Case Studies in Construction Materials 16 (2022) e00931

Contents lists available at ScienceDirect

Case Studies in Construction Materials

journal homepage: www.elsevier.com/locate/cscm

Retrofitting technique effectiveness and seismic performance of multi-rise RC buildings: A case study

Turgay Cosgun [a], Baris Sayin [a,*], Baris Gunes [a], Atakan Mangir [b]

[a] *Department of Civil Engineering, Istanbul University-Cerrahpasa, 34320 Istanbul, Turkey*
[b] *School of Engineering and Natural Sciences, Istanbul Medipol University, 34810 Istanbul, Turkey*

ARTICLE INFO

Keywords:
RC Buildings
Retrofitting
Field study
Laboratory tests
Numerical analysis
Seismic performance

ABSTRACT

This study aims to determine the effectiveness of a retrofitting technique based on the seismic performance analysis results comparison between two inactive multi-storey reinforced concrete (RC) buildings for a long period. Their seismic performance is determined according to the latest Turkish Building Earthquake Code (TBEC-2018) recommendations. The main difference between the examined buildings is the reinforcement (retrofit) technique applied to one of them with additional new shear walls. The whole process was demonstrated with a code-based approach, and the results were compared after the performance evaluation. The structural system geometry, member dimensions and plan layout of the buildings are checked with the original design projects by making on-site measurements. The material properties of the buildings are determined by field studies and laboratory studies. Buildings are modelled with finite element software, and numerical simulations are performed to determine the seismic performance levels according to existing code. As a result of the analysis, the effectiveness of the retrofit approach was examined by comparing the structural behaviour and seismic performance levels of both buildings. The results showed the importance of some parameters that should be taken into account in the retrofitting process with the addition of new shear walls to RC structures. While the applied retrofit process increased the stiffness of the structure and decreased the displacements, the results indicated a failure with a shear-critical condition.

1. Introduction

In Turkey, reinforced concrete structures are preferred more than other types due to economic reasons in terms of both construction time, ease of construction and material accessibility. Reinforced concrete material will inevitably experience a decrease in durability and strength as a result of exposure to environmental conditions. On the other hand, due to these deficiencies, aesthetic and structural improvements are made by the users over time. Such interventions ensure the structural safety level and comfort of use of building, as well as prolonging its economic life. The design phase of reinforced concrete structures, as in the design of other types of structures, is carried out, taking into account the intended use. These buildings are used functionally in accordance with their predetermined functions. Some structures in the building stock could not be put into service after their construction. This is mostly due to legal issues like disputes between landlords and users. Accordingly, these buildings can be out of use for many years without the need for maintenance. In this vulnerable period, these buildings may also be directly exposed to the harmful effects of human vandalism and

* Corresponding author.
E-mail address: barsayin@iuc.edu.tr (B. Sayin).

https://doi.org/10.1016/j.cscm.2022.e00931
Received 4 October 2021; Received in revised form 2 February 2022; Accepted 4 February 2022
Available online 7 February 2022

external environmental effects. Structural damages occur mainly during this dormant period. Among these damages, the most obvious ones are the significant decrease in concrete strength and the effect of corrosion on rebars. These deficiencies will severely affect the structures in earthquake-prone areas, especially after seismic activity. In this context, the most important before reusing these buildings is the structural safety determination on the earthquake performance levels. In the determination of the seismic performance level of such buildings, determining the existing strength of the concrete and considering the corrosion effects on the reinforcements in seismic analyses are vital to reveal the actual structure behaviour. While the current performance level of the building is found after the analyzes according to the current regulations, the retrofitting process is started for the buildings that have insufficient structural performance but can be strengthened by taking into account the existing material properties, strength level and total cost.

There are many studies in the literature to determine the earthquake performance of existing RC buildings. Chaulagain et al. [1] focused on the seismic behaviour determination of four different 3-storey RC residential buildings. Buildings are subjected to static pushover loading with various load patterns. In addition, nonlinear time history analysis and adaptive pushover analysis methods are also performed on these buildings. Cherifi et al. [2] also investigated the seismic performance of existing RC buildings. Capacity curves are provided for the buildings using the pushover analysis method. Mosleh et al. [3] proposed a methodology for the seismic assessment of existing buildings. In this study, two buildings are analyzed using pushover and time-history analysis methods. These analyses are made by considering the earthquakes with different return periods and seismic demand levels compared with the seismic codes' limits. Halder and Paul [4] suggested the seismic performance of a low-rise RC building. They used nonlinear static analysis to obtain the capacity curve of the building. The results showed that the damage level of the building ranged from moderate to severe damage. Wahyuni [5] studied the evaluation of the structural performance of RC buildings under seismic loadings. He recommended whether the buildings are still in a state of immediate occupancy, life safety, or collapse prevention. On the other hand, Melani et al. [6] studied the seismic assessment of low-rise RC buildings. Analyses are carried out with capacity design approaches, taking into account shear and bending strengths. Sobaih and Nazif [7] performed seismic performance analyses on existing RC buildings. They presented a methodology that has a significant impact on the seismic behaviour of the buildings. Ghobarah et al. [8] performed nonlinear static and dynamic analyses on RC frames. They adopted a probabilistic approach in which many artificially generated ground motion records are used as input motion for the structure. The results showed the probability of different levels of damage expectancy when the frames were exposed to various levels of ground motion. Yakut [9] presented another methodology to determine the seismic performance of existing RC buildings. The presented method classifies the buildings as safe or unsafe; this means that the building may not be seriously damaged or the life safety performance level would not be achieved. El-Betar [10] studied the seismic fragility of buildings to propose an approach to simulate the seismic behaviour of existing RC buildings. He conducted a pushover analysis to determine the performance level of the buildings. Hosseini et al. [11] responded to how to ensure the life safety performance level in RC buildings. For the seismic performance evaluation, in addition to plastic hinges formation and distribution, roof acceleration, displacement, and base shear values were calculated. The performances of the examined buildings exceeded life safety level and reached the collapse level under different earthquake records.

Numerous studies have been carried out to investigate the retrofitting procedures on RC buildings. Chaluagain et al. [12] examined four different structures using various retrofitting methods. They used the addition of new shear walls that support the structural system with steel elements and increased the cross-section size of the structural members. Yalciner and Hedayat [13] analyzed an existing RC building by comparing various retrofitting methods. Steel elements, along with shear wall insertion and cross-section increasing techniques, have determined the most efficient support for the structures. On the other hand, Ismaeil et al. [14,15] studied the addition of new shear walls in eight- and four-storey reinforced concrete buildings. At the end of their analyses, they determined that this retrofitting technique, which emphasises a minimum wall thickness of 15 cm, is one of the most suitable alternatives for similar structures. Hueste and Bai [16] studied the performance level of a five-storey reinforced concrete building built-in 1980. In order to retrofit the building that does not provide the required performance level, they tried several retrofitting methods such as the addition of new shear walls or steel members and increasing the cross-section dimensions. In addition, they determined that the shear wall attachment technique was superior to the others in terms of increased success in structural performance. Ghobarah et al. [17] studied the effects of retrofitting strategies on the structural performance for RC columns using the pushover analysis method. They emphasised the importance of a strength increase in vertical structural members on the overall structural performance of buildings. Altun et al. [18] investigated the damages in an RC building after the Marmara Earthquake (1999) and determined the most appropriate retrofitting method for this structure. Inoue and Youcef [19] examined three RC structures with different functions: residence, hospital, and school buildings. They determined the seismic behaviour of the buildings and prepared a retrofit plan according to the function of the buildings. Rocha et al. [20] evaluated retrofitting applications in terms of efficiency and feasibility. For this purpose, they analyzed the effect of retrofitting on the seismic behaviour of the structures by comparing the results obtained from different numerical methods with the experimental studies regarding the retrofit of RC frames. Varum et al. [21] aimed to evaluate RC buildings' seismic performance and determine the most appropriate reinforcement method. In this context, they analyzed and determined the performance level of four existing RC buildings and suggested the most appropriate strengthening method for these structures. On the other hand, Dogan et al. [22] evaluated the performance level of a four-story RC building. They implemented two methods to reinforce this building: new shear walls and steel cord reinforcement. They evaluated the efficiency of these two methods by comparing the performance level of the two reinforced states of the building.

In the literature, there are many studies that strengthen the RC buildings and increase the stiffness and strength. On the other hand, there are not enough studies in the literature focusing on the possible adverse effects of retrofitting procedures. Therefore, it is vital to reveal all the parameters that should be considered in the retrofit of reinforced concrete structures. Applied strengthening may not always help the building achieve its intended performance level. Both design errors and faulty manufacturing of the elements can cause situations where the building does not reach the targeted performance level and causes a more critical situation than its unreinforced condition. The authors wanted to include a new case study in the literature to demonstrate the potential adverse effects of new shear walls added to a reinforced concrete structure for retrofit purposes.

