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Strengthening of reinforced concrete frames with engineered cementitious composite panels

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Infill walls in reinforced concrete frames are susceptible to failure owing to their brittle nature. Their interaction with boundary columns during earthquakes may also cause shear damage in the columns. It is crucial to employ effective seismic strengthening strategies in order to mitigate the seismic risk induced by the infill walls. In this study, a new strengthening technique, conducted by bonding engineered cementitious composite (ECC) precast panels onto infill walls, was investigated. Three reinforced concrete frames, termed bare frame, infilled frame and strengthened frame, were tested by using the pseudo-dynamic testing method. The test specimens were three-storey, three-bay, half-scaled frames with the central bay having hollow clay brick infill walls. The specimens were subjected to three consecutive ground motions scaled to represent earthquakes with return periods of 144, 475 and 2475 years. The experimental results were compared to observe the influence of the infill walls on the reinforced concrete frames, and to demonstrate the performance of the proposed strengthening technique. It was found that application of ECC panels significantly increased the frame strength and stiffness while allowing the system to have a displacement ductility of about four.

1. Introduction

Infill walls increase the stiffness of reinforced concrete (RC) building frames at low displacement demands and thereby help to attain better deformation control (Fiorato *et al.*, 1970; Ockleston, 1955; Polyakov, 1956; Read, 1965; Smith, 1962). However, they usually sustain significant damage at large lateral displacement demands owing to frame–infill interaction (Fenerci *et al.*, 2016). In-plane motions can induce diagonal tension in the infills, which in turn cause wall cracking and sliding. In addition, the formation of compression struts may exert undesired additional shear forces to the columns and the beam–column joints. The structural contribution of the brick infill walls to the building response is generally ignored at the seismic design or assessment stage, despite the fact that recent earthquakes have demonstrated the life-threatening presence of infill walls (Saatcioglu *et al.*, 2001; Sezen *et al.*, 2003; Yakut *et al.*, 2005).

There are a large number of deficient RC frame buildings with infill walls in seismically active zones in countries such as Turkey, Italy and Greece that require immediate attention in order to reduce the seismic risk. The demolition and rebuilding of all buildings is not a viable option, hence feasible retrofit alternatives are needed for engineers to use. The most preferred rehabilitation approach has been the addition of a new lateral force-resisting system, such as RC infills or steel braces, to improve the stiffness, strength, energy dissipation, lateral drift and ductility (Altin *et al.*, 1992; Aoyama *et al.*, 1984; Canbay *et al.*, 2003; Ersoy and Uzsoy, 1971; Higashi and Kokusho,

1975; Higashi *et al.*, 1980; Jirsa and Kreger, 1989; Kahn and Hanson, 1979; Moehle, 2000; Sonuvar *et al.*, 2004). Nevertheless, this technique, when applied inside the frame, requires demolishing the infill walls and constructing a new lateral load system from the foundation up to the roof level. In order to overcome this limitation, recent studies have proposed occupant-friendly strengthening techniques without any demolition work by using fibre-reinforced polymers (FRPs) (Akin *et al.*, 2013; El-Dakhakhni *et al.*, 2004; Kurt *et al.*, 2011; Ozcebe *et al.*, 2006; Ozsayin *et al.*, 2011). The use of precast panels made of mesh-reinforced conventional concrete bonded to infill walls has been employed by Acun and Sucuoğlu (2006), Sevil *et al.* (2011), Baran *et al.* (2011) and Yaman and Canbay (2014) previously. These methods were mostly tested on one-bay, one-storey frame specimens with the purpose of demonstrating the applicability of the proposed procedure and they did not involve experiments under seismic demands. Furthermore, conventional concrete was used in preparing the precast panels, which resulted in brittle failure of the panels bonded to the infill walls.

