

## Article

# Seismic Response of a Damaged Multi-Story RC Building During the 6 February 2023 Kahramanmaraş Earthquakes

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## Abstract

Post-earthquake field investigations provide significant contributions to earthquake-resistant building design. Especially after devastating earthquakes, determining the damage levels in buildings is an important issue for structural engineering. The Kahramanmaraş earthquake, which occurred on 6 February 2023, had a devastating impact over a wide area. Numerous reinforced concrete buildings collapsed or were severely damaged. Significant damage was observed not only in older buildings but also in newer ones. Even buildings that were yet to be in service suffered significant damage. This study examines the structural damage of a newly constructed and severely damaged after the 2023 Kahramanmaraş earthquakes 13-story reinforced concrete building in Malatya province. The damage caused by the earthquake was investigated through field observations. Concrete core samples were taken from the damaged building, and reinforcement checks were performed using a rebar scanner. Nonlinear time-history analyses were conducted using the measured data. The results of the numerical analyses were compared with site observations of earthquake-induced damage. The results obtained from the nonlinear analyses appear to be consistent with the field observations.

**Keywords:** field investigation; Kahramanmaraş earthquakes; pounding effect; reinforced concrete buildings; structural damages

## 1. Introduction

On 6 February 2023, two devastating earthquakes occurred along the East Anatolian Fault Zone in Türkiye. These two earthquakes, centered in Pazarcık (Mw: 7.7) and Elbistan (7.6), occurred on the same day and close to each other (distance between epicenters is approximately 90 km). These earthquakes caused heavy damage or the collapse of many structures over a wide area. The official report published by Türkiye Disaster and Emergency Management Authority (DEMA) stated that more than 35,000 buildings had collapsed [1]. In addition, the official report published by the ministry stated that more than 200,000 buildings collapsed or were severely damaged [2]. The number of damaged buildings demonstrates the destructiveness of earthquakes.

It is of great importance to identify and evaluate the causes of damage in buildings damaged by earthquakes. Determining the causes of damage, especially in new buildings, makes a great contribution to earthquake-resistant building design. Many studies have been conducted on the causes of damage in reinforced concrete buildings after the 2023 Kahramanmaraş earthquakes. These studies categorize the causes of damage in reinforced



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concrete buildings under general headings. These studies emphasize poor material quality, poor soil conditions, inadequate transverse reinforcement, short column effects, weak and soft story effects, pounding effects, and strong beam-weak column effects. Vuran et al. (2025) examined the earthquake damage of reinforced concrete buildings and evaluated the buildings by taking into account the construction year before and after 2000 [3]. İnce (2024) evaluated the earthquake damage in reinforced concrete buildings in Adıyaman province after the Kahramanmaraş earthquakes, especially in terms of design and reinforcement details [4]. Doğan et al. examined the earthquake damages that occurred in reinforced concrete buildings in Hatay province in 2024 and compared whether a building complies with the 2018 Türkiye Building Earthquake Code (TBEC) regulations [5]. Altıok et al. (2024) investigated the seismic performance of reinforced and unreinforced concrete school buildings [6]. Işık et al. (2024) evaluated the damages in reinforced concrete buildings after the Kahramanmaraş earthquakes in terms of soil and structure [7]. Altunsu et al. (2024) investigated the damage mechanisms of reinforced concrete buildings in Hatay province [8]. İnce (2025) evaluated the performance of RC buildings with one-way joist slab after the Kahramanmaraş earthquake [9]. Özmen et al. (2025) investigated the structural damage of RC buildings, masonry buildings, precast and steel structures [10]. İnce et al. (2026) investigated the structural damage of RC structures after the 2023 Kahramanmaraş earthquakes [11]. Ozturk et al. (2023) investigated structural damages of school buildings [12]. Also, there are different studies that investigated structural damages in RC structures [13–15]. Many literature studies examined damages to different structural systems following the 2023 Kahramanmaraş earthquake. Studies on precast structures have generally highlighted non-ductile damage and inadequate connections [16–18]. Many studies have focused on the damage status of masonry and historical structures [19–23].

In addition to general post-earthquake field investigations, many studies analyze damage conditions of specifically designated buildings by numerical analysis. Kazaz et al. (2024) [24] studied the damage conditions of an 11-story reinforced concrete building in Malatya province. It was emphasized that the soft stories' irregularity and inadequate seismic shear wall ratio had an impact on the damage to the examined building [24]. Tura et al. (2024) [25] investigated the nonlinear behavior of a multi-story reinforced concrete building in Hatay Province. The building was reported to have been subjected to an earthquake exceeding its design earthquake. Transverse reinforcement defects were highlighted in the study [25]. Kazaz et al. (2024) assessed the damage mechanism of a 13-story building in Hatay Province using a numerical analysis [26]. Peker et al. (2025) analyzed a 14-story reinforced concrete building in Malatya province [27]. Tan et al. (2025) examined a damaged two-story reinforced concrete building and evaluated the effect of the infill wall by numerical analysis [28]. These studies examined the damage mechanisms of the buildings detected after the Kahramanmaraş earthquake in detail. Özüygür (2025) investigated the seismic performance of a slightly damaged RC building after the Kahramanmaraş earthquakes [29]. Tonyali (2025) investigated the seismic performance of a slightly damaged RC building with 6 stories (basement, ground and, 4 stories) after the Kahramanmaraş earthquakes in Hatay province [30]. Bassurucu (2025) investigated the seismic performance of a slightly damaged RC building with 8 stories after the Kahramanmaraş earthquakes in Adıyaman province [31]. Altıok (2025) investigated a collapsed retrofitted school building [32]. These studies examined the damage mechanisms of specific buildings after the Kahramanmaraş earthquake in detail.

