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Seismic assessment of existing RC buildings before and after shear-wall retrofitting

Authors:

Assist.Prof. **Onur Onat**, PhD. CE
Munzur University, Turkey
Faculty of Engineering
Department of Civil Engineering
onuronat@munzur.edu.tr

Assoc.Prof. **Burak Yön**, PhD. CE
Munzur University, Turkey
Faculty of Engineering
Department of Civil Engineering
burakyon@gmail.com

Prof. **Yusuf Calayıri**, PhD. CE
Fırat University, Turkey
Faculty of Engineering
Department of Civil Engineering
ycalayir@firat.edu.tr

Professional paper

Onur Onat, Burak Yön, Yusuf Calayır

Seismic assessment of existing RC buildings before and after shear-wall retrofitting

The existing reinforced concrete (RC) buildings are analysed in the paper before and after shear-wall retrofitting. For numerical application, an existing RC building was selected and retrofitted with shear walls. Incremental dynamic analyses and static pushover analysis of these buildings were performed using the distributed plastic hinge approach. The results show that seismic retrofitting with shear walls increases rigidity and capacity of the building, while decreasing lateral displacements and damage.

Key words:
structural analysis, damage, retrofitting, shear wall, reinforced concrete buildings

Stručni rad

Onur Onat, Burak Yön, Yusuf Calayır

Seizmičko ocjenjivanje postojećih AB građevina prije i nakon pojačavanja posmičnim zidovima

U ovom se radu ocjenjuju postojeće armiranobetonske (AB) građevine prije i nakon pojačanja posmičnim zidovima. Za potrebe numeričkog proračuna odabrana je postojeća AB građevina koja je zatim pojačana pomoću posmičnih zidova. Provedene su inkrementalne dinamičke analize i analize postupnim guranjem (pushover analiza), a za to je primijenjena metoda raspoređenog plastičnog zgloba. Prema dobivenim rezultatima, seizmičkim pojačanjem pomoću posmičnih zidova ne samo da se poboljšava krutost i nosivost građevina već se i smanjuju bočni pomaci i oštećenja.

Ključne riječi:
analiza konstrukcije, oštećenje, pojačanje, posmični zid, armiranobetonske građevine

Fachbericht

Onur Onat, Burak Yön, Yusuf Calayır

Seismische Bewertung der bestehenden Stahlbetongebäude vor und nach Verstärkung durch Schiebewände

In dieser Abhandlung werden die bestehenden Stahlbetongebäude vor und nach Verstärkung durch Schiebewände bewertet. Für die Zwecke der nummerischen Berechnung wurde ein bestehendes Stahlbetongebäude ausgewählt, das dann mithilfe von Schiebewänden verstärkt wurde. Durchgeführt wurden inkrementelle dynamische Analysen und Analysen durch schrittweises Schieben (Pushover-Methode), und dafür wurde die Methode des verteilten Plastikgelenks angewendet. Nach den erhaltenen Ergebnissen wird durch die seismische Verstärkung mithilfe von Schiebewänden nicht nur die Steifigkeit und Tragfähigkeit der Gebäude verbessert, sondern es werden auch seitliche Verschiebungen und Beschädigungen verringert.

Schlüsselwörter:
Analyse der Konstruktion, Beschädigung, Verstärkung, Schiebewand, Stahlbetongebäude

1. Introduction

Earthquake is the most forceful external effect on man-made structures. The response of RC structures to earthquake action depends on various parameters. Weight of structures, soil properties in the area in which the building is located, location and size of structural elements, architectural details, and material quality, influence the earthquake performance and level of structural damage [1, 2]. Capacity of the RC buildings needs to be increased before or after earthquakes to achieve the required performance level. Seismic retrofitting techniques that are usually considered include insertion of shear walls, insertion of RC column jackets, and confining the column plastic hinge regions by using steel plates externally attached to reinforced concrete buildings [3]. However, selection of proper investigation method is another problem. To decrease damage to structures, suitable retrofitting solutions need to be adopted and their seismic response needs to be evaluated. Here the main problem is how to adapt retrofitting practices to the structures, as reinforcing techniques will define structural performance during a future earthquake. For this reason, the retrofitting must be well defined and properly applied [4]. One of the most important methods for retrofitting RC structures is to add a RC shear wall to the structure. RC shear walls greatly increase the strength of high rise buildings and their resistance to lateral forces such as earthquake and wind loads [5]. In addition to vertical loads, horizontal loads also gain in importance in proportion to the height of the structures. This effect of horizontal load exposes columns and beams of RC buildings to extreme bending moments, shear stresses and second order moments in earthquake sensitive zones, depending on the height of the building. As a result, horizontal displacements along the height of the building reach unexpected levels. To limit horizontal displacements which lead to the evolution of second order moments, RC shear walls with higher bending stiffness must be used instead of conventional columns. The main task of a RC shear wall is to limit horizontal displacements between the floors by increasing horizontal stiffness with regard to reversible cyclic earthquake loads. The performance of these elements needs to be evaluated numerically so as to associate them effectively with the RC frame in order to prevent damage to non-structural elements. This assessment must be made both during construction of new buildings and in the scope of retrofitting activities for existing structures, in order to ensure greater safety.
The assessment of nonlinear seismic response of RC buildings requires a method which demonstrates behaviour of the building from linear elastic region to yielding stage or until it collapses. For multiple degree of freedom systems, determination of nonlinear response can be difficult due to the effect of higher modes. The Incremental Dynamic Analysis (IDA) is generally used to verify nonlinear response of buildings. This method proposes a set of ground motion acceleration records by selecting and scaling into multiple intensity domain levels to cover the whole array of structural behaviours, from elastic response to global dynamic instability [6-11].
However, another method that is used for estimation of building performance is known as the nonlinear static pushover analysis. This method is a practical procedure for estimating structural capacity of buildings in the post-elastic range. The capacity curve of a building shows relationship between the base shear force and the roof displacement. To obtain the capacity curves, lateral forces are increased proportionally until displacement at the top of the building reaches a certain level. [12-16].
In this study, the IDA and the pushover analysis are used to evaluate seismic response of existing reinforced concrete buildings before and after seismic retrofitting. Moreover, this study presents the distributed plastic hinge model that is used for nonlinear modelling of structural elements.

