Earthquakes and Structures, *Vol. 11, No. 6 (2016) 967-982*
DOI: http://dx.doi.org/10.12989/eas.2016.11.6.967

A preliminary case study of resilience and performance of rehabilitated buildings subjected to earthquakes

S. Ali Hadigheh[1a], S. Saeed Mahini[*2], Sujeeva Setunge[3b] and Stephen A. Mahin[4c]

[1]School of Civil Engineering, The University of Sydney, NSW, Australia
[2]Discipline of Civil and Environmental Engineering, University of New England, NSW, Australia
[3]Department of Civil Engineering, School of Engineering, RMIT University, VIC, Australia
[4]Department of Civil and Environmental Engineering, University of California, Berkeley, USA

(Received March 16, 2016, Revised June 27, 2016, Accepted August 24, 2016)

Abstract. Current codes design the buildings based on life safety criteria. In a performance-based design (PBD) approach, decisions are made based on demands, such as target displacement and performance of structure in use. This type of design prevents loss of life but does not limit damages or maintain functionality. As a newly developed method, resilience-based design (RBD) aims to maintain functionality of buildings and provide liveable conditions after strong ground movement. In this paper, the seismic performance of plain and strengthened RC frames (an eight-story and two low-rise) is evaluated. In order to evaluate earthquake performance of the frames, the performance points of the frames are calculated by the capacity spectrum method (CSM) of ATC-40. This method estimates earthquake-induced deformation of an inelastic system using a reduced response spectrum. Finally, the seismic performances of the frames are evaluated and the results are compared with a resilience-based design criterion.

Keywords: buildings; rehabilitation; resilience; performance; earthquake

1. Introduction

A large number of structures have been designed based on old codes and do not satisfy the requirements of the new seismic design criteria. Problems associated with poor structural design are one of the key factors that can lead to the loss of structural integrity during a seismic event. To date, earthquake is considered as part of the general loading requirements applicable to all regions of Australia, e.g., Melbourne and Sydney (AS1170.4 2007, Wilson *et al*. 2008). In conventional seismic assessment of structures, the trade-off strength demand is compared with the ductility (displacement) demand on structure (Lumantarna *et al*. 2010). Incidents during recent earthquakes necessitate the need for more sound effective designing procedures for structures.

Design of structures based on performance objectives (performance-based design) have been

*Corresponding author, Senior Lecturer, E-mail: smahini@une.edu.au
[a]Associate Lecturer, E-mail: ali.hadigheh@sydney.edu.au
[b]Professor, E-mail: sujeeva.setunge@rmit.edu.au
[c]Professor, E-mail: mahin@berkeley.edu

http://www.techno-press.com/journals/eas&subpage=7 ISSN: 2092-7614 (Print), 2092-7622 (Online)

practiced in the last decades. In this method, performance objective can be the target displacement as the response parameter of a substitute single-degree-of-freedom (SDOF) system which is so-called displacement-based design (DBD) method. It can be also related to strain-based limit state, and the level of damage (Ghobarah 2001) on a capacity spectrum curve (Hadigheh *et al*. 2014). Capacity spectrum method (CSM) is recommended by Applied Technology Council (ATC) (1996) for performance assessment of concrete structures. Although, DBD method prevents loss of life, the damage cannot be limited and functionality of structure after the event (e.g., earthquake) cannot be maintained.

In recent years, resilience is considered as an effective method for assessment of the seismic performance of structures (Cimellaro 2008). It involves the discovery and development of new knowledge and technologies in order to equip communities to become more disaster resilient during earthquakes and other extreme events. Therefore, the main objective of resilience-based design (RBD) is to make communities 'resilient'. It aims to develop actions and technologies that allow structure and/or community to recover its function as promptly as possible whenever a disaster occurs (Cimellaro 2013).

Cimellaro (2008) defines seismic 'resilience' as a decision variable that compares the seismic performance recovery with a given loss required in order to maintain the functionality of the system with minimal disruption. The seismic resilience framework compares losses and different pre- and post-event measures in order to verify if strategies and actions are able to reduce or eliminate disruptions during a seismic event (Cimellaro 2008). In this regard, Chang and Shinozuka (2004) proposed a series of quantitative measures of resilience and applied them to a case study of an actual community (seismic mitigation of a water system). Biondini *et al*. (2015) developed a probabilistic approach to the lifetime assessment of deteriorating concrete structures. Cimellaro *et al*. (2010) presented a quantitative evaluation of disaster resilience concepts, and a unified terminology for the evaluation of health care facilities subjected to earthquakes.

Several strategies have been proposed to upgrade a structure against seismic actions; e.g., steel reinforcing, steel bracing system, shotcrete, shear walls, dampers, and FRP strengthening. Among these, a number of studies have focused on the influence of retrofitted joints using fibre-reinforced polymers (FRPs) on the overall behaviour of reinforced concrete (RC) frames. Zou *et al*. (2007) investigated a 3-storey frame that was strengthened with FRP around its columns. They observed that this could increase the strength of the columns while marginally increasing their stiffness. Reducing stiffness is, however, more desirable for the overall stability of a frame, as stiffer columns are more vulnerable to higher seismic forces. Niroomandi *et al*. (2010) and Hadigheh *et al*. (2014) studied the seismic performance of RC ordinary moment resisting frames (OMRFs) retrofitted by FRPs or steel braces using DBD method. Hadigheh *et al*. (2014) showed that while using X-braces decreases the ductility of frames, FRP retrofitting increases the behaviour factor and maintain the ductility within a reasonable margin.

In their recent research, the authors (Mahini *et al*. 2015a, b) presented the fundamentals of the seismic resilience and evaluation method, and applied formulated frameworks to low and medium-rise retrofitted RC buildings in which the seismic performance has been already evaluated by the PBD method. Research in this field is extremely limited, particularly in Australia, and further developments are necessary to fully understand the disaster resilience of hospital networks. Therefore, a formulated framework for a hospital complex system is employed in this paper in order to assess the seismic behaviour of low-to-medium-rise RC buildings and possible rehabilitation techniques. Two different retrofitting methods are considered; steel bracing system and FRP composite application. Performance of the plain and retrofitted frames is evaluated based on performance- and resilience-based design. It can be seen that although PBD prevents loss of life, it cannot maintain functionality or limit damages.