In this study, seismic performance evaluation of two identical RC buildings that have not been active for many years has been made. Recently, one of these buildings has been retrofitted with new shear walls. The analysis results of these buildings were compared to determine the effectiveness of this retrofitting technique. The seismic performance levels of these buildings are determined through field studies, laboratory studies and numerical simulation models. Before any work, fieldwork and laboratory tests are conducted to determine the existing concrete strength and rebar arrangement of the old and new structural members. Then, 3D building analysis models are prepared in finite element software [23]. Next, nonlinear static analyzes of the buildings are made based on the rules presented in the latest Turkish Building Earthquake Code (TBEC-2018) and data collected through field studies and material tests. Finally, the seismic performance levels of the buildings are calculated, and the analysis results are compared to evaluate the effectiveness of the retrofitting procedure applied.

2. Seismic retrofitting techniques

RC structures may experience losses in stiffness and strength over time due to the effects they are exposed to. In these scenarios, the option of retrofitting comes to the fore to restore the lost stiffness and strength. In addition, due to the updates in the seismic codes, the need for retrofitting becomes a necessity. After determining the seismic performance level of the building, adopting an efficient retrofitting strategy ensures an adequate structural safety reserve and avoids the relatively high cost of rebuilding [24].

Retrofitting applications are broadly classified into two main parts: focusing on structural members and/or structural systems. Element-based retrofitting consists of applications made on damaged or undamaged structural members. The process of restoring the lost strength by repairing damaged elements or increasing the strength of undamaged elements can be defined as element-based retrofitting. Interventions to the building carrier system can be defined as structural system-based retrofitting. It includes adding new structural members, retrofitting the connections of structural members, and reducing the system mass and seismic isolation approaches.

Element-based retrofitting is mostly applied on columns, beams and partition walls [25–27]. The purpose of these operations on columns is to increase the shear, bending and compressive strengths. In this context, increasing the element cross-section size is a technique frequently applied to columns. The strength and ductility of existing columns can be increased significantly with the Jacketing technique, which provides additional material "Jacket" around the column. In the current practice, the plaster/mortar layer on the column surface is peeled off to ensure the adherence between the existing concrete and the new jacketing material. The retrofitting jackets can be made of steel or reinforced concrete material. A negative effect of this technique is an increase in the total mass of the structure, which will attract higher seismic loads. Another preferred technique in column retrofitting is fibre-reinforced polymer (FRP) wrap application. In this technique, the ductility of the columns can be significantly increased by the shear and compressive strengths as well as confinement effects. This technique is also used for increasing the bond strength in cases where the lap splice length of the rebars is insufficient [28]. Due to its practicality, the FRP application has become popular over the other two jacketing techniques. In this approach, the columns are wrapped by FRP materials and epoxy adhesives, resulting in an increase of concrete strength by up to 25% [29]. The efficiency of wrapping concrete columns with FRP considering continuous loads was investigated in the experimental study of Micelli et al. [30].

Element-based retrofitting methods are also applied in beams. The primary aim is to increase the shear strength and ductility of beams. One of the techniques applied is to add new stirrups on the facades of the beams. Another technique is to wrap the beams with FRP materials [31]. Shear strength and ductility of the beams can be increased significantly with these techniques. Increasing the number of FRP layers and applying it to the three facades of the beam provide additional efficiency in strength and ductility [32].

Retrofitting of partition walls is defined under element-based retrofitting methods. The main aim is to increase the shear strength and stiffness of the partition walls between the RC frames. One technique is the application of steel wire mesh, which includes the anchorage layers of wire mesh and mortar layers applied on both facades of the wall [33]. As a disadvantage to the mortar application, the increased overall mass of the structure can attract higher seismic loads. Fibre-reinforced polymers, which can be classified as lightweight, easy-to-use and corrosion-free materials, are also used as another technique to obtain higher strength and ductile behaviour [34].

The most commonly used techniques in structural system-based retrofitting include adding new shear walls and new frames, reducing the mass of the system, seismic isolation approaches, and steel braces [35–39]. Considering Turkey's reinforced building stock, the most preferred approach to increase the strength and stiffness of buildings with insufficient capacity is the addition of new shear walls to the load-bearing system. In general, an increase in the lateral stiffness and strength of the structural systems is expected from new shear walls. RC walls are positioned in the weaker direction of the structural system for the targeted performance level with this type of retrofitting application, as they significantly prevent the displacements due to earthquake in a direction. Adding RC shear walls to the frame plane shortens the natural vibration period, increases the lateral stiffness of buildings, and reduces the floor displacements [40]. However, the addition of new shear walls without proper design guidelines or manufacturing process adaptation can result in higher seismic forces with increased structure mass, and the building may fail in a shear-critical situation. Another technique is to add a new frame system into the existing one. In this technique, the sharing of the seismic forces is provided. The composite behaviour of new and old frames to ensure load transfer is the most critical parameter for the effectiveness of this technique. The combined mass of new and old frames can attract higher seismic loads as a disadvantage to efficiency. Reducing structural system mass is an indirect approach in structural system-based retrofitting. The mass of the structural system directly affects the angular frequency value. Thus the decrease in mass and the natural vibration period of the structure reduces the seismic forces acting on the structural system. Demolishing some floors, constructing a lighter roof instead of the existing roof, removing additional weights such as water tanks, replacing heavy balconies and partition walls with lighter elements are among some applications to reduce the mass of the structural system. In the seismic isolation approaches, structural system-based retrofitting can be made by placing rubber bearings flexible in the horizontal direction with a specific displacement capability. Seismic isolation aims to decrease the seismic effects on the structure by increasing the period and damping ratio of the building. Therefore, there are significant reductions in the relative floor drifts, floor accelerations and soil-structure interaction. Seismic isolation application in buildings can reduce storey drifts up to 50% and shear forces in vertical structural elements by up to 30% [41]. Structural system-based retrofitting can also be achieved by adding steel braces to the frame plane, thereby increasing strength and ductility under seismic loads without increasing the mass of the structure. In some cases, while the stiffness of the structural system is increased, the cross-section size of some members to which the steel braces are attached is also increased to prevent brittle failure due to internal force increases [42].

The main aim of the retrofitting techniques mentioned above for reinforced concrete buildings is to increase the strength and stiffness of the structures. On the other hand, it should be shown that the preferred element-based or structural system-based techniques improve the structural behaviour of buildings by taking all parameters into account. In other words, the efficiency and suitability of the preferred technique should be determined by numerical simulation models before implementation. In this process, additional torsion effects should be avoided by reducing the distance between the centres of mass and the rigidity. In addition, the building should not be made extremely rigid to prevent possible brittle behaviour. If these parameters are not considered in an appropriate retrofitting design process, the structure may be damaged instead of improving the structural strength, resulting in economic losses and worse structural safety. If the targeted seismic performance level cannot be achieved after retrofitting the structure, a preferred technique could not be assumed as efficient.

3. Field surveys and laboratory studies

The examined buildings are located in the Büyükçekmece region of Istanbul, Turkey. There are four similar 10–11 storey multi-rise RC buildings in the study area, Building A, B, C, and D (Fig. 1). These buildings have been idle for 30 years since the construction, and one of them (Building C) was recently retrofitted. Buildings have identical properties in terms of plan geometry and structural system layout. Building A and retrofitted Building C are examined in this paper to evaluate the effectiveness of the retrofitting technique applied (Figs. 2 and 6). Detailed field studies, laboratory tests and numerical analyses are carried out for these two buildings. In field surveys, on-site measurements are made, including the positions and dimensions of the structural elements, and the conformity of the structural system to the design project is checked. The existing concrete strength and rebar arrangement of old and new structural elements are examined by sampling and laboratory tests. In addition, damage on the structural members is visually checked by on-site observations.

Fig. 1. Aerial view of the buildings A to D.

Fig. 2. Overview of Building A.

3.1. Preliminary study and visual inspection

This section presents the preliminary study results with on-site visual inspections and defect investigations. The buildings surveyed were built in 1990 and have remained inactive ever since. Building C, with the same plan geometry, was retrofitted by the addition of new RC shear walls in 2005 while retaining the original form of Building A.

3.1.1. Preliminary study and visual inspection of Building A

Building A consists of 11 floors, including a basement, ground floor and nine standard floors (Fig. 2). The primary structural system of the building consists of an RC frame with shear walls. No discontinuities are observed in the structural members throughout the building. Building A sits on an area of 432 m^2 with 25.9 m x 16.2 m plan dimensions and has strip foundations in both directions. The storey height is measured as 2.80 m on all floors.

Table 1 shows the measured dimensions of the structural members in Building A. The results of these measurements are compared with those specified in the original design project of the same building. It has been determined that measured values are suitable with the original design project. A typical floor plan and strip foundation plan from the design project is shown in Fig. 3. The interior views of the building are given in Fig. 4. In the investigations, swelling and spalling were observed on the mortar and paint layers of some columns, beams, flooring, and wall surfaces due to the effect of moisture. Examples of some defects in the building are given in Fig. 5. The loss in the diameter of some rebars due to corrosion and deterioration of concrete in some walls are shown in Fig. 4. As a result of the moisture effects, swelling and spalling in the mortar and paint layers of some beams, columns and walls are shown in Fig. 5.

