Improvement of Earthquake Performance of Low Rise RC Buildings using Shotcrete Panels

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SUMMARY:

Application of shotcrete directly on the partition walls or creation of conventional shear walls in the vulnerable reinforced concrete (RC) structures are commonly used as retrofitting techniques in Turkey. Construction of shotcrete infill panels in a planar bare frame of a six-storey RC building for retrofitting is studied analytically here. Adaptive pushover and nonlinear time history dynamic analyses were conducted to investigate the performance of this frame with these panels. The displacement capacity of the retrofitted frame decreased by 2.8 times compared to the bare frame. The nonlinear time history analyses performed for the selected earthquake records indicate that several cross sections of the bare frame have attained the collapse state defined in Turkish Code for Earthquake Resistant Design (2007), however the retrofitted frame performs within the minimum damage state. The interstory drift ratios obtained for the retrofitted specimens are around 1%.

Keywords: retrofitting, earthquake performance, reinforced concrete frame, shotcrete panel

1. INTRODUCTION

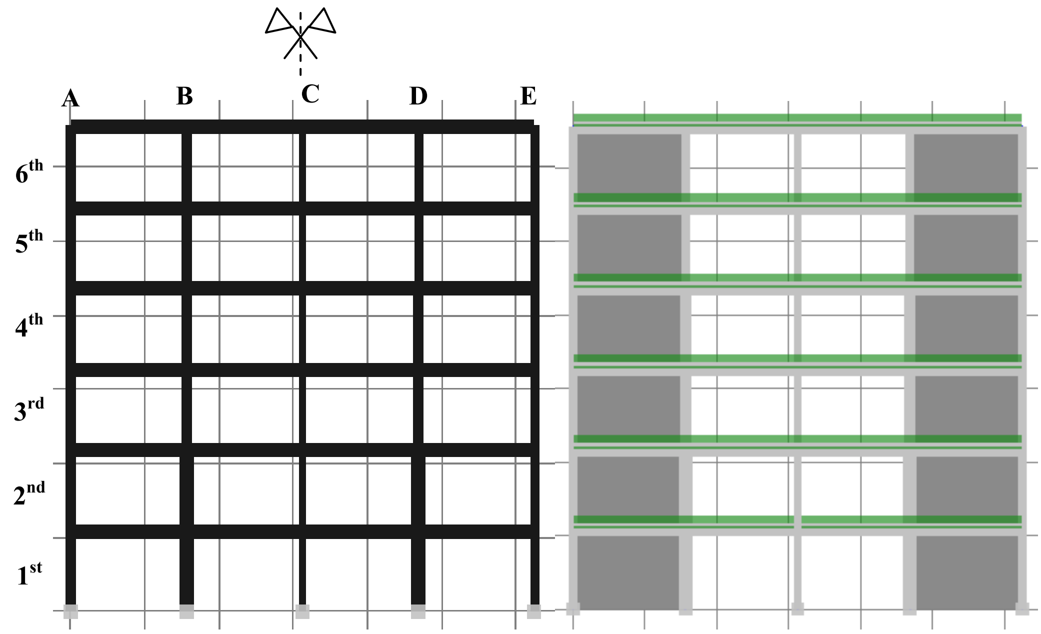
Teymur et al. 2008 and 2012 had conducted experiments on single story, single bay concrete frames retrofitted with cast-in-situ panels made from wet-mixed shotcrete. The RC frames were chosen to represent the vulnerable low-rise low-cost reinforced concrete structures in Turkey; especially constructed before the validation of the last two earthquake codes. The experimental studies showed that the wet-mixed shotcrete panels added to vulnerable RC frames increase the lateral load carrying capacity, the lateral rigidity and the energy dissipation capacity of the system. In this paper, the behavior of these shotcrete panels predicted from these experimental studies is adapted to the analytical studies. Two outer spans of a 2D bare frame of a building representing the typical RC frame type structures in Turkey are retrofitted with these shotcrete panels. Pushover and nonlinear dynamic time history analysis (NDTHA) analysis are performed using SeismoStruct to evaluate the effect of the retrofitting technique on the response of the frame.

2. APPLICATION OF THE RETROFITTING TECHNIQUE TO A REPRESENTATIVE FRAME

The retrofitting technique is applied on a 2D frame of a building representing the typical RC frame type structures in Turkey, (Girgin 1986, Yıldız 2008). The typical elevation of the frame can be seen in Figure 1a. The frame has six storeys with a total height of 21 m. Storey heights are equal and are 3.5 m and span lengths are 5 m. The slabs have a thickness of 15 cm.

