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Araştırma / Research

APPLICATION OF WET-MIXED SHOTCRETE PANELS FOR RETROFITTING PURPOSES

Pınar TEYMÜR (ORCID: 0000-0002-5961-1020)[*]
Sumru PALA (ORCID: 0000-0002-4255-6328)

İnşaat Mühendisliği Bölümü, İnşaat Fakültesi, İstanbul Teknik Üniversitesi, İstanbul, Türkiye

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ABSTRACT

The work presented in this study aimed to investigate the behaviour of vulnerable reinforced concrete frames after retrofitted with wet-mixed shotcrete panels. Application of this retrofitting technique on a representative bare frame is examined by pushover and nonlinear dynamic time history analysis (NDTHA). Depending on the capacity curves attained in the pushover analyses, maximum base shear capacity of the frame is increased from 16% to 39% of the seismic weights after retrofitted with shotcrete panels. On the other hand, the displacement capacity of the retrofitted frame decreased by 2.8 times compared to the bare frame. The NDTHA performed for the selected earthquake records produces the results in terms of strains of confined concrete and longitudinal reinforcement that several cross sections of the bare frame have attained the *collapse state* defined in Turkish Earthquake Code (TEC 2007) however the retrofitted frame performs within the *minimum damage state*.

Keywords: Wet-mixed shotcrete, strengthening, reinforced concrete frame, pushover analysis, nonlinear dynamic time history analysis

GÜÇLENDİRME AMACI İLE ISLAK KARIŞIMLI PÜSKÜRTME BETON PANELLERİN UYGULANMASI

ÖZ

Bu çalışma, ıslak karışımlı püskürtme beton paneller ile güçlendirilmiş zayıf betonarme çerçevelerin davranışlarını araştırmayı hedeflemektedir. Bu güçlendirme yönteminin temsili yalın bir çerçeve üzerine uygulaması, itme ve zaman tanım alanında doğrusal olmayan dinamik analizleri ile incelenmiştir. İtme analizi sonucunda elde edilen kapasite eğrileri; püskürtme beton paneller ile güçlendirilmiş çerçevenin maksimum taban kesme kuvveti kapasitesinin, sismik ağırlığının %16'dan %39'a çıktığı gözlemlenmiştir. Diğer taraftan güçlendirilmiş çerçevenin deplasman kapasitesi, yalın çerçeveye göre 2.8 kat azalmıştır. Seçilen deprem kayıtları ile yapılmış olan zaman tanım alanında doğrusal olmayan analiz ile üretilen sonuçlar; yalın çerçeveye ait çeşitli enkesitler sarılmış beton ve ana donatı davranışları bakımından incelendiğinde, Deprem Yönetmeliğinde tanımlanmış olan *göçme bölgesinde* olduğunu göstermiştir. Ancak çerçeve güçlendirildikten sonra kesitlerin, *minimum hasar bölgesinde* kaldıkları gözlemlenmiştir.

Anahtar Kelimeler: Islak karışımlı püskürtme beton, güçlendirme, betonarme çerçeve, itme analizi, zaman tanım alanında doğrusal olmayan dinamik analizi

[*]Corresponding author / Sorumlu yazar. Tel.: +90 212 285 3795; e-mail / e-posta: teymurp@itu.edu.tr

1. INTRODUCTION

As a retrofitting technique in vulnerable RC frames, construction of shotcrete panel is quite beneficial against conventional shear wall when formwork and workmanship is expensive and/or accessing to the work area is difficult. One layer of simply prepared formwork is adequate for the construction of shotcrete panel. The wet-mixed concrete is applied with a certain speed, and it will stick to the surface easily so that proper replacement of the wall is achieved.

Strengthening a damaged RC frame with forming a thin concrete wall on the existing masonry walls [1-3], or using shotcrete on special wall-like structures in lieu of masonry walls [4] showed that, these kinds of easily applicable retrofitting techniques increases lateral load carrying capacity and lateral rigidity of the structure.

Strengthening of infill walls using shotcrete is typically used in strengthening of damaged and/or undamaged masonry buildings in Turkey as stated in the studies [5-7].

Teymur et al. [8] used wet-mixed sprayed concrete to form a panel within vulnerable RC bare frames. Nearly ½ scale, one bay and one story specimens were tested by using the loads simulating earthquake effects, Figure 1. The experimental work is composed of testing one bare; one conventional shear wall retrofitted RC frames and two vulnerable RC frames retrofitted by wet-mixed shotcrete panels. The shear wall and the panel are connected to the frames through shear studs used at four edges of them to create strong bond. Also a model is proposed for the behavior of the frames that are retrofitted with wet-mixed shotcrete. The details of the model for the proposed retrofitting technique can be found in [8].

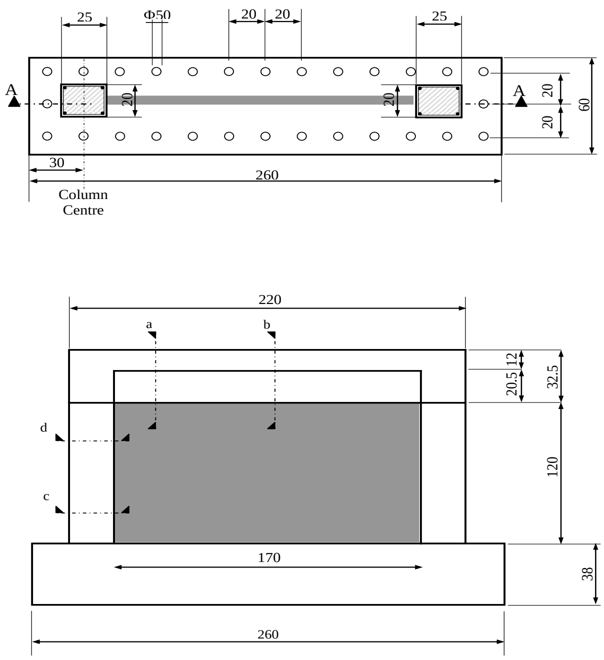


Figure 1. Geometry of the specimens [8]

In this paper, the investigated retrofitting technique has been applied on a planar frame of a building representing the typical RC frame type structures in Turkey. Two outer spans of this frame are retrofitted with shotcrete panels through the height of the frame continuously. The models, which represent the inelastic behavior of shotcrete panel used in this study, are adapted from the calibrated models that were developed for the test specimens in [8]. Pushover and nonlinear dynamic time history analysis have been performed on the bare and the retrofitted frames.

2. MATERIAL AND METHODS

2.1. Application of the Retrofitting Technique to a Representative Frame

In this section, the proposed retrofitting technique is applied on a 2D frame of a building representing the typical RC frame type structures in Turkey [9, 10]. The elevation of the frame can be seen in Figure 2. The frame has six storeys with a total height of 21 m. Storey heights are identical and equal to 3.5 m and span lengths are 5 m. The slabs have a thickness of 15 cm.

Two outer spans of the frame, namely AB and DE, are filled with 15 cm thick shotcrete panels as can be seen in Figure 3 for retrofitting purpose. Since the geometry of the representative frame is almost 3 times bigger than the tested specimens, the panel thickness is chosen as 15 cm.

Steel quality is S420a and concrete quality for frame and shotcrete panel are C16 and C20, respectively. The dimensions and longitudinal reinforcement data of columns are presented in Table 1. The longitudinal reinforcement data of beams are presented in Table 2. All the beams are 300 mm in width and 600 mm in depth. The concrete cover of beams and columns are selected as 40 mm. The lateral reinforcement of columns and beams are ϕ10/200.

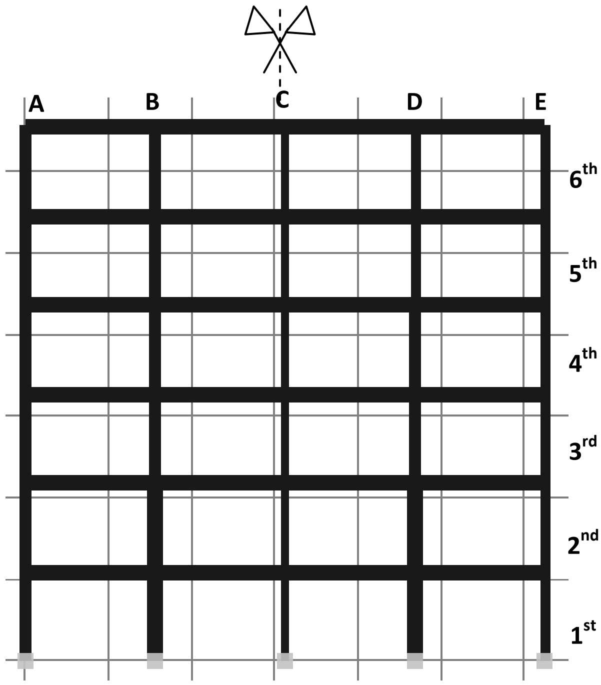


Figure 2. The representative frame

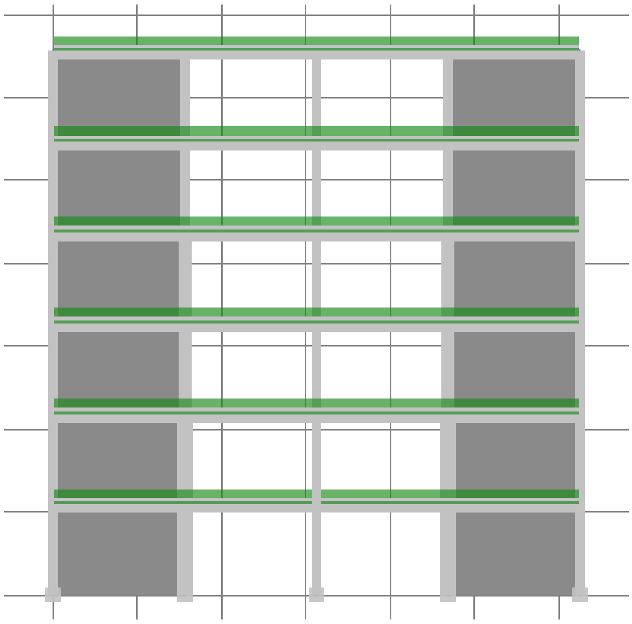


Figure 3. Retrofitting of the frame by shotcrete panels

Table 1. The dimensions and reinforcement of the columns

Story	Axes		
	A	**B**	**C**
5 - 6	300/400 mm 4Φ18 + 4Φ16	300/400 mm 4Φ16 + 4Φ16	400/300 mm 4Φ16 + 4Φ14
3 - 4	300/400 mm 4Φ18 + 4Φ16	300/500 mm 4Φ20 + 4Φ20	500/300 mm 4Φ20 + 4Φ20
1 - 2	300/400 mm 4Φ18 + 4Φ16	300/600 mm 4Φ22 + 4Φ20	600/300 mm 4Φ20 + 4Φ20