This study examines the seismic performance of a 13-story building that was exposed to the 2023 Kahramanmaraş earthquake. The investigated reinforced concrete building has been designed taking into account TBEC-2018 (Türkiye Building Earthquake Code) and TS500 regulations [33,34]. The investigated building has a basement (3 m), ground

story (4.5 m) and 11 stories (3 m). The basement was surrounded by RC walls on all sides. The total building height above the basement is 37.5 m.  $S_{DS}$  (Design Spectral Acceleration Parameter at Short Periods) value is 0.977 for this building. Because the building is used for residential purposes, the building importance factor is 1. Seismic Design Category and Building height class are 1 and 4, respectively.

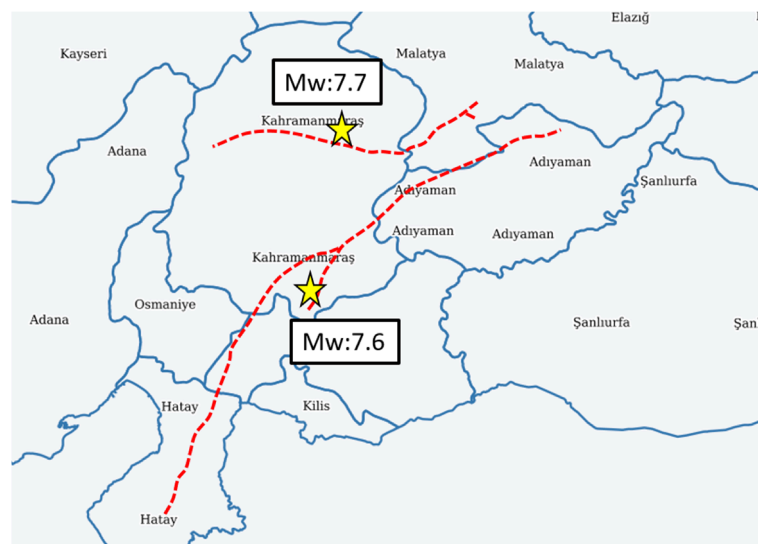
There are different structural irregularities and design deficiencies in the investigated multi-story RC building. The building's upper story height is 3 m, while the ground story height is designed to be 4.5 m. Furthermore, while the upper stories have infill walls between the frames, the amount of infill wall is decreased in the ground story. These factors create a risk of soft and weak story irregularities in the building. An examination of the building's structural elements reveals frame discontinuities at several points. Furthermore, an inadequate seismic gap distance between the ground story and the adjacent single-story building is observed. Concrete samples taken from the reinforced concrete building and on-site inspections indicate that the material quality and reinforcement details are generally consistent with the design. Assessing earthquake damage in such a structure is of remarkable importance from a structural engineering perspective.

Many studies in the literature have investigated soft and weak story irregularities, the pounding effect, and frame discontinuity. Increasing the height of the ground story and removing infill walls reduces its rigidity and increases displacements. These conditions, known as soft and weak stories, cause damage to be concentrated on these stories during an earthquake [35]. In reinforced concrete buildings, discontinuity of vertical load-bearing elements, in other words, frame discontinuity, negatively affects the seismic performance of the building [36]. The presence of insufficient seismic gap distance between reinforced concrete buildings causes these buildings to collide, and this situation is called the pounding effect. Especially when the story levels are different, the pounding effect causes destructive effects and high shear forces in the vertical load-bearing elements [37]. Shear walls are load-bearing elements with a long side at least six times the width of the short side. Due to their high rigidity, these elements resist the majority of horizontal loads during an earthquake [38–44]. In this study, shear walls were used for 1.3% and 0.8% of the plan area of the investigated building along the short and long sides, respectively.

The aim of this study is to investigate the structural damage of a newly constructed 13-story building in Malatya province after the Kahramanmaraş earthquake. For this purpose, firstly, the structural features of the examined building are given in detail. The earthquake damage that occurred in the building was marked on the structural plan. Important parameters such as the results of the concrete sample taken from the building are given within the scope of the study. Then, a nonlinear analysis of the reinforced concrete building was performed using the finite element method. The factors that are effective in damaging the reinforced concrete building have been evaluated. Additionally, within the scope of the study, the effects of the Kahramanmaraş earthquakes on the examined building were evaluated.

## 2. Characteristics of Kahramanmaraş Earthquakes

Pazarcık (Mw: 7.7) and Elbistan (Mw: 7.6) earthquakes occurred on the East Anatolian Fault on 6 February 2023. The first earthquake occurred in the Pazarcık district of Kahramanmaraş province at 04:17 local time. Then the second earthquake occurred in Elbistan, a district of Kahramanmaraş province, at 13:24 local time. The distance between the epicenters of these earthquakes was nearly 90 km. Figure 1 shows the epicenters and fault ruptures of the Kahramanmaraş earthquakes. Parameters of the Kahramanmaraş earthquakes were recorded by different institutions. Tables 1 and 2 show the parameters of the Pazarcık and Elbistan earthquakes, respectively.