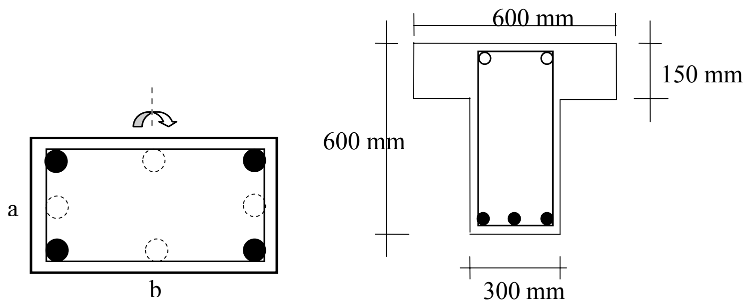
Two outer spans of the frame, namely AB and DE, are filled with 15 cm thick shotcrete panels as can be seen in Figure 1b for retrofitting purpose. Since the geometry of the representative frame is almost 3 times bigger than the tested specimens (Teymur et al. 2008 and 2012), the panel thickness is chosen as 15 cm.

Reinforcing steel strength used in this study is 420 MPa and concrete compressive strength of the frame and the shotcrete panel are 16 MPa and 20 MPa, respectively. The dimensions and longitudinal reinforcement data of columns are presented in Figure 2a and Table 1. The dimensions and longitudinal reinforcement data of beams are presented in Figure 2b and 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 $\phi10/200$.



a) The representative frame **b)** Retrofitting of the frame with shotcrete panels

Figure 1: Representative and retrofitted frames



a)Cross section of a column **b)** Cross section of a beam

Figure 2: Cross sections of the frame members

Table 1: The dimensions and reinforcement of the columns

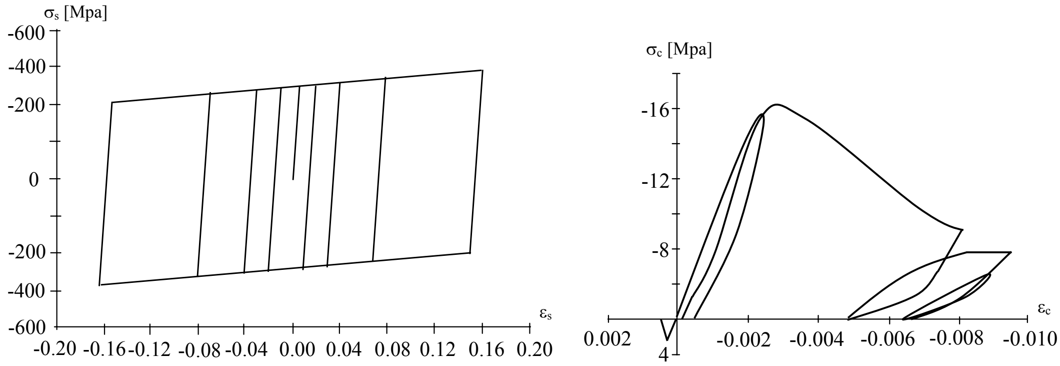
Story	Axes		
	A	B	C
5 - 6	a/b=300/400 mm 4Φ18 + 4Φ16	a/b=300/400 mm 4Φ16 + 4Φ16	b/a=400/300 mm 4Φ16 + 4Φ14
3 - 4	300/400 mm 4Φ18 + 4Φ16	300/500 mm 4Φ20 + 4Φ20	500/300 mm 4Φ20 + 4Φ20
1 - 2	300/400 mm 4Φ18 + 4Φ16	300/600 mm 4Φ22 + 4Φ20	600/300 mm 4Φ20 + 4Φ20

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

Analytical study is conducted by using SeismoStruct. Two types of analysis have been carried out namely; pushover and NDTHA. The response parameters defined for shotcrete panels from the experimental studies (Teymur et al. 2008 and 2012), are adapted here.

Bilinear steel model is used to model the behavior of reinforcement given in Figure 3a. Uniaxial nonlinear constant confinement concrete model is used for confined concrete seen in Figure 3b. The frame is idealized to have fixed type support in these analyses.



a) Steel **b)** Concrete

Figure 3: 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 considered here.

The building is assumed to be constructed on firm soil (Z2 type) at seismic zone 1 defined in Turkish Code for Earthquake Resistant Design (TEC), 2007.

