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Research Article

Performance investigation of a mixed slab building with beam and ribbed slabs

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ABSTRACT

Over time deterioration in material properties, unauthorized modifications, or construction defects can negatively affect the seismic behavior of structural members. Therefore, the in-situ identification and performance analysis of reinforced concrete elements, carried out to ensure the safety of the existing building stock, have become a fundamental part of examining structural characteristics and developing disaster risk reduction strategies. In this study, the structural performance analysis of an existing reinforced concrete building with a mixed slab system was conducted by using the STA4CAD V13.1 software. For this purpose, nonlinear pushover analysis was performed for each seismic direction. As the existing reinforced concrete building did not satisfy the requirements for the single-mode method prescribed in TBDY (2018), a multi-mode pushover analysis was conducted for the structure. Based on the results of nonlinear multi-mode pushover analysis, the damage states of the structural members were determined, and the performance level of the structure was evaluated. The targeted performance level of controlled damage, as defined in the Turkish Building Earthquake Code-2018 TBEC (2018), could not be achieved for the analyzed residential-type reinforced concrete building. However, the collapse prevention performance level was determined due to the damage occurrence in the structural elements.

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1. Introduction

Due to its geological location, Türkiye is situated on the Alpine-Himalayan earthquake belt, one of the most active seismic zones in the world. This condition exposes the country to significant seismic hazards. Numerous active fault lines are distributed across the country, with major seismic sources such as the North Anatolian Fault, East Anatolian Fault, and active fault systems in Western Anatolia capable of generating earthquakes that can cause widespread damage. These fault lines have historically produced destructive earthquakes, resulting in severe loss of life and property. The fact that a large portion of Türkiye's settlements are located in close proximity to active fault zones further increases the seismic risk and renders structural safety a top priority. In order to mitigate the impacts of earthquakes, one of the leading natural disasters causing significant loss of life and property, it is of great importance not only to design new buildings in earthquake-prone regions such as Türkiye to be earthquake-resistant but also to assess the structural safety of existing buildings. Therefore, in the context of addressing seismic hazards, the condition of the existing building stock, urban planning policies, and disaster management strategies must be addressed through an integrated and comprehensive approach.

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Earthquakes that have occurred during both the historical and instrumental periods clearly reveal the severity of Türkiye's seismic characteristics. This reality has necessitated the revision of seismic codes in line with scientific data. In particular, the 1999 Marmara Earthquake exposed serious structural deficiencies and led to significant regulatory changes through the issuance of the 2007 Earthquake Code for Buildings to be Constructed in Seismic Zones. In the following years, considering advances in scientific research, engineering practices, and the increasing availability of seismic records, the existing code was comprehensively revised, resulting in the Turkish Building Earthquake Code-2018 TBEC (2018), which came into effect on January 1, 2019. The new code incorporates the performance-based design approach and includes detailed provisions not only for the design of new buildings but also for the evaluation and, when necessary, retrofitting of existing structures. It is regarded as a significant step toward enhancing earthquake safety. Additionally, the Türkiye Earthquake Hazard Map, which expresses the seismic hazard of a given location in terms of peak ground acceleration based on geographical coordinates, was issued separately and implemented on a different date.

Structures subjected to seismic effects are exposed to sudden and significant lateral forces. The compliance and load-bearing capacities of existing reinforced concrete elements, particularly columns, beams, and slabs, with current seismic codes play a decisive role in the seismic performance of buildings. Reinforced concrete structural elements are the fundamental components responsible for safely transferring a building's vertical and horizontal loads to the ground. Elements such as columns, beams, slabs, and shear walls work together to ensure the strength and stiffness of the structure. The load transfer between these elements forms the basis of the structural design, improper load transmission can lead to significant deficiencies in structural performance. Therefore, the sizing of reinforced concrete elements, reinforcement detailing, and the design of connection regions must be carried out in accordance with current codes and standards. A properly designed structural system contributes to maintaining the integrity of the building not only under service loads but also under extreme loading conditions such as earthquakes. Compliance with design principles is essential for the construction of structures that are both safe and economically efficient.

Reinforced concrete slabs are an integral part of the structural system, ensuring the safe transfer of loads to vertical load-bearing elements such as beams and columns. They also contribute to the horizontal stiffness of the structure, significantly influencing its behavior under lateral loads such as those generated by earthquakes. Designing slabs with adequate thickness and proper reinforcement is critical for effective load transfer and for ensuring the integral action of the structural system. Within the scope of TBEC (2018), irregularities in plan, specifically type A2 irregularities referred to as slab discontinuities, are associated with the presence of openings or voids that may compromise the validity of the rigid diaphragm assumption in the slab plane TBEC (2018). Such discontinuities can lead to uncertainties in horizontal load transfer and reduced structural performance. Therefore, these effects must be taken into account during slab design, and the seismic safety of the structure should be thoroughly evaluated. Researchers have investigated the effects of voids on the slab behavior in the literature (Kalib 2021; Khajehdehi and Panahshahi 2016; Özbayrak 2021).

Structural performance refers to whether a building's behavior under a given level of loading remains within acceptable limits, and it is particularly evaluated in terms of meeting the performance level targets defined in seismic design codes. In the assessment of the seismic safety of buildings, nonlinear analysis methods are preferred as they provide more realistic results (Erdem 2016; Ricci et al. 2018; Ozkul et al. 2019; Lin and Chuang 2023; Kuria and Kegyes-Brassai 2023; Gupta and Gupta 2024; Ergin and Seçer 2025). One such method, nonlinear pushover analysis, enables the identification of the structure's lateral load capacity and collapse mechanism by revealing plastic hinge formations and displacement capacities. Therefore, nonlinear pushover analysis has become widely used in recent years for the performance evaluation of existing buildings, due to its practicality in engineering applications and its compatibility with code-based requirements (Golghate et al. 2013; Joyner and Sasani 2020; Bento and Simões 2021; Sullivan et al. 2021; Erdem and Karal 2022; Erdem and Uyan 2025).

A mixed slab system refers to a structural configuration in which different types of slabs such as one-way ribbed, flat, waffle, or conventional beam-supported slabs are used within the same building. These systems are often encountered in residential buildings where architectural design plays a prominent role. Due to the varying stiffness, mass, and load transfer characteristics of different slab types, the overall structural behavior becomes more complex and difficult to accurately represent using conventional analysis methods. This complexity hinders the reliable assessment of the overall performance of the load-bearing system. Therefore, in existing buildings with mixed slab systems, nonlinear analysis methods are preferred, as they offer a more realistic and reliable representation of structural behavior under seismic effects (Eşki et al. 2020; Mene and Nilawar 2022; Hulke and Solanke 2023; Shende et al. 2024).