**Figure 1.** Epicenters and fault rupture of Kahramanmaraş earthquakes.

**Table 1.** Seismic parameters of the Pazarcık earthquake reported by different institutions.

| Institution                                                                  | Mag. ( $M_w$ ) | Depth (km) | Long. (°) | Lat. (°) |
|------------------------------------------------------------------------------|----------------|------------|-----------|----------|
| Turkish Ministry of Interior Disaster and Emergency Management Agency (DEMA) | 7.7            | 8.6        | 37.04     | 37.28    |
| Kandilli Observatory and Earthquake Research Institute (KOERI)               | 7.7            | 5.0        | 37.12     | 37.11    |
| United States Geological Survey (USGS)                                       | 7.8            | 10.0       | 37.02     | 37.22    |

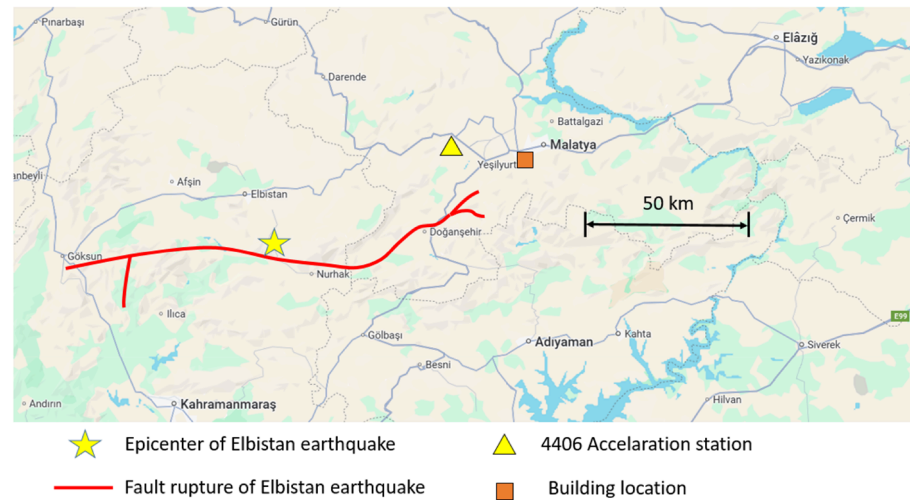
**Table 2.** Seismic parameters of the Elbistan earthquake reported by different institutions.

| Institution                                                                  | Mag. ( $M_w$ ) | Depth (km) | Long. (°) | Lat. (°) |
|------------------------------------------------------------------------------|----------------|------------|-----------|----------|
| Turkish Ministry of Interior Disaster and Emergency Management Agency (DEMA) | 7.6            | 7.0        | 37.24     | 38.09    |
| Kandilli Observatory and Earthquake Research Institute (KOERI)               | 7.6            | 5.5        | 37.21     | 38.07    |
| United States Geological Survey (USGS)                                       | 7.5            | 15.0       | 37.21     | 38.02    |

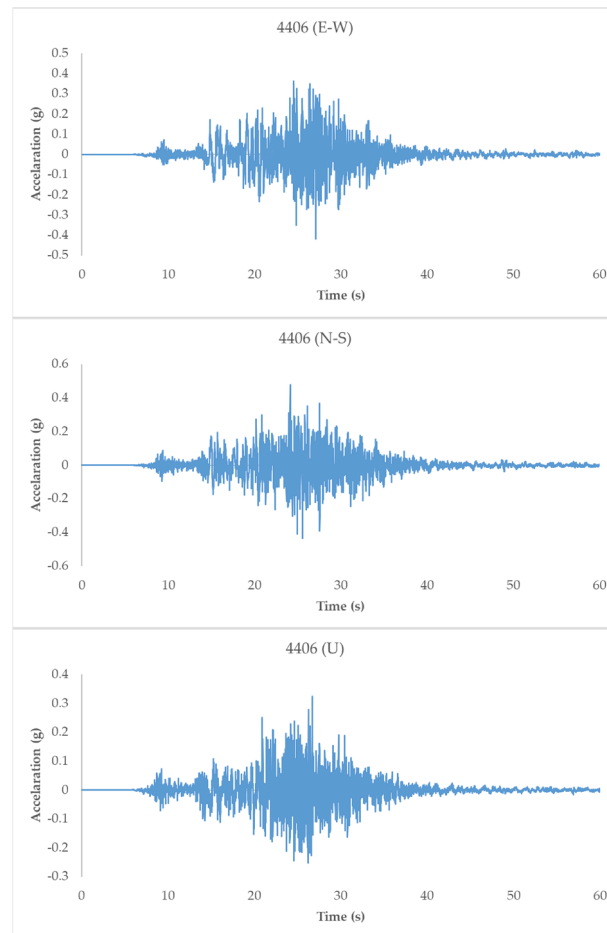
#### *Selection of the Earthquake Record for Time History Analysis*

The building investigated in this study is located in the Yeşilyurt district of Malatya province. Building location, fault rupture, and epicenter of the Elbistan earthquake are illustrated in Figure 2. When the acceleration records of the Pazarcık and Elbistan earthquakes are examined, it is seen that Malatya province was more affected by the Elbistan earthquake. Therefore, only the Elbistan earthquake record was used for the analysis of the building. The Elbistan earthquake was recorded by multiple acceleration stations. The closest acceleration station to the building is approximately 26 km away. The distance between the investigated building and the acceleration station to the fault rupture is approximately the same. Figure 3 shows the acceleration records of the 4406 acceleration

station for three directions. Also, Figure 4 shows the comparison of this station's records with the design earthquake in TBEC-2018.