Table 2. The reinforcement of the beams

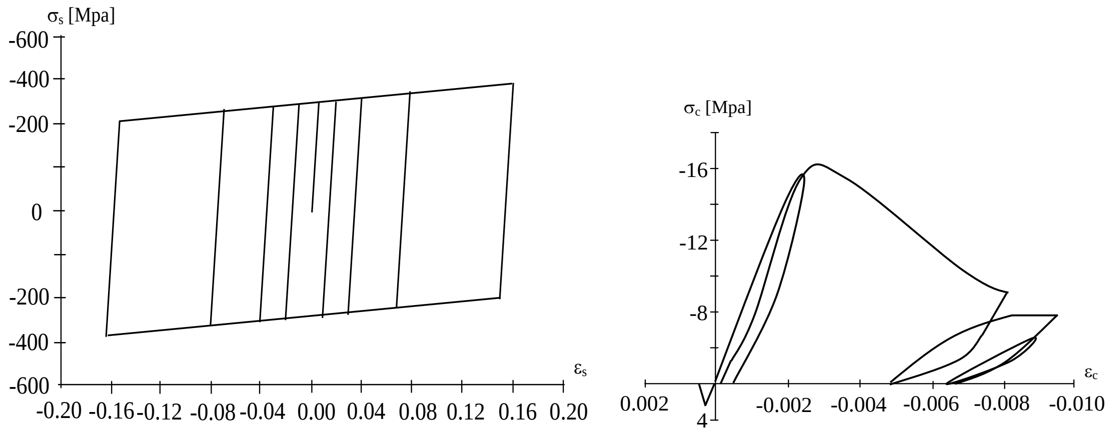
Story	Place of reinforcement	A – B Beam			B – C Beam		
		Left support	Span	Right support	Left support	Span	Right support
6 - 5	Top	2ϕ12+2ϕ18	2ϕ12	2ϕ12+2ϕ18	2ϕ12+2ϕ18	2ϕ12	2ϕ12+2ϕ14
	Bottom	3ϕ16	3ϕ16	3ϕ16	3ϕ16	3ϕ16	3ϕ16
4 – 3	Top	2ϕ14+3ϕ20	2ϕ14	2ϕ14+3ϕ20	2ϕ14+3ϕ20	2ϕ14	2ϕ14+2ϕ20
	Bottom	4ϕ16	4ϕ16	4ϕ16	4ϕ16	4ϕ16	4ϕ16
2 – 1	Top	3ϕ14+2ϕ22	3ϕ14	3ϕ14+3ϕ22	3ϕ14+3ϕ22	3ϕ14	3ϕ14+1ϕ22
	Bottom	4ϕ16	4ϕ16	4ϕ16	4ϕ16	4ϕ16	4ϕ16

2.2. Analytical Study

Analytical study is conducted by using SeismoStruct [11], a finite element program, to evaluate the effect of the retrofitting technique on the response of the frame. Two types of analysis have been carried out; pushover and NDTHA. The frame is idealized to have fixed type support in the analysis.

Bilinear steel model is used to model the reinforcement given in Figure 4a. This is a uniaxial bilinear stress-strain model with kinematic strain hardening, whereby the elastic range remains constant throughout the various loading stages, and the kinematic hardening rule for the yield surface is assumed as a linear function of the increment of plastic strain.

Uniaxial nonlinear constant confinement concrete model is used for confined concrete seen in Figure 3b.



a) Steel

b) Concrete

Figure 4. Constitutive models used in analytical study

Sum of the dead loads and 30% of the live loads are taken into account in the calculation of seismic weight. The mass values used in the analysis are given in Table 3 for two cases.

Table 3. Concentrated mass values at each floor levels

Storey	Storey mass [$kNsec^2/m$]	
	Bare frame	**Shotcreted frame**
6	49.0	55.6
5	78.7	92.0
4	79.7	93.0
3	80.3	93.6
2	81.2	94.6
1	80.9	94.2

The building is assumed to be constructed on firm soil (Z2 type) at seismic zone 1 defined in TEC [12].
The first natural vibrational periods for the bare frame and the shotcreted frame are T_1= 0.992 sec and T_1= 0.430 sec, respectively. First mode shapes of both cases can be seen in Figure 5.

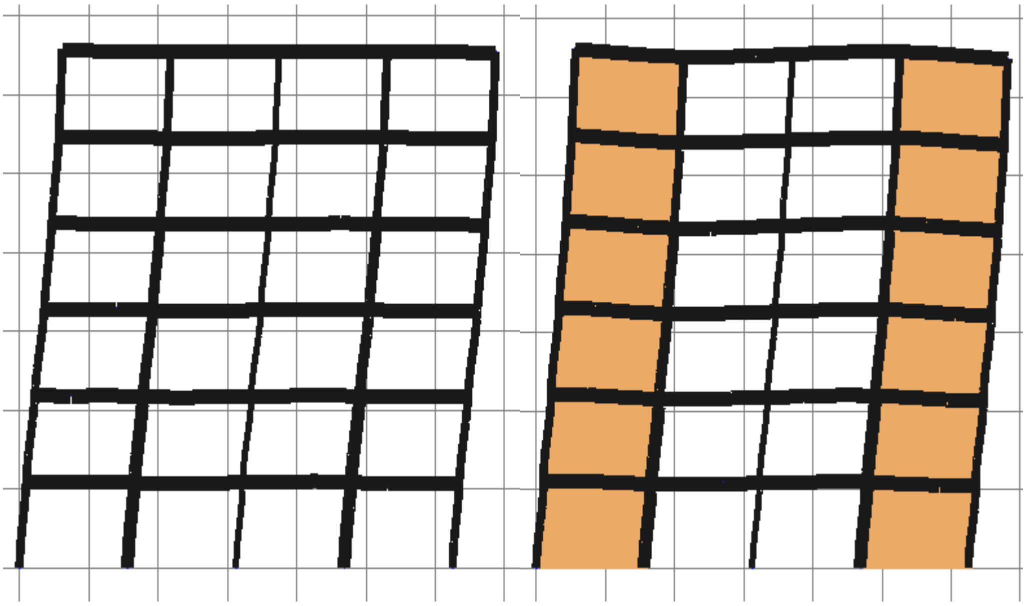


Figure 5. First mode shapes of bare and retrofitted frame

2.2.1. Pushover Analysis

In Table 4, the lateral load ratios to be used in the pushover analysis are calculated. They are obtained from the static moments of the storey seismic weights to the ground. The obtained lateral load distribution is close to 1st vibration mode of the frames.

2.2.2. Nonlinear Dynamic Time History Analysis

NDTHA is also performed. Three earthquake records were taken from Pacific Earthquake Engineering Research Centre, (PEER) [13] data bank to generate artificial accelerograms. The detailed information about the earthquakes is given in Table 5. The original earthquake acceleration records are drawn in Figure 6, 7 and 8.

Table 4. Forces applied during pushover analysis

a) Bare frame

Storey	W_i (kN)	H_i (m)	W_i*H_i	(W_i*H_i)/∑(W_i*H_i)
6	480.87	21.0	10098.27	0.197
5	772.25	17.5	13514.38	0.264
4	782.01	14.0	10948.14	0.214
3	787.73	10.5	8271.16	0.162
2	796.60	7.0	5576.20	0.109
1	793.38	3.5	2776.83	0.054
∑	4412.84		5217.63	1.000

b) Retrofitted frame

Storey	W_i (kN)	H_i (m)	W_i*H_i	(W_i*H_i)/∑(W_i*H_i)
6	546.50	21.0	11476.48	0.193
5	903.50	17.5	15811.25	0.266
4	913.26	14.0	12785.65	0.215
3	918.98	10.5	9649.25	0.162
2	927.85	7.0	6494.96	0.109
1	924.63	3.5	3236.20	0.054
∑	5134.72		6060.53	1.000

Table 5. Earthquake records

Earthquake	Date	Station / Direction	M	PGA (g)
Erzincan	13.03.1992	Erzincan / EW	6.9	0.496
İzmit	17.08.1999	İzmit / 090	7.4	0.220
Düzce	12.11.1999	Bolu / 090	7.1	0.822