In this study, the structural safety of an existing residential type reinforced concrete building with a mixed slab system was investigated. The STA4CAD V13.1 (2023) software was utilized to assess the seismic performance of the structure. As the analysis method, nonlinear incremental pushover analysis was applied to the building. Initially, a single-mode pushover analysis was performed. However, according to TBEC (2018), the mass participation ratio of the dominant vibration mode in the direction considered must exceed 0.70 for the single-mode pushover method to be valid. The analysis results showed that the mass participation ratio remained below 0.70. Furthermore, the torsional irregularity coefficient must be less than 1.40, as stipulated by the code. In the examined building, neither of these two conditions was satisfied. Therefore, a multi-mode pushover analysis was applied. Based on the performance analysis conducted for each seismic direction, the damage states of the structural members were identified, and the seismic performance level of the existing building was subsequently determined. Within the scope of this study, both the investigation of a reinforced concrete building with a mixed slab system and the implementation of multi-mode pushover analysis are considered to offer valuable contributions to the literature.

2. Materials and Method

As part of this study, the existing building whose seismic performance was investigated is a six-story residential reinforced concrete structure constructed in 1995. The building has a total floor area of 274.17 m^2 and includes one partial basement floor, one ground floor, three typical floors, and one roof floor. The story heights are 250 cm in the basement, 302 cm on the ground floor, 270 cm on the first and second floors, 302 cm on the third floor, and 260 cm in the roof floor. Based on the geotechnical investigation conducted for the building, the local soil class was identified as ZC, corresponding to very dense sand, gravel, and stiff clay layers. Examination of the slab system revealed the use of a mixed slab system. Ribbed (hollow block) slabs were used on the ground and third floors, while beam-and-slab systems were employed on the other floors. The front elevation of the building is shown in Fig. 1.

Following the on-site verification of the building's structural drawings, core sampling was carried out to determine the concrete compressive strength. For this purpose, a total of 13 core samples were extracted from the building. Three samples were taken from the ground floor, while two samples were taken from each of the remaining floors. Laboratory results were obtained from axial compression tests conducted on the core samples. The test results for each sample are presented in Table 1. In the table, the value F_d represents the correction factor for core diameter, and F_{kd} indicates the corrected compressive strength of the core sample.

Fig. 1. Front elevation of the residential building.

Table 1. Experimental results of concrete samples.

Sample no	Floor	Element no	Diameter and height of the sample (mm)	Test result (MPa)	F_d (MPa)	F_{kd} (MPa)
1	Basement	S107	94/94	11.01	1.03	11.3
2	Basement	P104	94/94	10.16	1.03	10.5
3	Ground	S217	94/94	9.66	1.03	9.9
4	Ground	S228	94/94	7.81	1.03	8.0
5	Ground	1S222	94/94	8.06	1.03	8.3
6	1. Floor	S317	94/94	9.88	1.03	10.2
7	1. Floor	S318	94/94	13.58	1.03	14.0
8	2. Floor	S417	94/94	16.06	1.03	16.5
9	2. Floor	S418	94/94	13.05	1.03	13.4
10	3. Floor	S517	94/94	8.25	1.03	8.5
11	3. Floor	S518	94/94	13.24	1.03	13.6
12	4. Floor	S617	94/94	8.48	1.03	8.7
13	4. Floor	S618	94/94	12.81	1.03	13.2

According to the laboratory test results conducted on identical core samples, as detailed in Table 1, the average compressive strength was calculated as 11.50 MPa, with a standard deviation of 2.70 MPa. The difference between these two values is 8.80 MPa. By applying a reduction factor of 0.85 to the average compressive strength, the final value was determined to be 9.80 MPa. In accordance with Article 15.2.5.3 of TBEC (2018), the concrete compressive strength to be used in the structural analysis software for all load-bearing elements was defined as C10 (10 MPa).

Following the determination of the compressive strength of concrete, a reinforcement detection study was conducted to identify the existing reinforcement. As an example, the rebar exposure process performed on one beam and one column is presented in Fig 2.

Fig. 2. Exposure of rebars.

The column dimensions used in the structural system are 25×50 cm, 25×60 cm, 30×60 cm, 25×100 cm, and 25×125 cm. The dimensions of columns S06, S07, S08, and S18 vary on the roof floor. Shear walls are present as vertical load-bearing elements in the basement floors and around the stairwell. Additionally, two shear walls are located around the elevator shaft. The dimensions of these elements are 25×200 cm and 20×290 cm. Columns with varying cross-sectional dimensions and their reinforcement layouts are presented in Fig. 3.

To determine the cross-sectional loss in the reinforcement of reinforced concrete columns, rebar exposure tests were conducted. As an example, the structural elements on the ground floor where reinforcement investigation was performed are marked in Fig. 4. Based on the investigation, the column reinforcements were identified as S420 ribbed reinforcement steel. Additionally, no signs of corrosion were observed in the column reinforcements.

The longitudinal reinforcement details of the reinforced concrete columns are presented in Table 2. Double-legged vertical stirrups were used as transverse reinforcement in the columns. The stirrups have a diameter of 8 mm and are spaced at 20 cm intervals. Additionally, it was determined that the stirrups do not have 135-degree hooks; instead, the hooks are implemented at 90 degrees.

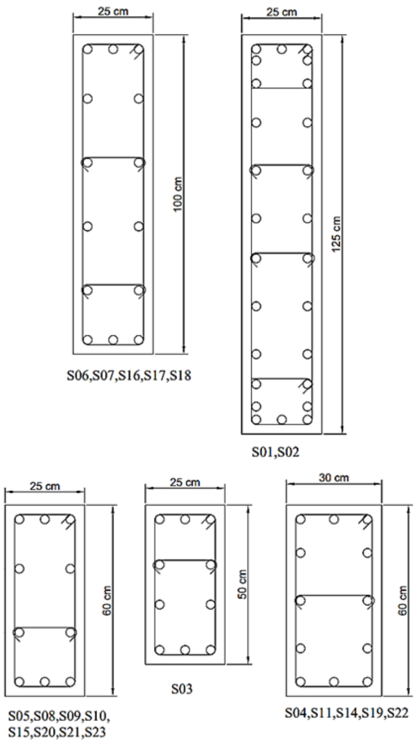

Fig. 3. Column sections.