2. Plastic hinge model used for nonlinear modelling

This model is defined as a fibre element model that is mainly concerned with plasticity. This plasticity is distributed throughout the cross-section and the length of the element. In this hinge model, the structural element is divided into three types of fibres. Some fibres are used for modelling longitudinal steel reinforcing rods; other fibres are used to define nonlinear behaviour of confined concrete, and the remaining fibres are defined for unconfined concrete, which includes cover concrete. Also, the stress/strain field is determined for each fibre in nonlinear range using constitutive laws according to defined materials. Typical fibre modelling for a rectangular reinforced concrete section is shown in Figure 1.

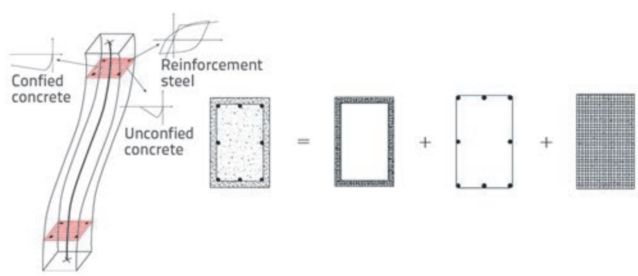

Figure 1. Typical fibre model of a RC element [17]

Researchers use the distributed plastic hinge model approach in their studies, since this hinge model is more accurate compared to lumped hinge models, especially when high axial force variations exist [18]. Dides and Llera compared plasticity models that include the fibre hinge model in dynamic analysis of buildings [19]. Duan and Hueste investigated earthquake behaviour of a five-story reinforced concrete building designed according to the requirements of the Chinese seismic code. They used the distributed hinge model in these analyses [20]. Kadid et al. investigated behaviour of reinforced concrete buildings under simultaneous ground motions using the fibre hinge approach [21]. Mwafy assessed earthquake design behaviour factors of concrete wall buildings. In this respect, five reference buildings, ranging from 20 to 60 storeys in height, were determined. Analyses of these structures were performed according to the fibre hinge approach [22]. Carvalho et al. compared miscellaneous hinge models by performing the pushover and time history analysis on a reinforced concrete structure [23]. Beigi et al. assessed seismic retrofitting of a soft frame using a novel gapped-inclined brace (GIB) system. They used the fibre hinge approach in nonlinear static analyses [24]. Yön and Calayır assessed the soil effect on seismic response of RC buildings by considering the distributed hinge model [25]. Yön et al. carried out a study about the effect of seismic zones and site conditions on seismic response of RC buildings [26]. Chaulagain et al. investigated seismic retrofitting solutions of the existing low-rise RC buildings in Nepal, and assessed seismic safety of the existing masonry infill structures in Nepal. For analyses, a half of the larger dimensions of the cross-section is considered as the plastic hinge length with fibre discretization [27, 28]. Sadraddin studied fragility assessment of high-rise reinforced concrete buildings considering the effects of shear wall contributions. A fibre method was used to model cross-sections of building members [29]. Khaloo et al. investigated the influence of earthquake record truncation on fragility curves of reinforced concrete frames with various damage indices. They used fibre hinge model approach in nonlinear analyses [30].

The fibre hinge approach is used in this study to evaluate existing reinforced concrete buildings before and after seismic retrofitting.

3. Numerical application

3.1. Description of building, adopted material models and properties

The existing reinforced concrete frame with 6 storeys and 5 bays was selected for numerical study. The same model was retrofitted by shear walls. Columns were not equipped with a jacket in this study. The models are presented in Figure 2. The buildings are 22 meters in total height. For the selected building, the ground floor is 4.5 m in height, and the upper storey heights are 3.5 m. Columns dimensions are 45/45 and 40/40 cm, and dimensions for beams amount to 25/60 and 25/50 cm for the existing building. The slab thickness is 12 cm. The nonlinear dynamic analyses of the buildings were performed using various ground accelerations (from 0.1g to 0.4g) for soil class Z3 according to the Turkish Seismic Code (TSC) [31]. The building importance coefficient is assumed to be 1.0, the specific compressive strength of concrete is 14 MPa, and the rebar yield strength is 220 MPa for the existing building. The compressive strength of the vast majority of buildings in Turkey, especially those constructed before the year 2000, is below 20 MPa. This compressive strength value was therefore considered in this situation. In addition to this, the compressive strength of concrete amounting to 25 MPa and the yield strength of rebar of 420 MPa, were selected for shear walls of retrofitted building. The boundary condition of the building was assumed to be a fixed support. Also, the soil differences and damping properties were not considered. The SeismoStruct [32] program was used for nonlinear analyses as this program is able to simulate the response of inelastic structural systems subjected to dynamic and static loads. Also, SeismoArtif [33] and SeismoSignal [34] programs were used to scale earthquake time series according to design spectrums.