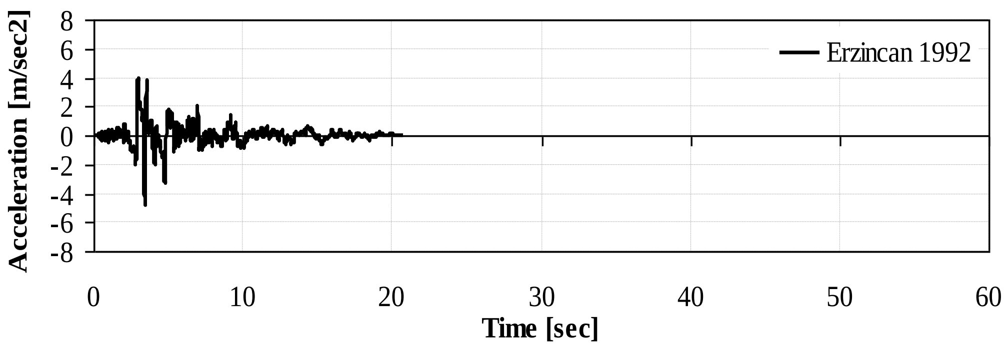


Figure 6. The acceleration record of Erzincan Earthquake

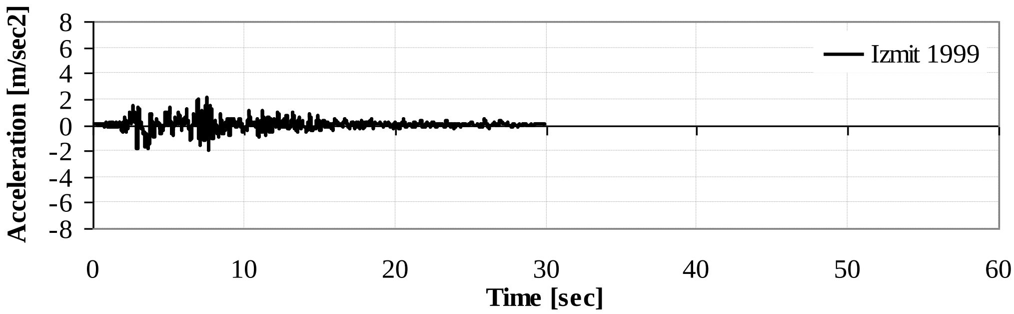


Figure 7. The acceleration record of İzmit Earthquake

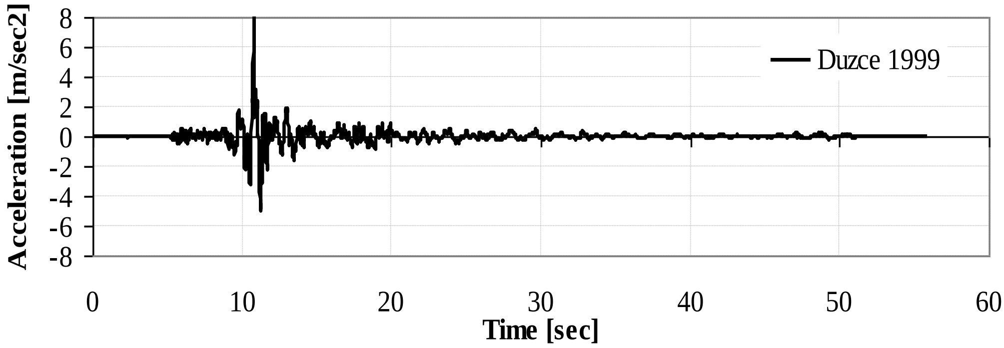


Figure 8. The acceleration record of Düzce Earthquake

The original acceleration records of the three earthquakes given above are modified respect to the acceleration spectra defined for Z2 type soil given in [12]. Two versions of the earthquakes are produced named as "service" and "design" earthquakes, [10].

The modified earthquake records obtained are given in Figure 9 as "service" and in Figure 10 as "design" earthquakes.

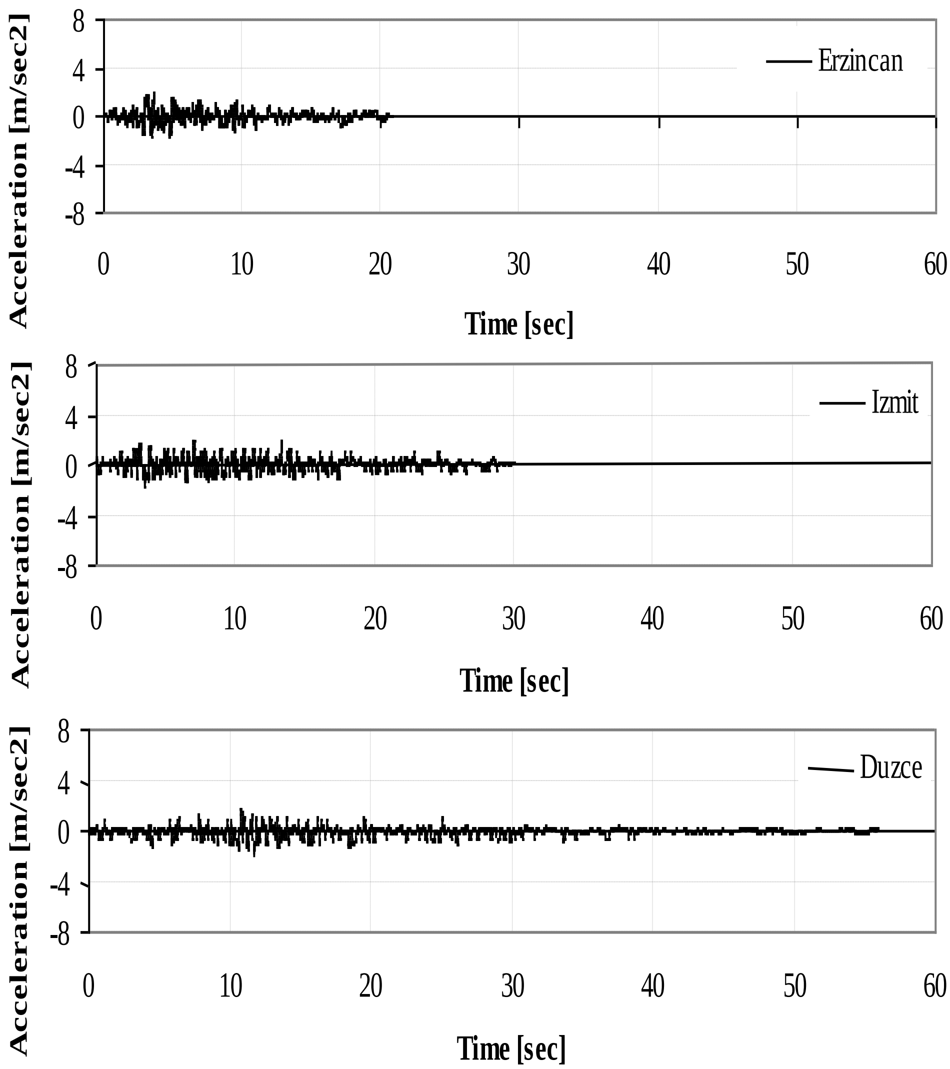


Figure 9. "Service" type acceleration records

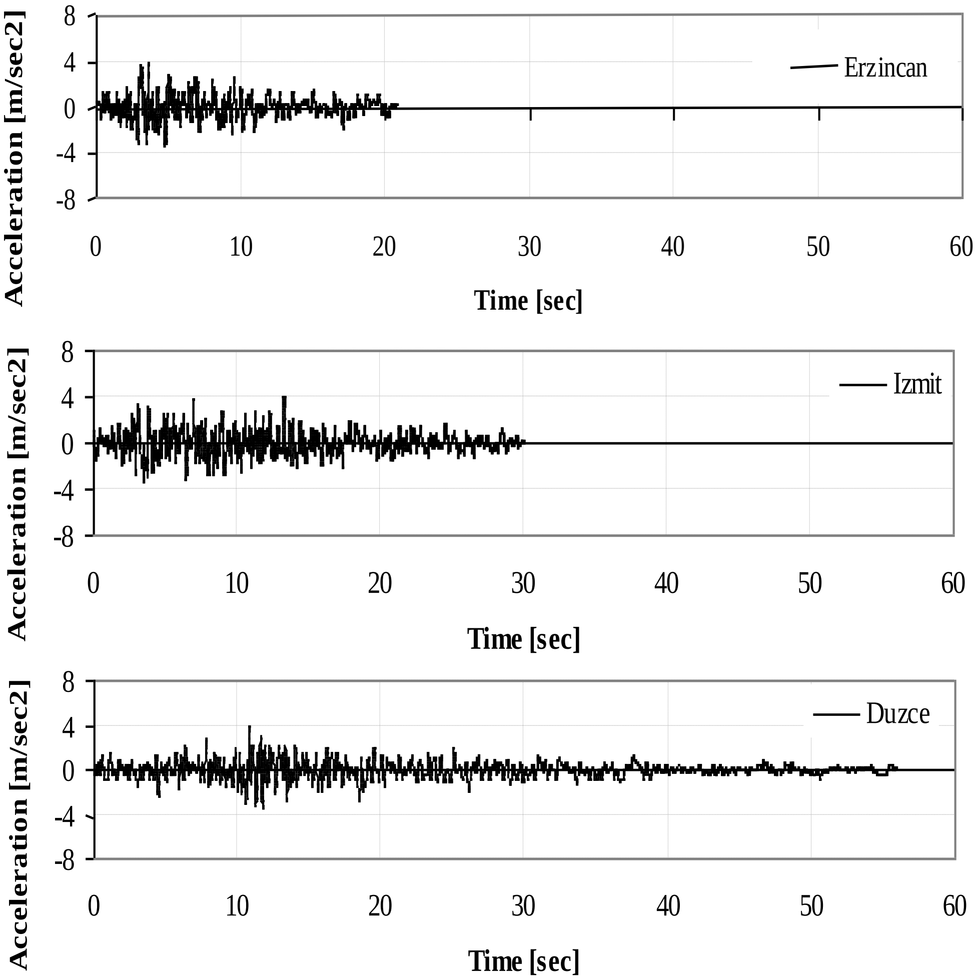


Figure 10. “Design” type acceleration records

The mean spectrum of the modified design earthquakes is drawn with the design spectrum for Z2 type soil in Figure 11.