After determining the column dimensions and reinforcement, the characteristics of the beams in the building were evaluated. According to the existing project, longitudinal reinforcement in the beams consists of 2Φ12 bars on floors with slab systems and 3Φ12, 4Φ12, or 5Φ12 bars on floors with hollow block slab systems. It was observed that the stirrup hooks used as transverse reinforcement were implemented at 90 degrees. Additionally, no stirrup confinement was provided at the beam end regions. One beam per floor was subjected to rebar exposure, and no signs of corrosion were detected in these beams' reinforcement. However, differences were noted between the beam reinforcement shown in the project and the reinforcement observed during the rebar exposure tests. The details of the beams subjected to rebar exposure are presented in Table 3. Additionally, as an example, the reinforcement layout of three beams located at grid line 6 on the ground floor is shown in Fig. 5.

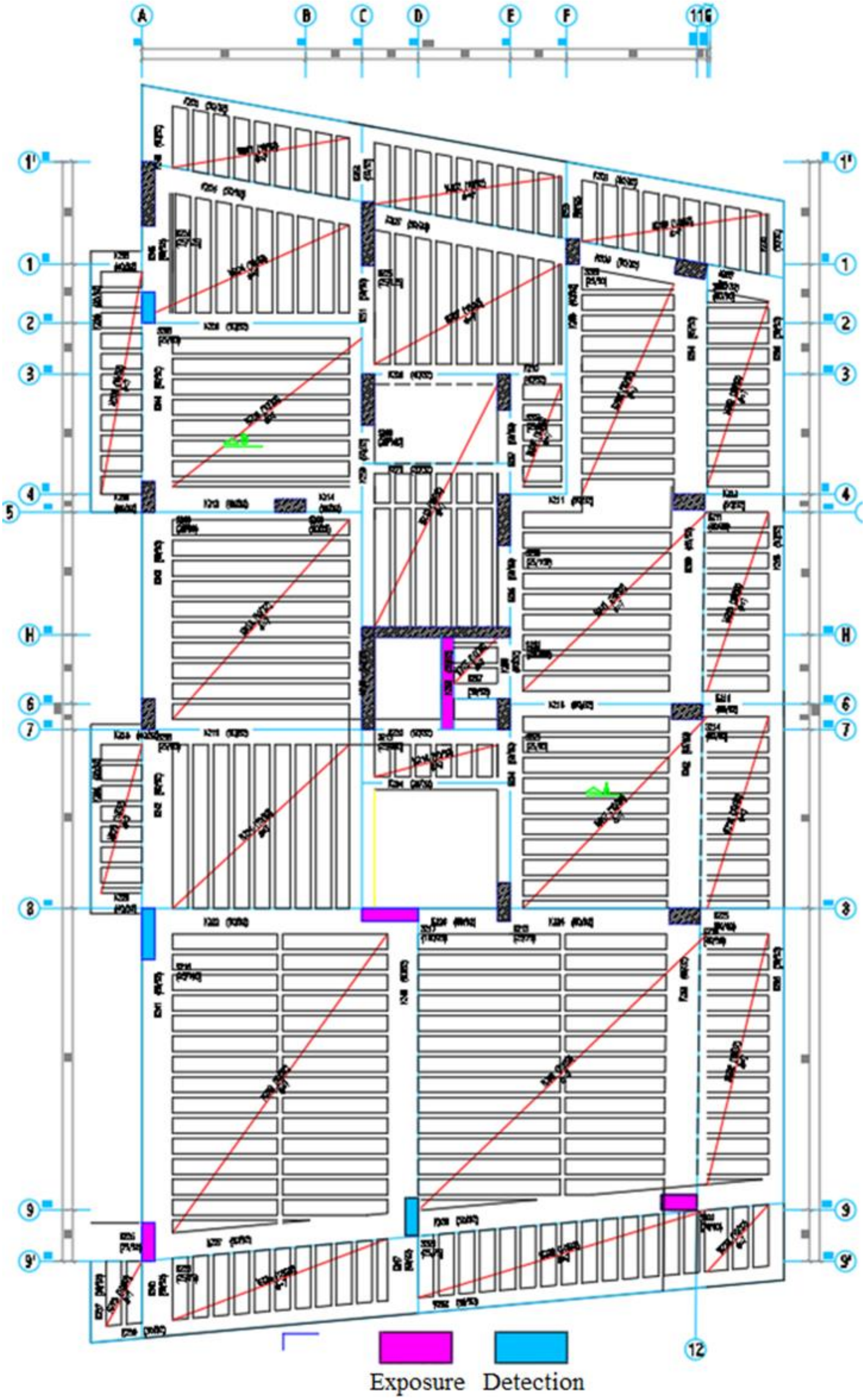

Fig. 4. Exposure and detection of rebars in ground floor.

Table 2. Dimension and reinforcement details of the columns.

Column no	Dimensions (cm)	Longitudinal reinforcement
Basement columns		
SB05, SB08, SB09, SB15	25×60	10Φ14
SB06	25×100	14Φ16
SB01, SB02	25×125	6Φ14 + 12Φ12
Ground floor columns		
SZ03	25×50	10Φ14
SZ05, SZ08, SZ09, SZ10, SZ15, SZ20, SZ21, SZ23	25×60	10Φ14
SZ04, SZ11, SZ14, SZ19, SZ22	30×60	12Φ14
SZ06, SZ07, SZ16, SZ17, SZ18	25×100	14Φ16
SZ01, SZ02	25×125	6Φ14 + 12Φ12
Roof floor columns		
SZ08	25×50	10Φ14
SZ06, SZ07, SZ18, SZ10, SZ15, SZ23	25×60	10Φ14
SZ14	30×60	12Φ14
SZ16, SZ17	25×100	14Φ16

Table 3. Dimension and reinforcement details of the beams.

No	Floor	Beam	Beam dimensions (cm)	Existing reinforcement (S420)			Project reinforcement (S420)		
				Longitudinal reinforcement (S420)		Stirrups (cm)	Longitudinal reinforcement (S420)		Stirrups (cm)
1	Basement	K101	20×50	Bottom	3Φ14	8Φ19	Top	1Φ12	Φ8/20/10
							Bottom	2Φ14	
2	Ground	K253	25×32	Bottom	3Φ12	8Φ14	Top	2Φ12	Φ8/20/10
							Bottom	3Φ12	
3	1. Floor	K314	20×50	Bottom	8Φ17	8Φ17	Top	1Φ12	Φ8/20/10
							Bottom	2Φ14	
4	2. Floor	K414	25×40	Bottom	8Φ12	8Φ12	Top	2Φ12	Φ8/16/8
							Bottom	2Φ12	
5	3. Floor	K514	20×60	Bottom	8Φ25	8Φ25	Top	2Φ12	Φ8/16/8
							Bottom	2Φ12	