2. Performance- and resilience-based design

2.1 Performance-based design

A limit state is a form of performance objective in which the target displacement is considered as the response parameter of a substitute SDOF system. The structural response in terms of displacement (i.e., displacement-based design method) can be also related to strain-based limit state, and the level of damage on a capacity spectrum curve (Ghobarah 2001).

To obtain capacity curve of a structure, the non-linear force-deformation relations of the structural components (material non-linearity) and P-Δ effect (geometric nonlinearity) need to be considered. The capacity curve can be obtained from the pushover analysis, which is carried out based on the first mode response of structure assuming that the fundamental mode of vibration is the predominant response. Pushover capacity curve exhibits the behavior of structure beyond the elastic limit under seismic loads.

The demand curve is normally derived by the standard elastic 5% damped design spectrum. These demand curves are presented by the constant acceleration and velocity ranges plotted in an acceleration versus period domain. The result is plotted in “acceleration-displacement response spectrum” (ADRS) format. Capacity curve can be transformed to a capacity spectrum by

$$S_a = \frac{V/W}{\alpha_1} \tag{1}$$

$$S_d = \frac{\Delta_{roof}}{(PF_1 \phi_{roof,1})} \tag{2}$$

where, S_a and S_d are spectral acceleration and spectral displacement. V and W are base shear, and dead load plus likely live loads, respectively. Δ_{roof} represents roof displacement. Modal participation factor for the first natural mode, PF_1, can be obtained from

$$PF_1 = \left\{ \frac{\sum_{i=1}^{N} w_i \phi_{i1} / g}{\sum_{i=1}^{N} w_i \varphi_{i1}^2 / g} \right\} \tag{3}$$

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N} w_i \phi_{i1} / g \right]^2}{\left[\sum_{i=1}^{N} w_i / g \right] \left[\sum_{i=1}^{N} w_i \varphi_{i1}^2 / g \right]} \tag{4}$$

N is the level of the uppermost main portion of structure. α_1, w_i/g, ϕ_{i1} are the effective mass coefficient for the first natural mode, mass assigned to level i, amplitude of mode 1 at level i, respectively.

2.2 Resilience-based design

Resilience is the capability of the system to sustain the effects, ΔQ, of the extreme event at time t_{oE} and to recover efficiently a target level of functionality, $Q(t)$, at time t_{oE} plus T_{LC} (Cimellaro *et al*. 2010). For a single event, resilience can be defined by the following equation

$$R = \int_{t_{OE1}}^{t_{OE1}+T_{RE1}} Q(t)/T_{LC}\,dt \qquad (5)$$

where

$$Q(t) = [1 - L(I, T_{RE})][H(t - t_{OE}) - H(t_{OE} + T_{RE}))]\{f_{\mathrm{Re}\,c}(t, t_{OE}, T_{RE})\} \qquad (1)$$

where t_{OE} is the time of occurrence of event, T_{LC} is the control time of the system E, $L(I,T_{RE})$ is the loss function. H and f_{Rec} $(t,\ t_{0E},\ T_{RE})$ represent the Heaviside step function, and the recovery function. T_{RE} is the recovery time from event E necessary to go back to pre-disaster condition evaluated starting from t_{0E}.

The Resilience can be illustrated as the normalized shaded area under the functionality function of a system, $Q(t)$, which is a non-stationary stochastic process and each ensemble is a piecewise continuous function as the one shown in Fig. 1. Here, the functionality, $Q(t)$, is measured as a nondimensional function of time.

In Fig. 1, L is the loss, or the drop of functionality right after the extreme event. Robustness is strength or the ability of elements, systems and other measures of analysis to withstand a given level of stress or demand without suffering degradation or loss of function. It is therefore the residual functionality right after the extreme event and can be obtained from

$$\text{Robustness (\%)} = 1\text{-L (mL, } \sigma\text{L)} \qquad (7)$$

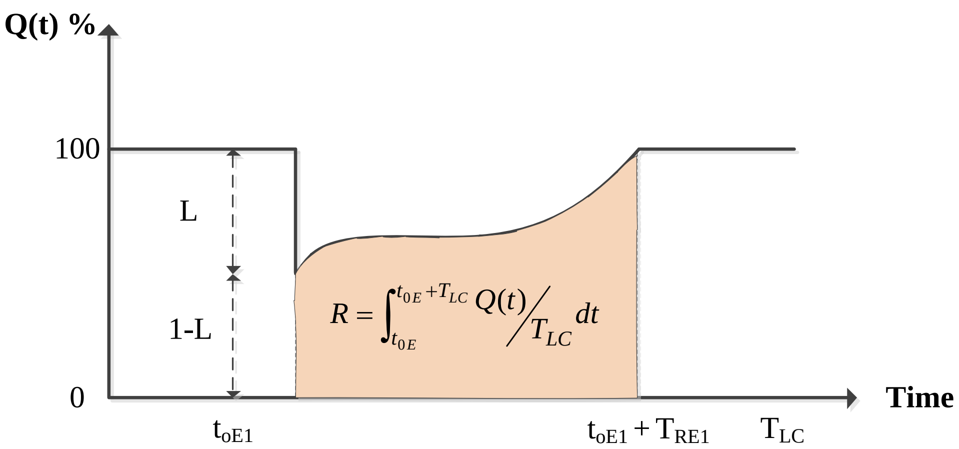


Fig. 1 Resilience-Functionality curve: average prepared community

where L is a random variable expressed as a function of the mean mL and the standard deviation σL (Cimellaroa *et al*. 2010).

Normally, three different types of recovery functions which are (i) linear, (ii) exponential and (iii) trigonometric can be selected depending on the system (resources and societal response) and societal preparedness. The simplest form; linear recovery function, is used herein as there is no information available regarding the preparedness, resources and societal response as follows