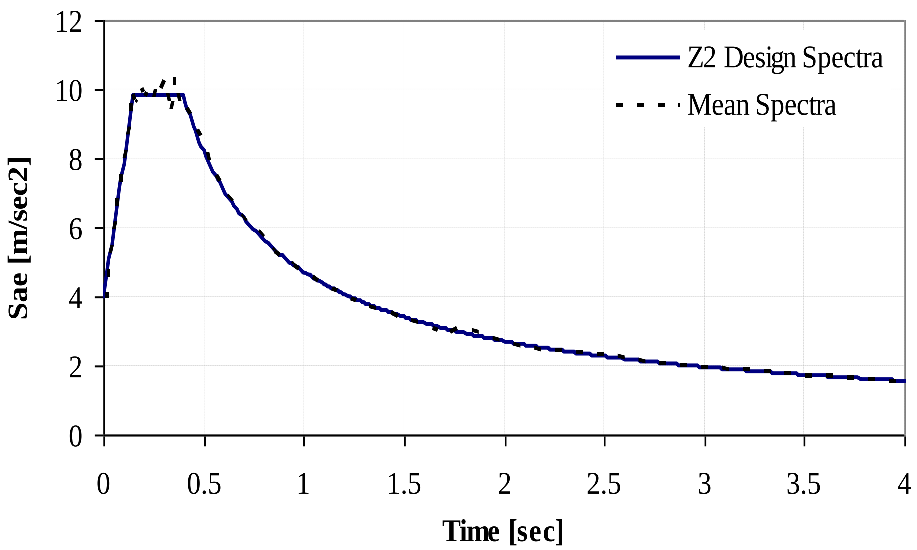


Figure 11. Design spectrum defined in TEC, 2007

3. RESULTS AND DISCUSSION

The base shear-top displacement relation determined by the pushover analysis is presented in Figure 12. The frame's resistance and rigidity has increased significantly after retrofitting with shotcrete panels. The stiffness and the maximum strength of the retrofitted frame are expectedly much larger. In the case of the bare frame and the retrofitted frame, maximum strength is about 0.16W and 0.39W, respectively. The displacement capacity of the retrofitted frame decreased by 2.8 times compared to the bare frame.

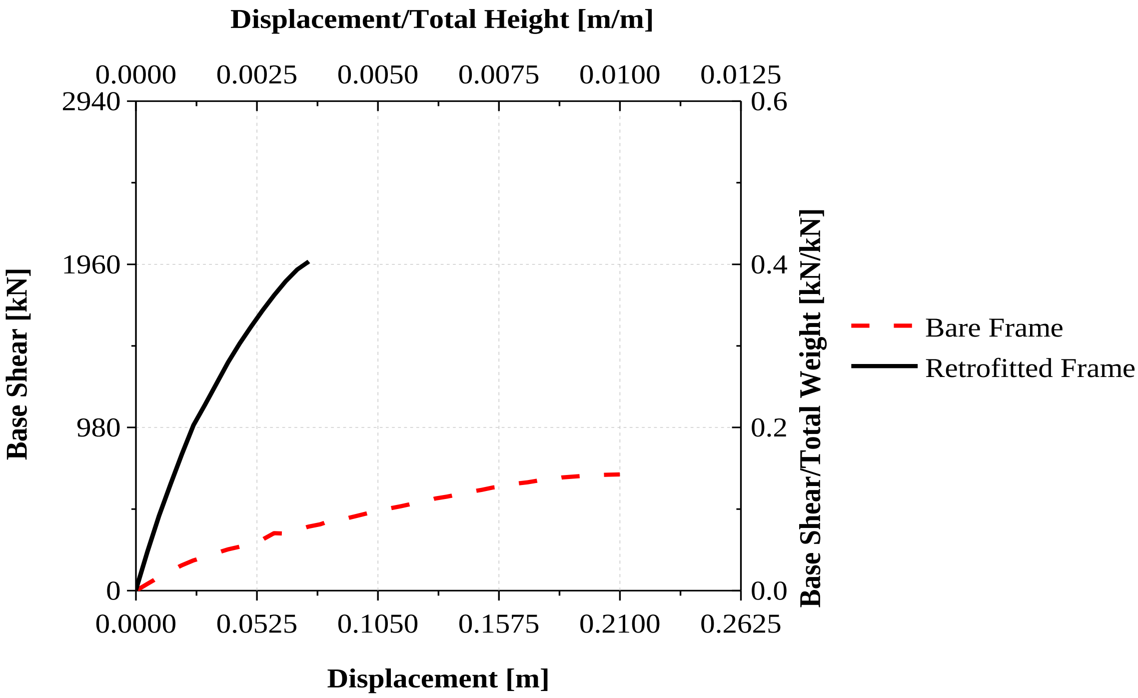


Figure 12. Base shear-top displacement and base shear/total weight-top displacement/total height diagram

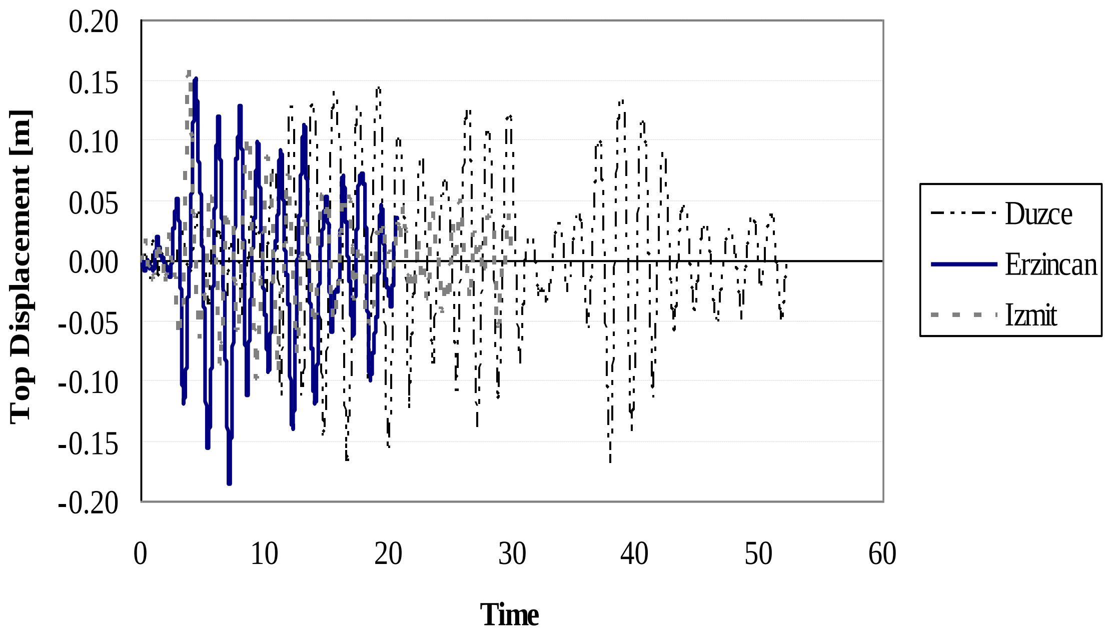
Top displacement-time graphs for the bare and retrofitted frame under service and design earthquakes are given in Figures 13 and 15, respectively. The displacement demand for bare frame is 0.18 m under service earthquake. After retrofitting, the displacement demand decrease to 0.04 m. Under design earthquakes the displacement of the bare frame increases to 0.32 m. After infilling of two spans with shotcrete panels, this demand decreases to 0.09 m.

Base shear-time graphs for the bare and retrofitted frame under service and design earthquakes are given in Figures 14 and 16, respectively. The shear force demand for bare frame is 592 kN under service earthquake. After retrofitting, the displacement demand becomes 1562 kN. Under design earthquakes the shear force demand of the bare frame increases to 715 kN. After infilling of two spans with shotcrete panels, this demand becomes 1963 kN.

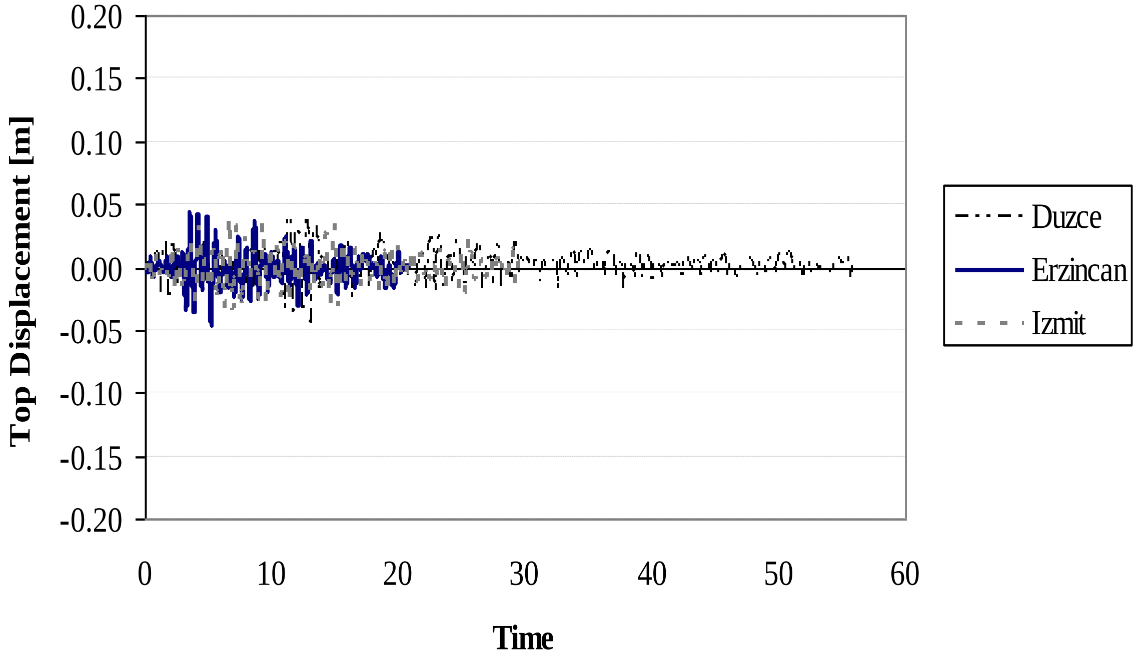
The nominal shear stress obtained for the design earthquake is 1026 kN/m^2 at the 1st floor, which is lower than the nominal shear stress, 2130 kN/m^2, obtained in the experimental study in [8].