The bilinear elastic plastic material model that includes kinematic strain hardening is used for reinforcing bars. Concrete material is defined by the uniaxial confinement concrete model (Figure 3) The confinement effect was calculated using the Mander model [35]. Parameters related to structural elements of both the existing and retrofitted structures are presented in Table 1.

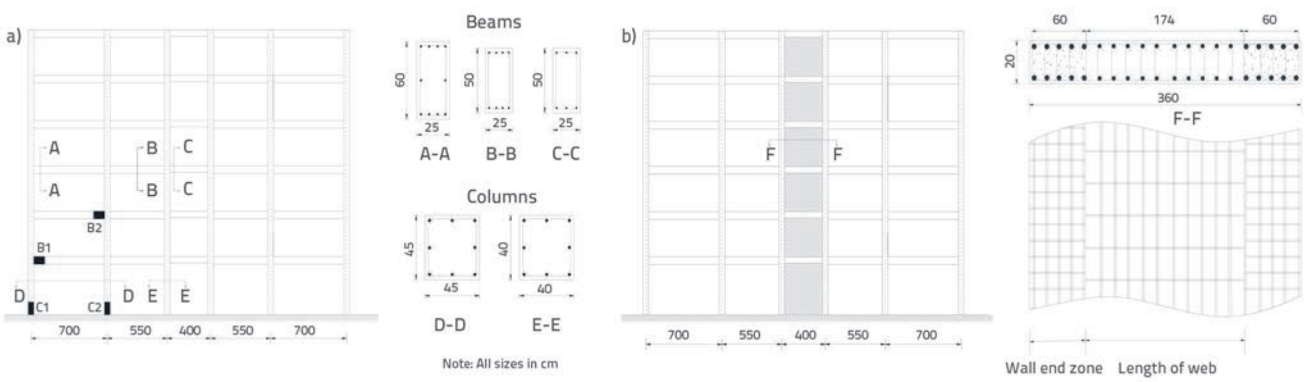

Figure 2. a) existing building with structural elements; b) retrofitted buildings with structural elements

Table 1. Reinforced concrete parameters related to structural elements

Models	Structural elements		Longitudinal reinforcement	Section	Transverse reinforcement spacing [cm]
Retrofitted building	Shear wall (20/360 cm)	Wall end zone	20Ø14	F-F	10
		Length of web	20Ø14		20
Existing building	Column (45/45 i 40/40 cm)	Confinement zone of column	8Ø16	D-D	15
		Central zone of column		E-E	20
	Beam (25/60 cm)	Confinement zone of beam	Top reinforcement 4Ø12	A-A	20
		Central zone of beam	Bottom reinforcement 4Ø12		
	Beam (25/50 cm)	Confinement zone of beam	Top reinforcement 4Ø12	B-B	20
		Central zone of beam	Bottom reinforcement 4Ø12		
	Beam (25/50 cm)	Confinement zone of beam	Top reinforcement 3Ø12	C-C	20
		Central zone of beam	Bottom reinforcement 3Ø12		

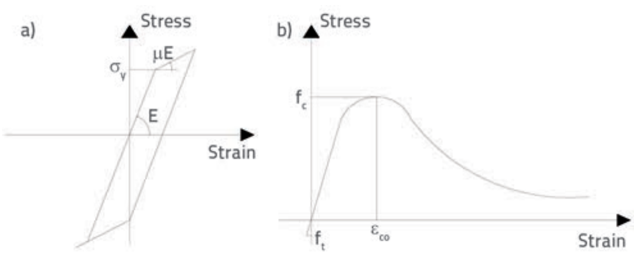

Figure 3. Material models a) bilinear elastic plastic material for steel bar; b) Mander [35] model for concrete

3.2. Earthquake parameters, site conditions and performance criteria

Selected earthquake time series are presented in Table 2. The seismic records have been provided by the PEER Database [36]. These records were scaled in frequency domain to be coherent with the design spectrum in compliance with various ground accelerations and soil class Z3 in TSC. Figure 4 shows scaled spectra compatible with the target design spectrum on the basis of miscellaneous ground accelerations and soil class Z3. According to TSC 2007, soil class Z3 is divided into two groups (C and D) on the basis of the topmost soil layer thickness. This thickness should be between 15 m and 50 m for Group C, and less than 10 m for Group D. Group C consists of the generally highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity, medium dense sand and gravel, and stiff-silty clay. Group D consists of the generally soft deep alluvial layers with high ground water level, loose sand, soft clay, and silty clay.