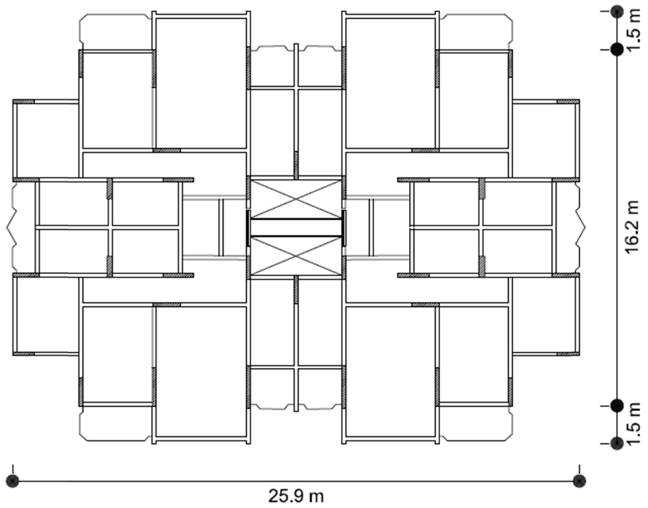
3.1.2. Preliminary study and visual inspection of Building C

Building C consists of 10 floors, including a basement, ground floor and eight standard floors (Fig. 6). The building's primary structural system, plan layout and floor heights are the same as Building A. No discontinuities are observed in structural members throughout the structure. The differences between the two buildings are the retrofitting process in Building C and the total number of floors (A-11, C-10). It is determined that the building is retrofitted with new reinforced concrete shear walls placed over the basement floor and at the four sides and corners of the building. As the retrofitting procedures were completed probably without any design project, on-site measurements helped to determine the length and thickness of the new walls. The new shear walls around the basement are 20 cm thick, while those in the four corners are 55–60 cm thick. It has been determined that the measured values for the dimensions of the existing structural members are in accordance with the original design project (Fig. 3). The interior views of the building are given in Fig. 7.

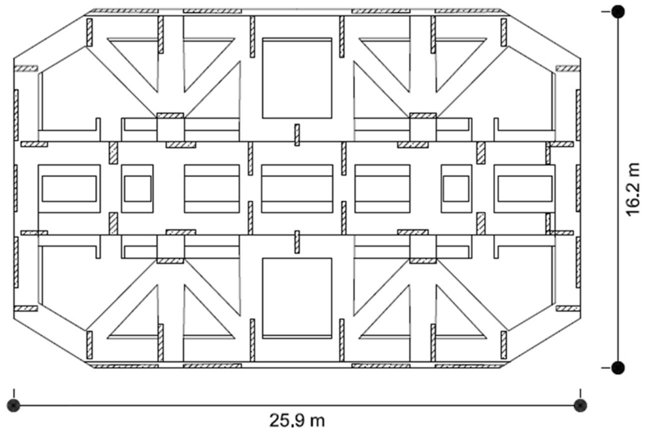
Table 1
Dimensions of the structural members (Building A).

Story	Columns	Shear wall (thickness)		Beams			Slab*	The width of foundations** (bottom/top)		
Basement	40 × 105	20	25	20 × 50	20 × 60		12	100/100	100/130	120/150
Ground	40 × 105	20	25	15 × 60	20 × 40	20 × 60		120/160	120/210	130/130
1–9	35 × 105 20 × 100	20		15 × 60	20 × 60			150/150	200/200	

Units in cm. *plate, **height: 100 cm



(a)



(b)

Fig. 3. Typical floor (a) and strip foundation (b) plans of Building A.

In the examinations made, swelling and spalling were observed in the mortar and paint layers of some columns, beams, flooring, and wall surfaces due to the effect of humidity. In addition, corrosion effects are observed on the rebars of some structural elements after concrete stripping. No significant damage was found in either building. Examples of some defects found in the building are given in Fig. 8. Swelling and spalling due to the effect of moisture in the mortar and paint layers of some beams, columns, walls and floors are shown in Fig. 7. In the slab shown in Fig. 8, losses in diameters of some rebars due to corrosion and deterioration of concrete are shown.

3.2. Laboratory studies

In order to determine the concrete compressive strengths of Buildings A and C, a sufficient number of core samples are taken from the buildings in light of the relevant standards [43,44] (Fig. 9a and 9d). Samples are taken from both the original members and new shear walls. The rebars are determined by taking into account the rules given in TBEC-2018 and TS708 [45,46]. Two different methods are adopted to determine the structural rebar arrangement in the rebar detection procedures. The first is rebar scanning, using X-ray rebar scanners to obtain the rebar diameter and spacing as shown in Fig. 9b and 9e. The second is concrete stripping, as in Fig. 9c and 9f, which allows visual inspections and measurements to observe and measure the spacing, diameter, and corrosion level of rebars. Raw after-test data of the test results are also presented in a technical report of an authorised structural test laboratory [47].

3.2.1. Core sampling and testing for concrete strength

Twenty-four core samples were collected from the existing vertical structural members in Building A, and twenty-two core samples from the existing vertical structural members in Building C were collected to determine the unconfined concrete compressive strength. Three samples are taken from both basement and ground floors, and two samples from each standard floor. In addition, four samples (two from the basement and ground floors) of the new shear walls added for retrofitting were collected around Building C. The

Fig. 4. Interior views from Building A.

Fig. 5. Example of some defects in Building A.

locations of the samples taken from both buildings and the compressive test results are presented in Tables 2 and 3, respectively. The compressive test results of the samples taken from the new shear walls are presented in Table 4. Since the height/diameter ratio of cylindrical core samples is 1 (90 mm/90 mm), the strengths of the tests are considered as cubic strength values. The code's approach and formulation are applied to find the existing cylindrical unconfined concrete strength of the buildings and the new shear walls in Building C [43,44]. The approach followed in code to find the existing cylindrical unconfined concrete strength is basically based on the conversion of unconfined cubic strength value (f_{cu}) by "0.85 x f_{cu}" multiplication, and the related calculations are also presented in Tables 5 and 6.

Examination of these results to find the existing concrete strength of the buildings is given in Tables 5 and 6. The existing concrete

Fig. 6. Overview of Building C (retrofitted with shear walls).

Fig. 7. Interior views from Building C.

strength of Building A has been calculated as 12.50 MPa. The existing concrete strength of structural members and new shear walls of Building C, are calculated as 12.24 MPa and 28.49 MPa, respectively.

3.2.2. Rebar detection

In total, 44 (4/each floor) vertical structural members and 22 (2/each floor) beams were scanned with rebar scanners in Building A, and 40 (4/each floor) vertical structural members and 20 (2/each floor) beams were scanned with rebar scanners in Building C to obtain rebar arrangement data. In addition, four new shear walls were scanned on both the basement and ground floors of Building C. Details of the rebar arrangement of some members are presented in Tables 7 and 8.

The concrete cover of one beam and three vertical structural members on each floor is stripped for visual inspections and measurements. With this method, a total of 33 vertical structural members and 11 beams were inspected in Building A, and 30 vertical structural members and 10 beams in Building C. In addition, in Building C, the concrete cover of these new members was stripped, and

Fig. 8. Example of some defects in Building C.

Fig. 9. Core sampling and rebar detection on Building A (a-c) and Building C (d-f).

four new shear walls were examined on both the basement and ground floors. A low corrosion damage level was observed in existing members (Table 9 and Table 10). It is determined that the inspected rebars for the existing structural members of both buildings were made of plain bars with S220 (f_y:220) steel type. The rebars of the new shear walls were made of ribbed bars with S420 (f_y:420) steel type [46]. The observed rebar arrangement in terms of locations, spacing and diameter for existing members is consistent with the

Table 2
Compressive test results of the collected samples from Building A.

Storey	Sample	Compressive (MPa)	Storey	Sample	Compressive (MPa)
Basement	01	20.56	4	13	22.67
	02	24.82		14	17.18
	03	19.95	5	15	12.00
Ground	04	15.46		16	17.91
	05	11.97	6	17	15.11
	06	12.08		18	25.47
1	07[a]	10.32	7	19	21.13
	08	16.15		20	21.40
2	09	13.69	8	21	10.36
	10	17.80		22	15.34
3	11	12.20	9	23	20.02
	12	23.09		24	11.55

[a] Neglected in the calculations since the code specifies that a group of samples' minimum value should not be less than 75% x average of the rest.

Table 3
Compressive test results of the samples collected from existing members of Building C.

Storey	Sample	Compressive (MPa)	Storey	Sample	Compressive (MPa)
Basement	01	16.65	4	13	18.80
	02	13.27		14[a]	7.86
	03	13.92	5	15	18.10
Ground	04	22.48		16	13.04
	05	17.99	6	17	22.28
	06	13.92		18	18.45
1	07	17.95	7	19	15.23
	08	22.40		20	14.61
2	09	23.70	8	21	10.74
	10	10.89		22	22.25
3	11	13.35			
	12	15.80			

[a] Neglected in the calculations since the code specifies that a group of samples' minimum value should not be less than 75% x average of the rest.