Displacement, interstorey drift ratio and shear force demands of retrofitted frames are compared with the bare frames'. In Figures 17, 18 and 19, the comparisons of the results obtained for service and design earthquake are given. The results given in the figures are the average of results of the three earthquakes.

The displacement demands of the bare frame under service and design earthquakes are 0.17 m and 0.29 m, respectively. After placing shotcrete panels in the two spans, these values decrease to 0.04 m and 0.12 m. The interstorey drift ratio is below 1% under service and design earthquakes. Under design earthquakes, it is slightly higher than 1% only at the 1st storey.

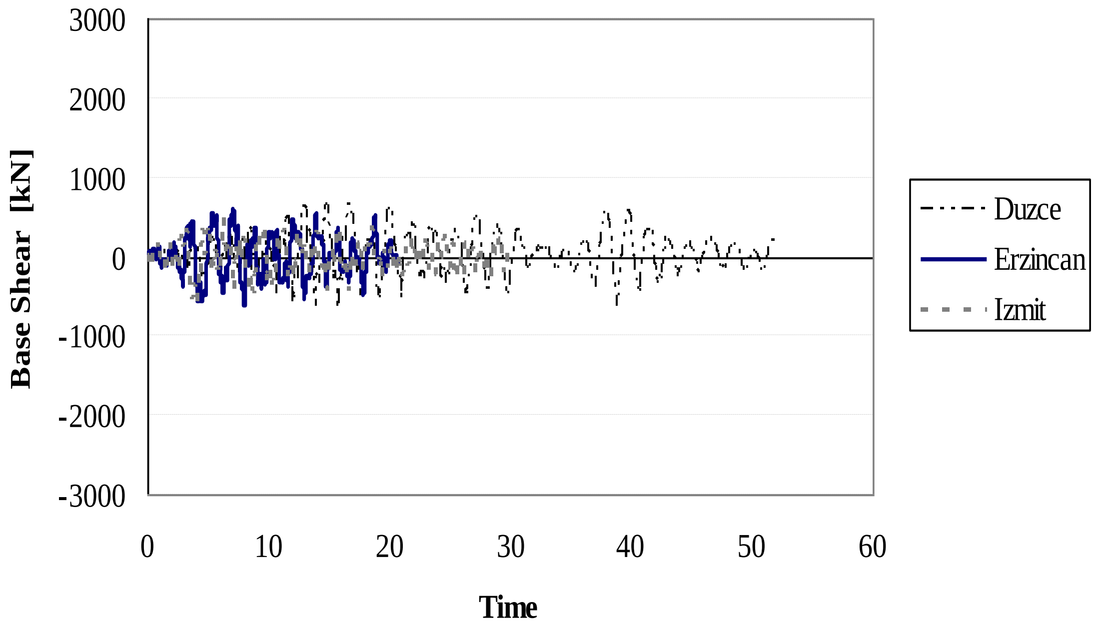


a) Bare frame

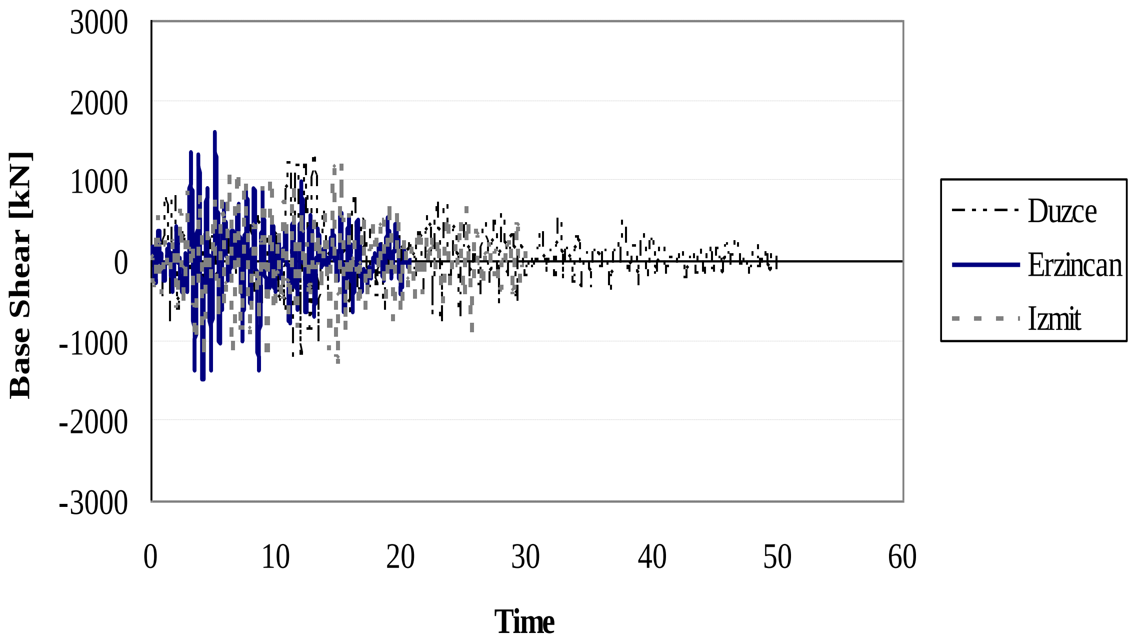


b) Retrofitted frame

Figure 13. Time versus top displacement graphs for bare and retrofitted frame under service earthquakes



a) Bare frame



b) Retrofitted frame

Figure 14. Time versus base shear force graphs for bare and retrofitted frame under service earthquakes