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 first natural vibration periods for the bare frame and the shotcreted frame are T_1= 0.992 sec and T_1= 0.430 sec, respectively.

In Table 4, the lateral load ratios to be used in the push-over analysis are listed. 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. The base shear-top displacement relation determined by the analysis is presented in Figure 4. 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.

Table 4: Forces applied during pushover analysis

a) Bare frame

Storey	W_i (kN)	H_i (m)	W_i*H_i	$(W_i*H_i)/\sum(W_i*H_i)$
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
$\sum$	4412.84		5217.63	1.000

b) Retrofitted frame

Storey	W (kN)	H_i (m)	W_i*H_i	$(W_i*H_i)/\sum(W_i*H_i)$
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
$\sum$	5134.72		6060.53	1.000

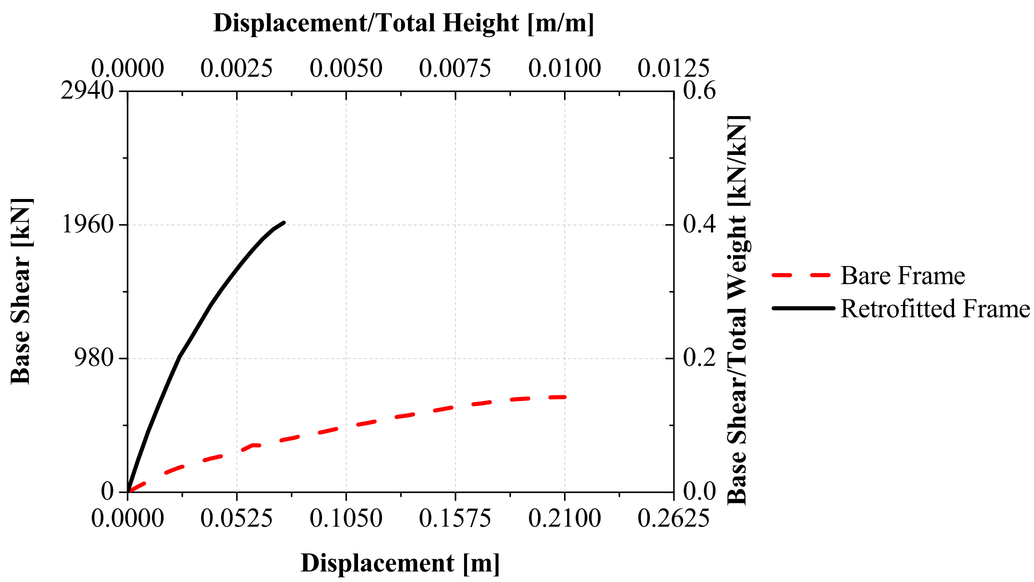


Figure 4: Base shear-top displacement and base shear/total weight-top displacement/total height diagram

NDTHA is also performed. Three earthquake records were taken from PEER (2007) 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 5.

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

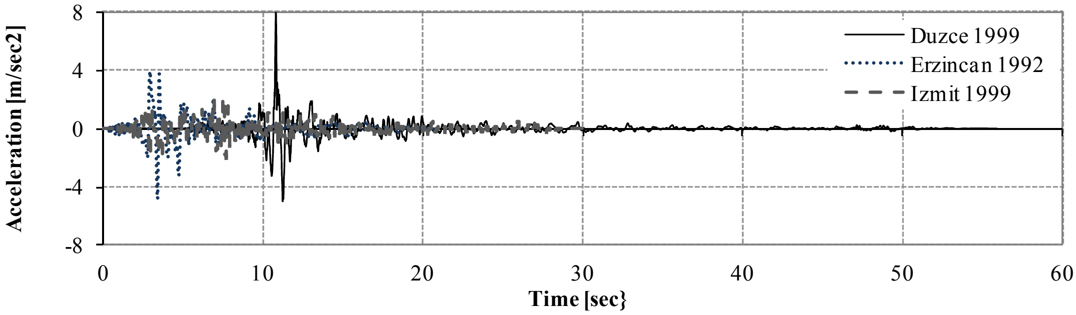


Figure 5: The acceleration record of Erzincan, İzmit and Düzce Earthquake, PEER (2007)

The original acceleration records of the three earthquakes given above are modified respect to the acceleration spectra defined for Z2 type soil given in TEC (2007). The acceleration records were modified to be compatible with the elastic design spectra which has a return period of 475 years, corresponding to a probability of exceedance of 10% in 50 years defined in TEC (2007). These modified records are referred as “Design earthquakes”, (Yıldız, 2008). Another version of the earthquakes is produced as “Service earthquakes” correspond to a probability of exceedance of 50% in 50 years. The ordinates of the spectra of this earthquake are defined as the half of the ordinates of the design spectra.