Engineered cementitious composite (ECC) is a mortar-based composite reinforced with fibres (Li, 1998). ECC is composed of water, cement, fine aggregates, fibre and some chemical additives. A low water-to-cement (w/c) ratio (0.5 or lower) and fibres (2% or less by volume) are the main factors which enable ECC to attain ductile behaviour and suitable structural behaviour (Li and Kanda, 1998). The mix proportions, along

with a uniform fibre distribution, are the key features of ECC (Ma *et al.*, 2015; Zhou *et al.*, 2012). ECC has a very high strain capacity, varying between 3 and 5% in tension and exhibiting a ductile response in tension (Şahmaran and Li, 2010), whereas conventional concrete has a strain capacity of approximately 0.01% with a sharp softening response. ECC was developed using micromechanics (Li *et al.*, 2004). The interaction between the fibres and cement matrix causes many micro-cracks with a limited crack width. When ECC cracks under tension, the bridging fibres continue carrying the load across the crack, which allows the formation of new cracks. This multiple cracking process continues until the maximum bridging stress is reached. This damage process occurs at increasing loads, resulting in a pseudo-strain-hardening behaviour for the composite (Li, 1993). The strain-hardening behaviour of ECC can lead to a high flexural strength-to-tensile (first cracking) strength ratio (Maalej and Li, 1994). High tensile strain capacity, damage tolerance with limited crack width and pseudo-hardening response in tension make ECC an attractive repair and retrofit material (Li *et al.*, 2000).

Sprayable ECC with welded wire fabric steel mesh was implemented as a strengthening method on the infill walls of one-storey, one-bay, 1/5-scale, non-ductile RC frames (Kyriakides and Billington, 2013). Frames under in-plane quasi-static cyclic lateral loading demonstrated that this retrofitting technique enhanced the lateral strength and stiffness of the frame and resulted in more ductile behaviour. Koutromanos *et al.* (2012) implemented the same strengthening technique for 2/3-scale, three-storey, two-bay, masonry-infilled RC frame with shake table tests. Maalej *et al.* (2010) retrofitted unreinforced masonry wall panels with ECC. This approach increased the ultimate load-carrying capacity of the masonry walls, improved their ductility, enhanced their resistance against multiple low-velocity impacts and prevented sudden failures. The idea of using highly ductile cementitious composite materials for precast panels in infill wall strengthening had not been investigated previously.

The objectives of this study are: (a) to develop cementitious composite precast panels for the seismic strengthening of infill walls; (b) to experimentally observe the effectiveness of the ECC strengthened infilled frame system under simulated ground motions by using pseudo-dynamic testing; (c) to demonstrate the beneficial influence of the proposed retrofit scheme by comparing the results with those obtained from the testing of bare and infilled frames. The key contribution of this study is to demonstrate experimentally the efficiency of prefabricated ECC panels for upgrading the seismic performance of deficient RC frame structures. The outcome of the research is believed to provide structural engineers with an innovative alternative for seismic strengthening.

2. ECC properties

In this study, 16 different mix proportions were tested by Keskin (2012) to investigate the mechanical and shrinkage

properties of ECC mortar with varying amounts of cement, fly ash, ground granulated blast furnace slag, silica sand and water. Although ECC mixtures with ground granulated blast furnace slag exhibited higher compressive strength than ECC mixtures with fly ash, all samples showed a similar flexural tensile strength response. Moreover, the maximum mid-span deflection was measured for the ECC mixture with fly ash with low mineral admixture cement ratio by the four-point bending test (Keskin, 2012). The composition of the ECC mortar used in the production of precast panels is given in Table 1. Ordinary Portland cement (CEM I 42.5 R corresponding to ASTM Type I) was used with an extremely low water/binder ratio (0.27) to obtain higher compressive strength. Class F (according to ASTM C618 (ASTM, 2003)) type fly ash was used in the ECC mix. The chemical composition of the cement and fly ash are given in Table 2. The main motivation of this part of the study was to enhance the tensile capacity of the ECC panels. For this purpose, polyvinyl acetate (PVA) fibres were added to the mortar. The fibre properties declared by the manufacturer are presented in Table 3. The maximum and average aggregate size used in this study were selected as 400 µm and 200 µm, respectively. A polycarboxylate ether type high-range water-reducing superplasticiser (BASF – Glenium