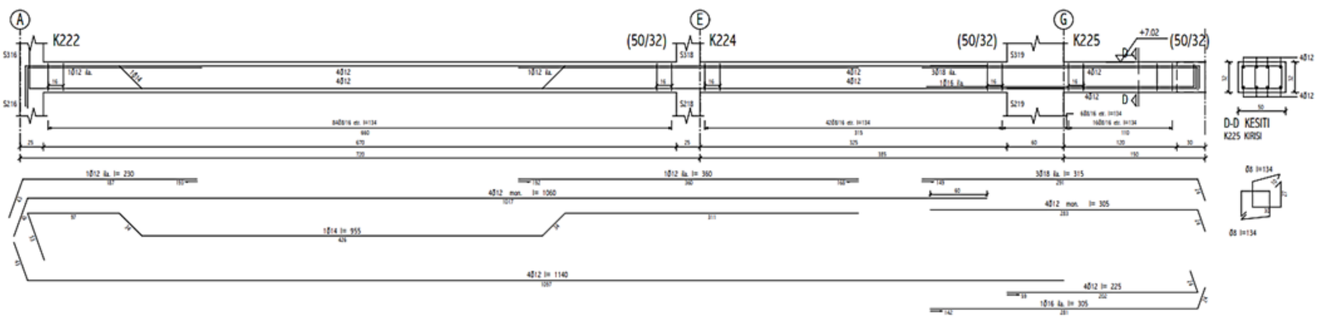

Fig. 5. Reinforcement details of K222, K224, and K225 beams.

An examination of the reinforced concrete beam-and-slab floor thickness revealed that the slab thickness is 12 cm on all floors and 15 cm on the roof floor. The hollow block slabs on the ground and third floors have a thickness of 7 cm, with rib heights measuring 32 cm. The live load value for the slabs was taken as 2.00 kN/m^2 (TS-498 1997). Additionally, exterior walls were modeled as 20 cm thick and interior walls as 10 cm thick in the structural analysis software. The beam-and-slab and ribbed slab systems used in the existing building are illustrated on the floor formwork plans shown in Figs. 6 and 7.

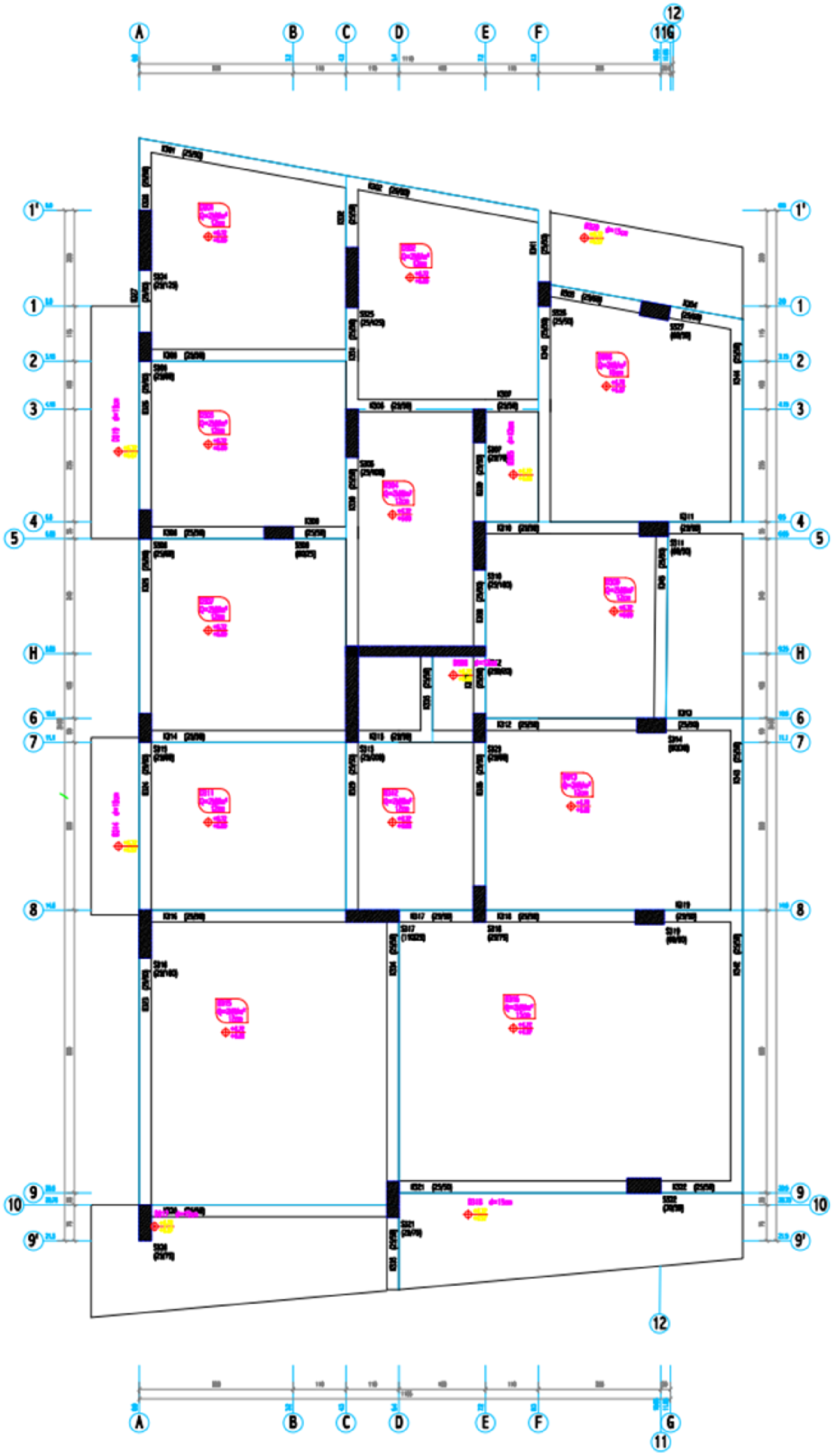

Fig. 6. Formwork plan of 1st floor.