The modified earthquake records obtained are given in Figure 6 as “service” and in Figure 7 as “design” earthquakes.

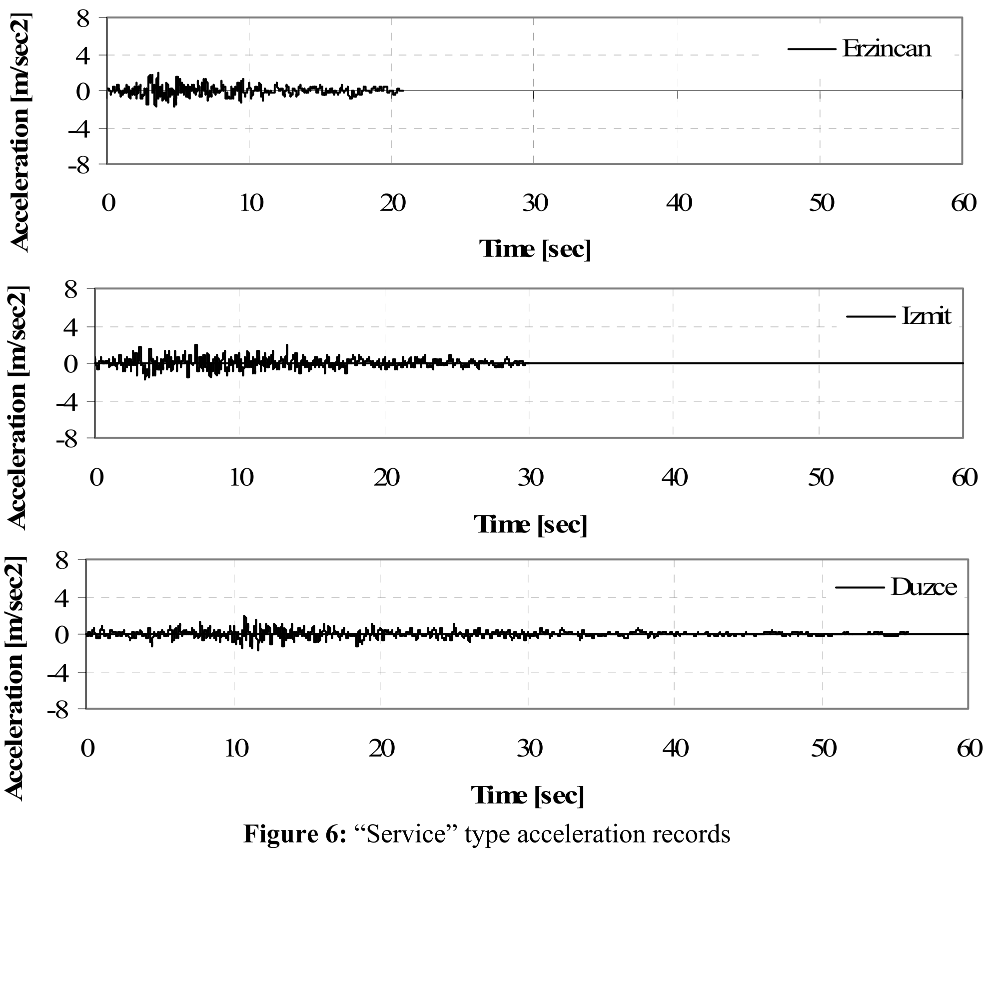


Figure 6: “Service” type acceleration records

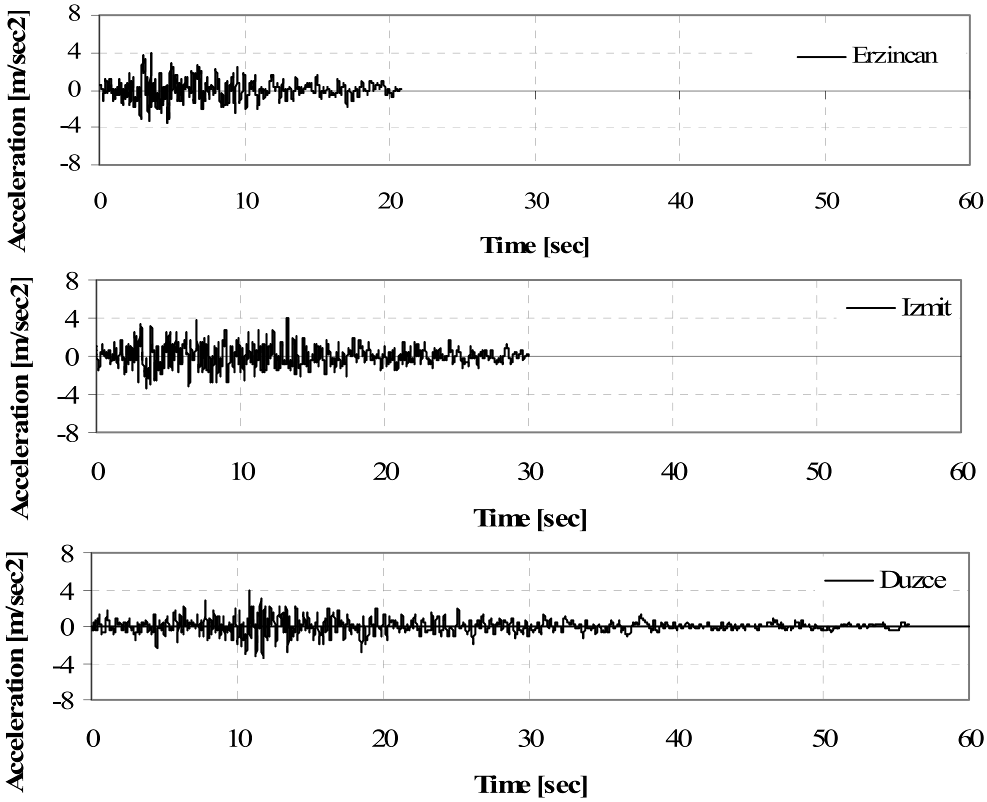