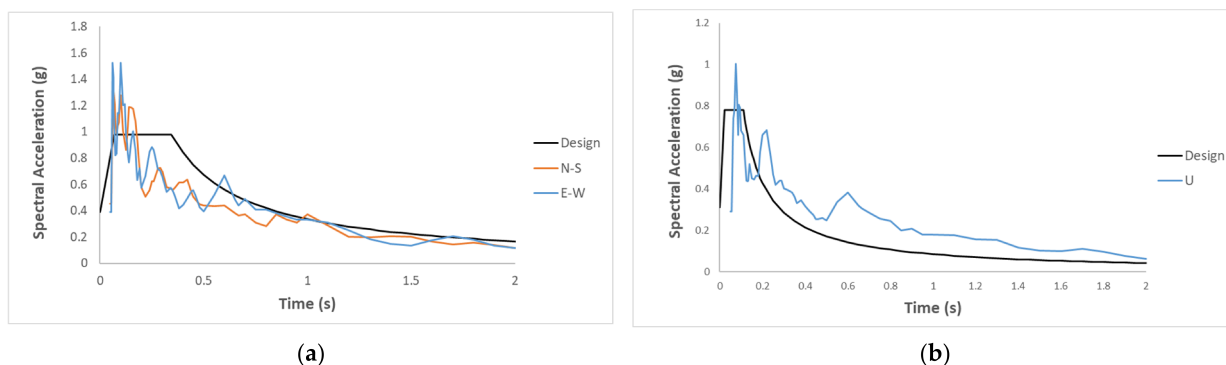
**Figure 2.** Location of the investigated building and acceleration station.



**Figure 3.** Acceleration records of the Elbistan earthquake recorded by the 4406 station.

According to the design report of the reinforced concrete building, the soil class is classified as ZC. The Peak Ground Acceleration (PGA) is 0.34 g for the design earthquake at the building's location, according to the Türkiye earthquake hazard map. Figure 4 presents a comparison of the design earthquake and the Elbistan earthquake, as recorded by Station 4406. No scaling procedure was applied to the acceleration records, and the recorded

ground motions were used in their original form to evaluate the actual seismic response and observed damage of the building.



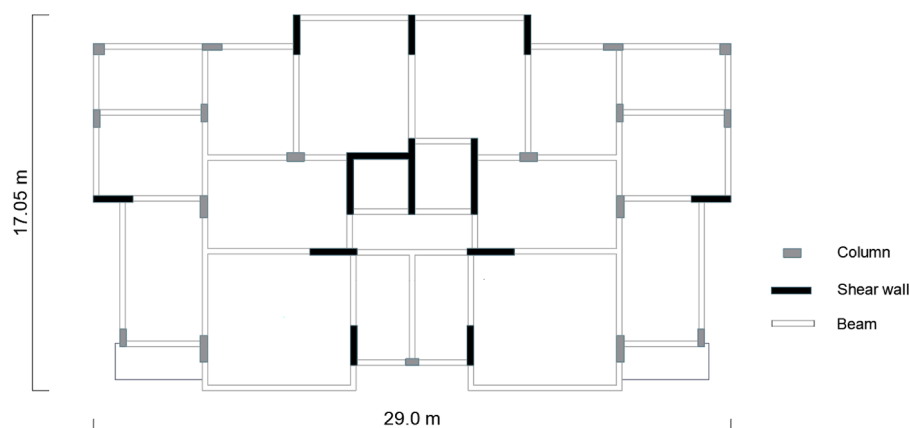
**Figure 4.** Comparison of design spectrum and 4406 records (a) horizontal (b) vertical.

### 3. Investigated RC Building

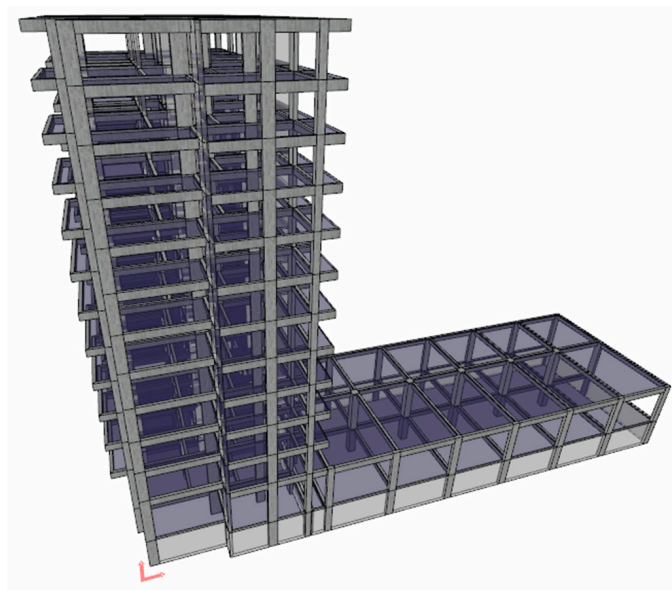
The investigated multi-story reinforced concrete building consists of 13 stories (basement + ground story + 11 stories). Also, there is a two-story building (basement and ground story) adjacent to the investigated building. The building has a 4.5 m ground story height and 3 m upper story height (Figure 5). At the time of the Kahramanmaraş earthquake, the construction of all stories of the building was completed. However, the building had not yet been put into service. The building was designed with shear walls in two directions. The structural plan of the building is given in Figure 6. Also, a 3D model of the investigated building is presented in Figure 7.