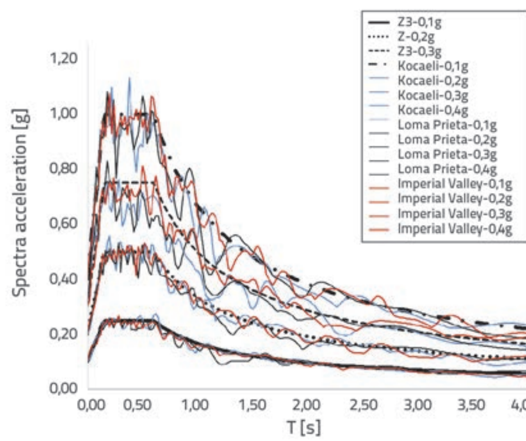

Figure 4. Response spectra belong to selected acceleration time series scaled in accordance with the elastic design spectrum for soil class Z3 and various ground motions

Table 2. Earthquake records used for nonlinear dynamic analysis

Earthquakes	Date	Station	Direction	Magnitude	PGA [cm/s²]
Imperial Valley	1940	El Centro Array	East-West	7.0	307.05
Loma Prieta	1989	Corralitos	East-West	6.9	631.76
Kocaeli	1999	Kocaeli	North-South	7.4	373.76
PGA - Peak Ground Acceleration					

Table 3. Performance criteria used in analyses

Damage level	Limit values for confined concrete	Limit values for unconfined concrete	Limit values for steel bars
Minimum damage limit (MN)	$(\varepsilon_{cu})_{MN} = 0.0035$	0.0035	$(\varepsilon_{s})_{MN} = 0.010$
Safety damage limit (GV)	$(\varepsilon_{cg})_{GV} = 0.0035 + 0.01(\rho_s/\rho_{sm}) \leq 0.0135$	0.0037	$(\varepsilon_{s})_{GV} = 0.040$
Collapse damage limit (GC)	$(\varepsilon_{cg})_{GC} = 0.004 + 0.014(\rho_s/\rho_{sm}) \leq 0.018$	0.0040	$(\varepsilon_{s})_{GC} = 0.060$

The performance criteria are used in TSC as basic criteria. The TSC requires three performance levels for seismic evaluation. The first performance level is the Minimum Damage Limit (MN) [Immediate Occupancy-IO], the second performance level is the Safety Damage Limit (GV) [Life Safety-LS] and the third performance level is the Collapse Damage Limit (GC) [Collapse Prevention-CP]. These performance limits are presented in Table 3.
In Table 3, ε_{cu}, ε_{cg}, and ε_s are the ultimate strain of unconfined concrete, ultimate strain of confined concrete, and deformation of reinforcement steel, respectively. Also, ρ_s denotes the volumetric ratio of spiral reinforcement placed in cross section, while ρ_{sm} defines the volumetric ratio of shear reinforcement placed in cross section.

4. Analysis results

Absolute maximum responses were obtained from IDA by means of scaled records. Maximum responses were fitted to obtain dynamic analysis curves. These responses and the dynamic analysis curves for the existing and retrofitted buildings are presented in Figure 5.

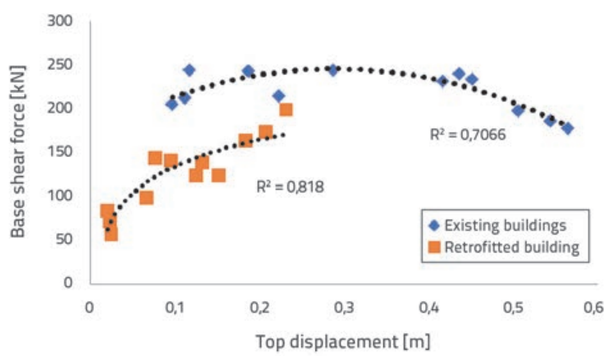

Figure 5. Comparison of maximum responses and dynamic analysis curves of the buildings

Antoniou and Pinho, suggested a procedure for obtaining absolute maximum displacements from the dynamic analysis results. According to this suggestion, dynamic analysis envelopes were plotted using the absolute maximum displacement versus the corresponding base shear (i.e., peak base shear within a ±0.5 s interval of the instance of maximum displacement) [37]. Dynamic analysis envelopes were determined by considering this situation. It can be seen from this figure that displacements and shear forces of the existing building are larger than the corresponding values for the retrofitted building. The shear walls of the retrofitted building limit displacements and provide additional rigidity to the buildings.
Capacity curves of the two compared buildings are presented in Figure 6. It can be seen that the base shear capacity of the retrofitted building is greater than the base shear capacity of the existing building. This graph proves contribution of shear walls as a conventional idea related to the capacity of the building.
Comparison of the capacity curve of the existing building and the maximum responses of the IDA with static pushover curves are presented in Figure 7.