Table 4
Compressive test results of the samples collected from new shear walls of Building C.

Storey	Sample	Compressive strength (MPa)
Basement	23	33.87
	24	33.33
Ground	25	40.35
	26	38.47

Table 5
Compressive concrete strength examination of Building A.

$f_{c,cube,av.}$	*σ	$f_{c,cube,1} = f_{c,cube,av} - \sigma$	$f_{c,cube,2} = 0.85 \cdot f_{c,cube,av.}$	$f_{c,cube,uc}$	$f_{c,cyl,uc} = 0.85 \cdot f_{c,cube,uc}$
17.30	4.47	12.83	14.71	14.71	12.50

All units: MPa, *σ: standard deviation, uc: unconfined, av: average, cyl:cylinder
$f_{c,cube,uc}$ is the highest one of $f_{c,cube,1}$ and $f_{c,cube,2}$

Table 6
Compressive concrete strength examination of Building C.

Members	$f_{c,cube,av.}$	*σ	$f_{c,cube,1} = f_{c,cube,av} - \sigma$	$f_{c,cube,2} = 0.85 \cdot f_{c,cube,av.}$	$f_{c,cube,uc}$	$f_{c,cyl,uc} = 0.85 \cdot f_{c,cube,uc}$
Existing	16.94	3.88	13.06	14.40	14.40	12.24
New	36.51	2.99	33.52	31.03	33.52	28.49

All units: MPa, *σ: standard deviation, uc: unconfined, av: average, cyl:cylinder
$f_{c,cube,uc}$ is the highest one of $f_{c,cube,1}$ and $f_{c,cube,2}$

Table 7
Rebar scanning results of some existing structural members in Building A.

Member type	Rebars		
	Longitudinal	Stirrup	Stirrup spacing (cm)
Column	ϕ14, ϕ16	ϕ8, ϕ10	20–35
Shear wall	ϕ14, ϕ16	ϕ8, ϕ10	14–28
Beam	ϕ12, ϕ14, ϕ16	ϕ8	16–37

Table 8
Rebar scanning results of some existing and new structural members in Building C.

Member type	Rebars	
	Longitudinal	Stirrup diameter/spacing (mm/cm)
Column	6ϕ16	ϕ8/22
Shear wall[a]	ϕ16/8	ϕ10/8
Beam	6ϕ14	ϕ8/22

Longitudinal bars representation: quantityϕdiameter/spacing (mm/cm)
[a] New member

Table 9
Concrete stripping results of some existing structural members.

Member	Dimensions (cm)	Longitudinal rebar diameter (mm)		Stirrup rebar diameter (mm)	
		Project	Measured	Project	Measured
Shear wall	20/100	ϕ16	16.31	ϕ8	8.52
	20/100	ϕ16	16.30	ϕ8	8.70
Column	40/105	ϕ16	16.07	ϕ8	8.74
Beam	20/60	ϕ14	13.01	ϕ8	8.78

Table 10
Concrete stripping results of some existing and new structural members.

Member	Dimensions (cm)	Longitudinal rebar diameter (mm)		Stirrup rebar diameter (mm)	
		Project	Measured	Project	Measured
Shear wall	20/100	ϕ16	15.97	ϕ8	9.38
	65/135[a]	–	16.93	–	9.87
Column	65/135	ϕ16	16.38	ϕ8	9.40
Beam	20/60	ϕ14	14.78	ϕ8	8.86

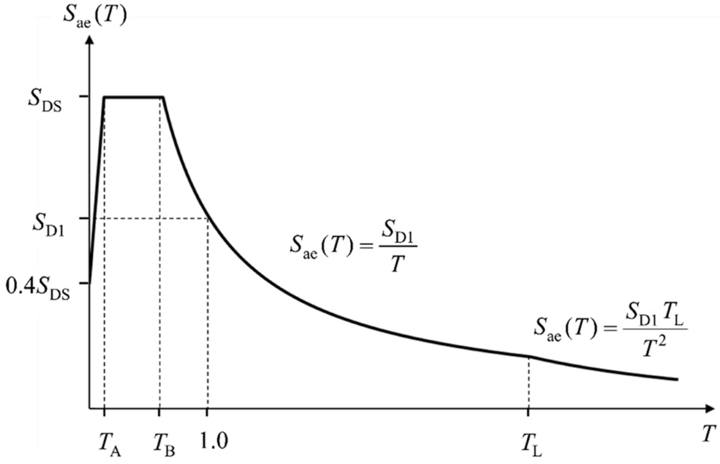
[a] New member.

original design project. Since the applied retrofitting procedure was not applied with a design project, the data from this study are used to model these new shear walls in the finite element software.

4. Analysis criteria

In finite element models, soil and earthquake parameters are determined by considering TBEC 2018 criteria [45]. Soil parameters are taken from the field study report [48] prepared by drilling works in the building area. Accordingly, the local soil class of the building is determined as “ZC”. Elastic acceleration spectrum characteristic values are selected according to the soil classification. Earthquake map spectral acceleration coefficients (S_S=1.142 g and S_1 =0.308 g) are determined based on the building locations in Turkey Earthquake Hazard Maps [49]. Then, local soil impact factors (F_s =1.2 and F_1 =1.5) are calculated based on map spectral acceleration coefficients and local soil class information as shown in the code. Accordingly, the spectral acceleration coefficients are found by multiplying the map spectral acceleration coefficients with the local soil impact factors as described in TBEC 2018 (S_{DS}=1.3704 and S_{D1}=0.462). Lateral elastic spectral acceleration values [S_{ae} (T)] and the horizontal elastic acceleration spectrum corner periods (T_A and T_B) are calculated based on the values calculated above and the criteria given in the regulation (Fig. 10). Accordingly, the lateral elastic acceleration spectrum is formed.

In the numerical analyses, the target performance level of these buildings is evaluated considering the DD-2 earthquake ground motion level (475 years return period, standard design) presented in TBEC (2018). Since the examined buildings are planned as residences, they are classified as “Other Buildings” in the code. Accordingly, the building’s usage class (BUC) is specified as “3”, and the building importance factor (I) is determined as “1”. The earthquake design class of the building has been determined as “1” considering



$$S_{ae}(T) = \left(0.4 + 0.6\frac{T}{T_A}\right) S_{DS} \qquad (0 \le T \le T_A)$$

$$S_{ae}(T) = S_{DS} \qquad (T_A \le T \le T_B)$$

$$S_{ae}(T) = \frac{S_{D1}}{T} \qquad (T_B \le T \le T_L)$$

$$S_{ae}(T) = \frac{S_{D1} T_L}{T^2} \qquad (T_L \le T)$$

$$T_A = 0.2\frac{S_{D1}}{S_{DS}} \quad ; \quad T_B = \frac{S_{D1}}{S_{DS}}$$

$$T_L = 6 \text{ s}$$

Fig. 10. Elastic spectrum criteria [45].

the “S_{DS}” and usage class of the building. Building height class (BHS) is determined as “4” considering the earthquake design class and height of the buildings. Analysis parameters in the study are given in Table 11.

The performance analysis of the building is made by considering the criteria in TBEC 2018 Section 15. Since sufficient data could be obtained from the structures, “Comprehensive knowledge level” is chosen, and the existing material strength values are used without applying any reduction factors. The “nonlinear pushover analysis” method is used in the analyses. In this method, seismic load reduction factors [R_a(T)] are not applied to the spectral acceleration values. Fibre hinges are assigned to vertical structural members, and rotational lumped hinges are assigned to beams. The maximum and minimum strain values are obtained directly from the fibre hinges and calculated from the curvature results of lumped hinges. It is noticed that the examined buildings should meet the “Controlled Damage Level (CD)” performance level considering the DD-2 earthquake. As stated in the code, a strain-based evaluation approach has been adopted in the evaluation. At the end of the analysis, the damage in structural elements is achieved through the internal strains, where the building was pushed to its target performance point.

Section damage limits and regions in the strain-based assessment approach are given in Fig. 11. Three damage limits are defined for the ductile members, limited damage (LD), controlled damage (CD) and pre-collapse damage (PC). The limited damage level corresponds to a limited plastic behaviour of a section. The controlled damage level corresponds to a permissible plastic behaviour of a section. The pre-collapse damage level corresponds to a high-level plastic behaviour of a section. This approach is not applicable to brittle members.

The performance level of the reinforced concrete buildings is determined according to the criteria presented in *Section 15.8.4*. The related criteria are summarised below;.

Buildings that meet the following conditions are considered to ensure the Controlled Damage (CD) Performance Level, provided that the elements with a brittle failure, if any, are strengthened:

- (1) At any floor, maximum 35% of the beams (excluding the secondary beams that are not part of the main structural system) and vertical structural members defined in (2) below can be in the Excessive Damage Region (ED) under the unidirectional seismic effects.
- (2) The total contribution of the vertical structural members in “ED” to the total story shear on each floor should be less than 20%. This limit could be increased to 40% on the top floor.