Figure 7: “Design” type acceleration records

The mean spectrum of the modified design earthquakes is drawn with the design spectrum for Z2 type soil in Figure 8.

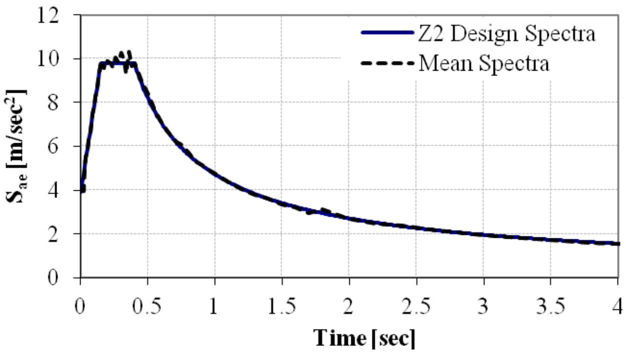


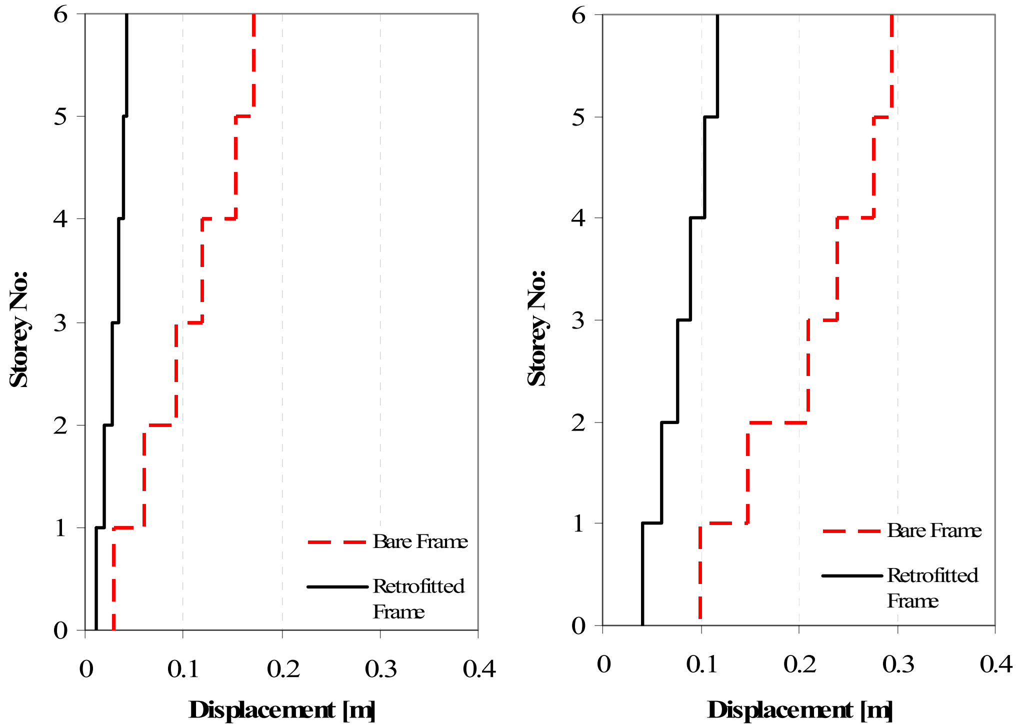
Figure 8: Design spectrum defined in TEC, 2007