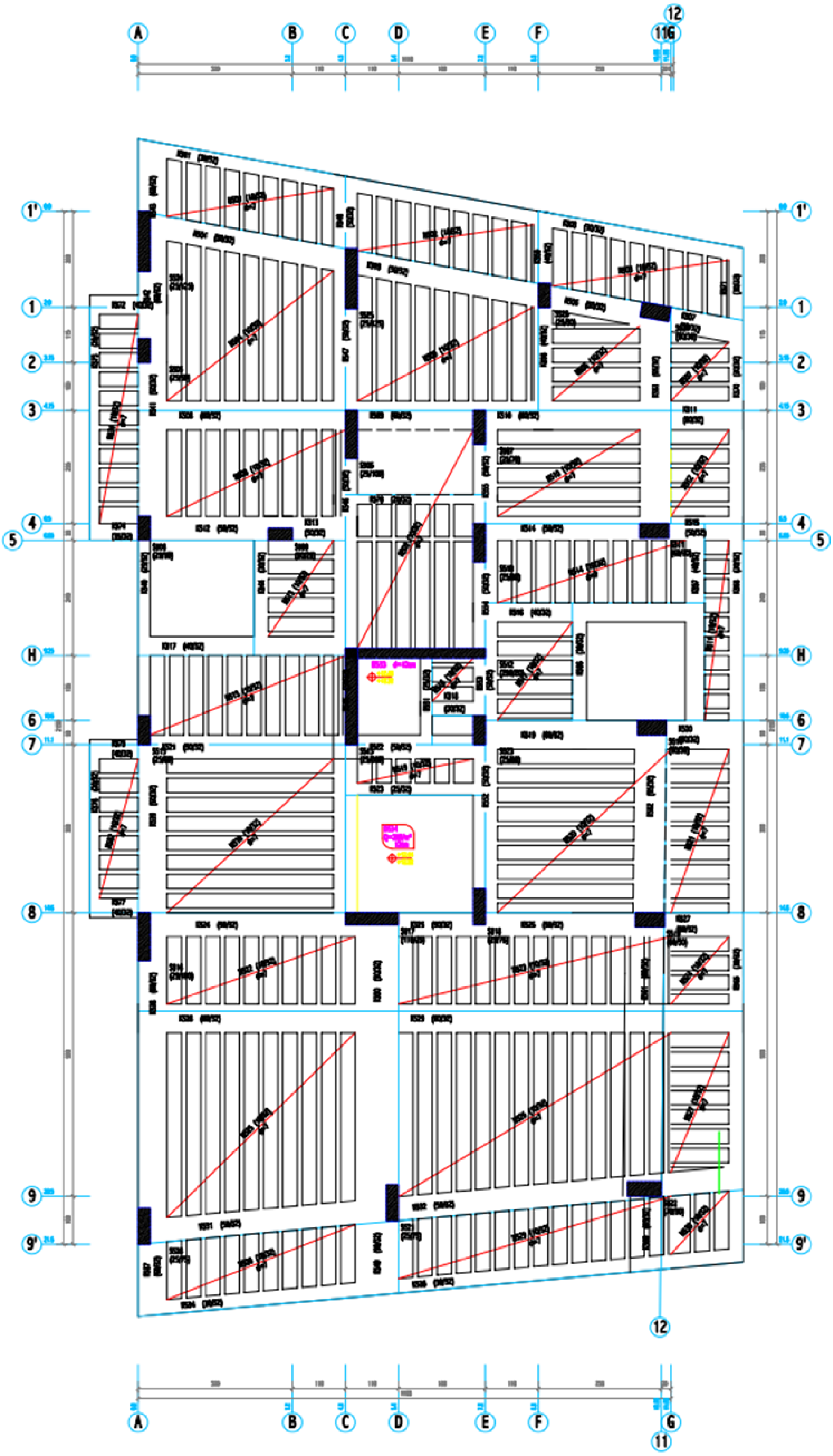

Fig. 7. Formwork plan of 3rd floor.

3. Performance Analysis

In this section, the structural performance analysis of the existing reinforced concrete building was conducted in accordance with TBEC (2018). The building was first modeled digitally using the STA4CAD V13.1 (2023) software. The beam elements and the three-dimensional solid model of the building are shown in Fig. 8.

Using the Türkiye Seismic Hazard Map application prepared for TBEC (2018), the building's location information, including latitude and longitude details, seismic ground motion level, and local soil classification were defined. The seismic ground motion level was applied as "DD2", corresponding to a 10% probability of exceedance in 50 years (return period of 475 years).

Following the modeling process, the building's location, soil, and seismic level data were entered into the system using the TDTH application, and the values obtained for the DD2 earthquake level from the interactive web application are presented in Table 4.

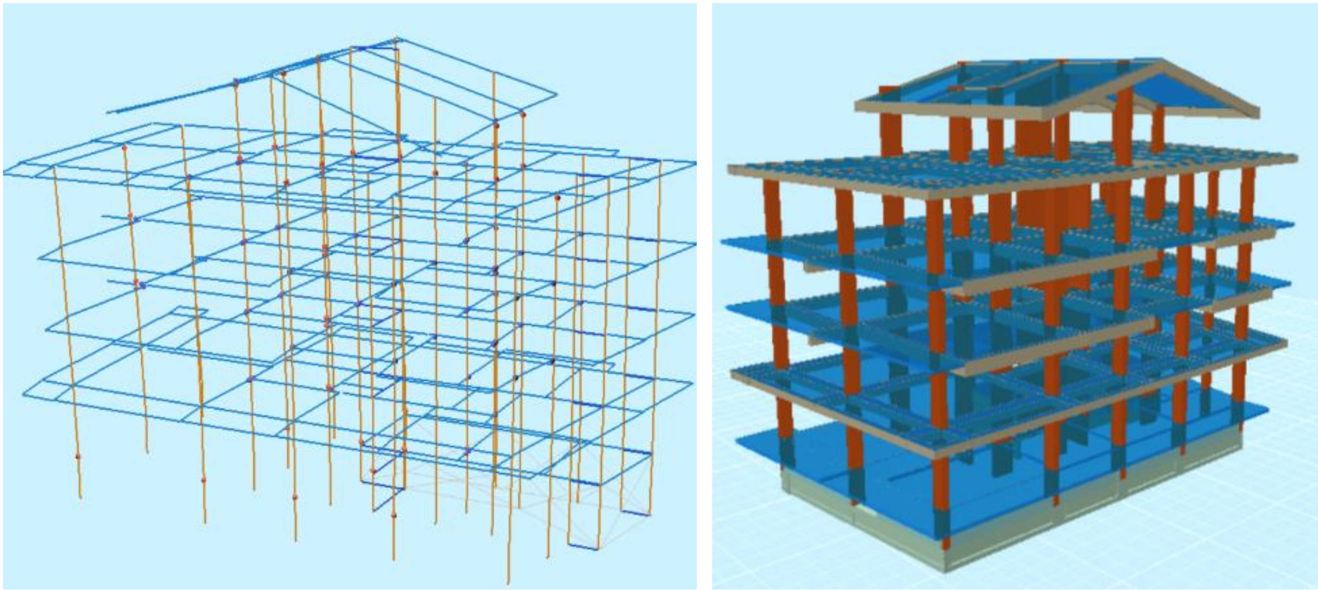

Fig. 8. Existing building model.