$$f_{\mathrm{Rec}}(t) = a\left(\frac{t - t_{OE}}{T_{RE}}\right) + b \tag{8}$$

3. Buildings under Investigation

3.1 Details of selected buildings

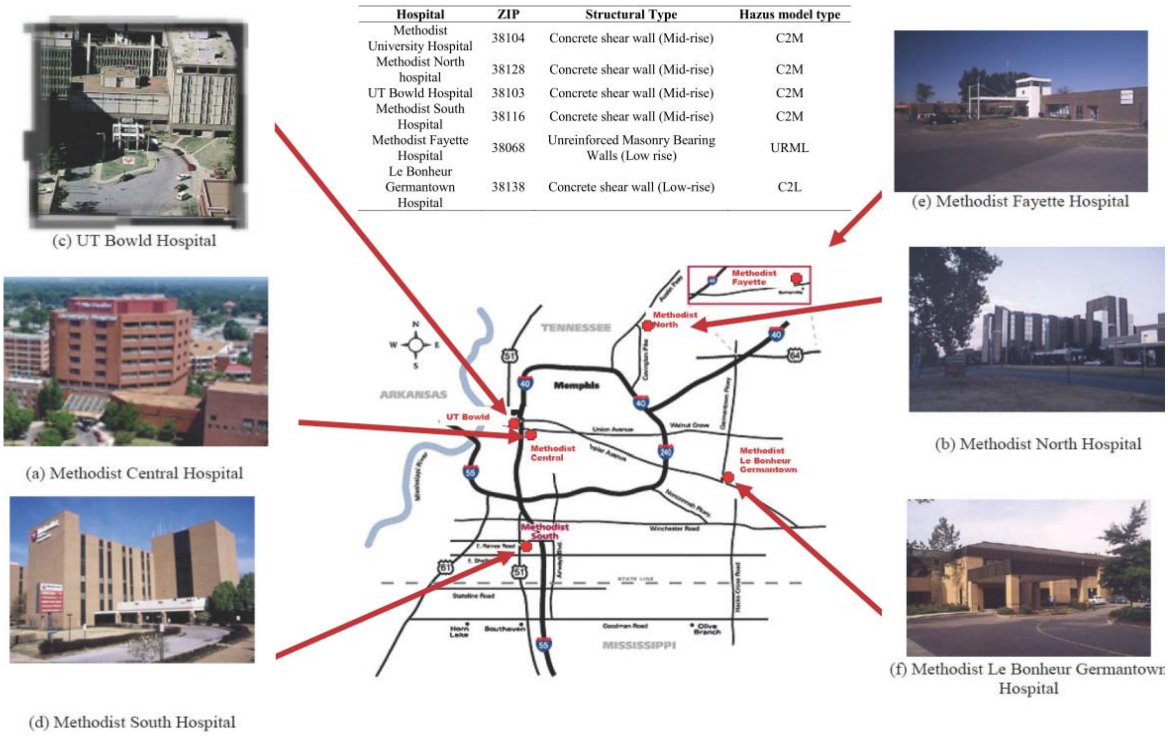
In this paper, two series of analyses on the rehabilitated buildings will be presented. In Series 1, resilience of rehabilitated hospital buildings in a pilot city under seismic events, reported by Cimellaro (2013), is reviewed and then, performance and resilience of several retrofitted reinforced concrete frames are discussed (Series 2). Therefore, resilience of rehabilitated hospital network (Series) is presented here to provide a platform for investigating resilience of RC frames which are strengthened by FRP and X-bracing system in Series 2.

3.1.1 Pilot study (Series 1)

In order to investigate capability of resilience-based design in functionality evaluation of buildings subjected to seismic events, a complex of six hospitals located in Memphis (Tennessee, USA) was selected as a pilot study (Fig. 2). It consists of a study aimed at the estimation of regional economic losses of several buildings within a geographical area like a city. Fig. 2 shows the locations by Zip codes that were used to define the seismic hazard and the structural type of the hospitals used to define the structural vulnerability.

This section presents methods which are used for rehabilitation strategies of aforementioned pilot study (Series 1). Four alternative seismic rehabilitation schemes are considered for hospital buildings, each structural type as per Federal Emergency Management Agency (FEMA- 276) (1999): 1) no action; 2) rehabilitation to life safety (LS) performance level; 3) rehabilitation to immediate occupancy (IO) performance level; 4) rebuild (RB), which are the target performance levels for rehabilitation against an earthquake.

Fragility curves for each rehabilitation alternative (Viti *et al*. 2006) are obtained directly correlating to the HAZUS code levels (2016). Fragility curves demonstrate the probability when the response of a structure exceeds threshold of the performance limit state (Cimellaroa *et al*. 2010). Therefore, the HAZUS code levels are assigned to the rehabilitation levels mentioned above with reasonable assumptions. For example, the “No Action” option corresponds to a slight damage while “retrofit to immediate occupancy level” indicates moderate damage condition. Fragility curves are developed for structural damage and non-structural damage of drift sensitive and accelerations sensitive components using the HAZUS approach. Fig. 3 shows fragility curves of structural damage for concrete shear walls mid-rise building type (C2M) versus return period.



Hospital	ZIP	Structural Type	Hazus model type
Methodist University Hospital	38104	Concrete shear wall (Mid-rise)	C2M
Methodist North hospital	38128	Concrete shear wall (Mid-rise)	C2M
UT Bowld Hospital	38103	Concrete shear wall (Mid-rise)	C2M
Methodist South Hospital	38116	Concrete shear wall (Mid-rise)	C2M
Methodist Fayette Hospital	38068	Unreinforced Masonry Bearing Walls (Low rise)	URML
Le Bonheur Germantown Hospital	38138	Concrete shear wall (Low-rise)	C2L

(c) UT Bowld Hospital

(a) Methodist Central Hospital

(d) Methodist South Hospital

(e) Methodist Fayette Hospital

(b) Methodist North Hospital

(f) Methodist Le Bonheur Germantown Hospital

Fig. 2 System definition by Cimellaro *et al*. (2010)

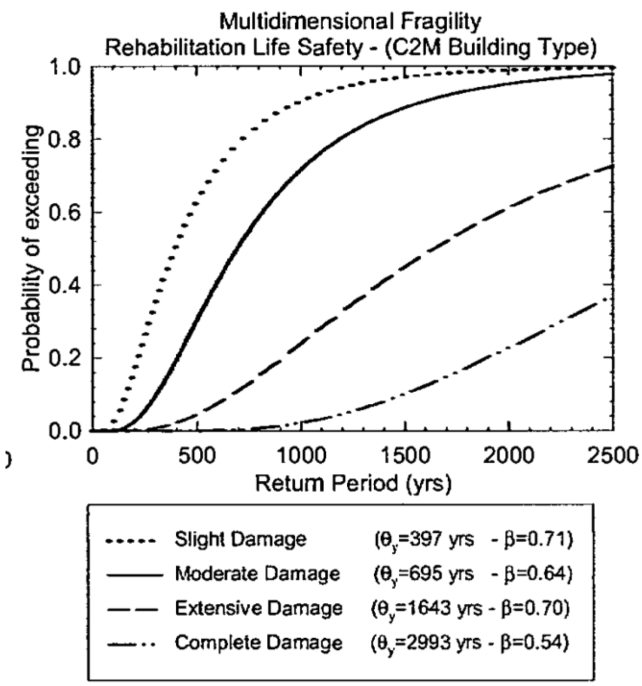


Fig. 3 Multidimensional fragility curves for C2M type structure, -rehabilitation to LS (Cimellaroa *et al*. 2010)