**Figure 5.** Investigated multi-story building.



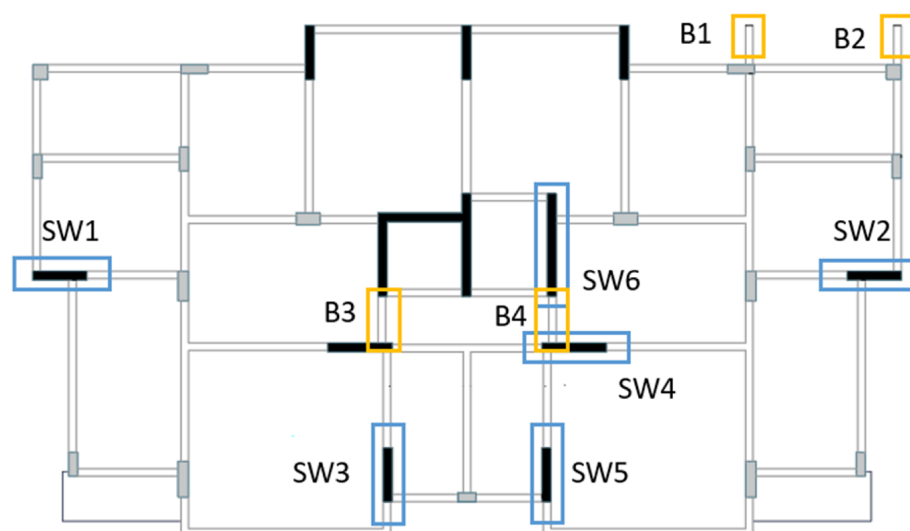
**Figure 6.** Plan of the building.



**Figure 7.** Three-dimensional model of the investigated building.

### 3.1. Observed Damage

The investigated building has severe damage to load-bearing elements. Figure 8 shows the ground floor plan and damage locations of the building. Crushing of the concrete cover and buckling of reinforcement were observed at the end zones of the six shear walls, as shown in Figure 9. Pounding occurred between the examined building and the adjacent building. Therefore, B1 and B2 beams have been damaged due to the pounding effect (Figure 10). The beam dimensions in the structural plan were designed as 25/60. However, the dimensions of beams B3 and B4 were taken as 25/45. Crushing of the concrete was observed in these beams located between the shear walls. In addition to these damages, shear cracks with widths less than 3 mm were observed in many load-bearing elements and connection zones. Furthermore, damage was noted in the building's infill walls.



**Figure 8.** Locations of observed damage marked on the ground floor plan.



**Figure 9.** Structural damage to the shear walls on the ground floor.



**Figure 10.** Observed damage due to the pounding effect.

### 3.2. Experimental Investigation

Core samples were taken to determine the concrete strength of the investigated building (Figure 11). Compressive strengths of the cylindrical samples are given separately for each story in Table 3. The building's design concrete compressive strength was specified as 30 MPa. Examination of core samples revealed that the concrete quality was consistent with the design. Also, removal of concrete cover and X-ray analyses were carried out on the columns to check the reinforcement in the load-bearing elements. As a result of the examinations, it was determined that the spacing of the transverse reinforcement in the confinement regions ranged between 10 and 14 cm.



Figure 11. Experimental study in the investigated building.

Table 3. Core concrete compressive strength (MPa).

| Sample No | Story Level |        |      |      |      |      |      |      |      |      |      |      |      |
|-----------|-------------|--------|------|------|------|------|------|------|------|------|------|------|------|
|           | Basement    | Ground | 1    | 2    | 3    | 4    | 5    | 6    | 7    | 8    | 9    | 10   | 11   |
| 1         | 43.6        | 44     | 47.7 | 49.2 | 51.5 | 36.6 | 48.5 | 51   | 31.3 | 41.4 | 41.3 | 40.8 | 39.4 |
| 2         | 50.2        | 36.4   | 50.7 | 48.1 | 57.6 | 50   | 49.2 | 48.6 | 29.9 | 49   | 33.8 | 36.5 | 34.4 |
| 3         | 51.8        | 47.7   |      |      |      |      |      |      |      |      |      |      |      |

#### 4. Numerical Analysis of the Building

Nonlinear time history analysis of the investigated building was conducted with the SAP2000 v25 software. The fiber element method was used to reflect the nonlinear behavior of reinforced concrete elements. Fiber element hinges were defined at both ends of each element. Slabs were not included in the model, and their loads were applied to the beams. Rigid diaphragms were defined at each story level to represent the rigid diaphragm effect provided by the slabs. A three-dimensional finite element model of the building is presented in Figure 12.

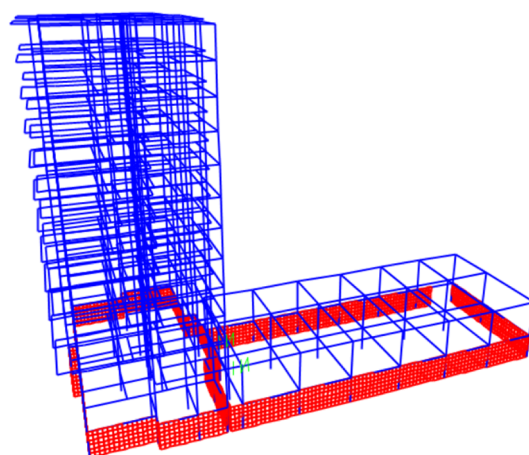
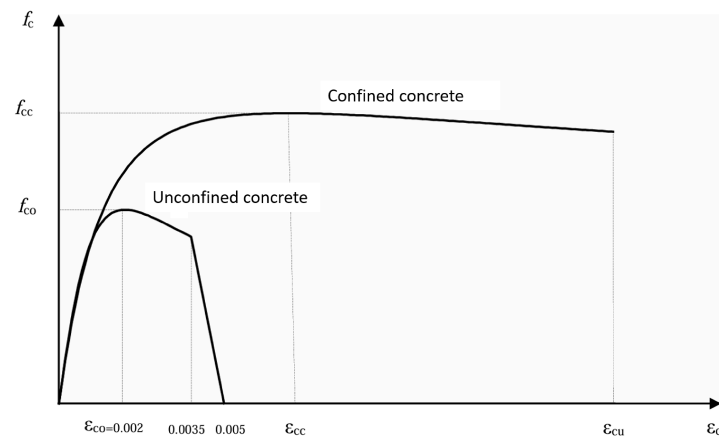