Table 4. Spectral parameters.

Parameter	Value
Maximum ground acceleration, PGA	0.409
Maximum ground velocity, PGV (cm/s)	23.102
Short-period spectral acceleration coefficient, S_s	0.977
Spectral acceleration coefficient for 1.0 s period, S_1	0.234
Short-period design spectral acceleration coefficient, S_{ds}	1.172
Design spectral acceleration coefficient for 1.0 s period, S_{d1}	0.351

Since the existing building is used for residential purposes, it falls under the "other buildings" category according to TBEC (2018). Therefore, the building use class was assigned as 3, and the building importance factor was taken as 1.0. The fundamental periods of the building were determined as 0.88 seconds and 0.71 seconds in the x and y directions, respectively. For the building's performance analysis, a deformation-based assessment and design approach was applied, requiring the achievement of the controlled damage performance objective for the DD2 earthquake level.

The global performance limits of are determined based on the damage levels observed in structural elements as a result of nonlinear pushover analysis, and these limits are largely consistent across various seismic design codes. As illustrated in Fig. 9, these performance limits defined within the internal force–deformation relationship are categorized as Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). At the Immediate Occupancy level, minor cracking may be observed in structural elements, yet the overall integrity and functionality of the structure are preserved. The Life Safety limit is characterized by more extensive damage, with certain elements exhibiting significant deformation within designated damage zones, yet structural collapse is still avoided. The Collapse Prevention limit represents a near-collapse condition, where severe damage compromises the structural system's capacity to resist further loading. Exceeding this threshold results in global structural collapse TBEC (2018).

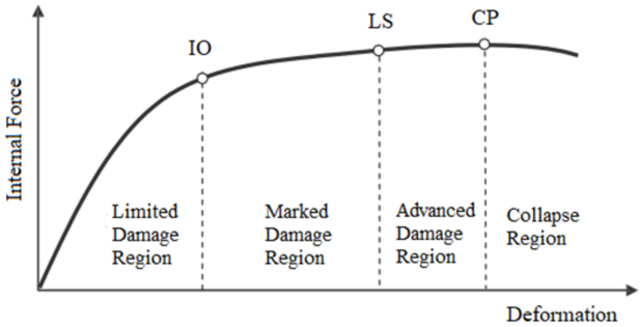

Fig. 9. Damage regions.

Recent studies in earthquake engineering have revealed the limitations of traditional force-based design approaches, leading to an increasing shift toward displacement-based design methods. This approach provides a more appropriate framework for evaluating the realistic responses of structures to seismic demands. Particularly in determining the seismic performance of newly designed or existing reinforced concrete buildings, nonlinear static or dynamic analysis methods which yield more reliable results compared to linear analysis techniques have become prominent. However, nonlinear static analysis methods are more frequently preferred in practical engineering applications due to their ease of implementation, reduced computational effort, and compatibility with widely used engineering software. This preference highlights the prominence of static methods in performance-based assessments of existing structures.

The nonlinear incremental pushover analysis is a widely used method in performance-based seismic evaluation of structures. In this approach, lateral loads are incrementally applied to a structural model until a target displacement is reached, allowing the assessment of the structure's behavior beyond the elastic range. The analysis captures the sequence of yielding in structural elements and provides insight into damage progression and failure mechanisms.

One of the main advantages of the nonlinear pushover analysis is its relative simplicity compared to fully nonlinear dynamic analyses, while still offering valuable information about global and local performance levels. It enables engineers to estimate the capacity curve of the structure, determine the target performance points, and evaluate the distribution of possible damage regions, making it an effective and practical tool for the seismic assessment of both new and existing buildings. The modal capacity diagram is obtained using the coordinate transformation defined in TBEC (2018). As shown in Fig. 10, the modal capacity diagram is plotted together with the earthquake spectrum.

Nonlinear pushover analyses were performed in both directions of the existing building for the relevant seismic level. Target displacement values were determined as 7.9 cm and 6.6 cm for the *x* and *y* directions, respectively. The damage states of the structural system elements were evaluated as the building was pushed up to the defined performance points. The base shear force–roof displacement curves for the building are presented in Figs. 11 and 12.

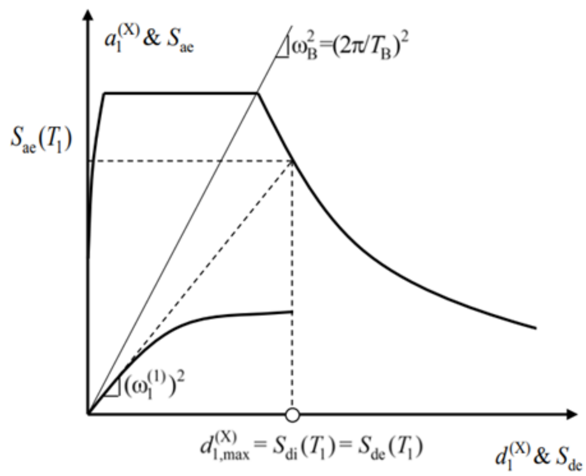

Fig. 10. Modal capacity diagram.

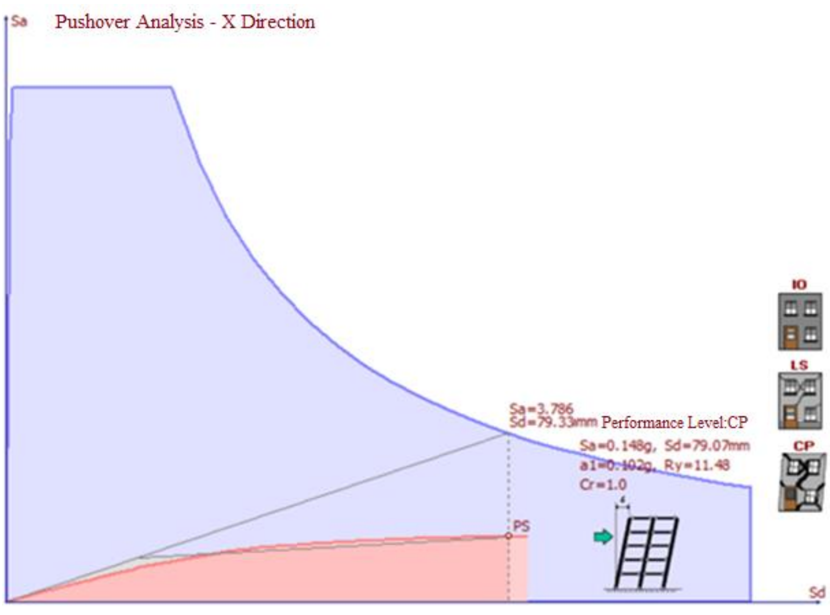

Fig. 11. Pushover curve in *x* direction.

