Journal of Structural Engineering & Applied Mechanics (2023) 6(1):10-26
DOI 10.31462/jseam.2023.01010026

RESEARCH ARTICLE

Nonlinear time history analysis of a low-story RC building: Comparison of the existing and retrofitted states

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Article History

Received 22 November 2022
Accepted 01 February 2023

Keywords

RC building
Seismic performance
Retrofitting technique
Nonlinear time history analysis

Abstract

The current paper presents a seismic performance assessment of a low-story RC building through the time history analysis and a retrofitting proposal accordingly. The seismic analyses were repeated for the retrofitted building model and the effectiveness of the retrofitting proposal was determined. In the first stage of the study, the design project and material characteristics of the building were examined. Using these data, a 3D model of the building was prepared on the Midas Gen program. Next, a total of 11 earthquake records were selected and scaled for nonlinear time history analysis. Using the analysis results with the codes written on the Matlab program, the damage states of the load-bearing members were determined and the building's performance was measured. The results indicated that some load-bearing members have insufficient strength and a retrofitting proposal was made accordingly. A building model was then prepared considering the retrofitting proposal and the retrofitted building model was found to satisfy controlled damage performance criteria envisaged in the local seismic code. It is believed that the methodology presented in this study can be effectively used in similar buildings with poor structural strength for identifying structural performance and selecting proper retrofitting practices.

1. Introduction

As a natural disaster, earthquakes account for thousands of human deaths. Thanks to recent developments in engineering, buildings constructed according to the new seismic design codes mostly have sufficient strength against seismic effects, and earthquake-caused deaths are reduced. However, existing buildings constructed many years of age are at risk due to poor structural strength. Therefore, such buildings are either demolished and rebuilt or retrofitted using modern engineering methods. Since retrofitting can provide the building with sufficient seismic strength, it stands out considering time and cost compared to the rebuilding method. Accordingly, to determine the proper retrofitting method for an RC building, the current performance of the building should be determined using some analysis methods. For example, the finite element method can be used as an effective way to determine the nonlinear behavior of a structure [1, 2]. Analysis programs based on this method nonlinearly examine buildings and determine their performance considering the purpose of

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the building and the ground motion levels. Therefore, evaluating the performance level of the building, the required intervention method -demolishing&rebuilding or retrofitting- is precisely determined [3-6].

The retrofitting method is preferred considering its contribution to the structural performance, economic practicality, materials, and frame system [7-9]. If the retrofitting cost is above 40% of the rebuilding cost, rebuilding is suggested considering the age of the building [10, 11]. In case of a retrofitting decision, member-scale strengthening methods such as enlarging column-beam sections and building-scale retrofitting like adding RC shear walls can be used. Column jacketing and adding new reinforced concrete shear walls to the system in low-rise buildings are considered the most effective strengthening methods [56]. Increasing column-beam sections is preferred when there is a need to increase the bending capacity of members. On the other hand, adding RC shear walls is also a widely-used method since it reduces inter-story drifts. RC shear walls are placed within or adjacent to the frame system. In some cases, external steel plate bonding is considered in addition to RC shear walls [12-30]. For column strengthening, steel wrapping is used to increase shear and compressive strength. For steel wrapping, vertical brackets are placed at the four corners of the column and lateral flat bars are bonded to present prevent buckling. Columns can also be strengthened by fiber-added polymer wrapping since it is practical and increases bonding strength [31, 32]. Fiber-added polymer materials are also used for beam strengthening since they improve shear strength and ductility. Furthermore, adding external stirrups is also a method to increase the shear strength of beams [33, 34].

Diagonal steel bracing applied to external frames is also a building-scale retrofitting method that increases lateral strength without causing additional mass. Due to the possibility of brittle fracture as a result of the internal force increase after adding diagonal steel elements to the frame system, section enlargement can be considered in columns to which steel elements are bonded [35, 36]. The partition walls that are vertically continuous in the RC frame are strengthened to increase the shear strength and stiffness. This member-scale strengthening method is implemented by applying steel mesh-reinforced plaster and fibrous polymer material on the partition wall surfaces [37, 38]. In addition to the above-mentioned methods, RC structures can be strengthened by adding new frames to the building, reducing building mass, and application of seismic isolation. Since seismic isolation significantly reduces earthquake-induced inter-story drifts and story accelerations, it is considered an effective method for both rebuilding and retrofitting practices [39].