Table 1. Mix design of ECC

Material	Weight for 1 m ³ : kg
Cement	563
Fly ash	676
Water	329
Silica sand	450
PVA fibre	26
Superplasticiser (BASF – Glenium 51)	18.5–25.0

Table 2. Chemical composition of cement and fly ash

Chemical composition	Portland cement	Fly ash
Calcium oxide (CaO): %	61.43	1.64
Silicon dioxide (SiO ₂): %	20.77	56.22
Aluminium oxide (Al ₂ O ₃): %	5.55	25.34
Iron (III) oxide (Fe ₂ O ₃): %	3.35	7.65
Magnesium oxide (MgO): %	2.49	1.8
Sulfur trioxide (SO ₃): %	2.49	0.32
Potassium oxide (K ₂ O): %	0.77	1.88
Sodium oxide (Na ₂ O): %	0.19	1.13
Loss on ignition: %	2.2	2.1
SiO ₂ + Al ₂ O ₃ + Fe ₂ O ₃	29.37	89.21

Table 3. Properties of PVA fibres as provided by the manufacturer

Nominal strength: MPa	1620
Apparent strength: MPa	1092
Diameter: µm	39
Length: mm	8
Young's modulus: GPa	42.8
Elongation: %	6
Density: kg/m ³	1300
Melting temperature: °C	230

51) was used to enhance the workability of ECC. First, the fly ash and cement were mixed and water was later added to this mix. Afterwards, the superplasticiser was added to increase workability and flowability. In this way, the mixing process was accelerated to achieve the intended consistency. Finally, PVA fibres were added to the mortar and mixed further. At the end, the ECC was placed into the moulds. The mechanical properties of ECC were determined from standard tests, such as uniaxial compressive strength, elastic modulus and bending tests. Three 100 mm dia. and 200 mm high cylinder specimens were tested to determine the compressive strength of the ECC. The average compressive strength and modulus of elasticity of the ECC were obtained as 47.4 MPa and 14 800 MPa, respectively. Three 315 mm × 75 mm × 75 mm and three 510 mm × 75 mm × 25 mm rectangular prismatic specimens were tested under four-point loading. Thin specimen dimensions were selected to simulate the ECC panel behaviour (Figure 1). Simply supported beams were tested under four-point loading. The average flexural strengths of standard and thin specimens were found as 10.60 MPa and 9.66 MPa, respectively. The flexural tensile stress–mid-span displacement response of the panels is presented in Figure 1 along with photographs of the bent specimens. It can be observed that the ECC panels exhibited a ductile response with significant deformation capacity. The ductility of the panel specimens (i.e. with dimensions 510 mm × 75 mm × 25 mm) were higher than prismatic specimens (i.e. with dimensions 315 mm × 75 mm × 75 mm) owing to their purely flexural dominated response.

3. Pseudo-dynamic tests

3.1 Test specimens

Test frames were chosen from the middle axis of a three-storey prototype building designed for gravity loads (Figure 2). The