Table 11
Analysis parameters.

Parameter	Value
Local soil class	ZC
Spectral acceleration coefficients from the earthquake risk map (g)	$S_S = 1.142$, $S_1 = 0.308$
Earthquake ground motion level	DD-2
Building usage class (BUC)	3
Building importance factor (I)	1.0
Live load mass participation factor (n)	0.30
Earthquake design class	1
Building height class (BHS)	4
Local soil class	ZC

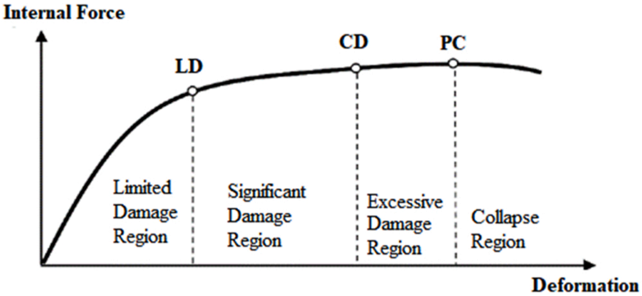


Fig. 11. Section damage regions [45].

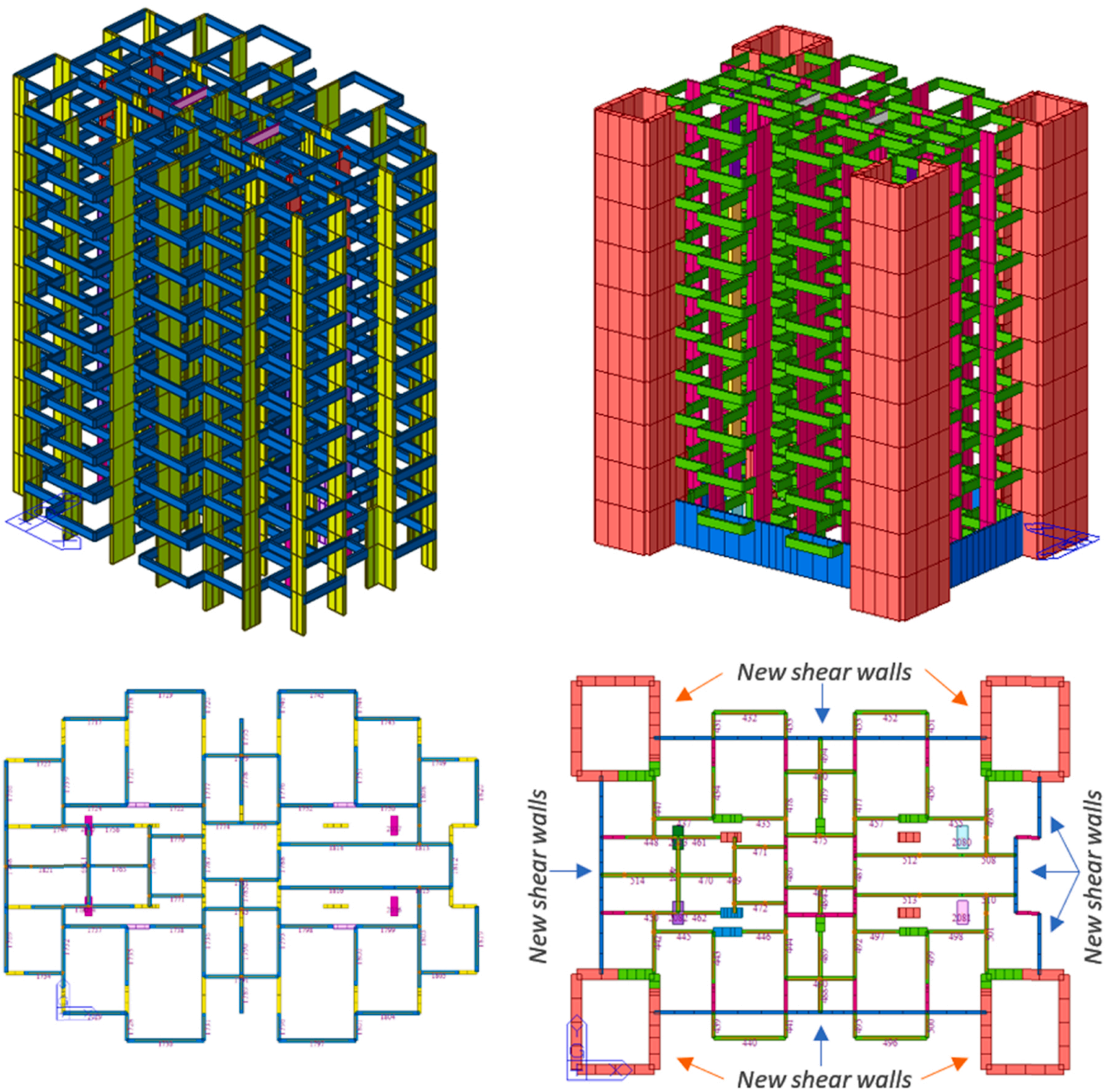


Fig. 12. Three-dimensional finite element model of Building A (left) and Building C (right).

- (3) The rest of the structural members must be in Limited Damage (LD) or Significant Damage (SD) Regions. However, if both the upper and lower parts of the vertical members exceed the significant damage (SD) limit, the ratio of total shear forces of these members to the total shear on any floor should not exceed 30%.

5. Finite element modelling and results

This section presents the details of the finite element models of both buildings and analysis results. Finite element models are prepared by considering the data obtained from field studies and laboratory tests. The building plan geometry of these buildings is the same except that new shear walls have been added in the finite element model of Building C. Also, Building A is one floor (standard story) higher than Building C. Fig. 12 shows the three-dimensional finite element models of Building A (left) and Building C (right), presented with their plan layout. The plan geometry of a standard story is shown under Building A, while the basement floor plan geometry is presented under Building C to show all shear walls added for retrofitting. Since the new shear walls were modelled in the finite element model of Building C, increased floor area and floor masses were taken into account in the analyses. In the finite element models, code rules are applied for material degradation of existing members and corrosion rate obtained for reduction of the diameter of rebars. The masses and weights of the slabs are defined in the software with the two-way floor loading option, and rigid diaphragm constraints are defined for each floor. The ground connections of the structural system are modelled as fixed supports. Beam-column connections are assumed as rigid connections. Dead and live loads are defined as two-way floor loading at each floor level as specified in the regulation. Since no significant damage was found in either building, which can affect the structural finite element modelling in the preliminary study and visual inspection, the models are formed without a physical damage definition on structural elements. First, modal analysis is performed for the seismic loads and response spectrum analysis is performed to consider the effects of different modes on the structure. Then, the structural behaviour is obtained as a result of the response spectrum loading. This modal response behaviour is converted to inertia loads at floor levels by an algorithm in the analysis software, and initial pushover loading is formed according to these inertia loads. The maximum and minimum strain values are obtained directly from the defined fibre hinges and calculated from the curvature results of lumped hinges. This method is accepted in the literature and gives very consistent results with experiments. In the study by Keun-Ho Cho [50], the results of nonlinear analysis of a column defined by fibre hinges in the software were compared with an experimental study [51]. The results were found to be quite compatible.

Some members are selected from both buildings, and a validation procedure is conducted based on the last stage of nonlinear analysis results. The selected members are modelled and analyzed in XTRACT software, which is an interactive and adaptive programme for the analysis of cross-sections step by step, started as an academic and research tool at the University of California at Berkeley [52]. In the last step of pushover loading for the selected members, compression strain of concrete fibre at the edge of cross-section and tension strain of the steel bar at the opposite edge of the cross-section are obtained from both the FEM and XTRACT models under the same moment and axial force. These values are compared to check whether our FEM models' results are valid. The obtained strain values are found in compliance with each other considering the same forces in the last step of pushover analysis. The comparison of results for the validation of four selected members from two buildings is given in Tables 12 and 13, respectively. The numerical difference is assumed to satisfy the validation if it is less than 3%.

Modal capacity curves of buildings are prepared using the pushover curves for both directions. The curves are drawn with the demand spectrum, and the performance point is determined according to the recommendations given in the code. At the performance point, the base shear/total weight and top displacement/total height, and demand spectrum and modal capacity curves (spectral acc./spectral disp.) under DD-2 earthquake are presented in Figs. 13 and 14 for two buildings. Lateral displacement contours in both directions at performance points of the buildings are shown in Figs. 15 and 16. When the behaviour of the two structures is examined, it is seen that Building A has reached the base shear values of 3645 kN and 3956 kN in X and Y directions, respectively.