Some previous studies reported the use of nonlinear time history analysis to determine the structural behavior of different buildings. For example, Nguyen et al. [40] introduced a practical, effective, and successful approach for nonlinear time-history seismic analysis of spatial steel frames. Their fiber plastic hinge method models just one element for each member and obtains the time-history dynamic behavior of steel frames accurately like sophisticated plastic zone methods. Mokarram and Banan [41] introduced a new metaheuristic surrogate model Surrogate FC-MOPSO and reported an important decrease in computational costs of solving structural multi-objective optimization problems. The approach they proposed was an extension of the FC-MOPSO algorithm and it combines NLTHA and pushover analysis to examine system responses. Karimzadeh et al. [42] reported non-linear time history analyses for reinforced concrete structures considering past seismic recordings. The authors used synthetic records of an earthquake with a magnitude of 6.3occurred in Italy employing both the Hybrid Integral-Composite and the Stochastic Finite-Fault methods. They compared actual data and model results for maximum displacement, acceleration, and plastic beam rotation for each story. Thai and Kim [43] reported a nonlinear inelastic time-history analysis for truss structures having both geometric and material nonlinearities. They represented geometric nonlinearity using an enhanced Lagrangian formulation, whereas the material nonlinearity was determined using an empirical stress-strain relationship. The authors found that the truss model can capture some failure modes including buckling, yielding, inelastic post-buckling, unloading, and reloading. Husseini et al. [44] tried to determine the role of IBC 2009 and ACI 318-2014 codes in providing the LS PL in reinforced concrete multi-story structures with a special moment frame lateral load-bearing system. The authors created some multi-story structure models in a region having high earthquake hazards in Iran using these codes. They performed nonlinear time history analyses and determined roof displacements and accelerations, as well as base shear forces. Their findings indicated that the buildings exceed LS PL, and even they reach collapse level under some seismic loadings. Tahmasebi and Rahimi [45] performed seismic analysis on three different steel structures having five, eight, and fifteen stories corresponding to the multi-story, middle-rise, and small high-rise buildings, respectively. The authors used SAP2000 and Opensees programs for designing steel buildings and nonlinear analysis, respectively. In seismic analyses, they considered the FEMA-P750, Standard Code 2800, FEMA 356, and FEMA-P695. Mazza and Mazza [46] performed nonlinear incremental dynamic analyses on two traditional RC-framed buildings based on the simplified and refined force-displacement laws of HDRBs. The authors used 9 different earthquakes representing strong near-fault seismic events obtained from the Pacific Earthquake Engineering Research center database. Their numerical findings indicated that the nonlinear HDRB behaviors can be defined equivalently by employing an upper and lower bound approach. Furthermore, Fahjan et al. [47] conducted a time-history analysis of an RC building and determined that more accurate results are obtained as the number of earthquake acceleration records increases. They also found that the method generates successful results similar to the actual nonlinear behavior of the building.

Although many studies focused on the seismic performance of RC buildings, only a few reports have examined the effectiveness of retrofitting approaches. In addition, studies in the literature are generally based on old seismic codes. Updated seismic codes need new studies that can be referenced accordingly. In addition, applying time history analysis and scaling the records by selecting and using proper computer programs would make a valuable contribution to the relevant literature. The current study, therefore, presents a seismic performance analysis of an existing RC building and the determination of the required retrofitting method considering the results obtained. In the first stage of the study, damaged regions of the structural elements were determined through nonlinear time history analysis, and the building's performance level was identified. A retrofitted building model was then designed and examined. It is believed that the presented approach can be effectively used for RC buildings.

2. Nonlinear time history analysis

Non-linear time history analysis (NLTHA) can be performed either as a seismic code requirement or upon a client's request aiming to ensure a higher structural performance than the envisaged in the seismic code. As an engineering approach, NLTHA decreases the assumptions in the modeling, therefore, makes the expected structural response prediction more accurate. Although NLTHA takes more time and requires more expertise considering the frame system and other factors, it can reduce new construction or retrofitting costs [48].

Furthermore, the non-linear time history analysis method is suitable for all structural designs including high-rise buildings, and can accurately determine structural performance as it allows applying of real seismic records to a building. Therefore, nonlinear time history analysis is considered the most accurate and advanced method. In NLTHA, earthquake acceleration varying in the x and y directions are simultaneously applied to the building. To accurately determine the structural behavior using this method, at least 11 different earthquake acceleration records should be used. Attention must be paid to choosing earthquake acceleration records from a location with similar soil characteristics to the location of the examined building.

2.1. Design stages for flexural strengthening Earthquake acceleration records

The proper selection and scaling of the earthquake acceleration records are important for this method. Some studies were conducted for this purpose [57-62]. The selected records should be suitable to the soil characteristics of the examined building, the distance between the building and fault lines, and the magnitudes of the potential earthquakes that can occur along these fault lines. If the earthquake records will be used for a 3D examination, as a first step, the resultant horizontal spectrum is calculated by taking the square root of the sum of squares of the spectra of the lateral components of earthquake records. The average of the resultant horizontal spectra and the scale coefficient is determined considering the principle that this average should be higher than 1.3 times the corresponding ordinate of the target spectrum in the period range from $0.2T_n$ to $1.5T_n$. The records are then scaled. For the current study, a Matlab [39] script was coded for scaling earthquake records considering this principle. Scaled and unscaled earthquake records are shown in Fig. 1. The earthquake records obtained from the PEER [49] database for the analysis are given in Table 1.