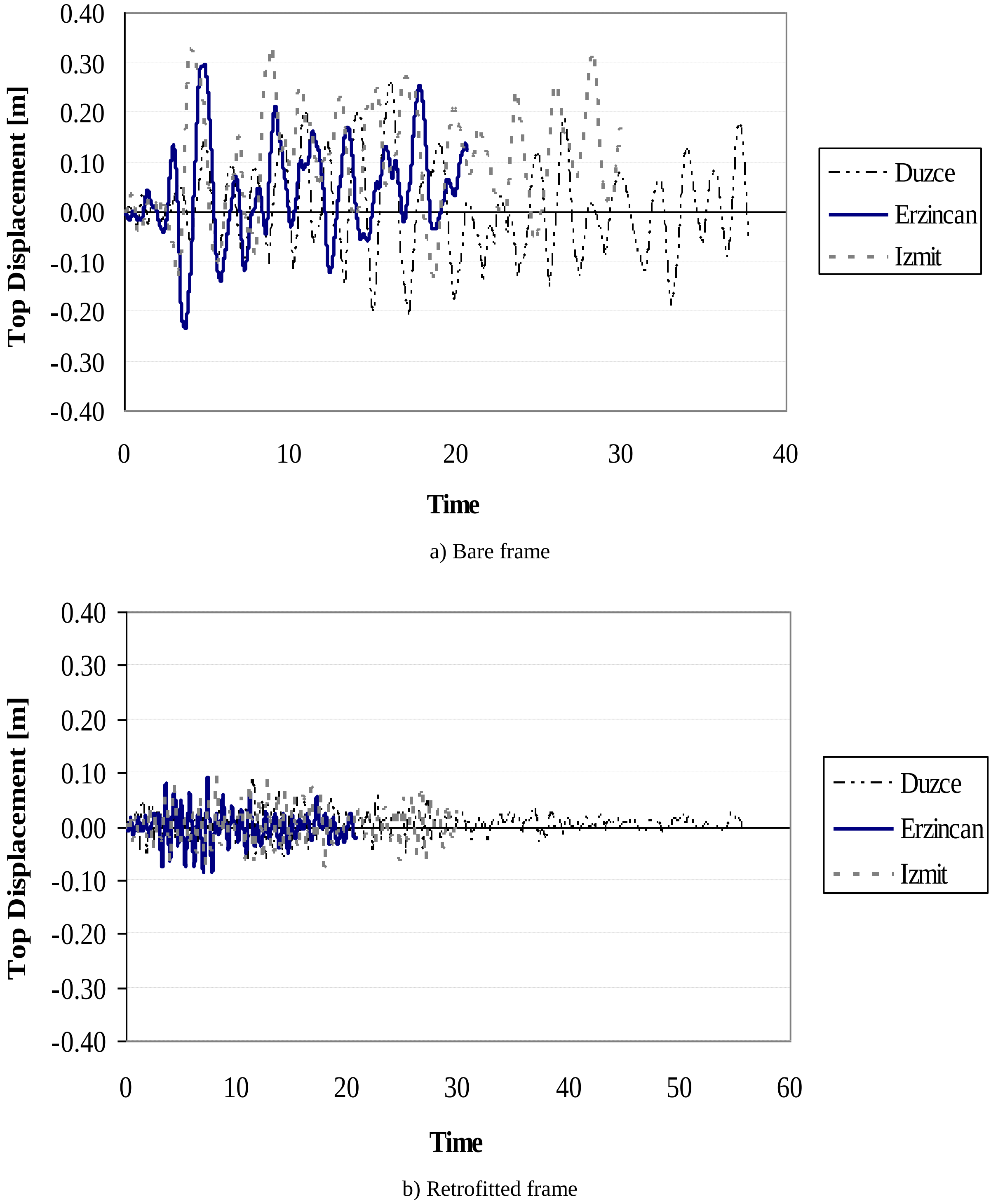
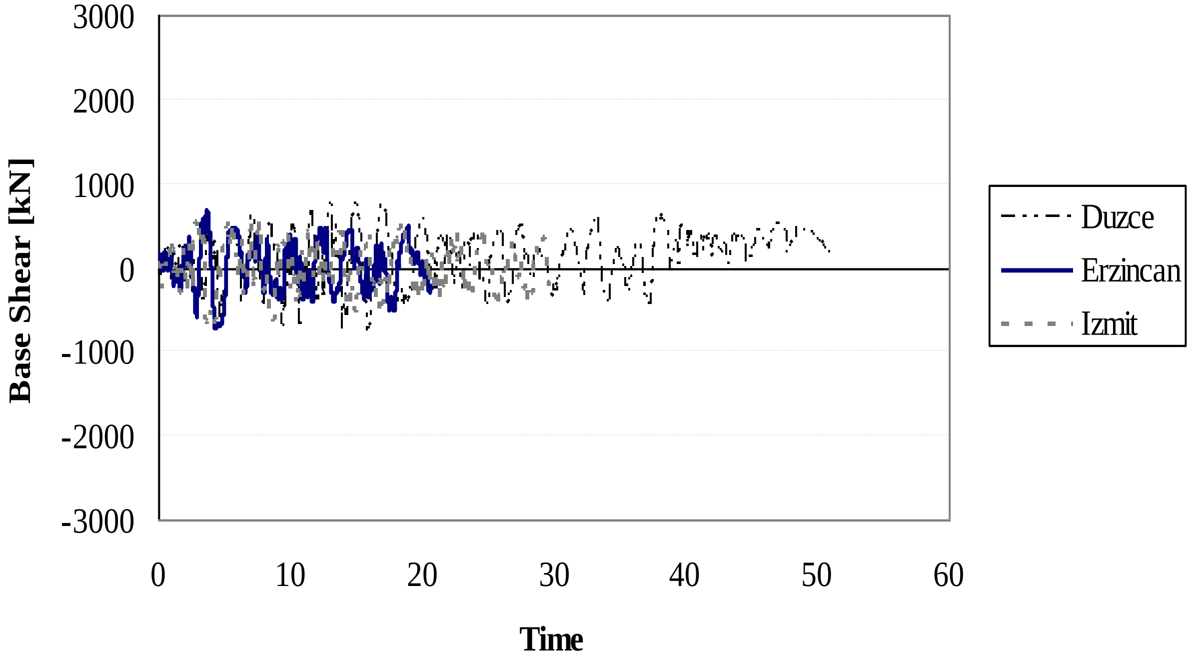
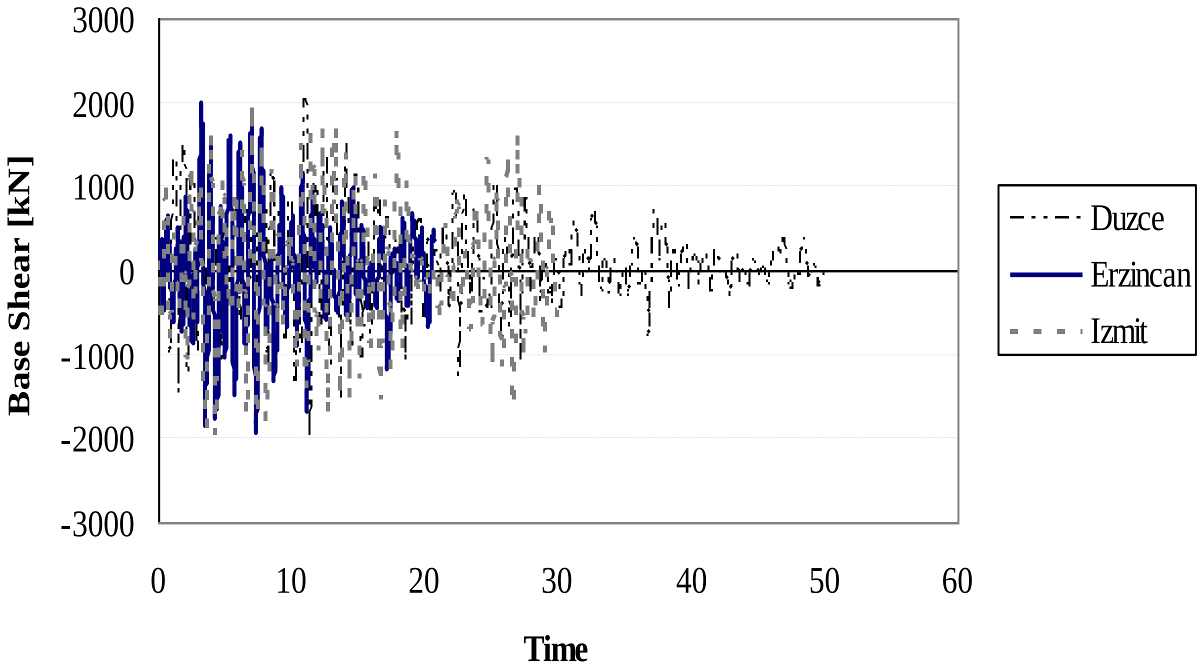


Figure 15. Time versus top displacement graphs for bare and retrofitted frame under design earthquakes

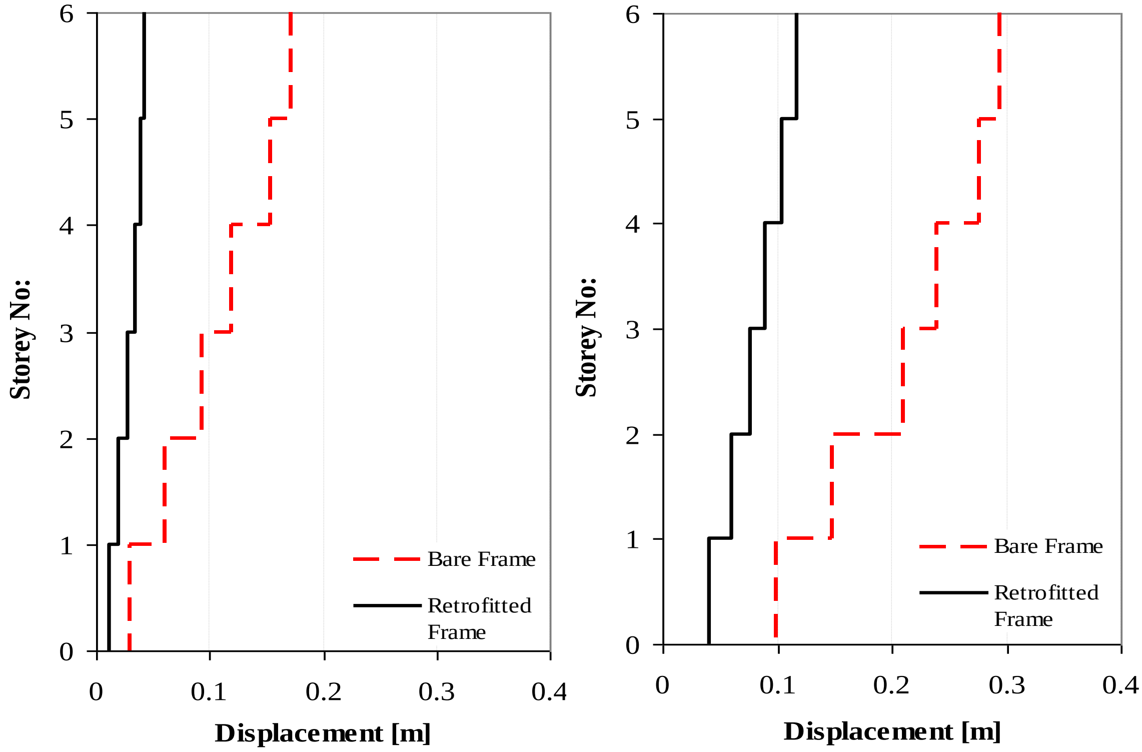


a) Bare frame



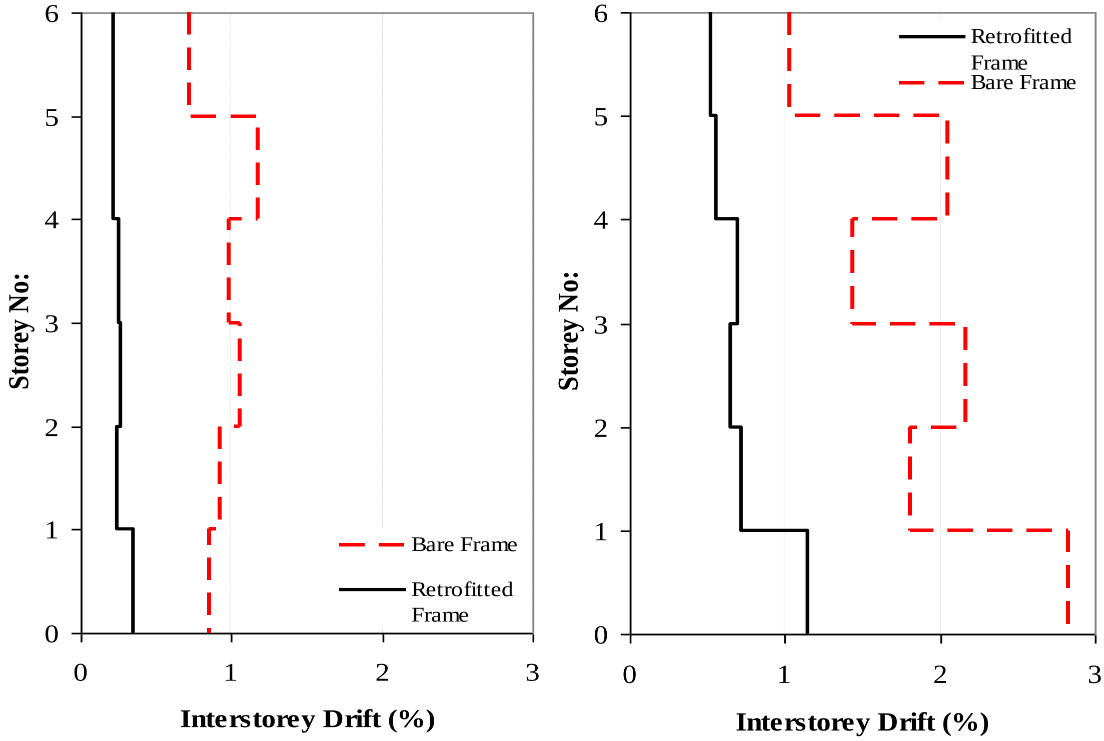
b) Retrofitted frame

Figure 16. Time versus base shear force graphs for bare and retrofitted frame under design earthquakes



a) Service earthquake b) Design earthquake

Figure 17. Comparison of the maximum story displacements of the frame with and without shotcrete panel for service and design earthquakes



a) Service earthquake b) Design earthquake

Figure 18. Comparison of the maximum interstorey drift of the frame with and without shotcrete panel for service and design earthquakes