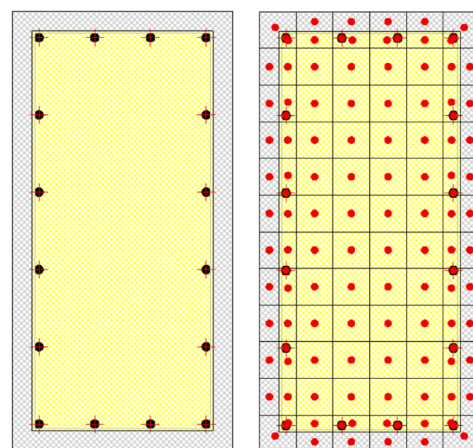
Figure 12. Three-dimensional finite element model of the building.

Concrete with a compressive strength of 35 MPa was used as the material in the numerical model. The confinement effect was taken into account for the columns and shear walls. The stress–strain graphs of confined and unconfined concrete were calculated using the equations defined in TBEC-2018 [33]. The damage limits of the concrete material were also determined using the equations given in TBEC-2018. Figure 13 shows the nonlinear behavior of confined and unconfined concrete. In the graph,  $f_{co}$  represents the

compressive strength of unconfined concrete, and  $f_{cc}$  represents the compressive strength of confined concrete.  $\varepsilon_{cc}$  and  $\varepsilon_{cu}$  vary depending on the confinement effect and represent the concrete strains corresponding to the maximum compressive strength and ultimate failure, respectively. The section of the column and the discretization of the fiber elements used in the numerical model are given in Figure 14. In the figure, the gray color represents unconfined concrete, the yellow color represents confined concrete, and the red points show the center of the fiber elements.



**Figure 13.** Behavior of confined and unconfined concrete [33].



**Figure 14.** An example section of a column and fiber element model.

In numerical model, shear walls were modeled as frame elements. Therefore, shear walls were connected to the frame system with rigid links. The U-shaped shear wall around the elevator was modeled with 3 frame elements and rigid links. Plastic hinges are defined to these frame elements.

Three damage limits were defined for concrete and reinforcing steel in TBEC-2018: Collapse Prevention (CP), Life Safety (LS), and Immediate Occupancy (IO). The confinement effect on concrete alters these damage limits. Therefore, damage limits were calculated separately for confined and unconfined concrete. Table 4 shows the damage limit in terms of strain of materials. The CP level of confined concrete is calculated using Equations (1) and (2).

$$\omega_{we} = \alpha_{se} \rho_{sh,min} \frac{f_{ywe}}{f_{ce}} \quad (1)$$

where  $\alpha_{se}$  is the effectiveness coefficient of the confinement reinforcement,  $\rho_{sh,min}$  is the minimum of the volumetric transverse reinforcement ratio,  $f_{ywe}$  is the expected yield stress of the confinement steel,  $f_{ce}$  is the expected concrete compressive strength.

$$\alpha_{se} = \left(1 - \frac{\sum a_i^2}{6b_o h_o}\right) \left(1 - \frac{s}{2b_o}\right) \left(1 - \frac{s}{2h_o}\right) \quad (2)$$

where  $b_o$  and  $h_o$  are the confined concrete dimensions,  $a$  is the distance between each longitudinal reinforcement in the section, and  $s$  is the transverse reinforcement spacing.

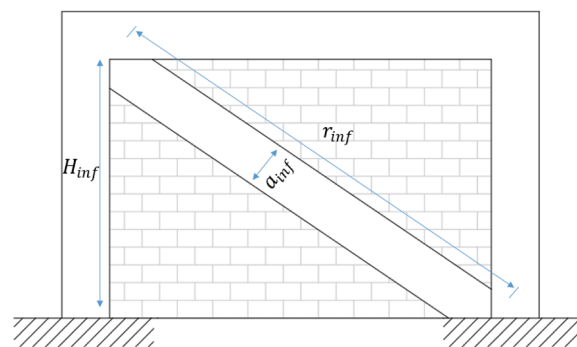
**Table 4.** Damage limit strains of materials.

| Damage Limit             | Unconfined Concrete | Confined Concrete                                            | Steel  |
|--------------------------|---------------------|--------------------------------------------------------------|--------|
| Immediate Occupancy (IO) | 0.0025              | 0.0025                                                       | 0.0075 |
| Life Safety (LS)         | 0.002625            | $0.75\varepsilon_c$                                          | 0.024  |
| Collapse Prevention (CP) | 0.0035              | $\varepsilon_c = 0.0035 + 0.04\sqrt{\omega_{we}} \leq 0.018$ | 0.032  |