2.2. Analysis assumptions

The code envisages three different damage limits for ductile elements in the strain-based evaluation. According to the code, if there is any brittle fault, the building under investigation is considered not to satisfy the desired performance level. These damage limits are Limited Damage (LD), Controlled Damage (CD), and Pre-collapse (PC). The damage regions determined by these limits are shown in Fig. 2.

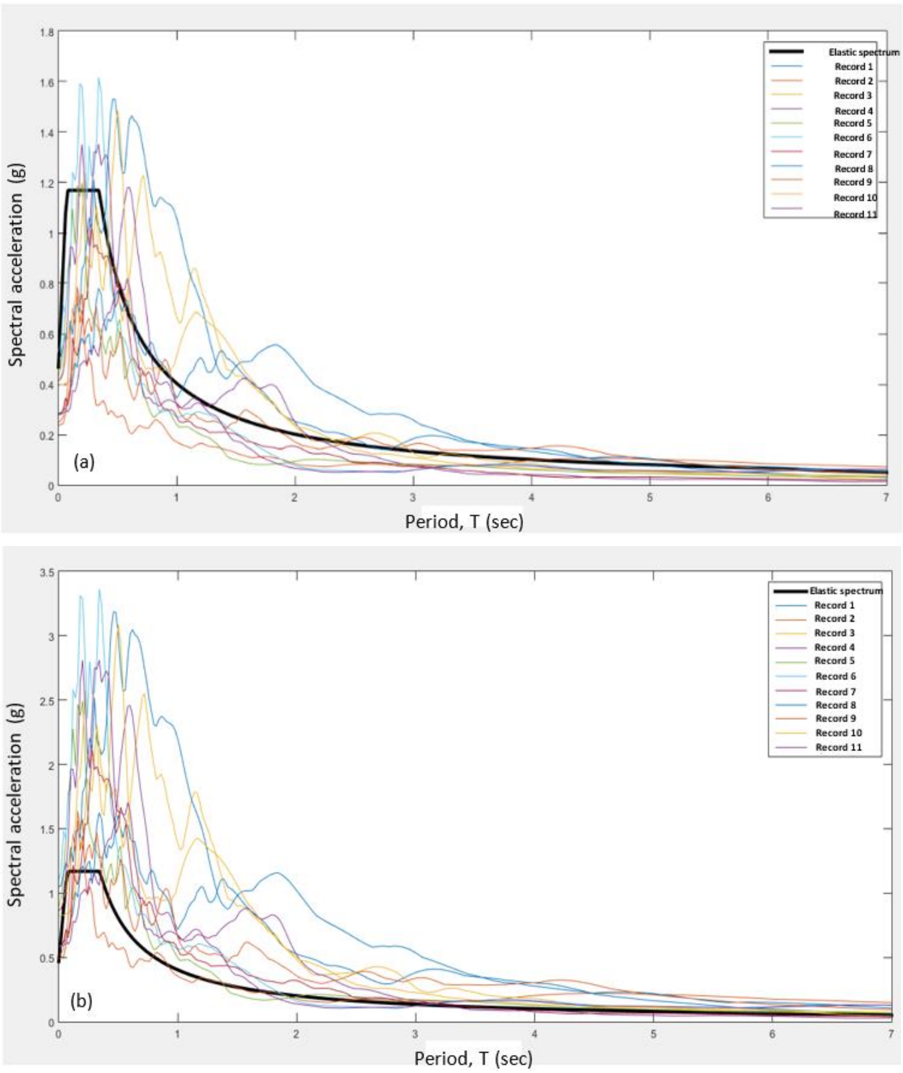

Fig. 1. Earthquake spectra and resultant spectral acceleration records. (a) Unscaled, (b) Scaled

Table 1. The selected earthquake records

Earthquake	Record no	Year	Magnitude	Fault mechanisms	Station	The epicentral distance (R_{jb}) (km)	Shortest distance to the fault (R_{rup}) (km)	Shear wave velocity (m/s)	Record duration (s)
Taiwan Chi Chi	1	1999	7.62	Reverse Oblique	CHY074	0.7	10.8	553.43	90
Kocaeli	2	1999	7.51	Strike Slip	Arçelik	10.56	13.49	523	30
Landers	3	1992	7.28	Strike Slip	Joshua Tree	11.03	11.03	379.32	44
Landers	4	1992	7.28	Strike Slip	Morengo Valley Fire Station	17.36	17.36	396.41	56
Darfield, New Zeland	5	2010	7	Strike Slip	LPCC	25.21	25.67	649.67	53
Düzce	6	1999	7.14	Strike Slip	Irigm 498	3.58	3.58	425	35
Taiwan Chi Chi	7	1999	7.62	Reverse Oblique	CHY010	19.93	19.66	538.69	132
Taiwan Chi Chi	8	1999	7.62	Reverse Oblique	CHY006	9.76	9.76	438.19	150
Hector Mine	9	1999	7.13	Strike Slip	Amboy	41.81	43.05	382.93	60
Hector Mine	10	1999	7.13	Strike Slip	Hector	10.35	11.66	726	45
Düzce	11	1999	7.14	Strike Slip	Irigm 487	2.65	2.65	690	55