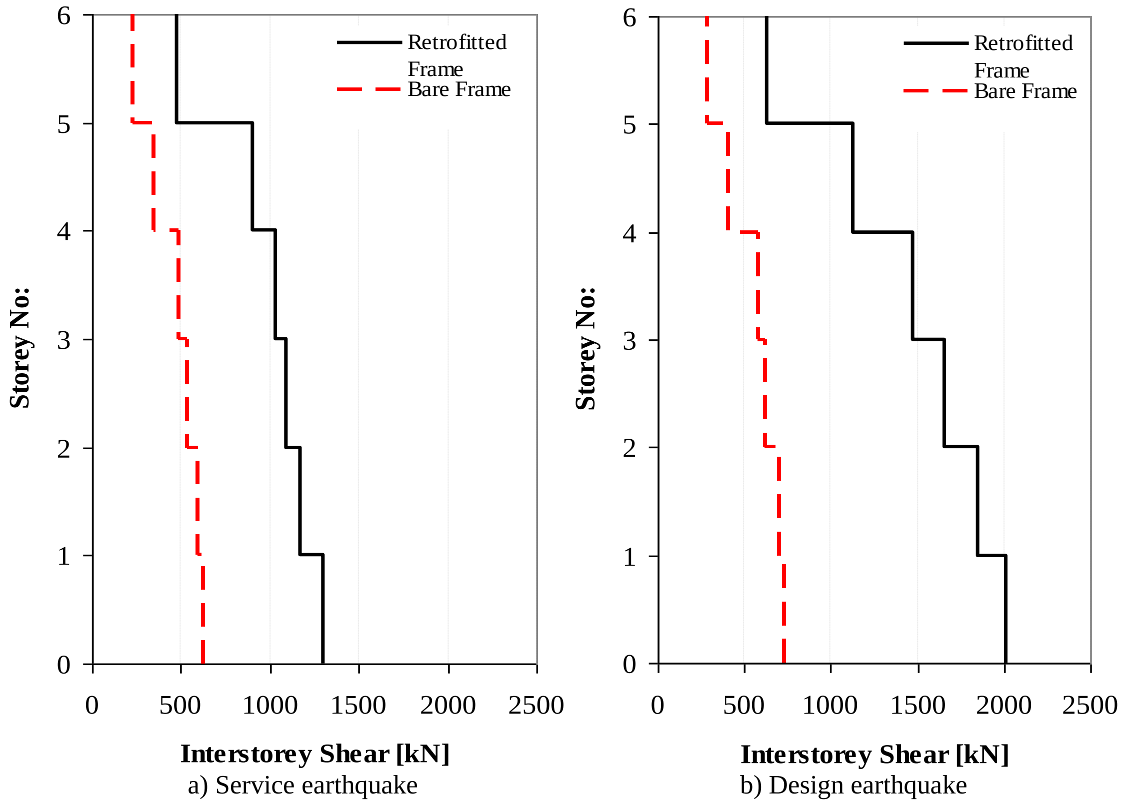


Figure 19. Comprasion of the maximum interstorey shear force of the frame with and without shotcrete panel for service and design earthquakes

The observed sectional performances of the bare and retrofitted systems in the case of the design earthquakes are illustrated in Figure 20. Various colours used in this figure correspond to performance levels defined in [12] presented in Table 6.

Table 6. Limit strain values to define damage states in structural members according to TEC [12]

Damage States in Structural Members	Strain Limit	Place of Deformation
Minimum Damage Limit (MNc)	<- 0.0035	The outer-most fibre of the concrete of the section
Minimum Damage Limit (MNs)	> 0.01	Longitudinal reinforcement
Safety Limit (GVc)	< -0.0085	The outer fibre of the concrete within the transversal reinforcement
Safety Limit (GVs)	> 0.040	Longitudinal reinforcement
Collapse Limit (GCc)	< -0.011	The outer fibre of the concrete within the transversal reinforcement
Collapse Limit (GCs)	> 0.060	Longitudinal reinforcement

c: concrete, s: steel,
compression (-), tension (+) in strain limits

It can be concluded that in the case of the bare frame, vulnerability of the ground floor is relatively higher than the upper floors. The story mechanism for the ground floor is the common damage pattern observed for the used earthquake records. After introducing the shotcrete panels to the outer spans; even though yielding of reinforcement for some critical section has been observed, both beams and columns stayed in performance level named as *Minimum Damage State*.

For the bare and retrofitted cases, the strain history of the main reinforcement and the confined concrete at the bottom section of the column, which is located in Axis C at 1st floor, is given for design earthquakes in Figures 21 and 22, respectively.

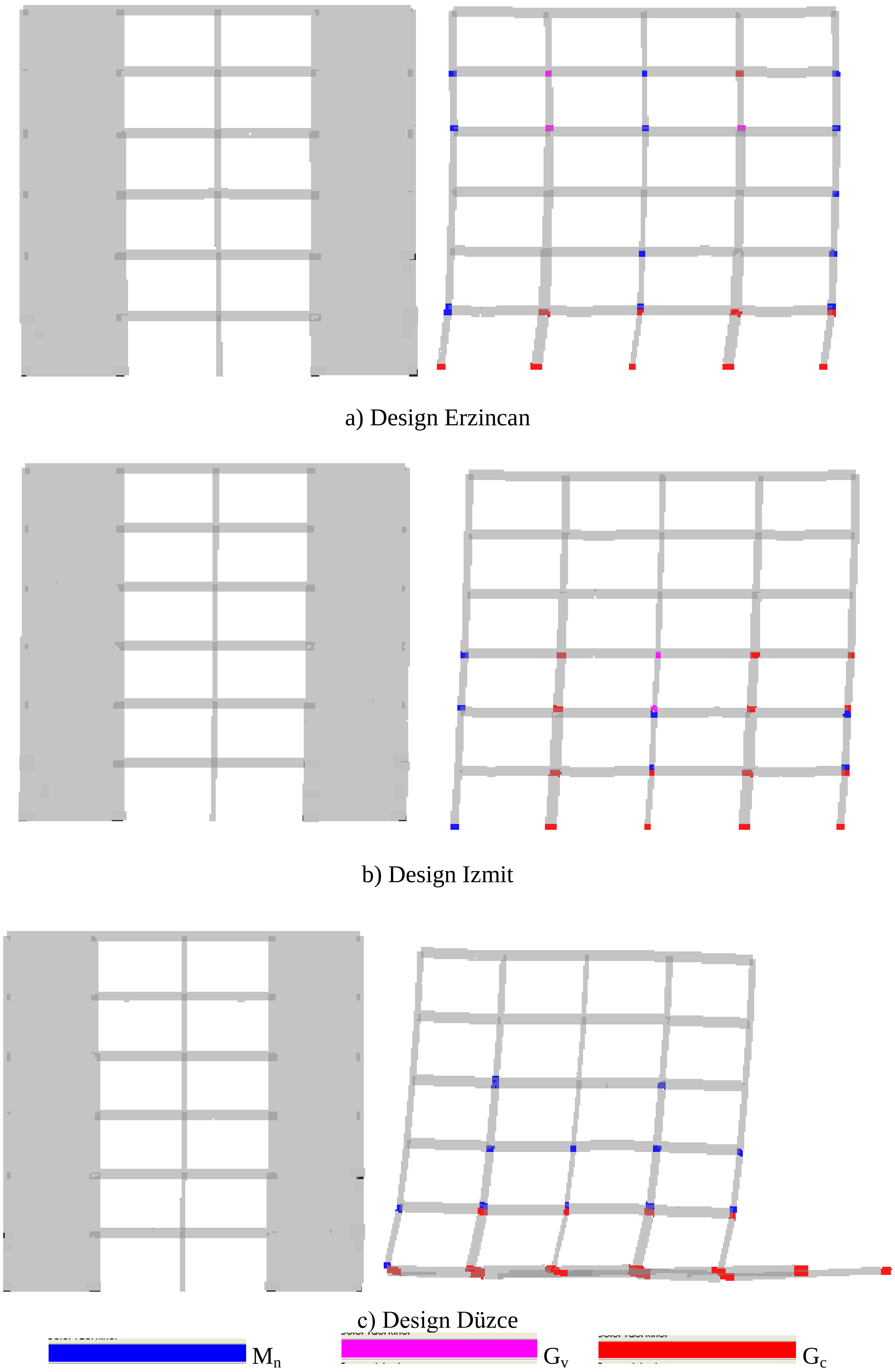
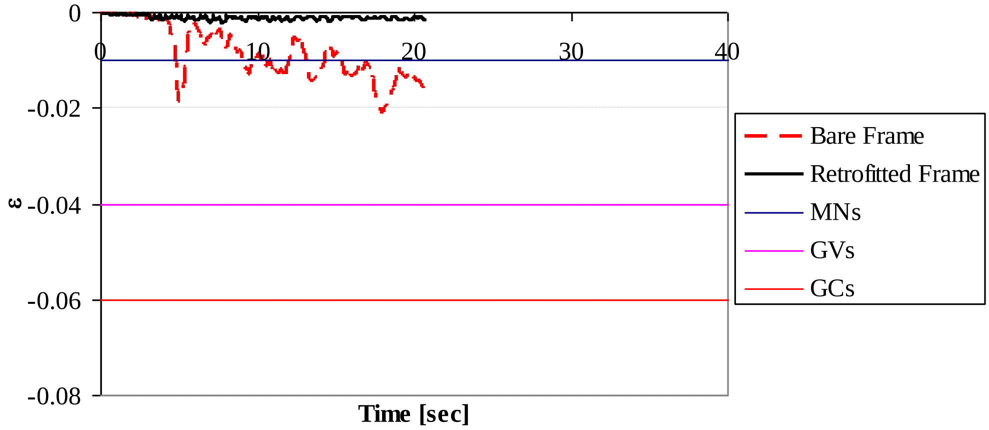
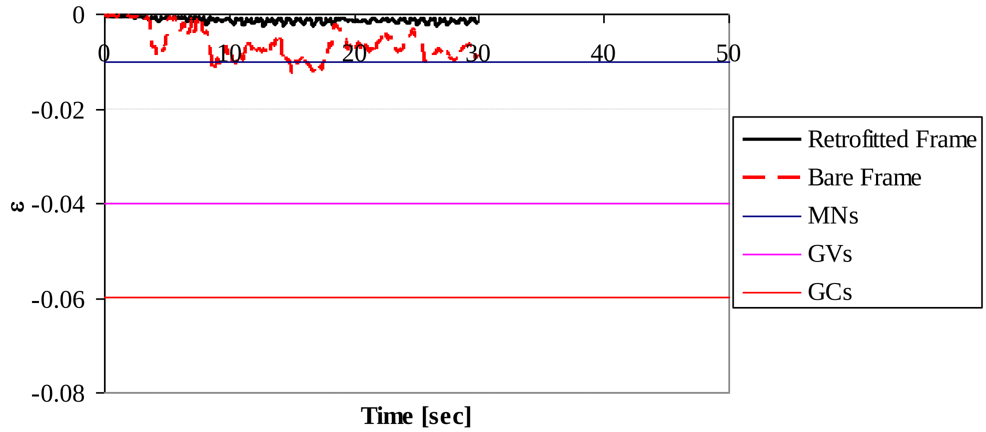