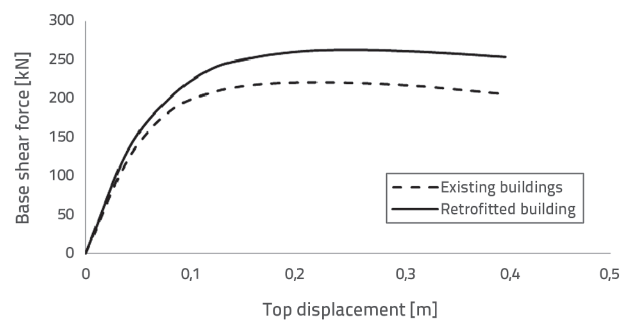

Figure 6. Comparison of capacity curves of buildings

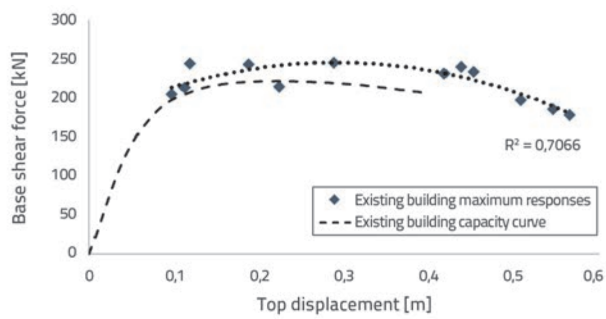

Figure 7. Comparison of capacity curve and maximum responses with dynamic analysis curve for existing building

The Figure 7 shows that the responses of the existing structure during the low amplitude, maximum responses, are close to the capacity curve of the building. Reponses obtained from high ground accelerations have larger displacement and smaller base shear force, whereas the other responses exceed the capacity curve. This situation shows that high ground accelerations cause collapse of the existing building.

Figure 8 shows comparison of the capacity curve of the retrofitted building and the maximum responses of the IDA. It can be seen that the responses of the retrofitted building are too close and under the capacity curve for low ground accelerations. However, the responses obtained from high ground accelerations do not exceed capacity curve of the building. This situation shows that the additional shear wall contributes to the capacity of the building.

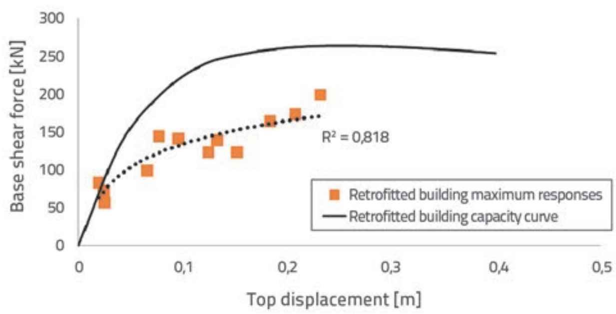

Figure 8. Comparison of capacity curve and maximum responses with dynamic analysis curve for retrofitted building

Figures 9-11 show interstorey drifts plotted on the basis of the scaled Imperial Valley, Loma Prieta and Kocaeli earthquakes, respectively, for the existing building. According to the Imperial Valley and Kocaeli earthquakes, maximum interstorey drift ratios were derived from soil class Z3 with 0.3g. The interstorey drift levels exceed the GV (Safety Damage Limit) performance level. The interstorey drifts were below this performance level at the other ground motions for soil class Z3. For Loma Prieta earthquake, the interstorey drifts exceeded the GV performance level for both Z3-0.3g and Z3-0.4g.

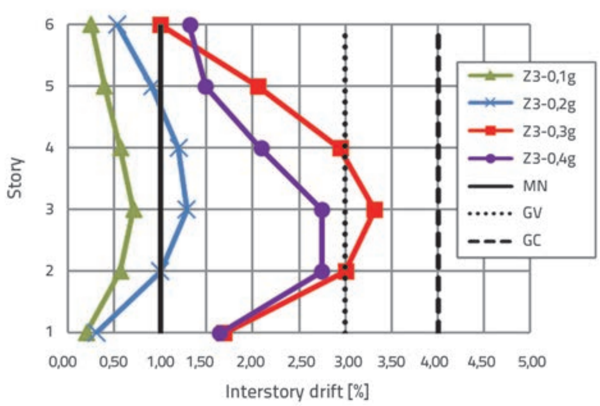

Figure 9. Interstorey drift according to scaled Imperial Valley earthquake for existing building

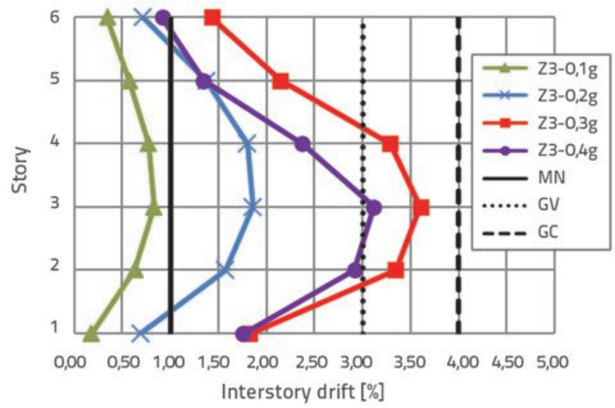

Figure 10. Interstorey drift according to scaled Loma Prieta earthquake for existing building

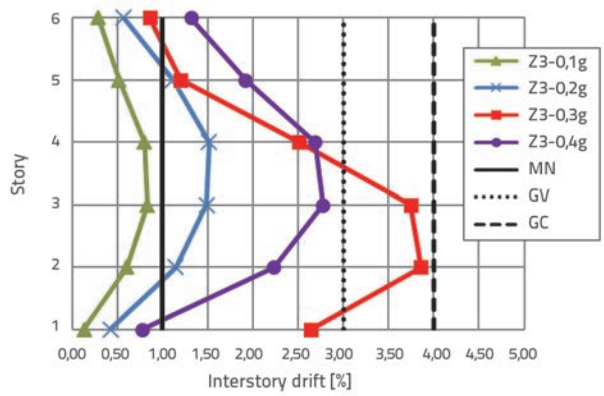

Figure 11. Interstorey drift according to scaled Kocaeli earthquake for existing building