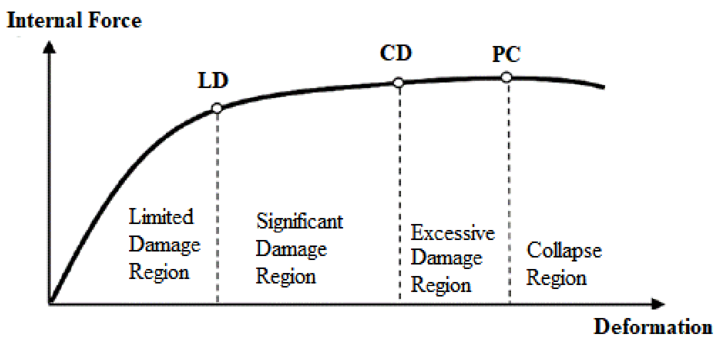

Fig. 2. Sectional damage limits and regions [50]

For the buildings with the target performance of pre-collapse damage and modeled according to the distributed hinge approach, the shortening of the concrete unit and rebar strain are calculated using Eqs. 1-4 for rectangular sections.

$$\epsilon_c^{(PC)} = 0.0035 + 0.04\sqrt{w_{we}} \leq 0.018 \tag{1}$$

$$w_{we} = \alpha_{se}\, \rho_{sh,min} \frac{f_{ywe}}{f_{ce}} \tag{2}$$

$$\alpha_{se} = \left(1 - \frac{\sum a_i^2}{6b_o h_o}\right)\left(1 - \frac{s}{2b_o}\right)\left(1 - \frac{s}{2h_o}\right) \tag{3}$$

$$\rho_{sh} = \frac{A_{sh}}{b_k\, s} \tag{4}$$

Shortening of concrete unit calculations for controlled and limited damage performance levels are made using Eqs. 5 and 6, respectively.

$$\epsilon_c^{(CD)} = 0.75\, \epsilon_c^{(PC)} \tag{5}$$

$$\epsilon_c^{(LD)} = 0.0025 \tag{6}$$

Calculations for the pre-collapse, controlled damage, and limited damage performance values for the distributed hinge approach are done with Eqs. 7-9:

$$\epsilon_s^{(PC)} = 0.4\, \epsilon_{su} \tag{7}$$

$$\epsilon_s^{(CD)} = 0.75\, \epsilon_s^{(PC)} \tag{8}$$

$$\epsilon_s^{(LD)} = 0.0075 \tag{9}$$

If the shear force ratio $V_e/b_w d f_{ctm}$ of the examined section is above 1.3, strain upper limits are decreased by multiplying by 0.5. If the ratio is between 0.65 and 1.30, linear interpolation is applied. On the other hand, if the shear force ratio is below 0.65, the strain upper limits in the equations are used without any change. If non-ribbed rebars are used in the building, the calculated strain upper limits are multiplied by 1.5 and then compared with the strain upper limits given in the code.

To conduct a time history analysis on the building, the inelastic properties of concrete and steel were defined as shown in Table 2. For this purpose, Mander [51] and Kent Park [52] models based on the inelastic properties of wrapped and unwrapped concrete were used. TBEC 2018 recommends using the equations given in this behavioral model. The definitions made in the Midas Gen [53] program are shown in Fig. 4.

After defining concrete and steel rebar properties, fiber joints were defined on the program according to the distributed hinge approach. Finally, the scaled earthquake acceleration records were entered into the program and the analysis was performed. After these procedures, a script was coded on the Matlab [54] program to calculate the average of the highest absolute values obtained from the time history analyses and compare them with the limit values in the seismic code. Time history data vs. time is shown in Fig. 3.

3. Examined building

The examined building was a 7-story structure, composed of 1 basement, 1 ground story, 4 normal stories, and a penthouse. The story heights were 2.4 m for the basement, 3.5 m for the ground story, 3.3 m for the normal stories, and 2.5 m for the penthouse. The foundation of the building is composed of continuous footings and individual footings. The continuous footings were connected with 25×40 cm tie beams. The building has columns with varying cross sections of 30-70 cm and 40-65 cm in the *x*- and *y*-directions, respectively. The width and heights of the beam sections vary from 20-60 cm and 30-60 cm, respectively. Twenty-cm-thick RC shear walls were used around the basement story and the elevator core. A 3D view of the existing building and column application plan for the normal stories are given in Fig. 4. Rebar examination performed in the building is given in Fig. 5.

