committee permission or any special permission.

The Declaration of Research and Publication Ethics
The authors of the paper declare that they comply with the scientific, ethical and quotation rules of SAUJS in all processes of the paper and that they do not make any falsification on the data collected. In addition, they declare that Sakarya University Journal of Science and its editorial board have no responsibility for any ethical violations that may be encountered and that this study has not been evaluated in any academic publication environment other than Sakarya University Journal of Science.

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