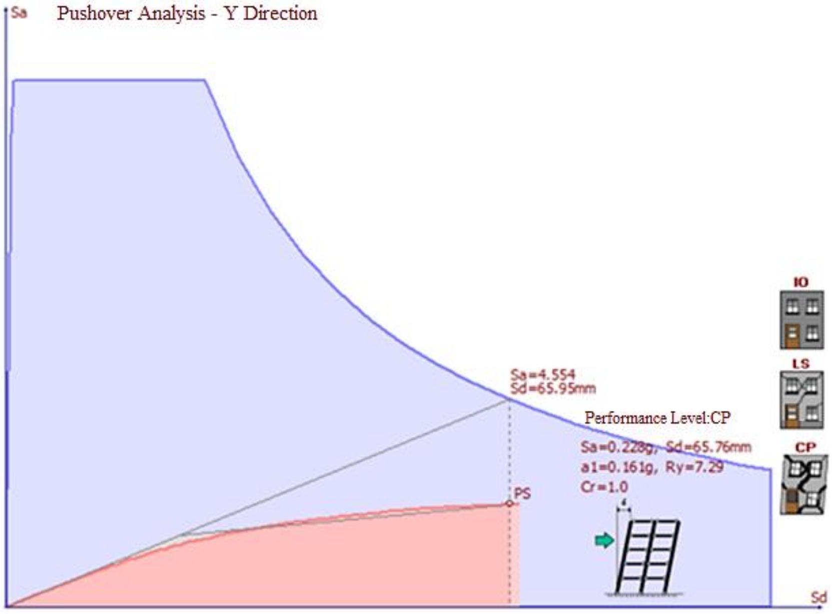

Fig. 12. Pushover curve in *y* direction.

The mass participation ratios were calculated as 66.9% and 69.4% in the x and y directions, respectively. These values fall below the 70% threshold. Additionally, the torsional irregularity coefficient was found to be 1.45, exceeding the limit of 1.40. Since the conditions for applying the single-mode pushover method were not met according to TBEC (2018), a multi-mode pushover analysis was conducted on the building. In this method, the structure is incrementally pushed using a generalized load vector until a specific story reaches its target inter-story drift ratio.

As a result of the multi-mode pushover analyses performed for each earthquake direction, the damage regions in the structural system elements were identified. The damage states for each floor are presented in Tables 5 and 6. Columns and shear wall elements are referred to as vertical elements in these tables.

Table 5. Damage regions in *x* direction.

Element	Floor	LDR	MDR	MDR	CR
Vertical	Basement	11	3	0	0
	Ground	8	15	0	0
	1. Floor	11	12	0	0
	2. Floor	8	15	0	0
	3. Floor	16	7	0	0
	4. Floor	4	6	0	1
Beam	Basement	3	0	0	0
	Ground	36	0	0	0
	1. Floor	18	4	0	0
	2. Floor	19	3	0	0
	3. Floor	39	2	0	0
	4. Floor	6	2	2	0

Table 6. Damage regions in *y* direction.

Element	Floor	LDR	MDR	MDR	CR
Vertical	Basement	10	4	0	0
	Ground	4	19	0	0
	1. Floor	5	18	0	0
	2. Floor	7	16	0	0
	3. Floor	17	6	0	0
	4. Floor	1	10	0	0
Beam	Basement	0	0	0	0
	Ground	32	2	0	0
	1. Floor	8	15	0	0
	2. Floor	9	14	0	0
	3. Floor	32	3	0	0
	4. Floor	7	5	0	0

To visually present the damage states occurring in each direction, the percentage distribution of damage regions for the structural system elements on each floor is shown in Figs. 13–16.

The structural safety of the existing reinforced concrete building is determined based on the damage states of its vertical and horizontal load-bearing elements. As a result of the analyses, the fourth floor i.e., the roof level, where the lowest stiffness is observed and changes in column dimensions occur, has been identified as the most critical story in terms of performance level. For the Design Earthquake Level 2 (DD2), the building is expected to meet the performance target of life safety. This performance level corresponds to a state where the structural elements sustain damage that is not excessively severe and, in most cases, repairable. According to the analysis results, it has been determined that the building does not meet the targeted performance level and remains at the "collapse prevention" performance level.

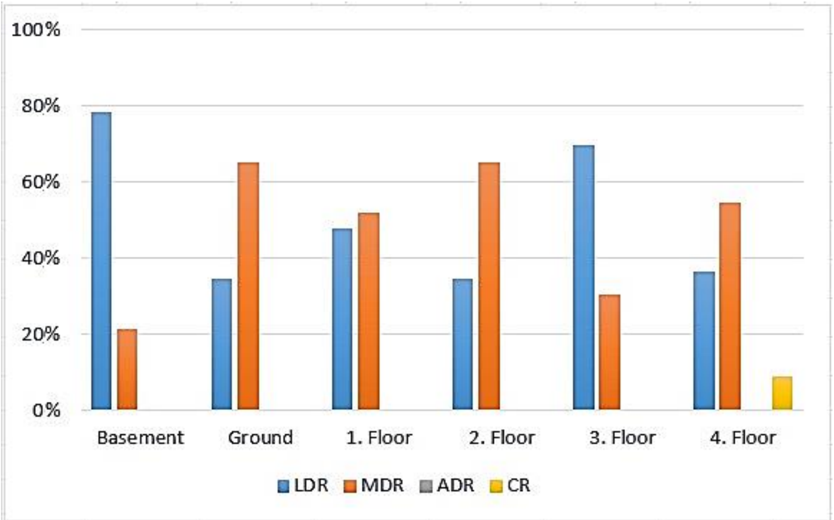

Fig. 13. Damage distributions of vertical elements in *x* direction.