Table 2. The selected earthquake records

	Parameter	Value
Mander model	Unconfined concrete strength	14.96 MPa
	Unconfined concrete strain	0.0002
	Elastic modulus of concrete	19339 MPa
	Tensile strength of concrete	2.40 MPa
	Tensile strain of concrete	0.000124
	Effective lateral confining stress on the concrete	0.36 MPa (*y*-axis) 0.54 MPa (*z*-axis)
	Area of effective concrete core	0.2380 m^2
	Total area of effectively confined core concrete	0.0471 m^2
	Strength of confined concrete	15.25 MPa
	Strain of confined concrete	0.002194
	Ultimate strain for confined concrete	0.027048
	Raito of the volume of transverse confining steel to that of confined concrete core	0.002083
Park model	Yield stress of steel	220 MPa
	Ultimate stress of steel	264 MPa
	Elastic modulus of steel	200,000 MPa
	Yield strain of steel	0.0011
	Strain at the onset of strain hardening	0.011
	Strain at the steel rupture	0.12

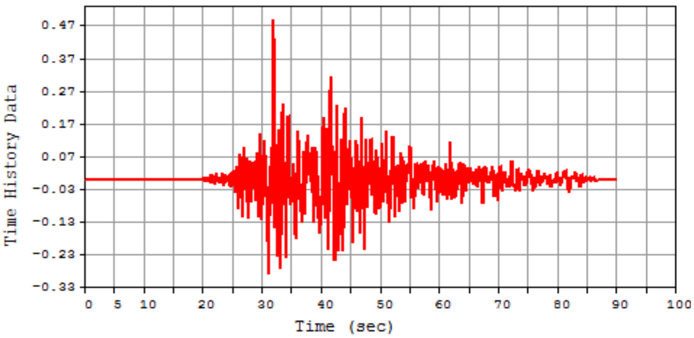

Fig. 3. Time history data vs. time

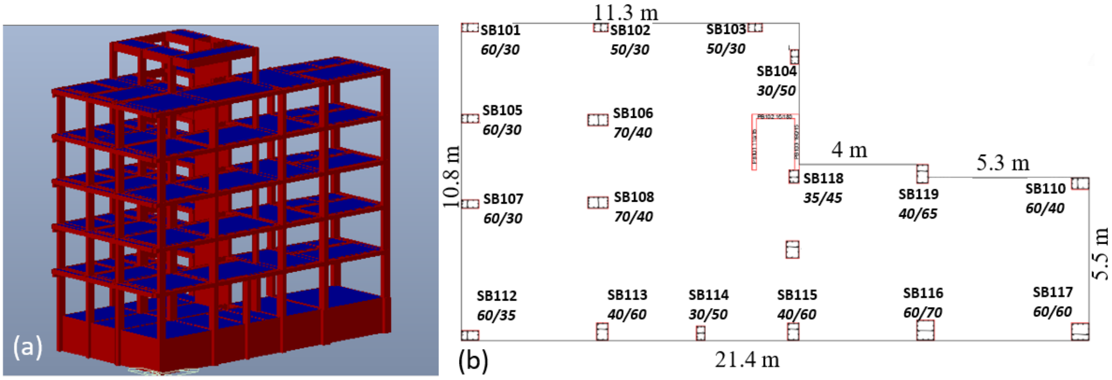

Fig. 4. (a) The 3D view of the existing building, (b) Column application plan for the normal stories

Table 3. Concrete compressive strength test results

Sample	Story	Fracture load (kN)	$f_{c,cube}$ (MPa)
SB16		157.8	20.1
SB11	Basement	186.4	23.7
SB06		139.0	17.7
SZ16		77.3	9.8
SZ15	Ground	119.9	15.3
SZ11		93.6	11.9
S108		136.5	17.4
S118	First	184.6	23.5
S109		121.0	15.4
S208		350.8	44.7
S218	Second	174.0	22.2
S209		223.6	28.5
S308		135.5	17.3
S318	Third	117.3	14.9
S309		157.2	20.0
S409		161.4	20.6
S418	Fourth	136.5	17.4
S408		105.0	13.4
S501		163.9	20.9
S502	Penthouse	202.5	25.8
S503		187.5	23.9
	Average		20.2 MPa

For all samples, Cylindrical, 100×100 mm, Cross-sectional area = 78.5 cm^2

Table 4. The evaluation of concrete compressive test results

$f_{c,cube,av.}$	σ (MPa)	$f_{c,cube} = f_{c,cube,av} - \sigma$	$f_{c,cyl} = 0.85 \cdot f_{c,cube}$	E_c (MPa)
20.2	2.6	17.6	14.96	26,750