Although it is assumed that infill walls do not carry loads in the design of reinforced concrete buildings, in fact, infill walls increase the rigidity of the building under the effect of horizontal loads. TBEC-2018 includes an equivalent diagonal strut model for modeling infill walls. TBEC-2018 recommends Equation (3) to determine the equivalent width of the diagonal strut (Figure 15).

$$a_{inf} = 0.175 \cdot \left( \sqrt[4]{\frac{E_{inf} \cdot t_{inf} \cdot \sin 2\theta}{E_f \cdot I_{col} \cdot H_{inf}}} \cdot h_{col} \right)^{-0.4} \cdot r_{inf} \quad (3)$$

where  $a_{inf}$ ,  $E_{inf}$ ,  $t_{inf}$ ,  $H_{inf}$  and  $r_{inf}$  are the effective width, the elasticity modulus, the thickness, the height, and the diagonal length of the infill wall, respectively.  $E_f$  is the elasticity modulus of the RC frame and  $h_{col}$  is the height of the column.



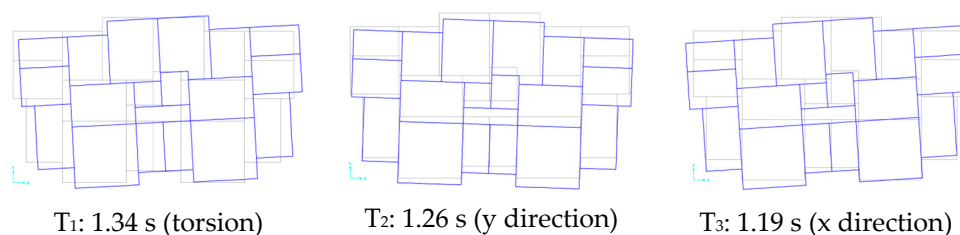
**Figure 15.** Modeling of infill wall as diagonal strut.

#### 4.1. Modal Analysis of the Building

Cracked section stiffness of reinforced concrete elements was used for modal analysis. The effective section stiffness is determined according to TBEC-2018. Modal periods and modal participating mass ratios of the investigated building are given in Table 5. Also, the first three mode shapes of the building are presented in Figure 16.

**Table 5.** Modal periods and modal participating mass ratios.

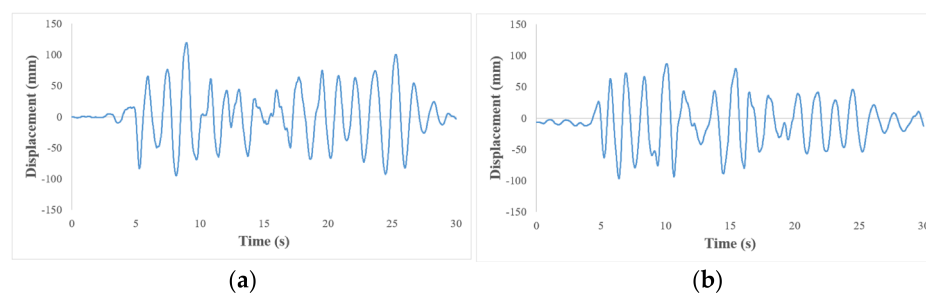
| Mode No | T (s) | U <sub>x</sub> | U <sub>y</sub> | R <sub>z</sub> |
|---------|-------|----------------|----------------|----------------|
| 1       | 1.34  | 0.16           | 0.00           | 0.44           |
| 2       | 1.26  | 0.12           | 0.55           | 0.00           |
| 3       | 1.19  | 0.43           | 0.15           | 0.03           |
| 4       | 0.43  | 0.01           | 0.00           | 0.06           |
| 5       | 0.37  | 0.04           | 0.07           | 0.00           |
| 6       | 0.35  | 0.07           | 0.04           | 0.00           |
| 7       | 0.32  | 0.07           | 0.00           | 0.41           |
| 8       | 0.28  | 0.01           | 0.03           | 0.02           |
| 9       | 0.27  | 0.00           | 0.05           | 0.00           |
| 10      | 0.23  | 0.00           | 0.00           | 0.02           |
| 11      | 0.18  | 0.03           | 0.01           | 0.00           |
| 12      | 0.18  | 0.01           | 0.03           | 0.00           |

**Figure 16.** Mode shapes of the building.

#### 4.2. Time History Analysis

In this section, the time history analysis results of the RC building were presented. The nonlinear analysis was performed with the effective 30 s of the Elbistan earthquake record. The nonlinear time-history analyses were performed using earthquake records with a time step of 0.01 s. Each record consisted of 3000 steps, corresponding to a total duration of 30 s. P-Δ effects were considered for the analyses. Also, a 5% damping ratio was assumed for the dynamic analyses.

Figure 17 shows the top displacement values of the RC building during the analysis. Figure 18 shows the lateral drift ratio of the building for two directions. The results show that there is a significant displacement difference between the two directions of the building. This can be explained by the shear wall ratios in the x and y directions. The shear wall ratios are 1.3% and 0.8% of the building's plan area in the y and x directions, respectively. Figure 19 shows shear forces applied to stories. The shear walls have a significant influence on the seismic behavior of the building. At the ground-floor level, the shear walls resist 68% and 70% of the total lateral shear force acting on the building in the X and Y directions, respectively.