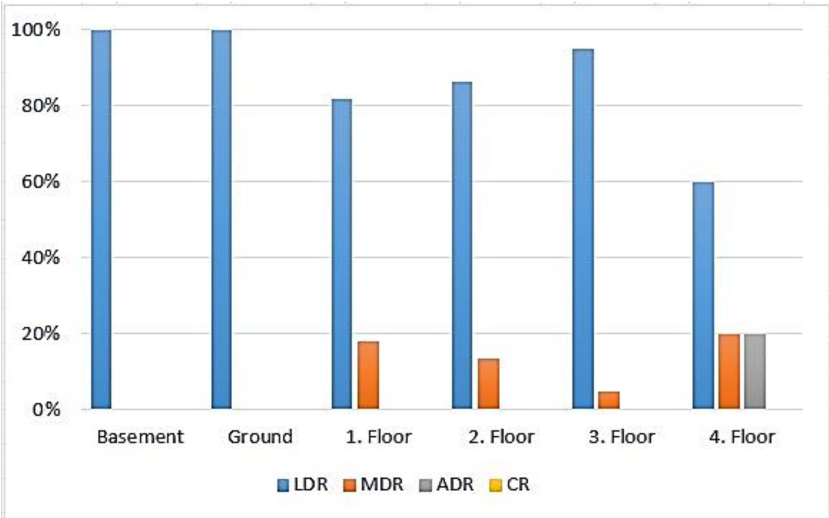

Fig. 14. Damage distributions of beams in *x* direction.

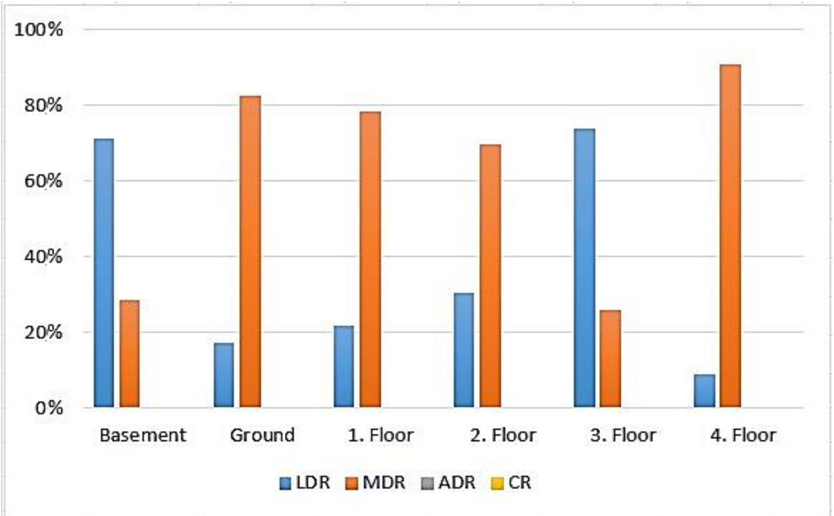

Fig. 15. Damage distributions of vertical elements in y direction.

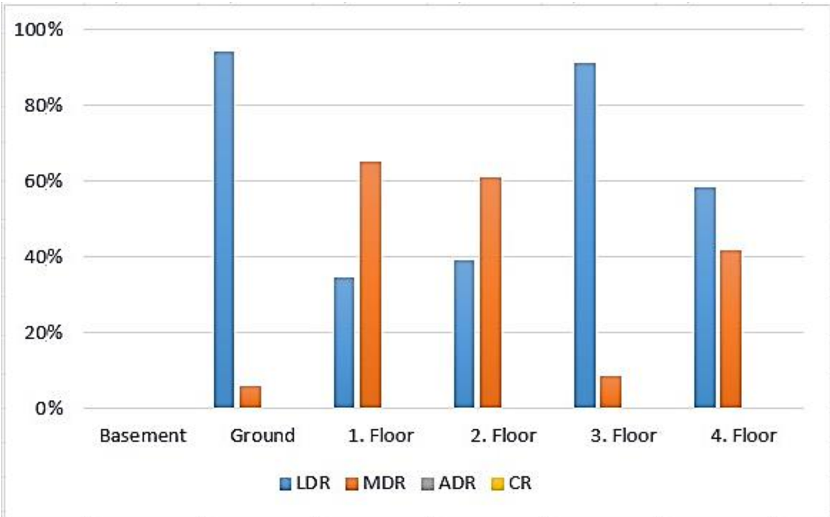

Fig. 16. Damage distributions of beams in *y* direction.

4. Conclusions

In this study, the seismic performance of an existing residential reinforced concrete building with a mixed slab system was investigated. Initially, in-situ inspections of the structural system elements were conducted. Subsequently, the soil classification of the site was determined. The compressive strength of the concrete was calculated based on the results of core sample tests. Through surface stripping and rebar scanning studies performed on the structural elements, the longitudinal and transverse reinforcement layout was identified. As a result, the concrete class of the building was determined to be C10, and the reinforcement steel was classified as S420 ribbed bars.

The building was modeled in the computer environment via STA4CAD V13.1 (2023) software. As a result of the analysis, since the obtained mass participation ratio and torsional irregularity coefficient were outside the limits permitted by the Turkish Building Earthquake Code TBEC (2018), a multi-mode nonlinear analysis was conducted instead of a single-mode analysis. For each earthquake direction, target displacement values were determined for the DD2 earthquake level. The damage states of all vertical and horizontal structural elements on each floor were identified. Based on the obtained damage ratios, the seismic performance of the existing building was evaluated.

Since the examined building is a residential structure, it is expected to meet the life safety performance level under the relevant earthquake scenario. The damage conditions of the reinforced concrete beams, columns, and shear walls in the building were investigated. Despite the presence of a partial basement floor, low concrete compressive strength, variations in column cross-section dimensions between floors, insufficient stirrup confinement, and the improper implementation of 135-degree stirrup hook angles, these factors have nonetheless contributed positively to the building's overall performance. Except for one column located in the collapse region of the critical story, no other vertical load-bearing element was found within the collapse zones. Additionally, two beams were observed to fall within the severe damage region on the critical floor. However, due to the presence of a vertical structural element within the collapse region, the life safety performance level targeted for the DD2 earthquake level could not be achieved.

The seismic performance of existing reinforced concrete buildings can be assessed using linear and nonlinear methods. In this study, a nonlinear pushover analysis was applied to the building to achieve a more realistic evaluation. Since the conditions permitted by TBEC (2018) were not met, the analysis was conducted in a multi-mode framework. It is considered that the findings obtained from this study will guide future research on the design and performance evaluation of buildings with mixed slab systems and contribute to the literature in the field of earthquake engineering.

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Conflict of Interest

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Author Contributions

All of the authors made substantial contributions to conception and design, or acquisition of data, or analysis and interpretation of data; were involved in drafting the manuscript or revising it critically for important intellectual content; and gave final approval of the version to be published.

Data Availability

The datasets created and/or analyzed during the current study are not publicly available, but are available from the corresponding author upon reasonable request.

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