σ = Standard deviation, E_c = Young's modulus of concrete

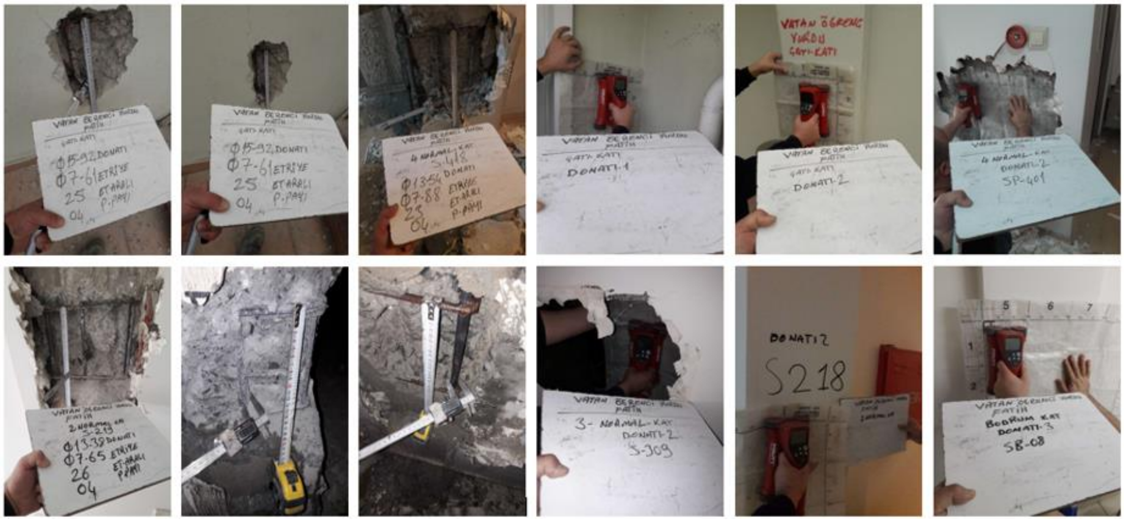

Fig. 5. Rebar examination using destructive and non-destructive techniques

3.1. Concrete compressive strength

Concrete core samples were taken from the vertical structural elements to assess the average compressive strength of the concrete in the building. Three different concrete samples were taken from each story. The 28-day strengths of the concrete were determined using the samples. These strengths correspond to the cubic strengths as defined in the TBEC 2018. The average cubic strengths were then converted to cylindrical compressive strengths after multiplying by 0.85 as specified in the code. The concrete strength test results are given in Table 3. The calculations for the compressive test results are shown in Table 4.

3.2. Rebar properties

According to TBEC 2018, two techniques can be used to examine rebars in concrete members. The first method is the rebar scanning method. The second is an observational rebar measurement approach implemented by stripping cover concrete. Both methods were used in the current study to evaluate the rebar configuration in the structural members. The concrete covers on one column and one beam on each floor were removed to examine the rebars. The rebar examinations showed that S220 (f_y = 220 MPa) plain steel bars were used in the rebars.

3.3. Seismic parameters of the soil

The seismic characteristics of the soil in the building's location were obtained using the Soil Classification Map under TBEC 2018. The seismic characteristics of the soil are given in Table 5. Turkey Earthquake Risk Map [55] was used and the spectral parameters corresponding to the soil class were determined. A spectrum curve was created on the Matlab program using these spectral values as shown in Fig. 6.

Table 5. Soil characteristics and seismic parameters of the region

Definition	Value / Class
Effective ground acceleration coefficient	$A_o = 0.40$
Spectrum characteristic periods	T_A= 0.15 s., T_B=0.60 s.
Spectrum coefficient	S(T1) = 2.5
Local soil class	Z3
Seismic zone	I
Modulus of subgrade reaction (vertical) (kN/m^3)	15000
Allowable bearing capacity, σ_z (kN/m^2)	165

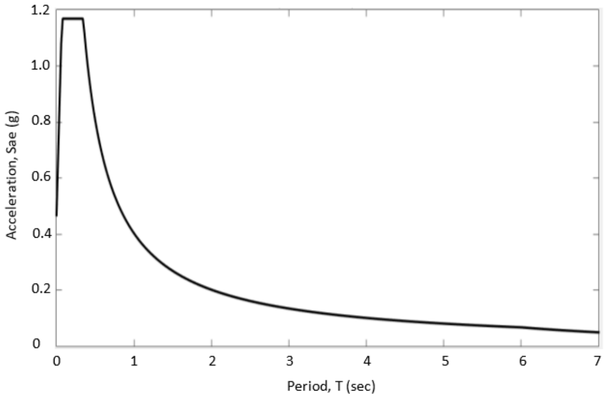

Fig. 6. Elastic design spectrum curve

4. Results and discussion

4.1. Results for the current state of the building

According to the analysis results, the natural vibration period of the current building was found to be 0.93 s. The results obtained are shown in Table 6. The ratio of the number of beams beyond the limited damage region to the number of all beams in the same story was calculated for each story. This ratio in the second story was calculated as 88.2%. Since 88.2%>20% and some load-bearing members went beyond the limited damage region, the building was found to not satisfy the "Limited Damage" performance criteria. On the other hand, the ratio of the number of beams beyond the significant damage region to the number of all beams in the same story was found to be 14.7% in the first story. Furthermore, in the ground story, the ratio of the shear force (in the y) direction carried by the vertical load-bearing members beyond the distinctive damage to the total shear force carried by all vertical bearing members in the same story was found to be 68.5%. Moreover, in the first story, the ratio of the shear force carried by the vertical load-bearing members beyond the significant damage -both in the lower and upper sections- to the total shear force carried by all vertical-bearing members in the same story was found to be 15.7%. Although 15.7%<30%, the building did not satisfy the “Limited Damage” performance level since 68.5% is above 20%. The building also met the “Prevention of Collapse” performance criteria since 15.7%<%30 and there are no beams -in all stories- beyond the collapse state. In conclusion, since some load-bearing members went beyond the significant damage region and the ratio of shear force carried by these members was high, the building was found to not satisfy the "Controlled Damage" performance criteria envisaged by TBEC 2018 for the existing RC buildings.