test bay was a three-bay, three-storey, half-scale, seismically deficient RC frame with column and beam dimensions of 150 × 200 mm and 175 × 150 mm, respectively. As shown in Figure 2, a 500 mm wide and 60 mm thick flange was constructed to represent the slab of the floor and carry the steel blocks placed for dead weight. The columns had eight plain bars (8Φ8) as the longitudinal reinforcement and had no confined zones at the ends, with 4 mm dia. plain bars provided every 100 mm (Φ4/100). Such reinforcement detailing represents typical 1970s construction practice in Turkey. The beam bottom reinforcements were three 10 mm dia. (3Φ10) plain bars. Beam top reinforcement was 2Φ10 at the span region and increased to 4Φ10 at support regions. Φ4/50 stirrups were provided at the 350 mm confined end zones of the beams and the tie spacing was increased to 80 mm (Φ4/80) at the mid-region of the beams. The Turkish earthquake code (TEC, 2007) stipulates the use of 135° hooks at the end of ties. The test frame, however, had transverse reinforcement with 90° hooks, which thus represented a common deficiency observed in old buildings. Furthermore, there was no transverse reinforcement in the beam–column joint regions. The same dimensions, reinforcement details and similar material properties were used in all three frame specimens, namely, the bare frame, infilled frame and strengthened frame. A general view and pictures of the test specimens are given in Figure 2.

Individual footings were constructed under the external columns, whereas a continuous footing was used for the interior columns in order to construct the infill wall. The footings were tightly clamped to the strong floor by means of post-tensioned bars. During testing no sliding and uplifting was observed on the footings. In order to represent non-structural partition walls in buildings, the middle bay of each storey for infilled and strengthened frames was filled with hollow brick

Figure 1. Bending tests: (a) 315 × 75 × 75 mm specimens; (b) 510 × 75 × 25 mm specimens; (c) bending test results

Figure 2. (a) Prototype building floor plan; (b) reinforcement details; (c) bare frame; (d) infilled frame; (e) strengthened frame; (f) test specimen; (g) infill wall

masonry. Storey loads were employed by using steel blocks. The mass of each storey of the test specimens is provided in Table 4.

In the strengthened specimen, ECC panels were applied to the infill wall prior to testing. Nine precast ECC panels 25 mm thick were bonded on the infill walls from one side at each floor with epoxy adhesive mortar (Sikadur 31). Reinforcement details of the panels and the panel locations are presented in Figure 3. The epoxy adhesive mortar had compressive and tensile strength values of 55 MPa and 6 MPa based on the manufacturer's test results. Panel dimensions were 510 × 435 mm, rectangular in shape. In order to enable the proper force transfer between the strengthened infill wall and surrounding frame, 10 mm anchorages were used. Holes of 14 mm dia. were drilled diagonally into the beam and column

Table 4. Weight distribution of specimens on each floor

	First storey: kg	Second storey: kg	Third storey: kg
Bare frame	10 410-00	10 410-00	6960-00
Infilled frame	9430-00	9430-00	8880-00
Strengthened frame	9430-00	9430-00	8880-00

cores, and anchor bars were attached with epoxy to the frame. The application details of the anchor bars are provided in Figure 3.

3.2 Materials

All specimens were constructed in the structural mechanics laboratory of the Middle East Technical University. Specimens

Figure 3. (a) ECC panel dimensions; (b) assembled ECC panels on wall; (c) anchorages; (d) implementation of ECC panels on wall; (e) ECC panels produced; (f) anchorage details (dimensions in mm)

were cast in place by using steel formworks. Footing concrete was delivered from a ready-mix company. The 28 d concrete compressive strength was determined as 38 MPa for the footing concrete. The concrete of the frame was prepared in the laboratory. The mix design of the concrete is given in Table 5. Proper curing was provided by using wet burlap. Since

all storeys were constructed in sequence, inevitably some minor variance was observed in the concrete strength of storeys (Table 6). The target concrete strength was selected as 12 MPa to simulate the poor in situ concrete strength that is commonly observed in the Turkish building stock remaining from the 1970s.