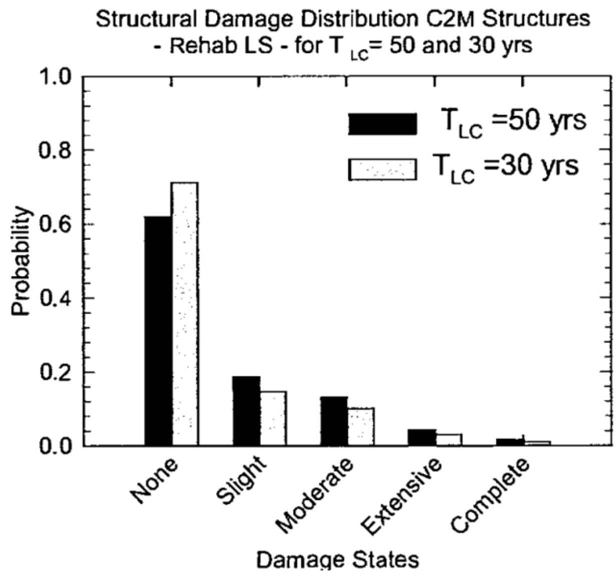


Fig. 1 Structural damage distribution for different rehabilitation strategies (T_{lc}=30 years) for C2M type structure -rehabilitation to LS (Cimellaro *et al*. 2010)

The control period, T_{LC}, for a decision analysis is chosen based on the decision maker's interest for assessment of the retrofit options. Generally, the longer time period of the building, the better justification for system rehabilitation. On the other hand a decision maker may prefer to retrofit structure when the rehabilitation is justified with shorter time period. Therefore, it is assumed T_{LC} of 30 years and a discount annual rate r of 6% (Cimellaroa *et al*. 2010).

A comparison of structural damage distributions for C2M type structures for two time control periods, T_{LC}=30 years and T_{LC}=50 years, is shown in Fig. 4. As expected the probability of having no damage increases for the shorter time periods.

The seismic input is normalized using four different hazard levels for simplicity. These levels of earthquakes in the region include earthquakes with 2%, 5% 10% and 20% probability of exceedance P in 50 years.

In order to improve the disaster resilience of the hospital system, four different schemes were considered for this reference case study: (a) moment resisting frames; (b) buckling restrained braces; (c) shear walls, and (d) weakening and damping (Cimellaro *et al*. 2010).

3.1.2 Low-to Medium-Rise frames (Series 2-this study)

To investigate performance and resilience of RC frames under seismic loads, an eight storey four bay OMRF is selected. The frame is designed and analysed according to the Australian Concrete Code (AS3600 2001). A 1/2.2 scale model of the frame (Fig. 5(a)) is then formed by the application of the similitude requirements of the Buckingham's theorem (Noor and Boswell 1992). According to Mahini (2005), performing the tests on a full-scale joint of the selected frames was impossible considering the limitations of the equipment sizes and capacities. Scaling down the size of samples is therefore required. In this research, the scale down frame behaviour is investigated before and after retrofitting due to the available data previously developed by Hadigheh *et al*. (2014).