Interstorey drifts were plotted for retrofitted building in Figure 12-14 on the basis of the scaled Imperial Valley, Loma Prieta, and Kocaeli earthquakes, respectively. According to the scaled earthquakes, maximum interstorey drift ratios were derived from Z3 soil class with 0.4g. The registered interstorey drift levels did not exceed the MN (Minimum Damage Limit) performance level except Z3-0.4g for all scaled earthquakes.

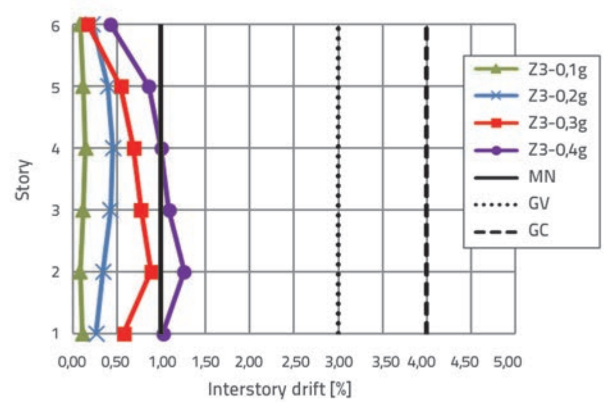

Figure 12. Interstorey drift according to scaled Imperial Valley earthquake for retrofitted building

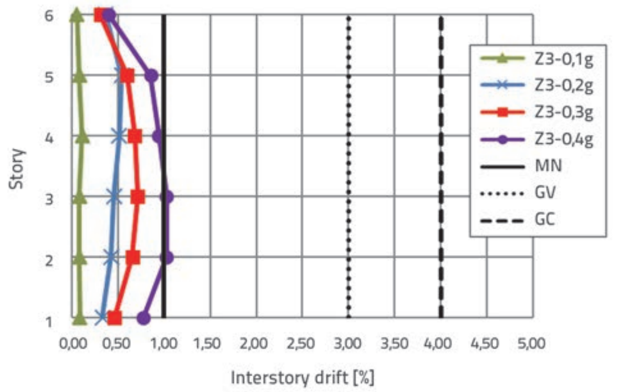

Figure 13. Interstorey drift according to scaled Loma Prieta earthquake for retrofitted building

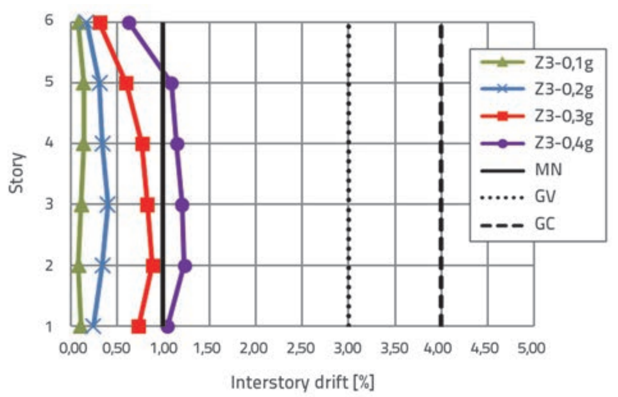

Figure 14. Interstorey drift according to scaled Kocaeli earthquake for retrofitted building

The damage levels are given according to the deformation levels defined for confined concrete in TSC. Figures 15-16 show damage at the lower end of columns C1 and C2 for the existing building. These figures clearly show that the deformations derived from the 0.3g and 0.4g ground acceleration levels for Z3 soil class pass beyond the collapse damage limit (GC) for the lower end of these confined concrete columns. However, deformations registered below the 0.2g ground acceleration are unable to retort the safety damage limit (GV), while for 0.1g the deformations do not exceed the minimum damage limit (MN) for confined concrete.

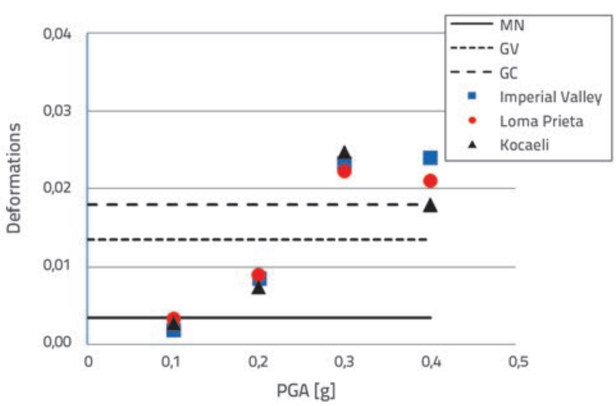

Figure 15. Existing building damage at the lower end of column C1 for various earthquakes