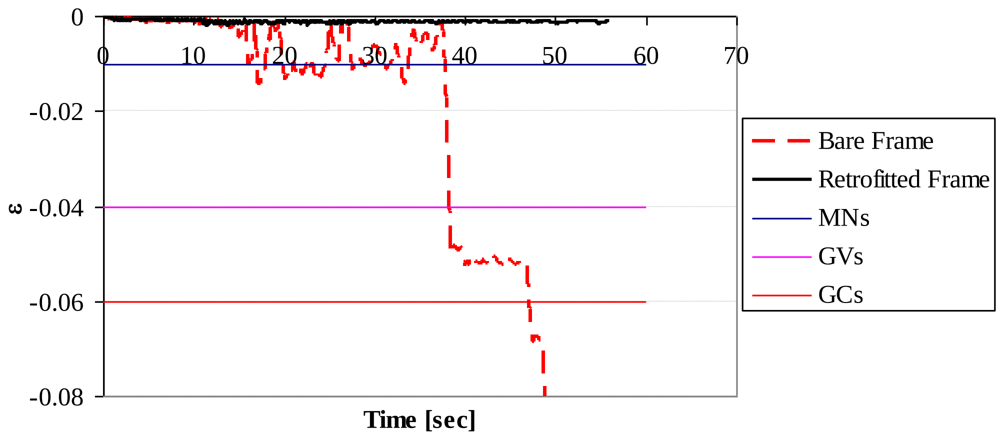
Figure 20. Performance of the bare and retrofitted systems under design earthquakes



a) Erzincan earthquake



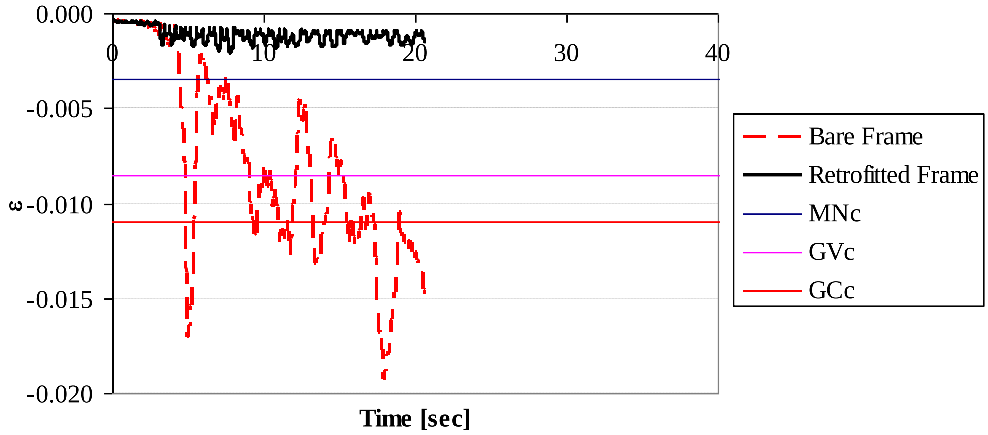
b) Izmit earthquake



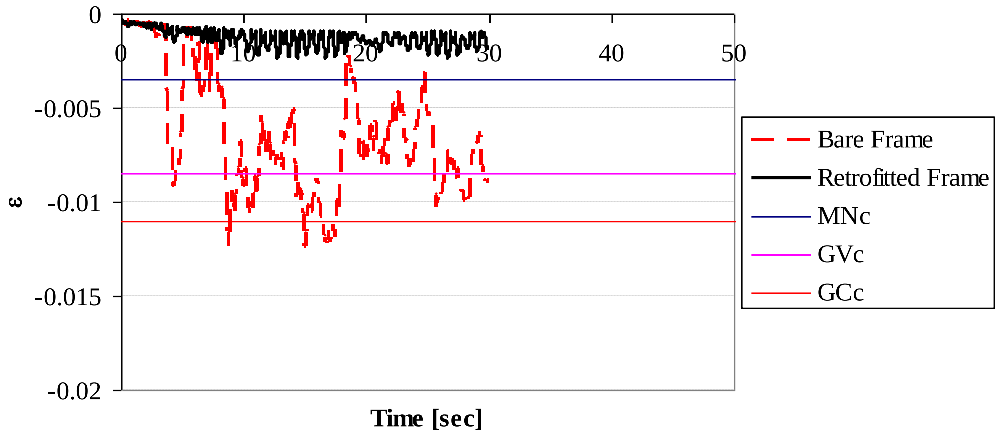
c) Düzce earthquake

Figure 21. Comparison of the reinforcement strains with and without shotcrete panel for design earthquakes

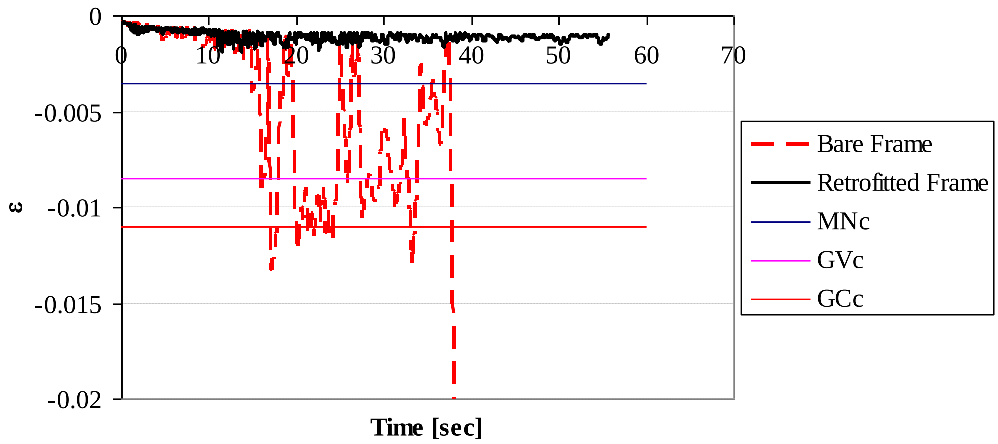
In the case of bare frame, the performance level of the reinforcement is attained to the *Collapse State* for Düzce Earthquake. However, for Erzincan and Izmit Earthquakes the performance levels correspond to *Safety State*. After introducing the shotcrete panels to the system, the reinforcement performs in the *Minimum Damage State* for all design earthquakes.



a) Erzincan earthquake



b) Izmit earthquake



c) Düzce earthquake

Figure 22. Comparison of the confined concrete strains with and without shotcrete panel for design earthquakes

In the case of bare frame, the performance level of the confined concrete is attained to the *Collapse State* for all design earthquakes. After introducing the shotcrete panels to the system, the confined concrete performs in the *Minimum Damage State* for all design earthquakes.

4. CONCLUSIONS

A planar frame of a building representing the typical RC frame type structures in Turkey is retrofitted with wet-mixed shotcrete panels. Two outer spans of the bare frame are retrofitted with shotcrete panels. Pushover and nonlinear dynamic time history analysis have been performed on the bare and the retrofitted frames. The frame's resistance and rigidity has increased significantly after retrofitting with shotcrete panels. These retrofitted frames can also carry the design loads defined in [12].

Future studies can focus on different number of stories and variations of infilled spans to validate the result of this study.

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