On the other hand, Building C has reached the base shear values of 26,731 kN and 31,882 kN, respectively, in the same directions. The controllable performance points in the roof displacement values are changed from 220 mm to 65 mm, and from 180 mm to 60 mm levels in the X and Y directions when the displacement demands of two buildings are compared. It can be understandable that the new shear walls provide increased stiffness. The variation observed between the base shear values is 733% in the X direction and up to 806% in the Y direction. This increase shows the importance of ductility in the retrofitting design phase. Unfortunately, an increase in this amount strongly invites brittle failure mechanisms of reinforced concrete members. In other words, retrofitting is applied to

Table 12
Strain results obtained from FEM and XTRACT models.

Member	Building	FEM Model				XTRACT Model			
		Concrete Fibre[a]		Steel Fibre[b]		Concrete Fibre[a]		Steel Fibre[b]	
		Stress (kPa)	Strain (mm/mm)	Stress (kPa)	Strain (mm/mm)	Stress (kPa)	Strain (mm/mm)	Stress (kPa)	Strain (mm/mm)
2021	A	7278	0.0005014	13,960	0.0000698	7127	0.0004881	14,270	0.0000713
2017	A	12,200	0.0014120	220,000	0.0013670	12,240	0.0014310	220,000	0.0013490
2076	C	7583	0.0005458	57,660	0.0002883	7567	0.0005303	57,610	0.0002880
2078	C	8064	0.0005957	61,910	0.0003095	8094	0.0005846	61,450	0.0003072

[a] Compression.
[b] Tension.

Table 13
Comparison of strain results for the validation of FEM model.

Member	Building	Difference between stress/strain results of FEM and XTRACT models				Status
		Concrete Fibre[a]		Steel Fibre[b]		
		Stresses	Strains	Stresses	Strains	
2021	A	2.07%	2.65%	2.17%	2.10%	OK
2017	A	0.33%	1.33%	0.00%	1.32%	OK
2076	C	0.21%	2.84%	0.09%	0.10%	OK
2078	C	0.37%	1.86%	0.74%	0.74%	OK

[a] Compression.
[b] Tension.

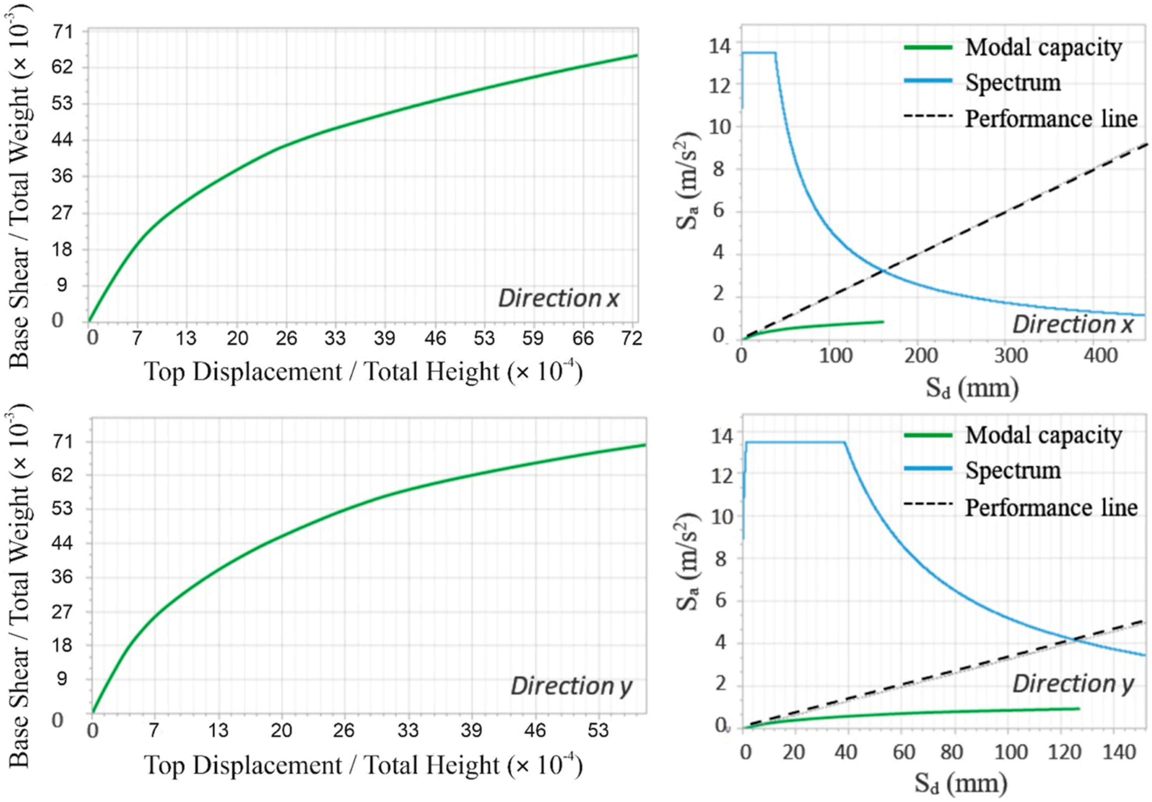


Fig. 13. Normalised base shear vs. top displacement and demand vs. capacity graphs of Building A.

increase the strength of the building, but the structure has become too rigid to overcome seismic effects and may face brittle failure. In the seismic performance evaluation of these two buildings, it is clear that a significant portion of the vertical structural members in Building C could not withstand excessive shear loads, and shear becomes critical. Accordingly, the building is subjected to brittle failure by becoming shear-critical.

Damage assessment of structural members, including all stories, is given in Tables 14 and 15 for beams and vertical structural members. Accordingly, when beams and vertical structural members in both buildings are compared in terms of damage level, brittle behaviour and target performance level, it is determined that the damage levels of beams decreased. While ductile behaviour is obtained with the retrofitting, brittle behaviour became dominant in vertical structural elements and had a negative effect. In addition, it is concluded that both buildings did not meet the "Controlled Damage" performance level for DD-2 earthquake with a 10% probability of exceedance in 50 years (return period 475 years) specified in TBEC-2018. The dominant numerical failure mode is “shear failure” for both buildings.

6. Conclusion and discussion

The advantages of reinforced concrete (RC) compared to other building materials under vertical and lateral loads are quite high in

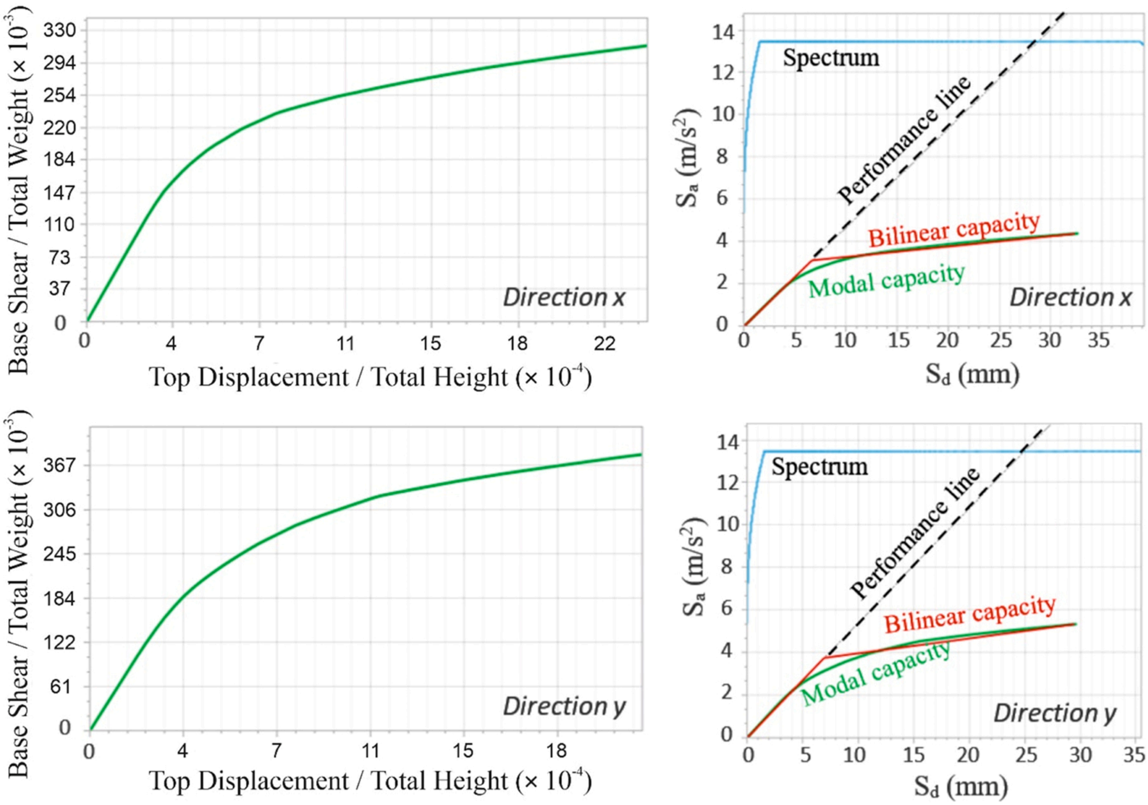


Fig. 14. Normalised base shear vs. top displacement and demand vs. capacity graphs of Building C.