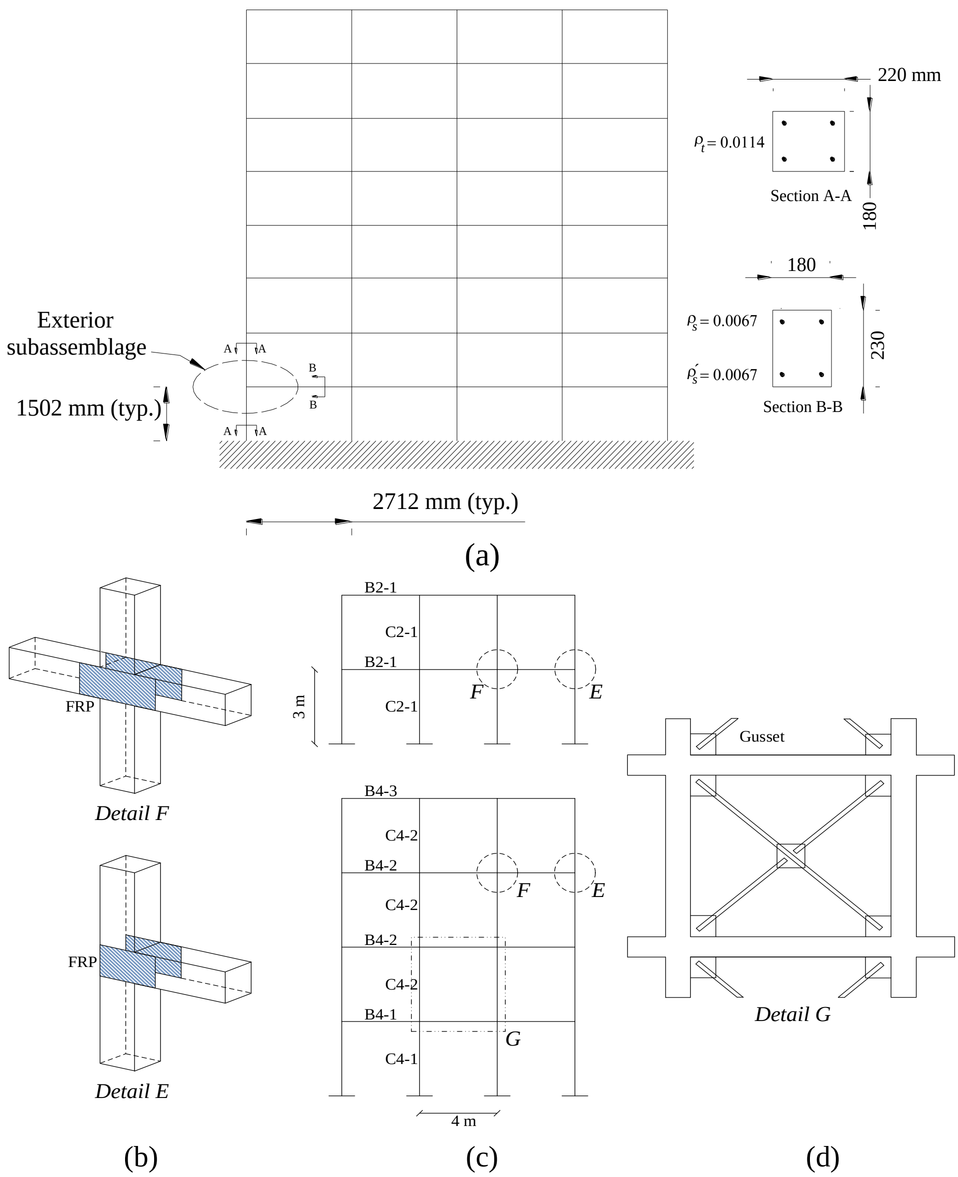


Fig. 5 Studied frames, (a) geometry of eight-storey frame (Mahini 2005), (b) FRP strengthening detail, (c) geometry of two and four-storey frames, and (d) steel bracing system (Hadigheh *et al*. 2014)

Four N12 (ϕ 12 mm) rebars are used for both the column vertical reinforcement and the beam longitudinal reinforcement. R6.5 bars (ϕ 6.5 mm) are used for stirrups at a spacing of 150 mm in both beam and column. A 30 mm concrete cover is considered for the beam and column reinforcements which is about half of the corresponding covers in prototype (Table).

The tensile properties of various deformed N12 reinforcing steel bars and plain R6.5 stirrups and ties are obtained from coupon tests in a Universal Testing Machine using a mechanical extensometer of 20 mm gage length. The average yield strengths of deformed N12 reinforcing steel bars and plain R6.5 mm stirrups and ties, are 507 MPa and 382 MPa, respectively. The modulus of elasticity of reinforcements is 200 GPa. Four N12 rebars are used for both the column vertical reinforcement and the beam longitudinal reinforcement. R6.5 bars are used for stirrups with a spacing of 150 mm in column and beam (Mahini 2005, Mahini and Ronagh 2009, 2011).

In order to study the seismic behaviour of low-rise frames, a 2-storey and a 4-storey frame (Fig. 5(c)) are designed and analysed based on Standard No. 2800 (2005) and ABA (2005). The column and beam dimensions are presented in Table 2. The vertical gravity load is calculated as dead load (D.L.=21.6 kN/m) and live load (L.L.=13.7 kN/m). Equivalent static earthquake lateral loads on the frames are derived using the design response spectrum of Standard No. 2800 (2005).

The compressive strength of the concrete and the yield stress of the steel reinforcements are assumed to be 40 MPa and 340 MPa, respectively. For all sections, the minimum and maximum values of the steel reinforcement is checked against the ABA (2005).

Since earthquake hazards in Australia are lower than in Iran, the response spectrum of Standard No. 2800 (2005) is higher than the Australian counterpart (AS1170.4 1993). Therefore, 8-storey frame is designed based on Australian standard while 2- and 4-storey frames designed and analysed according to Standard No. 2800 (2005) to cover both cases of low and high seismic hazards.

In the case of retrofitting of low-rise RC frames, two different techniques are applied, namely steel bracing and FRP techniques. Braces are placed in the middle bay of the frames. To estimate the seismic load, it is assumed that the frames are located in a zone with a high seismic hazard, 0.35 g. The seismic reduction factor, R (see Section 4), is initially adopted as R=4, and 75% of the lateral load, 0.75 V, is applied to design each RC frame. However after adding the X-steel brace to the RC frames 100% of the lateral load is applied, V. Therefore, the steel braces are designed to withstand a 25% share of the lateral load. Details of the bracing system are presented in Hadigheh *et al*. (2014).

Table 1 Column and beam dimensions of 8-storey frame

Section	Height (mm)	Width (mm)	Longitudinal Reinforcement	Stirrups	Concrete Cover (mm)
A-A	180	220	4N12	R6.5@150 mm	30
B-B	230	180	4N12	R6.5@150 mm	30

Table 2 Column and beam dimensions of 2- and 4-storey frames

Section	Height (mm)	Width (mm)	Longitudinal Reinforcement (%)
C2-1, 2	300	300	2.053
B2-1, 2	400	300	1.166 (Top) 0.528 (Bottom)
C4-1	450	450	1.007
C4-2, 3, 4	350	350	1.312
B4-1	450	450	0.638 (Top) 0.503 (Bottom)
B4-2, 3	350	350	1.396 (Top) 0.684 (Bottom)
B4-4	300	300	1.286 (Top) 0.986 (Bottom)

For FRP retrofitting technique, all of the joints (except the joints of the last floor) are retrofitted on their web by FRP sheets with overall 2 mm thickness and a length of 350 mm (Fig. 5(b)). The frames are designed with 75% *V* and retrofitted with FRP sheets. More details regarding FRP-retrofitting design can be found in Hadigheh *et al*. (2014). For simplicity, FRP retrofitting scheme is designed based on the critical joint at the first floor. Then, the same FRP retrofitting is used for other levels. This will guarantee that the plastic hinge relocation occurs in the upper levels which can prove the practicality of the proposed retrofitting system in a real-world application.

4. Seismic response of rehabilitated buildings

For the pilot study (Series 1), the disaster resilience value is calculated according to Eq. (5). The expected equivalent earthquake losses for each rehabilitation scheme were obtained considering the probability of each level of the earthquake, along with the initial rehabilitation costs, followed by the total expected losses considering an observation period, T_{LC}, of 30 years.

The recovery time and resilience values are shown in Fig. 6. For this case study, it is shown that the "rebuild" option has the largest disaster resilience of 96.5%, when compared with the other three strategies, but it is also the most expensive solution. However, if "No Action" is taken, the disaster resilience is still reasonably high (81.9%). Cimellaro (2008) reported that the initial investment and resilience are not linearly related.

When functionality is very high, a very large amount of investment is required to slightly improve functionality compared with the case when the functionality of the system is low. Although this is obvious, the procedure presented herein can be used by decision makers.