Table 5. Mix proportion of one batch concrete

	Weight: kg
Cement (CEM II/B-M (P-L) 32.5 R)	70
Aggregate (sand 0–3 mm)	140
Aggregate (crushed stone 7–15 mm)	215
Water	50
Total	475

Table 6. Uniaxial compressive strength of the cylinders of frame concrete

Uniaxial compressive strength: MPa	First storey	Second storey	Third storey
Bare frame	14.0	11.7	10.1
Infilled frame	12.7	13.9	14.0
Strengthened frame	15.0	15.8	13.8

Table 7. Mix proportion of mortar and plaster

	Weight: kg
Cement (CEM II/B-M (P-L) 32.5 R)	15
Aggregate (sand 0–3 mm)	64.8
Lime	5.5
Water	14.7
Total	100

Table 8. Properties of reinforcement

Bar diameter: mm	f_y : MPa	f_u : MPa	Type
10	355	555	Plain
8	320	460	Plain
4	240	340	Plain

Scaled special brick units with dimensions $190 \times 85 \times 95$ mm were used in the infill walls. The compressive strength of these units was determined as 3.1 MPa. The mix design of the mortar and plaster is given in Table 7. The thickness of the mortar between masonry units was approximately 8 mm and the thickness of the plaster on the infill wall was nearly 10 mm. The total infill wall thickness was roughly 105 mm. A 45 mm indentation was left between the column face and plaster for anchor bars and ECC panels. The average compressive strength of mortar and plaster was found as 8.4 MPa. Plain bars were used as the reinforcement in all columns and beams. The diameter of the plain bars of beams, columns and stirrups was 10 mm, 8 mm and 4 mm, respectively. The yield and ultimate strengths of the bars were determined from coupon tensile tests, and are provided in Table 8.

3.3 Instrumentation and test procedure

Lateral displacements were measured at every floor level by means of linear variable differential transducers (LVDTs). The

applied lateral loads were measured by load cells at every floor level. The average shear deformations of the infill walls were measured by means of diagonally placed displacement transducers on the infill walls. Displacement measurements on beams and columns were conducted in order to calculate curvatures, to measure slip, overturning and lifting. Additionally, tilt sensors were used to measure joint rotations. In all, three load cells, six dial gauges, 64 LVDTs and eight tilt sensors were used during testing (Figure 4). Two in-house-produced force transducers were placed at the bases of the outer columns. These force transducers were able to measure in-plane shear force, axial force and moments. Each transducer required six channels from the data acquisition system. A detailed explanation of the force transducers can be found in Canbay *et al.* (2004).

The testing system was composed of the reaction wall, strong floor, specimens, foundations, computer controlled actuators, instrumentation (i.e. LVDTs and potentiometers) and the data acquisition system. In addition, a steel frame was constructed around the test frame for safety reasons and to prevent out-of-plane displacements. Steel blocks resting on the beams were also loosely hung by means of steel cables to ensure the safety frame. Lateral loads were applied to the frame by three servo-controlled actuators attached between the reaction wall and frame. The actuator of the first floor had a capacity of 250 kN (Figure 4). Actuators at the second and third floors had 500 kN pull and push capacities. The actuators were aligned to the mass centre of the beams and attached by two hinges at both ends. The actuators applied a pushing force to the frame by direct bearing. In order to prevent tension in the beams during the pulling action of the actuators, the pulling force was transferred to the opposite side of the frame by means of four 7 m long tie rods to impose a pushing force from the other side, at the back of the frames.

3.4 Pseudo-dynamic testing

The tests in this study were conducted by means of a continuous pseudo-dynamic testing technique. This technique combines static testing with numerical integration to simulate the earthquake response of structures in the time domain (Molina *et al.*, 1999). Hence, the method is a hybrid computer-driven testing procedure by the simultaneous application of desired ground motions. The structural system is represented as a discrete spring–mass system and its dynamic response to earthquakes is solved numerically using direct explicit integration. The restoring forces of the system are directly obtained from the test specimen in parallel to the direct integration. Non-linear reaction forces are measured directly by means of load cells attached to the actuators. Target displacements at each step are computed by way of a discrete parameter model of mass and damping. The numerical integration of a second-order differential equation of motion is carried out to obtain the displacement history for

Figure 4. Test apparatus and instrumentation

further steps, with restoring forces as input values of the equation (Figure 5).