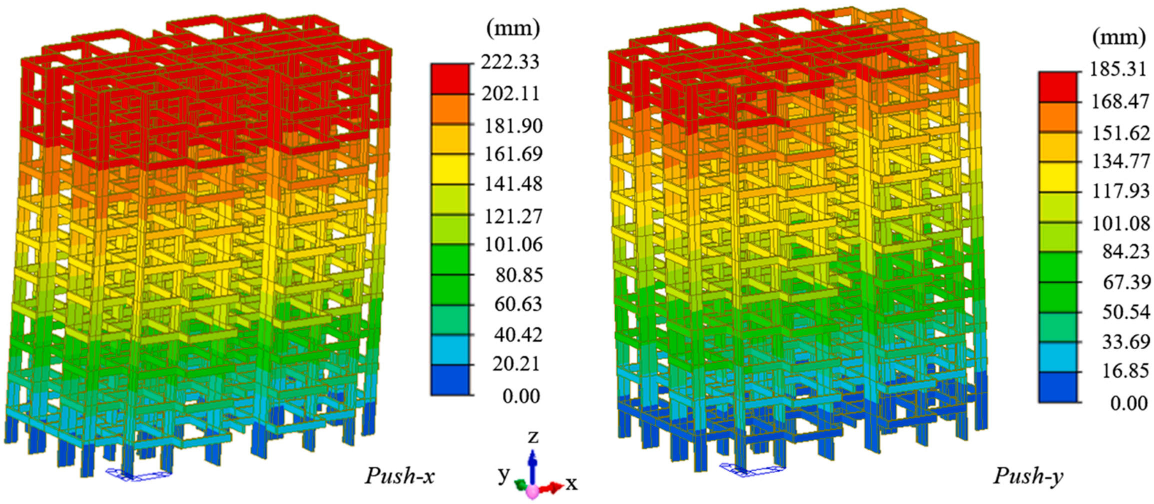


Fig. 15. Lateral displacements under the earthquake loading for Building A.

terms of construction time, material supply, cost and technology. For example, RC structures resist against shear force, bending and torsion moments, even under the impact of seismic loads, provided that the requirements of the codes are taken into account. The material strength and durability features of RC structures can degrade over time at the expense of structural safety. In particular, the stability of old and ageing buildings is in danger of collapse due to earthquakes if the current seismic code implementations are not appropriately applied. Demolishment, reconstruction, or retrofitting applications should be implemented for these structures according to current regulatory conditions to improve current performance levels. A retrofitting alternative is preferred in practical applications due to economic, social and legal conditions. Retrofitting can be applied based on system or element renewal systems. The

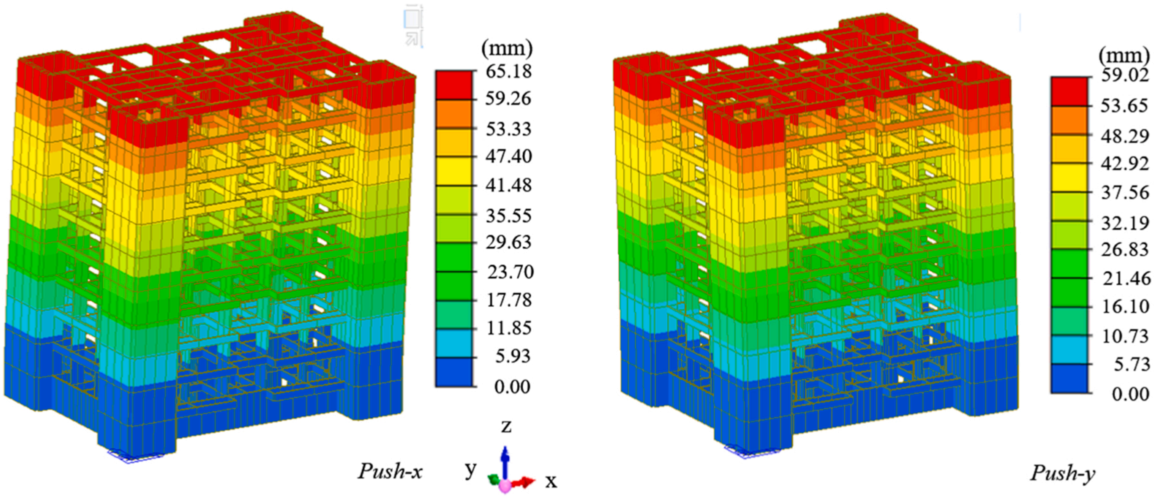


Fig. 16. Lateral displacements under the earthquake loading for Building C.

Table 14
Performance assessment results for beams in Buildings A and C.

Storey	Strain check		Brittle failure check		Target performance level (Controlled Damage)	
	Condition[a]					
	A	*C*	*A*	*C*	*A*	*C*
Basement	×	√	×	√	×	√
Ground	×	×	×	√	×	×
1	×	√	×	√	×	√
2	×	√	×	√	×	√
3	×	√	√	√	×	√
4	×	√	√	√	×	√
5	×	√	√	√	×	√
6	√	√	√	√	√	√
7	√	√	√	√	√	√
8	√	√	√	√	√	√
9	√	–	√	–	√	–

[a] Section 4 - Article (1)

Table 15
Performance assessment results for vertical structural members in Buildings A and C.

Storey	Strain check				Brittle failure check		Target performance level (Controlled Damage)	
	Condition[a]		Condition[b]					
	A	*C*	*A*	*C*	*A*	*C*	*A*	*C*
Basement	×	√	√	√	×	√	×	√
Ground	√	×	√	√	×	×	×	×
1	√	√	√	√	√	×	√	×
2	√	√	√	√	√	×	√	×
3	√	√	√	√	√	×	√	×
4	√	√	√	√	√	√	√	√
5	√	√	√	√	√	√	√	√
6	√	√	√	√	√	√	√	√
7	√	√	√	√	√	√	√	√
8	√	√	√	√	√	√	√	√
9	√	–	√	–	√	–	√	–

[a] Section 4 - Article (2).
[b] Section 4 - Article (3).

mix of old and new systems helps to handle seismic effects in terms of strength and stiffness parameters in addition to energy absorptions. Although the idea of additional RC elements for retrofitting helps to improve the structural behaviour, possible sources of error are explained in this study as a result of the lack of scientific rule enforcement.

The main purpose of this paper is to evaluate the effectiveness of the retrofitting technique and seismic performance of two mid-height inactive RC buildings based on numerical analysis supported by field studies and laboratory tests. A traditional approach to one of these buildings was recently retrofitted (addition of shear walls) to reduce the seismic action effect so as to reduce storey drifts. Finally, the seismic performances of these buildings are compared to determine the effectiveness of the retrofitting application. The approach showed that the storey drifts were reduced, and shear forces were partially transferred to the shear walls. It has also been observed that the retrofitted building does not meet the full seismic performance level. It was also concluded that significant shear forces are transferred on the shear walls in addition to storey drifts. The application of the retrofitting technique has reduced the structural ductile behaviour that may cause danger in the event of an earthquake. For best retrofitting technique practice, compatible new shear walls are recommendable to support existing frame systems to avoid irregularities and excessive torsional forces. One of the main points to consider is that the centres of mass and rigidity should overlap each other as much as possible so as not to cause excessive shear forces.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