For low-rise and short-period frames (Series 2), nonlinear static (pushover) analysis is more appropriate than inelastic dynamic analysis (Mwafy and Elnashai 2001, Mwafy and Elnashai 2002). Makarios (2012) presented a new seismic nonlinear static (pushover) procedure to obtain

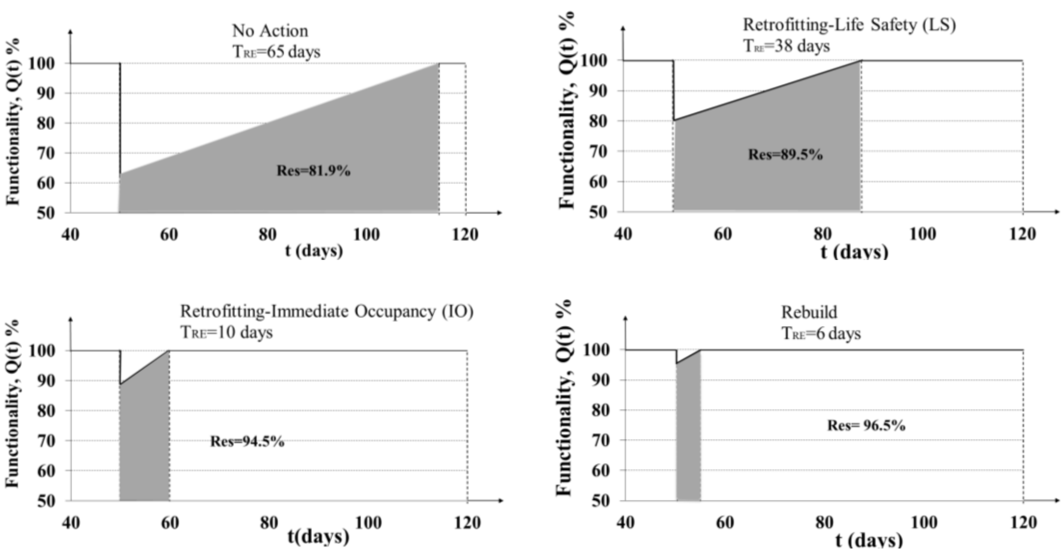


Fig. 6 Resilience for different retrofitting strategies, adapted from (Cimellaro 2008)

the seismic demands and the available behaviour factors of spatial asymmetric multi-story RC buildings. Due to simplicity of nonlinear static procedure compare to nonlinear dynamic analysis, a pushover analysis is employed in this research.

The nonlinear pushover analysis is performed for both the original and the retrofitted frames. The moment-rotation relationship of the joints (obtained from ABAQUS) is then incorporated into the finite element (FE) models of the frames and pushover analyses are carried out. More information regarding ABAQUS model and implementation of moment-rotation curves into pushover analysis is given in Hadigheh *et al*. (2014). The lateral load distribution is proportional to the product of the storey mass and the first mode shape of structure. The mass source of the frame is assumed to be the dead load plus 20% of the live load, according to Standard No. 2800 (2005).

To evaluate the ductility ratios of the frames, bilinear curves are fitted to the pushover curves of the original and retrofitted frames. The ductility ratio, μ, is calculated using the following relationship

$$\mu = \Delta_{\max} / \Delta_y \tag{9}$$

where $\Delta_{\max}$ is the ultimate displacement and Δ_y is the idealised structural yield displacement evaluated from the bilinear curve.

The behaviour factor, R, is a force reduction factor used to reduce the linear elastic response spectra (the elastic acceleration spectrum for defining the seismic hazard of a site) to the inelastic response spectra (the inelastic spectrum used to determine the seismic design forces)

$$R = (S_a)_d^{el} / (S_a)_d^{in} \tag{10}$$

where $(S_a)_d^{el}$ and $(S_a)_d^{in}$ are the elastic and inelastic design spectral acceleration values, respectively.

The behaviour factor consists of three different components (Eq. (1)): the ductility reduction factor, R_μ, the overstrength factor, R_s, and the allowable stress factor, Y

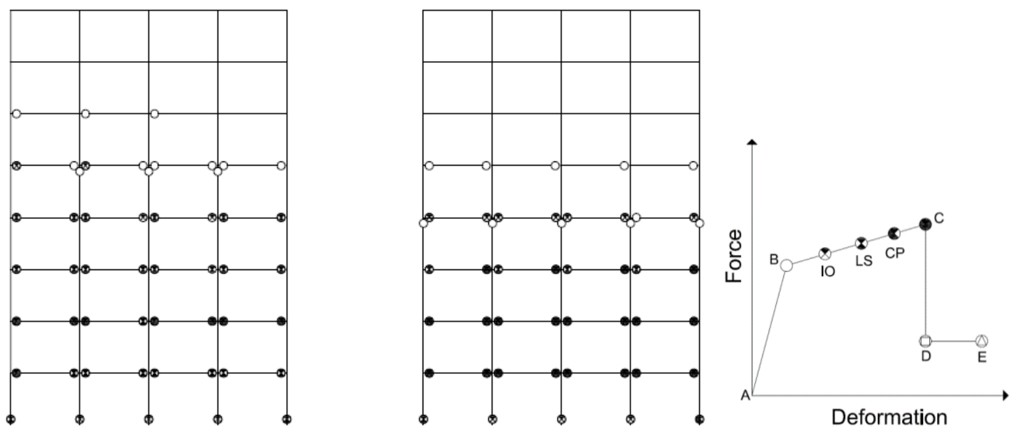
$$R = R_\mu . R_s . Y \tag{8}$$

Because of using the ultimate strength method, the allowable stress factor Y is assumed unity and R_s is evaluated as the ratio between the supply and the design resistances (Borzi and Elnashai 2000)

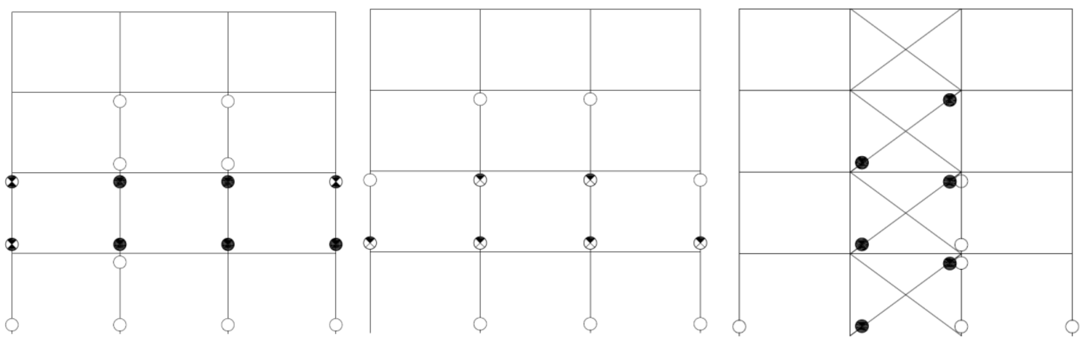
$$R_s = \frac{V_y}{V_s} \tag{9}$$

where V_y is the idealised yield strength and V_s is the base shear at which the first plastic hinge is formed in the structure.