In NDTHA, the direct integration of the equations of motion is accomplished using the numerically dissipative α-integration algorithm Hilber et al. (1977) with automatic time-step adjustment for optimum accuracy and efficiency, (SeismoStruct, 2007).

The results obtained from NDTHA; displacement, interstorey drift ratio and shear force demands of shotcreted frames are compared with the bare frames’. In Figures 9, 10 and 11, 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.

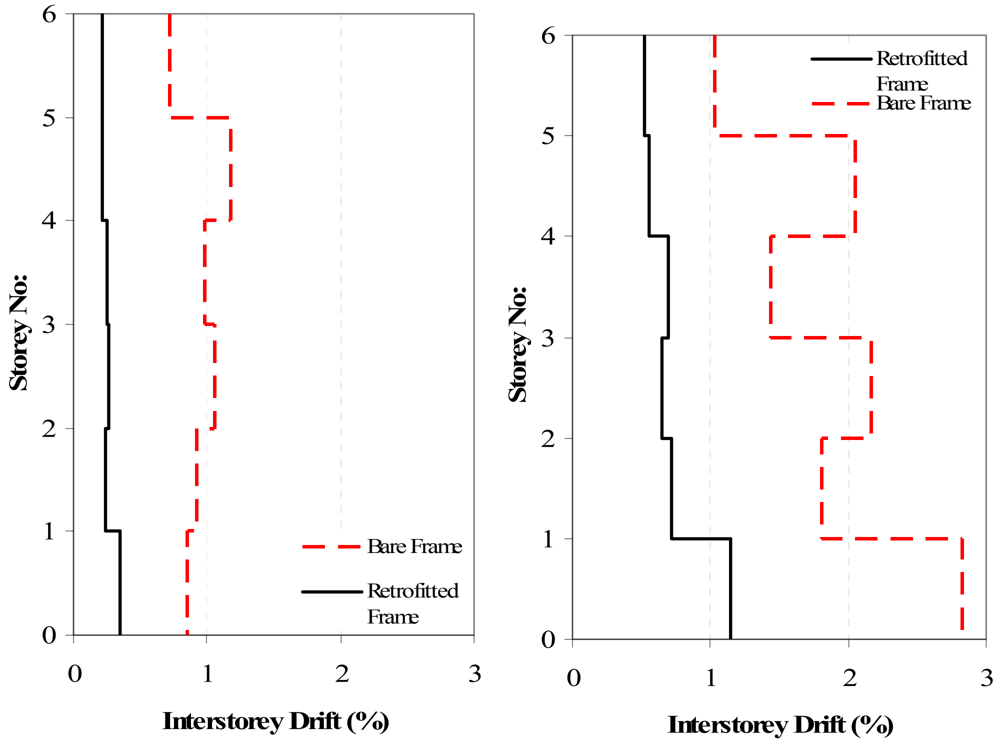
The displacement demands of the bare frame under service and design earthquakes are 0.17 m and 0.29 m, respectively. After placing shotcrete panels in the two spans, these values decrease to 0.04 m and 0.12 m. For the retrofitted frame, the interstorey drift ratio is below 1% under service earthquakes. Under design earthquakes, it is slightly higher than 1% only at the 1st storey. The shear force demand for bare frame is 592 kN under the service earthquakes. After retrofitting, the shear demand becomes

1300 kN. Under the design earthquakes, the shear force demand of the bare frame increases to 627 kN. After infilling of two spans with shotcrete panels, this demand becomes 2011 kN.