The three synthetic earthquake ground motions used in the experiments were generated using the response spectrum matching method of Hancock *et al.* (2006), which employs a wavelet adjustment technique. The three motions were designed to match the design spectrum of the Turkish earthquake code (TEC, 2007) and they are named as follows (Figure 6)

- D1: exceedance probability of 50% in 50 years for local site class Z1/rock
- D2: exceedance probability of 10% in 50 years for local site class Z1/rock
- D3: exceedance probability of 10% in 50 years for local site class Z3/soft soil.

4. Test results

4.1 Bare frame

No considerable damage was observed during the D1 motion on the bare frame. The D1 motion caused 9 mm maximum roof displacement and 0.27% maximum inter-storey drift ratio (Figure 7). There was almost no residual displacement at the end of the D1 motion. During the D2 earthquake, the roof displacement increased to 49 mm. There were local flexural cracks in both exterior and interior columns and also diagonal shear cracks were monitored at joint regions (Figure 7). It should be noted that the columns had no confined zones at the ends and there was no transverse reinforcement in the beam–column joint regions. The maximum inter-storey drift ratio was 1.29% during D2. The base shear–roof displacement curve exhibited non-linear behaviour during this earthquake

Figure 5. Pseudo-dynamic testing loop

Figure 6. (a) Ground acceleration–time history; (b) response spectra of ground motions

excitation. The D3 earthquake resulted in a roof displacement of 206 mm and a maximum inter-storey drift ratio of 5.28%. There were severe flexural cracks at the beam ends and the base of the exterior and interior columns. The tops of the third

floor columns had serious flexural cracks. Joint diagonal cracks spread even into the slab. Large residual displacements (first storey: 3.6 cm; second storey: 8.4 cm; and third storey: 12.5 cm) were measured at the end of the experiment. The

Figure 7. (a) Drift ratio plotted against time curves of bare frame; (b) observed damage of the bare frame

Table 9. Maximum column end rotations of bare frame

	First storey		Second storey	
	Bottom	Top	Bottom	Top
D1	0.00136	0.00137	0.00079	0.00148
D2	0.01088	0.00738	0.00596	0.00707
D3	0.04272	0.02192	0.01639	0.03422

maximum column end rotations of the first and second storeys are represented in Table 9. Peak rotations measured at D3 ground motion pointed out significant flexural plastic damage. Joint deformations and column rotations contributed to the lateral displacements of the bare frame.

4.2 Infilled frame

The D1 motion caused 4 mm maximum roof displacement and a maximum drift ratio of 0.1% (Figure 8). The masonry

infill wall reduced the displacement demands significantly under the same earthquake as compared to the bare frame. Minor boundary cracks formed in the first floor level between the interior columns and the infill wall. Furthermore, hairline flexural cracks were observed in the first storey beams. The D1 earthquake caused almost no residual displacement. The beneficial effect of the infill walls at small deformation demands was confirmed with these observations. During the D2 earthquake, the maximum roof displacement and drift ratio were 21 mm and 0.49%, respectively. These values were less than half of those measured in the bare frame test. The corner of the plaster on the infill wall crushed during this earthquake level (Figure 8). Diagonal shear cracks were visible on the infill wall due to the strut-and-tie mechanism. The compression strut caused diagonal shear cracks at the column ends. The maximum roof displacement and drift ratio were 41 mm and 1.86% during the D3 ground motion. The maximum drift was observed at the first floor. The maximum drift ratios at the

Figure 8. (a) Drift ratio plotted against time curves of infilled frame; (b) observed damage of the infilled frame

second and third floors were 0.74% and 0.68%, respectively. The separation and closure between the infill wall and neighbouring column were significant during the D3 motion. There were severe shear cracks at the top of the first storey columns. After spalling of the cover concrete in this region the longitudinal reinforcements buckled (Figure 8). While the infill walls increased the stiffness of the frame, they also resulted in an increase in the base shear demand. The maximum base shear was measured as 77 kN during the bare frame test, whereas the maximum base shear was 184 kN during the infilled frame test. In the D3 motion, the first storey column end rotations were significantly higher than those measured in the second storey (Table 10). This unbalanced distribution of column end rotations indicated the formation of a soft storey upon failure of the infill wall.