Table 6. Damage regions for the current state of the building (%)

Story	A	B	C	D	E	F	G
Basement	–	–	–	–	–	–	–
Ground	88.2	0.0	15.1	68.5	–	–	–
First	85.3	14.7	6.9	45.3	1.9	15.7	–
Second	87.9	9.1	2.6	22.0	–	–	–
Third	82.4	–	–	–	–	–	–
Fourth	64.7	–	–	–	–	–	–
Penthouse	20.0	–	–	–	–	–	–

A: The ratio of the beams beyond the limited damage region, **B**: The ratio of the beams beyond the significant damage region, **C**: The ratio of the shear force in the x direction carried by the vertical load-bearing members beyond the significant damage **D**: The ratio of the shear force in the y direction carried by the vertical load-bearing members beyond the significant damage, **E**: The ratio of the shear force in the x direction carried by the vertical load-bearing members beyond the controlled damage -both in the lower and upper sections- to the total shear force in the same story, **F**: The ratio of the shear force in the y direction carried by the vertical load-bearing members beyond the controlled damage -both in the lower and upper sections- to the total shear force in the same story, **G**: The ratio of the beams beyond the collapse region

Table 7. The story drifts to the current state of the building (m)

Story	A	B
Basement	-	-
Ground	0.026	0.0007
First	0.0257	0.0352
Second	0.0828	0.0415
Third	0.0331	0.0419
Fourth	0.026	0.0388
Penthouse	0.0189	0.028

A: Story drift in the *x*-direction, B: Story drift in the *y*-direction

In addition to these, the obtained story drifts results are shown in Table 7. Accordingly, for the current state of the building, the story drifts in the *x*-direction were 0.0189 m for the penthouse, 0.026 m for the fourth story, 0.0331 m for the third story, 0.0828 m for the second story, 0.0257 m for the first story, and finally, 0.026 m for the ground story level. In the *y*-direction, the story drifts were calculated as 0.028 m for the penthouse, 0.0388 m for the fourth story, 0.0419 m for the third story, 0.0415 m for the second story, 0.0352 m for the first story, and finally, 0.0007 m for the ground story level.

4.2. Results for the retrofitted building model

According to the performance analysis, the building needs to be strengthened to satisfy the target performance criteria. Therefore, it was decided that both member-scale and building-scale strengthening practices should be performed. The columns beyond the collapse regions were decided to be strengthened with section enlargement. Plus, since the shear walls in the core sections did not have sufficient strength, adding shear walls both within and near the frame system was concluded. Therefore, the load-bearing members beyond the collapse state can be maintained within the significant damage region and shear forces on the building can be carried out by more load-bearing members as new bearing members are added. For this purpose, a new building model was prepared with retrofitting interventions considering the strengthening needs discussed above. In this model, jacketing was applied to the whole building, and new shear members were added. The strengthening plan for the basement story is given in Fig. 7 as an example. This strengthening plan was implemented in other stories in the building model.

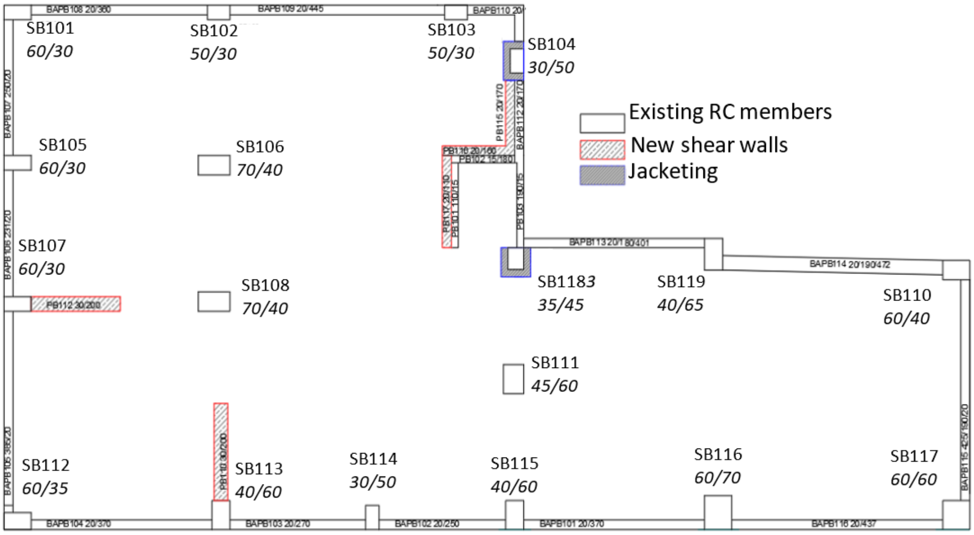

Fig. 7. Strengthening plan in the basement story

Table 8. Damage ratios of the beams for the strengthened building model

Story	A’	B’
Basement	–	–
Ground	84.8	6.1
First	87.9	3.0
Second	81.3	3.1
Third	72.7	3.0
Fourth	66.7	3,0
Penthouse	14.3	–