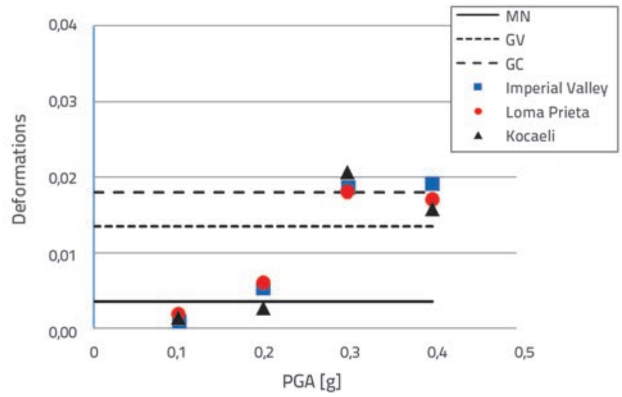

Figure 16. Existing building damage at the lower end of column C2 for various earthquakes

Figures 17-18 show confined concrete damage levels at the left end of beam B1 and the right end of beam B2 for the existing building. According to these figures, the deformations obtained for the ends of selected beams from the 0.3g and 0.4g ground accelerations are far above the collapse damage limit (GC) for confined concrete. However, deformations that occurred under the 0.2g ground acceleration exceed the safety damage limit (GV) except for the scaled Kocaeli earthquake. For 0.1g the damage registered for confined concrete exceeds the minimum damage limit (MN) except at the scaled Imperial Valley earthquake.

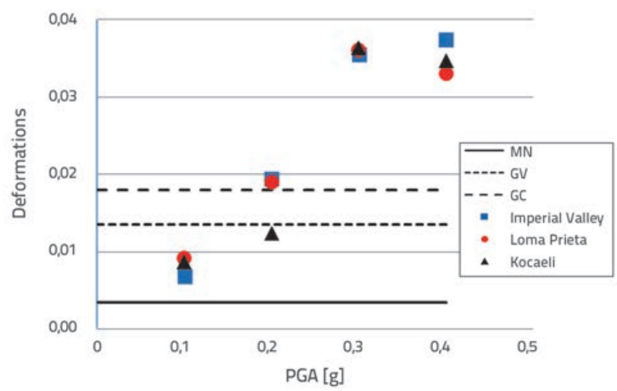

Figure 17. Existing building damage at the left end of beam B1 for various earthquakes

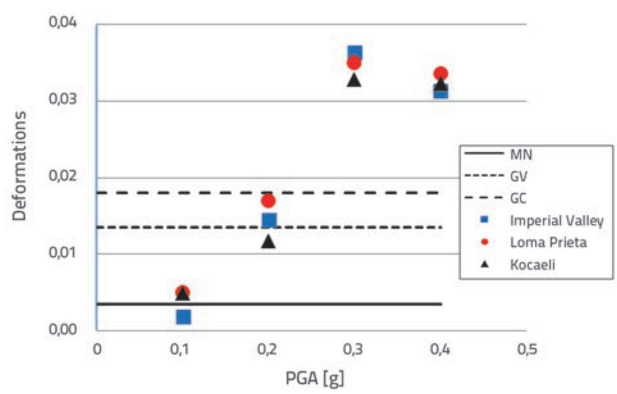

Figure 18. Existing building damage at the right end of beam B2 for various earthquakes

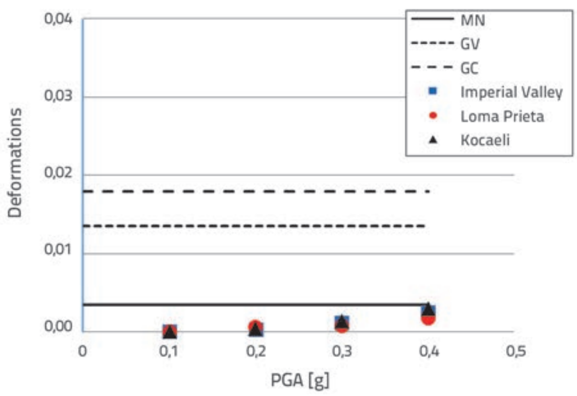

Figure 19. Retrofitted building damage at the lower end of column C1 for various earthquakes

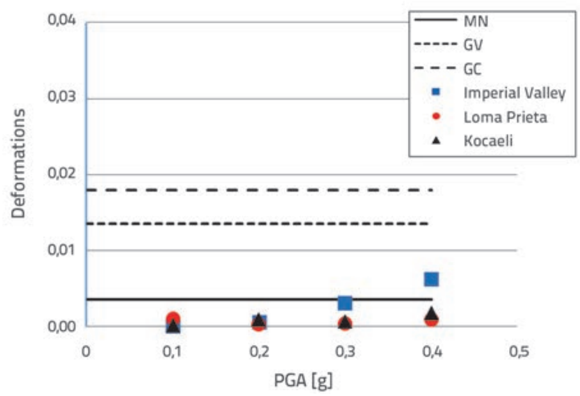

Figure 20. Retrofitted building damage at the lower end of column C2 for various earthquakes