4.3 Strengthened frame

This specimen was strengthened by means of ECC panels attached to the brick infill wall. The D1 earthquake caused

Table 10. Maximum column end rotations of infilled frame

	First storey		Second storey	
	Bottom	Top	Bottom	Top
D1	0.00053	0.00073	0.00060	0.00076
D2	0.00425	0.00551	0.00341	0.00357
D3	0.02038	0.02609	0.00757	0.00495

3.7 mm maximum roof displacement and 0.09% maximum drift ratio. Boundary cracks were observed between the ECC panels and surrounding structural elements (Figure 9). The specimen displayed elastic behaviour during this ground motion with no visible damage. ECC panels were effective in eliminating infill cracking at small deformation demands. The D2 earthquake caused a 25 mm roof displacement. The maximum drift ratios of the first, second and third floors were 0.51, 0.60 and 0.55%, respectively. The main damage was the flexural cracks on columns and cracks in the anchorage

Figure 9. (a) Drift ratio plotted against time curves of strengthened frame; (b) observed damage of the strengthened frame

regions of the bottom corner panels. The ECC panels did not sustain any damage, thereby proving the strength and stiffness enhancement obtained by the proposed strengthening procedure. The D3 motion resulted in an increase of the roof displacement to 43 mm and drift ratio to 0.86% at the first storey. The second and third storey drift ratios were 1.04 and 0.96%. The column flexural cracks became widespread and the crack widths increased. At the base of the infill wall, cracks widened and a sliding behaviour was observed. Despite the corner anchorage failures of panels, the struts of the retrofitted wall increased the base shear capacity of the frame. The base shear force reached 249 kN, as can be observed in Figure 10. No strength decrease was observed, even at the D3 earthquake level. It can be stated that the ECC panels provided good deformation control and strength increase, and were able to sustain inelastic deformations without loss of lateral strength. Moreover, the strengthening method reduced the peak column end rotations, as shown in

Table 11. It can be stated that the ECC panel retrofit was successful in reducing the storey displacement and column end rotation demands while increasing the base shear capacity of the system.

5. Discussion of test results

A summary of test results is presented in Table 12. A comparison of the specimens indicated that the infilled specimen and strengthened specimen experienced similar displacement demands under the D1 and D2 earthquakes. However, the ECC panels helped to avoid the cracking damage observed in the infill walls. It can be stated that ECC panel application was quite successful in keeping the structure undamaged and in removing the need for repair under small to moderate motions.

All storeys of the bare frame exhibited at least 3% residual drift after the D3 earthquake. The presence of the brick infill

Figure 10. Base shear plotted against roof displacement curves of specimens: (a) bare frame; (b) infilled frame; (c) strengthened frame

wall limited the drift ratio demands of the upper storeys during the D3 earthquake. However, the first floor drift ratio reached to a maximum value of around 2% level following the corner crushing of the infill wall. Strengthening of the infill

Table 11. Maximum column end rotations of strengthened frame

	First storey		Second storey	
	Bottom	Top	Bottom	Top
D1	0.00076	0.00119	0.00090	0.00099
D2	0.00820	0.00279	0.00201	0.00295
D3	0.01356	0.00412	0.00303	0.00559

walls with ECC panels positively affected the drift ratio demands by keeping them below the 1% level. The most important beneficial effect was the relatively uniform drift ratio distribution when retrofitting with the ECC panels. Furthermore, ECC panels helped the infill wall to remain intact, even during the D3 earthquake, and consequently the residual drifts of the strengthened frame were limited during the D3 earthquake. Moreover, contrary to the presence of observed shear cracks on columns of infilled frame, no column shear crack was observed in the strengthened frame. Columns and infill walls with ECC panels acted together as an integral unit, which increased the deformability without loss of lateral strength.