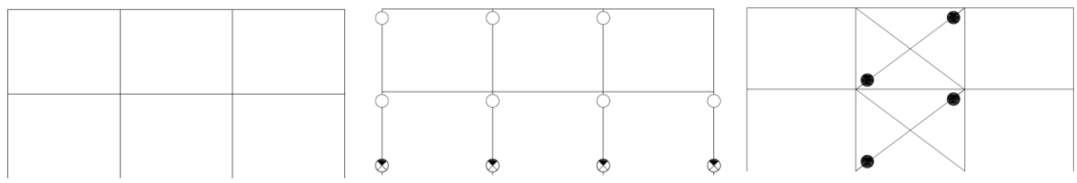
The state of building damage under earthquake excitation defines the performance level of a building by considering the vertical and horizontal lateral-force-resisting elements. These performance levels consist of three main objectives; immediate occupancy (IO), life safety (LS) and collapse prevention (CP). LS is defined as the post-earthquake state that includes damage to the elements but retains a margin against the onset of a partial or total collapse in structure (Federal Emergency Management Agency (FEMA- 356) 2000). For instance, Fig. 7 represents the plastic hinge distributions at the specific performance levels of the original and retrofitted frames.



(a) (b)



(a) (b) (c)



(a) (b) (c)

Fig. 7 Plastic hinge formation in (a) original frame, (b) FRP retrofitted frame, (c) steel-braced frame

In 8-storey frame, no plastic hinge is formed on the 6th floor after retrofitting. In plain eight-storey frame, approximately 74 per cent of plastic hinging occurs on the beams, whereas FRP retrofitted frames exhibit a 5 per cent improvement in beam hinging. This trend was also observed in previous research (Niroomandi *et al*. 2010) for plain frames.

Although no beam hinging occurred in low-rise frames after FRP retrofitting, the plastic hinging improved in the 4-storey frame, which was reclassified from collapse, C, to the acceptance criteria of immediate occupancy, IO. This trend was also observed for a 2-story frame. However, the plastic hinge numbers increased after FRP retrofitting. Again, the steel braced system exhibits lower plastic hinge formation, and a lower ductility demand is therefore expected for this system.

Table 1 The performance points of original and retrofitted frames

Type of the frame	Performance point (S_d, S_a)		
	2-storey	4-storey	8-storey
Plain	(4.00, 0.36 g)	(2.66, 0.27 g)	NPP
Retrofitted with FRP laminates	(3.79, 0.38 g)	NPP	(12.27,0.126 g)
Retrofitted with X-braced steel	(1.00, 0.70 g)	(0.92, 0.66 g)	-

*NPP: no performance point observed

Table 2 Recovery time and resilience of RC frames for rehabilitation strategies (T_{LC}=30 years).

Rehabilitation Alternatives	Type of the frame / Performance Level*	2-storey	4-storey	8-storey	Recovery Time, T_{RE} (days)	Resilience, Res [%]
Plain	NA	●	●	●	65	81.9
	LS	-	-	-	-	-
	IO	-	-	-	-	-
	RB	●	●	●	6	96.5
FRP-Retrofitted	LS	-	-	●	38	89.5
	IO	●	●	-	10	94.5
X-braced steel	LS	●	●	●	38	89.5
	IO	-	-	-	-	-

*NA: no action

Based on the pushover results, retrofitted frames meet the performance objectives of LS. This retrofitting technique can improve the behaviour of the frame under earthquake motions to the desired level. It should be noted that response spectrum of Standard No. 2800 (2005) is much higher than the response spectrum of the Australian Standard (AS1170.4 1993) because earthquake hazards in Australia are lower than in Iran. As the original frame was designed according to the Australian seismic code the shortfall of the frame is explained by the differences between the Iranian and the Australian response spectra. However, FRP retrofitting of the joints upgraded the frame to satisfy the LS performance level of FEMA- 356.

Table 3 presents the performance points of frames. According to this table, FRP-retrofitting of the 4-storey frame failed to upgrade the frame to satisfy the life-safety performance demand of the selected Standard No. 2800 (2005) earthquake, indicating insufficient thickness for FRP laminates. However the steel bracing of the frames considerably enhanced the performance to meet the required LS demands by substantially increasing capacities of the frames at the expense of highly reduced ductility.

Table 4 presents values of resilience for the two different retrofit techniques (FRP and X-braces) and for different low-to-medium-rise RC frames. This table shows that the best improvement in terms of resilience is obtained using a FRP retrofit strategy for 2 and 4 storey frames. However with the 8-storey building both retrofitting strategies led to the same improvement in shifting the building performance to the LS level. For more clarity, performance levels of the frames are shown by black circles in Table 4.

Although the difference in resilience of the frames is small, loss term of the ductility (complementary to resilience) indicates the advantage of FRP scheme. Based on Table 4 and force-deformation diagram in Fig. 7, it can be seen that the frames in immediate occupancy condition show lower deformation in compare with those in LS condition. Therefore, FRP retrofitting technique reduces displacements and maintains the ductility, regardless of the number of stories. Since reduction in ductility of existing RC OMRFs is not desirable, the ductility should be maintained in the seismic performance.

5. Conclusions

Resilience-based design integrates information from various fields such as earthquake engineering, social science and economics into a unique function. It implements uninformed intuitions/preconceived notions of risk which lead to unbiased results.

This paper provided a quantitative definition of seismic resilience versus the more conventional DBD method for RC frames retrofitted with FRPs and steel bracing technique. In this rationale, an analytical function was applied that can fit both technical and organizational issues. A regional complex of six hospitals was used as a reference to illustrate the applicability of the framework and to assess the seismic resilience of selected plain and retrofitted RC frames versus the DBD method.

It was shown that FRP retrofitting can improve resilience of RC frames by 15% (from 89.5% to 94.5%). The steel bracing considerably enhances the performance of the frames to meet the required life-safety demands by substantially increasing strength capacity of the frames at the expense of highly reduced ductility and lower seismic resilience. Therefore, FRP retrofit is more effective in terms of improving performance and the ductility in low-rise RC buildings, as the measured resilience illustrated an enhanced value compared to un-retrofitted structure. Results presented in this research are from a preliminary study (two case studies) and require further investigation on performance parameters and resilience factors of the structures. Based on this fact, the assumptions made herein are only representative of the presented cases.