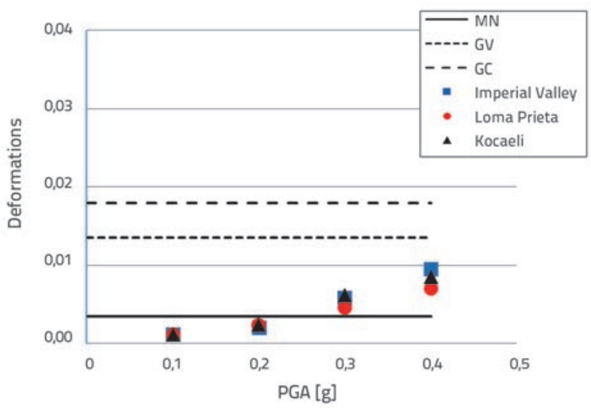

Figure 21. Retrofitted building damage at the left end of beam B1 for various earthquakes

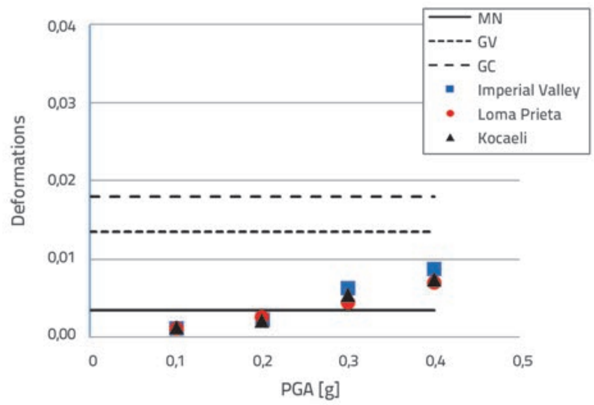

Figure 22. Retrofitted building damage at the right end of beam B2 for various earthquakes

The confined concrete damage at the lower end of columns C1 and C2 is given in Figures 19-20 for the retrofitted building. According to these figures, the damages remain under the MN damage limit for the lower end of these columns except for column C2 for the scaled 0.4g Imperial Valley earthquake. In terms of investigated beams, the registered damage is parallel with the columns. However, deformations remain under the GV performance level while exceeding the MN performance level for selected scaled 0.3g and 0.4g earthquakes. This situation is shown in Figures 21 and 22.

5. Conclusion

Seismic behaviour of existing reinforced concrete buildings is investigated in this paper before and after seismic retrofitting. An existing reinforced concrete building was selected and retrofitted by shear walls. Both incremental dynamic analyses and static pushover analyses were performed on the building for two cases using the distributed plastic hinge approach. For the nonlinear dynamic analyses, three acceleration time series of earthquake records were selected and scaled according to the design spectrum determined in the Turkish Seismic Code. Maximum responses and capacity curves of the existing and retrofitted buildings were obtained and plotted. Dynamic analysis curves were obtained by considering maximum responses. Interstorey drifts and damage were evaluated for selected elements of the buildings on the basis of the current code. The following can be concluded based on the results obtained in this study:

- Capacity curve of the retrofitted building is larger than that of the existing building. This situation shows that the shear wall increases the capacity of the building.
- For the existing building, responses of the existing structure during the low amplitude ground motion are close to the capacity curve of the same model. However, responses obtained from high ground accelerations have larger displacement and lower base shear force. This situation shows that the high seismic excitation causes collapse of the non-retrofitted building.
- In terms of retrofitted building, the responses of the structure obtained from dynamic analysis are too close to each other. Moreover, capacity curve values and seismic response of the structure under the low magnitude seismic motion are nearly the same. But, responses obtained from high amplitude seismic motion did not exceed the capacity curve of the retrofitted building. This situation shows the contribution of additional shear wall to the capacity of building and damping factor of the shear wall to overall system.

- Interstorey drifts obtained from soil class Z3 at 0.3g level exceed the Safety Damage Limit (GV) for the existing building according to the Imperial Valley and Kocaeli earthquakes. However, Z3-0.3g and Z3-0.4g interstorey drifts exceed the GV performance level for the Loma Prieta earthquake.
- Since all analysis results were investigated in terms of retrofitted building, the maximum interstorey drift ratios were obtained from soil class Z3 with 0.4g. The registered interstorey drift levels did not exceed the MN performance level except for Z3-0.4g.
- The deformations registered for the 0.3g and 0.4g ground accelerations exceeded the collapse damage limit (GC) for confined concrete at the lower end of the selected columns of the existing building. In addition to this, deformations that occurred under the 0.2g ground acceleration were unable to retort the safety damage limit (GV), whereas deformations do not pass beyond the minimum damage limit (MN) for confined concrete at 0.1g ground acceleration. Moreover, with regard to confined concrete damage levels at the end of the selected beams for the existing building, the deformations obtained from the 0.3g and 0.4g ground accelerations were far above the collapse damage limit (GC) for core concrete.
- The confined concrete damage located at the lower end of these columns remained under the Minimum Damage Limit (MN) for all records at the end of the selected columns except for the 0.4g Imperial Valley earthquake for C2 column for retrofitted building. In terms of investigated beams, the registered damage is parallel to the selected columns. However, deformations remained under the GV performance level, while deformations exceeded the MN performance level for selected earthquakes scaled to 0.3g and 0.4g.

As a result, additional shear walls increased rigidity and capacity of the building, limited lateral displacements, and decreased damage, because they were properly retrofitted and well detailed. On the basis of this detailed study, dynamic analysis envelopes should be obtained and compared with static pushover curves so as to increase reliability of evaluation of the existing reinforced concrete buildings before and after retrofitting.

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