References

[1] H. Chaulagain, H. Rodrigues, J. Jara, E. Spacone, H. Varum, Seismic response of current RC buildings in Nepal: a comparative analysis of different design/construction, Eng. Struct. 49 (2013) 284–294.
[2] F. Cherifi, M.-N. Farsi, S. Kaci, O. Belaidi, F. Taouchei-Kheloui, Seismic vulnerability of reinforced concrete structures in Tizi-Ouzou City (Algeria), Procedia Eng. 114 (2015) 838–845.
[3] A. Mosleh, H. Rodrigues, H. Varum, A. Costa, A. Arede, Seismic behavior of RC building structures designed according to current codes, Structures 7 (2016) 1–13.
[4] L. Halder, S. Paul, Seismic damage evaluation of gravity load designed low rise RC building using nonlinear static method, Procedia Eng. 144 (2016) 1373–1380.
[5] E. Wahyuni, Seismic vulnerability evaluation of existing R.C. buildings, Procedia Earth Planet. Sci. 14 (2015) 76–82.
[6] A. Melani, R.K. Khare, R.P. Dhakal, J.B. Mander, Seismic risk assessment of low rise RC frame structure, Structures 5 (2016) 13–22.
[7] M.E. Sobaih, M.A. Nazif, A proposed methodology for seismic risk evaluation of existing reinforced school buildings, HBRC J. 8 (3) (2012) 204–211.
[8] A. Ghobarah, N.M. Aly, M. El-Attar, Seismic reliability assessment of existing reinforced concrete buildings, J. Earthq. Eng. 2 (4) (1998) 569–592.
[9] A. Yakut, Preliminary seismic performance assessment procedure for existing RC buildings, Eng. Struct. 26 (2004) 1467–1481.
[10] S.A. El-Betar, Seismic vulnerability evaluation of existing R.C. buildings, HBRC J. 14 (2) (2016) 189–197.
[11] M. Hosseini, B. Hashemi, Z. Safi, Seismic design evaluation of reinforced concrete buildings for near-source earthquakes by using nonlinear time history analyses, Procedia Eng. 199 (2017) 176–181.
[12] H. Chaluagain, H. Rodrigues, E. Spacone, H. Varum, Assessment of seismic strengthening solutions for existing low-rise RC buildings in Nepal, Earthq. Struct. 8 (3) (2015) 511–539.
[13] H. Yalciner, A.A. Hedayat, Repairing and strengthening of an existing reinforced concrete building: a North Cyprus Perspective, Am. J. Eng. Appl. Sci. 3 (1) (2010) 109–116.
[14] M.A. Ismaeil, Ali Hassaballa, M.Y. Laissy, I.E. Kamal, F. Adam, Evaluation and strengthening of an eight-storey existing reinforced concrete building in Abha City KSA, Adv. Civ. Eng. Technol. 2 (5) (2019) 1–18.
[15] M.A. Ismaeil, M.E. Sobaih, A.E. Hassaballa, Assessment of seismic performance and strengthening of RC existing residual buildings in the Sudan, J. Adv. Sci. Eng. Res. 3 (3) (2013) 209–225.
[16] M.B.D. Hueste, J.W. Bai, Seismic retrofit of a reinforced concrete flat-slab structure: Part I Seismic performance evaluation, Eng. Struct. 29 (6) (2007) 1165–1177.
[17] A. Ghobarah, M. Attar, El, N.M. Aly, Evaluation of retrofit strategies for reinforced concrete columns: A case study, Eng. Struct. 22 (5) (2000) 490–501.
[18] F. Altun, H.B. Kara, E. Uncuoğlu, O. Karahan, Earthquake damages in reinforced-concrete structures and retrofitting work on a 6 storey building, Gazi Univ. J. Sci. 16 (2) (2003) 309–318 (in Turkish).
[19] Inoue A., Youcef M. (2008) Seismic evaluation and retrofit plan of existing Reinforced concrete buildings of Algiers in Algeria. The 14th World Conference on Earthquake Engineering, 12–17 October 2008, Beijing.
[20] P. Rocha, P. Delgado, A. Costa, R. Delgado, Seismic retrofit of RC frames, Comput. Struct. 82 (17–19) (2004) 1523–1534.
[21] Varum H., Chaulagain H., Rodrigues H., Spacone E. (2013) Seismic assesment and retrofitting of existing RC buildings in Kathmandu, CINPAR, 2–5 June 2013 Joao Pessoa, Brasil.
[22] O. Dogan, I. Akdemir, S. Er Bakirci, Computer aided seismic analysis of RC frame building strengthened with steel cables, Int. J. Res. Dev. 3 (1) (2011) 72–76 (in Turkish).
[23] Midas Gen, Integrated Solution System for Building and General Structures, MIDAS Information Technology Co, 2018.
[24] S. Sayar, B. Sayin, Systematic strengthening scenarios for determination of most suitable retrofitting approach in a low-rise RC building, Case Stud. Constr. Mater. 15 (2021), e00782.
[25] M. Baran, D. Okuyucu, T. Tankut, Seismic strengthening of reinforced concrete frame structures using concrete panels, Int. J. Eng. Res. Dev. 1 (2009) 1 (in Turkish).
[26] M.A. Çakıroglu, S. Terzi, Strengthening with jacketed system of reinforced concrete beams, e-J. N. World Sci. Acad. 5 (4) (2010) 658–670 (in Turkish).
[27] H. Can, M.H. Gogüs, Y. Demirel, Reinforced beam repairing/strengthening by midspan column addition method, J. Fac. Eng. Archit. Gazi Univ. 17 (3) (2002) 121–135 (in Turkish).
[28] Navdar M.B., Çağlar N., 2019. Numerical study on behavior of RC columns strengthened with RC jacketing. 3rd International Symposium on Natural Hazards and Disaster Management, 25–27 October 2019 Van, 2(2), 257–264 (in Turkish).
[29] K.V. Çelik, H. Karaşin, Structural strengthening with carbon fiber, Dicle Univ. J. Fac. Eng. 5 (1) (2014) 1–11.
[30] Micelli F., Cascardi A., Aiello M.A. (2018). A study on frp-confined concrete in presence of different preload levels. Proceedings of the 9th international conference on fibre-reinforced polymer (FRP) composites in civil engineering - CICE, Paris, France (pp. 17–19).
[31] N. Çetinkaya, H. Kaplan, Ş.M. Şenel, Repair and strengthening of reinforced concrete beams using fibre reinforced polimer (FRP) materials, Pamukkale Univ. J. Eng. Sci. 10 (3) (2004) 291–298.

[32] Z.F. Alemdar, N. İnkaya, Parametric finite element analyses of RC beams strengthened with CFRP. Dicle University, J. Fac. Eng. 10 (3) (2019) 1073–1085.
[33] H. Tekeli, O. Akyürek, M. Deniz, E. Hersat, N. Kara, U. Tosun, F. Kaya, Strengthening with steel wire mesh of infill wall in RC frames, Bitlis Eren Univ. J. Sci. 3 (2) (2014) 179–191.
[34] H. Tekeli, C. Avcı, M. Üreyen, V. Sasa, Experimental investigation of strengthening infill walls with CFRP, J. Eng. Sci. 8 (1) (2019) 158–170 (in Turkish).
[35] M.A. Ismaeil, Ali Hassaballa, M.Y. Laissy, I.E. Kamal, F. Adam, Evaluation and strengthening of an eight-storey existing reinforced concrete building in Abha city, KSA Adv. Civ. Eng. Technol. 2 (5) (2019) 1–18.
[36] M.A. Ismaeil, M.E. Sobaih, A.E. Hassaballa, Assessment of seismic performance and strengthening of rc existing residual buildings in the Sudan, J. Adv. Sci. Eng. Res. 3 (3) (2013) 209–225.
[37] H. Chaluagain, H. Rodrigues, E. Spacone, H. Varum, Assessment of seismic strengthening solutions for existing low-rise RC buildings in Nepal, Earthq. Struct. 8 (3) (2015) 511–539.
[38] H. Yalciner, A.A. Hedayat, Repairing and strengthening of an existing reinforced concrete building: a North Cyprus perspective, Am. J. Eng. Appl. Sci. 3 (1) (2010) 109–116.
[39] M.B.D. Hueste, J.W. Bai, Seismic retrofit of a reinforced concrete flat-slab structure: Part I Seismic performance evaluation, Eng. Struct. 29 (6) (2007) 1165–1177.
[40] T. Uçar, S.G. Toumatari, Y. Ertutar, Investigation of the effects of shear walls installed within the frame plane on structural characteristics of R/C buildings, J. Adv. Technol. Sci. 3 (1) (2014) 56–68.
[41] H.S. Şengel, H. Erol, E. Yavuz, Seismic isolation technique and sample experiment related to its use, J. Eng. Archit. Fac. Eskişeh. Osman. Univ. 22 (2) (2009) 165–178 (in Turkish).
[42] Atalay H.M., Aynur S. (2017) Strengthening of reinforced concrete frames using steel bracing. 4th International Conference on Earthquake Engineering and Seismology, 11–13 October 2017 Eskişehir.
[43] TS EN 13791 (2010) Assessment of in-situ compressive strength in structures and precast concrete components, Turkish Standards Institute, Ankara, Turkey.
[44] TS EN 12504–1 (2010) Testing concrete in structures-part 1: cored specimens—taking, examining, and testing in compression, Turkish Standards Institute, Ankara, Turkey.
[45] TBEC 2018 (2018) Turkey Building Earthquake Code: Rules for design of buildings under earthquake effect, Official Gazette, 18.03.2018, 30364.
[46] TS708 (2010) Steel for the reinforcement of concrete–Reinforcing steel, TSE (Turkish Standards Institute), Ankara, Turkey (in Turkish).
[47] Technical Report (2019) Tests for concrete compressive strength and rebar arrangement. Bersu test laboratory.
[48] Üstündağ İ. (2019) The report of soil characteristics (in Turkish).
[49] AFAD (2020) Turkey Earthquake Risk Map Interactive Web Application. Available at: tdth.afad.gov.tr/TDTH/main.xhtml.
[50] Keun-Ho Cho (2004). Verification of fiber model in MIDAS comparing with experimental results, NLTH-1.
[51] Keun-Ho Cho, An Experimental and Analytical Study on the Seismic Behavior of RC Piers using High-Strength Concrete and High-Strength Rebars, Seoul National University, 2004. Master's Thesis.
[52] C.B. Chadwell, R.A. Imbsen, XTRACT: a tool for axial force-ultimate curvature interactions. Structures 2004: Building on the Past, Securing the Future, 2004, pp. 1–9.