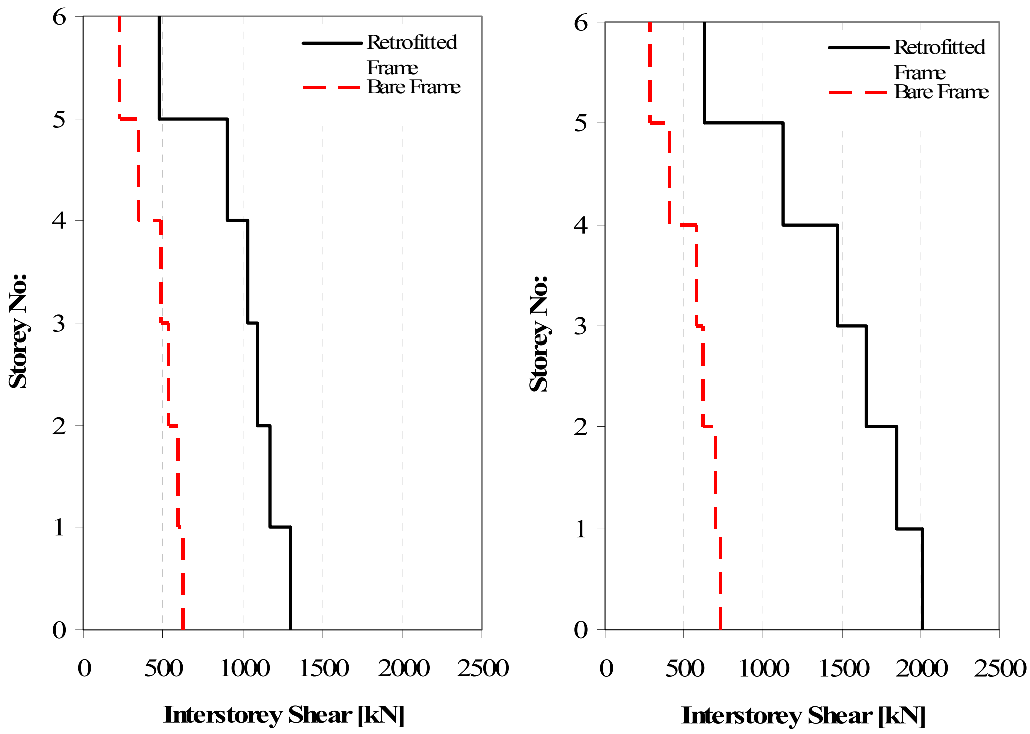
a) Service earthquake b) Design earthquake

Figure 9: 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 10: Comparison of the maximum interstorey drift of the frame with and without shotcrete panel for "service" and "design" earthquakes



a) Service earthquake b) Design earthquake

Figure 11: Comparison of the maximum interstorey shear force of the frame with and without shotcrete panel for "service" and "design" earthquakes

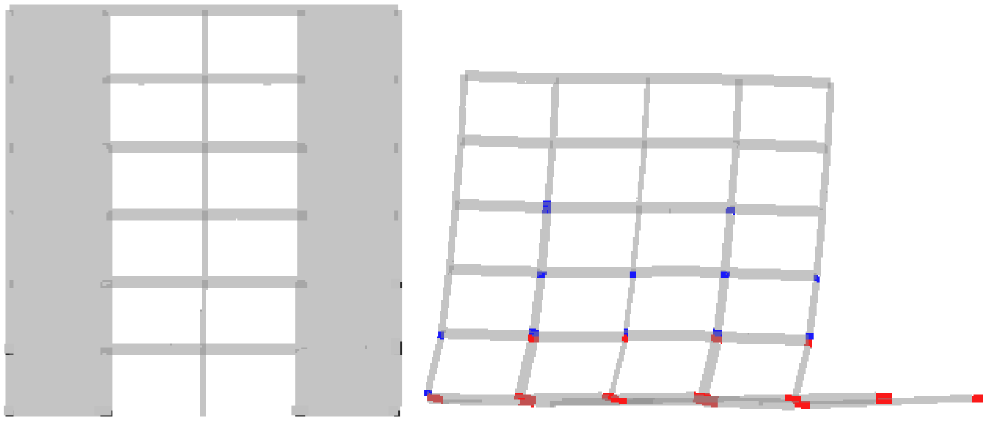
The observed sectional performances of the bare and retrofitted systems in the case of the design earthquakes are illustrated in Figure 12. Damage states for structural members defined in TEC (2007) are introduced to the program. Strain limits used to define damage states in structural members are given in Table 6. Various colours used in this figure correspond to performance levels defined in Table 6. 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.