The maximum plastic rotations for columns were obtained with the assumed plastic hinge length of half of the section depth. No plastic rotations were observed for the specimens during the D1 earthquake. The highest plastic rotation demand was measured for the bare frame during the D2 and D3 motions. Excessive roof displacement for the bare frame resulted in an increased ductility demand. The ductility demands of the infilled and strengthened frame were similar. However, the base shear capacity of the strengthened frame was about 35% higher than that of the infilled frame due to the panel application on the infill wall. It can also be observed that shear deformations of the infill wall were significantly reduced as a result of the ECC panel application (Table 12).

The energy dissipation of the specimens is demonstrated in Figure 11. The cumulative dissipated energy was calculated by integration of the load plotted against storey displacement curve. The ECC panel application improved the energy dissipation characteristics of the specimen. Although all specimens showed similar energy dissipation characteristics, the strengthened frame dissipated the energy with much higher displacement demands (Figure 11).

6. Conclusions

The main objective of this study was to develop a practicable and economical strengthening technique for structural RC frames. The high tensile strength and ductile response of ECC motivated the use of ECC panels as a strengthening method in this study. Within this scope, a suitable mix proportion for the production of ECC was prepared with local materials, as no

Table 12. Summary of test results

Specimen	EQ	IDRmax: %	Roof disp.: mm	Max. base shear: kN	Ductility demand	Max. column plastic rotation: rad	Max. infill shear def.: mm	Observed damage and failure mode
BF	D1	0.27	9.39	36.60	0.7	—	—	Flexural cracking
	D2	1.29	48.68	66.58	3.7	0.00309	—	Flexural and Joint cracking
	D3	5.28	205.95	76.71	19.0	0.01742	—	Cover spalling and extensive steel yielding
IF	D1	0.10	3.61	71.42	0.4	—	0.24	Flexural cracking and interface cracking
	D2	0.49	20.84	163.40	2.1	0.00035	1.13	Shear cracking of the infill wall
	D3	1.86	40.51	184.49	3.2	0.00786	14.20	Shear cracking of the infill wall and the boundary columns
SF	D1	0.09	3.69	100.48	0.3	—	0.20	Flexural cracking
	D2	0.60	24.85	202.45	2.0	0.00185	0.46	Flexural cracking
	D3	1.04	42.62	248.87	3.4	0.00430	0.66	Flexural cracking and ECC panel separation

Figure 11. Energy dissipation

clear and unique recipe for ECC in the literature is available. The following conclusions are based on the results of the tests and analyses.

- The deficient bare frame attained a maximum inter-storey drift ratio of above 5% with a residual deformation of above 3%. This specimen experienced cracking damage under low to moderate motions, leaving the structure in need of repair.
- Although the infilled frame showed a stiffer response compared to the bare frame, it experienced cracking under the D2 motion. This suggests that RC frames with infill walls are expected to be in need of repair under small to moderate motions. Upon failure of the infill wall, first storey columns, the column rotation demands in the first storey increased significantly. This result exhibits the change of structural frame response upon sudden failure of the infill walls.
- Upon retrofit with ECC panels, the following important structural benefits were obtained: (a) no visible damage

under low to moderate motions; (b) reduced inter-storey drift ratios up to 50%; (c) increased base shear capacity up to 35%; (d) a more uniform distribution of inter-storey drift deformation demands; (e) intact nature of the retrofitted infill wall, which is also less susceptible to out-of-plane collapse.

The application of precast ECC panels on plastered infill walls of frames was found to be an effective and practical technique for strengthening seismically vulnerable RC structures. Further studies are needed to establish detailed design guidelines of the retrofit scheme.

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