**Figure 17.** Displacement in the (a) x direction and (b) y direction.

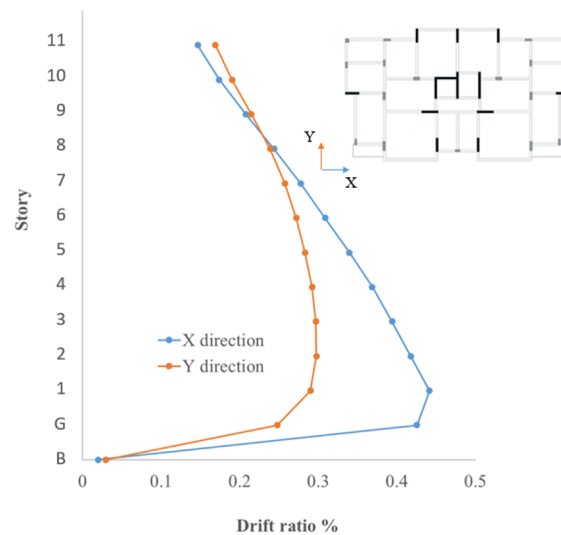


Figure 18. Lateral drift ratio of the building.

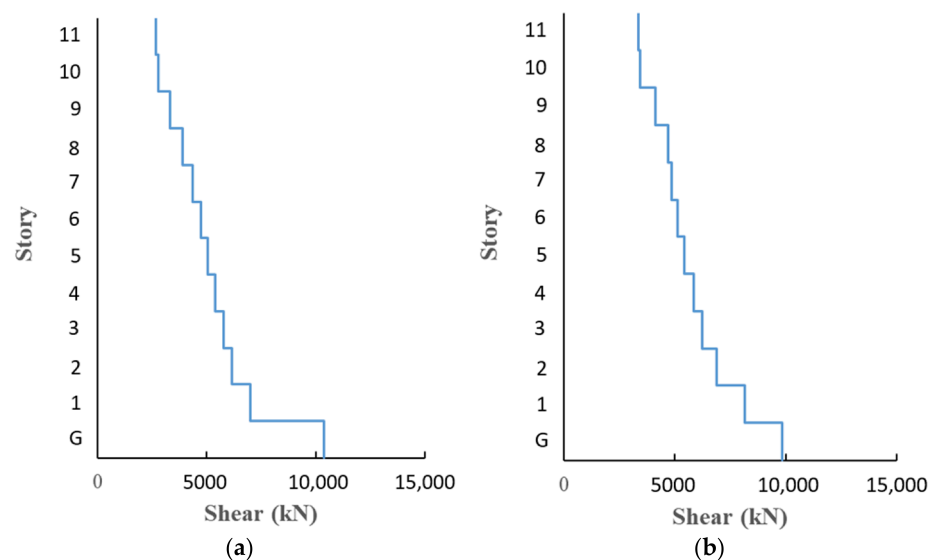


Figure 19. Story shear of the investigated building: (a) X direction; (b) Y direction.

The nonlinear analyses indicated that no significant damage occurred within the confined regions of the vertical structural members enclosed by transverse reinforcement. In the shear walls, the unconfined concrete along the strong axis approached the crushing limit but did not exceed it. The numerical analyses revealed that some of the beams connected to the elevator core walls exceeded the collapse limit. Overall, the damage patterns obtained from the numerical analyses were generally consistent with those observed during the post-earthquake field investigations. However, it should be noted that no ground motion was recorded at the building site; the actual ground motion accelerations experienced by the building during the earthquake could not be determined precisely. Therefore, the records from the nearest available ground motion were used for the nonlinear time-history analyses. Although these records cannot fully represent the site-specific ground motion at the building location, the numerical results obtained using these records provide a reasonable approximation of the observed seismic response and damage pattern of the investigated building.

## 5. Conclusions

This study examines the seismic performance of a 13-story RC building that was exposed to the Kahramanmaraş earthquake. The construction of the load-bearing elements and infill walls of the building has been completed, but it has not yet been in service during the Kahramanmaraş Earthquakes. Damage to the structural elements of the examined building was marked on the building's plan. Furthermore, the building was numerically analyzed using nonlinear modeling. The results obtained from damage assessments and numerical analyses are listed below.

- To obtain the compressive strength of the concrete, core samples were taken and subjected to a compressive strength test. The results of the experimental test are compatible with the values considered in the design of the building. The vertical structural elements were investigated on site by removing concrete cover and X-ray analyses to obtain the reinforcement details.
- There is insufficient seismic gap distance between the 13-story investigated RC building and the adjacent 2-story RC building. Although a collision occurred at floor level between these two buildings, only two beams sustained damage. The same floor level prevented further damage from pounding.
- Beams connecting the vertical load-bearing elements must have sufficient strength. Especially, beams connected to shear walls are subjected to greater loads. In the 13-story building, all beams were designed to be 60 cm high, while the two beams connecting the U-shaped elevator shear wall were designed to be 45 cm high. This caused significant damage to these two beams, even on the upper stories.
- The ground story height of the investigated building is 4.5 m, while all other stories were designed to be 3 m high. Furthermore, the infill walls on the upper stories were largely removed from the ground story. These two factors created a soft and weak story effect in the building. This is consistent with the fact that the damage to the vertical load-bearing elements occurred only on the ground story.
- Reinforcement buckling occurring at the end regions of the ground story shear walls indicates that more attention should be paid to the distance of transverse reinforcement in these regions.
- In this study, shear walls were used for 1.3% and 0.8% of the plan area of the building examined along the long and short sides, respectively. There is a significant displacement difference between the two directions of the building. This situation can be explained by the shear wall ratios of the building in the X and Y directions.

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