Table 6: Limit strain values to define damage states in structural members according to TEC 2007

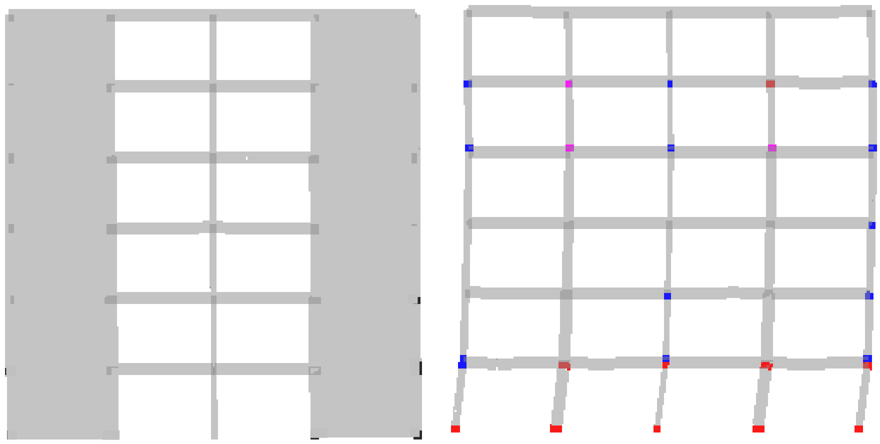
Damage States in Structural Members	Strain Limit	Place of Deformation
Minimum Damage Limit (MNc)	<- 0.0035	The outer-most fibre of the concrete of the section
Minimum Damage Limit (MNs)	> 0.01	Longitudinal reinforcement
Safety Limit (GVc)	< -0.0085	The outer fibre of the concrete within the transversal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

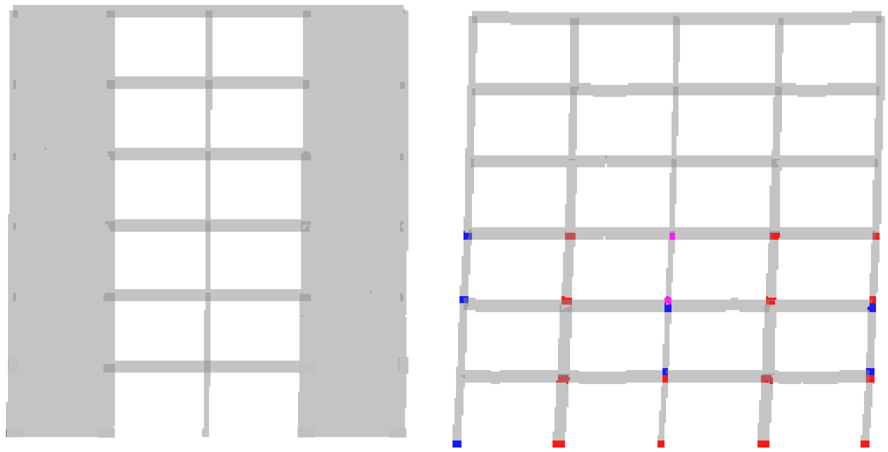
After introducing the shotcrete panels to the outer spans; even though yielding of reinforcement for some critical section has been observed, both beams and columns stayed in performance level named as *Minimum Damage State*.



a) Design Duzce



b) Design Erzincan



c) Design Izmit

Figure 12: Performance of the bare and retrofitted systems under 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.

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.

3. CONCLUSIONS

The behavior of the shotcrete panels predicted from experiments 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 behaviour of shotcrete panel used in this study, are adapted from the calibrated models that were developed for the test specimens. Pushover and NDTHA have been performed on the bare and the retrofitted frames. It is observed that the frame's resistance and rigidity has increased significantly after retrofitting with shotcrete panels. 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 at bare and retrofitted cases. On the other hand, the displacement capacity of the retrofitted frame decreased by 2.8 times compared to the bare frame.

The nonlinear time history analyses performed for the selected earthquake records yield out 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 TEC 2007, however the retrofitted frame performs within the minimum damage state. The interstory drift ratios obtained for the retrofitted specimens are around 1% which is the value observed from experimental study that corresponds to minor damages on the system.

Under these results it can be concluded that; after the retrofitting of the typical RC frame with shotcrete panels, this frame can carry the design loads defined in TEC 2007.

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