A’: The ratio of beams beyond the limited damage region, B’: The ratio of beams beyond the significant damage region

Table 9. The story drifts for the strengthened building model (m)

Story	A	B
Basement	-	-
Ground	0.0001	0.0001
First	0.0018	0.0013
Second	0.0021	0.0017
Third	0.0019	0.0019
Fourth	0.0018	0.0019
Penthouse	0.0014	0.0017

A: Story drifts in the *x*-direction, B: Story drifts in the *y*-direction

After the strengthening, the natural vibration period of the building model was calculated as 0.7078 s. The analysis results of the lateral members are given in Table 8. No vertical load-bearing member -in any story- went beyond the significant damage region.

Since the ratio of the beams beyond the limited damage region was 87.9% and above 35%, the building did not satisfy the “Limited Damage” performance criteria. Furthermore, the building also met the “Controlled Damage” performance criteria envisaged in the local seismic code since the ratio of beams beyond the significant damage region was 6.1%, below 20%, no beams went beyond the significant damage region in the penthouse, and all vertical load-bearing members were in the significant damage region.

For the building model representing the strengthened state, the obtained story drifts results are shown in Table 9. As seen in the table, the story drifts in the *x*-direction model were calculated as 0.0014 m for the penthouse, 0.0018 m for the fourth story, 0.0019 m for the third story, 0.0021 m for the second story, 0.0018 m for the first story, and finally, 0.0001 m for the ground story level. In the *y*-direction, the story drifts were found to be 0.0017 m for the penthouse, 0.0019 m for the fourth story, 0.0019 m for the third story, 0.0017 m for the second story, 0.0013 m for the first story, and finally, 0.0001 m for the ground story level.

5. Concluding remarks

It is known that a majority of the current building stock did not satisfy the requirements stated in the latest seismic code. Therefore, retrofitting proposals are made after conducting a nonlinear analysis. Among analysis methods, nonlinear time history analysis is a widely-preferred method since it generates more accurate results by using fewer assumptions. However, properly selecting earthquake records and scaling are important factors for determining the actual structural behavior.

In the current study, the structural strength of an existing RC building was determined using Midas Gen and Matlab computer programs. Accordingly, a 3D model of the existing RC building was prepared on the Midas program and examined through nonlinear time history analysis. Since the analysis results indicated that the building’s structural performance is poor, a strengthened building model was prepared with jacketing and new shear members. The mass and rigidity centers were set relatively close to one another; thus, the formation of the torsion effect and additional moments were prevented.

The analysis results showed that a majority of the beams -both in the current and strengthened building model- went beyond the limited damage region. Furthermore, after strengthening, the ratio of the beams beyond the significant damage region reduced from 14% to 3% in the first story and from 9% to 3.1% in the second story. The positive impact of the strengthening was also observed among the vertical load-bearing members. In the current building, there were vertical load-bearing members beyond the significant damage region. The ratio of the shear force carried by these members was 2.6% to 68.5% in the *x*- and *y*-directions, respectively. Moreover, the ratio of the shear force carried by the members beyond the significant damage region both in the lower and upper sections was 1.9% and 15.7% in the *x*- and *y*-directions, respectively. According to the story drift results, it can be argued that walls have a positive impact on reducing inter-story drifts. In the strengthened building model, on the other hand, no vertical load-bearing members went beyond the significant damage region, and also story drifts are reduced. Accordingly, the strengthened building model satisfied the controlled damage performance criteria.

In conclusion, the current paper presents the structural evaluation of a low-story RC building through the time history analysis method and a retrofitting proposal. It is concluded that the methodology presented in this study can be effectively used in similar buildings.

Acknowledgments

This paper is based on the data of Mehmet Fırat Karapınar's Master thesis entitled "Determination of Performance Level and Strengthening Proposal of an Existing Reinforced Concrete according to 2018 TSC". This thesis was supervised by Associate Professor Baris Sayin and Assistant Professor Baris Gunes.

Conflict of interests

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

Funding

This research received no external funding.

Author contributions

Mehmet Firat Karapinar: Conceptualization, Software. Baris Gunes: Data curation, Writing-Original Draft Preparation, Supervision. Baris Sayin: Methodology, Visualization, Investigation, Supervision, Writing-Reviewing and Editing, Validation.

Data availability statement

The data presented in this study are available upon request from the corresponding author.

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