References

ABA (2005), *Iranian Concrete Code*, Islamic Republic of Iran Management and Planning Organization.

Applied Technology Council (ATC) (1996), *Seismic Evaluation and Retrofit of Concrete Buildings*, Report No. ATC-40, Redwood City, California.

AS1170.4 (1993), *Structural design Actions Part 4: Earthquake Actions in Australia*, Standards Australia, Sydney, Australia.

AS1170.4 (2007), *Structural design Actions Part 4: Earthquake Actions in Australia*, Standards Australia, Sydney, Australia.

AS3600 (2001), *Concrete Structures*, Standards Australia, Homebush Bay, Australia.

Biondini, F., Camnasio, E. and Titi, A. (2015), "Seismic resilience of concrete structures under corrosion", *Earthq. Eng. Struct. Dyn.*, **44**(14), 2445-2466.

Borzi, B. and Elnashai, A.S. (2000), "Refined force reduction factors for seismic design", *Eng. Struct.*, **22**(10), 1244-1260.

Chang, S. and Shinozuka, M. (2004), "Measuring improvements in the disaster resilience of communities", *Earthq. Spectra*, **20**(3), 739-755.

Cimellaro, G.P. (2013), *Resilience-based design (RBD) modelling of civil infrastructure to assess seismic hazards*, in Handbook of Seismic Risk Analysis and Management of Civil Infrastructure Systems, Woodhead Publishing, 912.

Cimellaro, G.P., Reinhorn, A.M. and Bruneau, M. (2010), "Framework for analytical quantification of disaster resilience", *Eng. Struct.*, **32**(11), 3639-3649.

Cimellaro, G.P. (2008), "Seismic resilience of a regional system of hospitals, Multidisciplinary Center for Earthquake Engineering Research (MCEER) publication, <http://mceer.buffalo.edu/publications/resaccom/07-SP05/01Cimellaro.pdf>, Buffalo University, USA.

Cimellaroa, G.P., Reinhornb, A.M. and Bruneau, M. (2010), "Seismic resilience of a hospital system", *Struct. Infrastruct. Eng.*, **6**(1-2), 127-144.

Federal Emergency Management Agency (FEMA- 276) (1999), *Example applications of the NEHRP guidelines for the seismic rehabilitation of buildings*, Federal Emergency Management Agency, Washington, D.C.

Federal Emergency Management Agency (FEMA- 356) (2000), *Prestandard and commentary for the seismic rehabilitation of buildings*, Federal Emergency Management Agency, Washington, D.C.

Ghobarah, A. (2001), "Performance-based design in earthquake engineering: state of development, *Eng. Struct.*, **23**(8), 878-884.

Hadigheh, S.A., Mahini, S.S. and Maheri, M.R. (2014), "Seismic behaviour of FRP-retrofitted reinforced concrete frames, *J. Earthq. Eng.*, **18**(8), 1171-1197.

HAZUS-MH (2016), *Technical Manual*, Department of Homeland Security Federal Emergency Management Agency Mitigation Division Washington, D.C.

Lumantarna, E., Lam, N., Wilson, J. and Griffith, M. (2010), "Inelastic displacement demand of strength-degraded structures", *J. Earthq. Eng.*, **14**(4), 487-511.

Mahini, S.S. (2005), "Rehabilitation of exterior RC beam-column joints using web-bonded FRP sheets", Ph.D. Thesis, The University of Queensland.

Mahini, S.S., Hadigheh, S.A. and Setunge, S. (2015a), "Seismic resilience of retrofitted reinforced concrete buildings", *Proceedings of the Second International Conference on Performance-based and Life-cycle Structural Engineering (PLSE 2015)*, Brisbane, Australia.

Mahini, S.S., Setunge, S. and Hadigheh, S.A. (2015b), "Performance vs resilience-based earthquake design for low and medium-rise retrofitted RC buildings", *Proceedings of the Tenth Pacific Conference on Earthquake Engineering Building an Earthquake-Resilient Pacific (PCEE2015)*, Sydney, Australia.

Mahini, S.S. and Ronagh, H.R. (2011), "Numerical modelling for monitoring the hysteretic behaviour of CFRP-retrofitted RC exterior beam-column joints", *Struct. Eng. Mech.*, **38**(1), 27-37.

Mahini, S.S. and Ronagh, H.R. (2009), "Numerical modelling of FRP strengthened RC beam-column joints", *Struct. Eng. Mech.*, **32**(5), 649-665.

Makarios, T.K. (2012), "Seismic nonlinear static new method of spatial asymmetric multi-storey reinforced concrete buildings", *Struct. Des. Tall Spec. Build.*, **21**(11), 800-823.

Mwafy, A.M. and Elnashai, A.S. (2001), "Static pushover versus dynamic collapse analysis of RC buildings", *Eng. Struct.*, **23**(5), 407-424.

Mwafy, A.M. and Elnashai, A.S. (2002), "Calibration of force reduction factors of RC buildings", *J. Earthq. Eng.*, **6**(2), 239-273.

Niroomandi, A., Maheri, A., Maheri, M.R. and Mahini, S.S. (2010), "Seismic performance of ordinary RC frames retrofitted at joints by FRP sheets", *Eng. Struct*, **32**(8), 2326-2336.

Noor, F.A. and Boswell, L.F. (1992), Elsevier Science Pub. Co., 357.

Standard No. 2800 (2005), *Iranian code of practice for seismic resistant design of buildings*, Building and Housing Research Center, Tehran, Iran.

Viti, S., Cimellaro, G.P. and Reinhorn, A.M. (2006), "Retrofit of a hospital through strength reduction and enhanced damping", *Smart Struct. Syst.*, **2**(4), 339-355.

Wilson, J.L., Lam, N.T.K. and Pham, L. (2008), "Development of the new Australian earthquake loading standard", *EJSE Special Issue: Earthquake Engineering in the low and moderate seismic regions of*

Southeast Asia and Australia, 25-31.
Zou, X.K., Teng, J.G., De Lorenzis, L. and Xia, S.H. (2007), “Optimal performance-based design of FRP jackets for seismic retrofit of reinforced concrete frames”, *Composites Part B: Engineering*, **38**(5-6), 584-597.

JL