Proceedings of the Tenth Pacific Conference on Earthquake Engineering
Building an Earthquake-Resilient Pacific
6-8 November 2015, Sydney, Australia

Performance vs Resilience-based Earthquake Design for Low and Medium-rise retrofitted RC Buildings

S.S. Mahini

Discipline of Civil and Environmental Engineering, School of Environmental & Rural Science, University of New England, NSW, Australia Email: smahini@une.edu.au (corresponding author)

S. Setunge & S.A. Hadigheh

School of Civil, Environmental and Chemical Engineering, RMIT University, VIC, Australia

ABSTRACT: A large number of reinforced concrete (RC) structures designed according to older codes do not satisfy the requirements of new seismic design standards. 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 the performance of the 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 Earthquake Design aims to maintain functionality of buildings and provide liveable conditions after strong ground movement. In fact, Resilience-Based Design (RBD) can be considered as the next generation of PBD. In this paper, seismic performances of a scaled eight-story frame and two full-scaled low-rise RC frames are evaluated. In order to evaluate the earthquake performance of the frames, the performance points of the two frames are calculated by the Capacity Spectrum Method (CSM) of ATC-40. This method estimates the maximum response of a structure by expressing both structure capacity and the ground motion demand in terms of spectral acceleration and displacement. Finally, the seismic performances of the frames are evaluated and the results are compared with a resilience-based design criterion.

1 INTRODUCTION

In conventional seismic assessment of structures, the trade-off strength demand is compared with the ductility (displacement) demand on the structure (Lumantarna, et al., 2010). Designing the structure based on performance objectives have been practiced in the last decades. In recent years, the performance-based design method has been added to seismic design codes (Mahini, 2015). A limit state is a form of performance objective in which the target displacement is considered as the response parameter of a substitute single- degree-of-freedom (SDOF) system. The structural response in terms of displacement (i.e. displacement-based design method, DBDM) can be also related to strain-based limit state, and the level of damage on a capacity spectrum curve (Ghobarah, 2001). In ATC-40 (1996), it is so-called the Capacity Spectrum Method (CSM) (Hadigeh et al., 2015). Although, DBDM prevents loss of life, it cannot limit damages or maintain functionality.

‘Resilience’ is a performance assessment methodology primarily developed by the Multidisciplinary Center for Earthquake Engineering Research (MCEER) to improve ‘decision making’ procedures for the assessment of seismic performance of structural systems. It involves the discovery and development of new knowledge and technologies for equipping communities to become more disaster resilient in regard to earthquakes and other extreme events. In other word, 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).

The MCEER terminology defines seismic resilience as a decision variable (DV) 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 can compare losses and different pre and post event measures in order to verify if strategies and actions can reduce or eliminate disruptions during seismic events (Cimellaro, 2008). In this regard, Chang et al. (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 seismic resilience of deteriorating concrete structures. Cimellaro et al. (2010) presented a quantitative evaluation of the concepts of disaster resilience and a unified terminology for a common reference framework for the evaluation of health care facilities subjected to earthquakes.

In this paper a formulated framework for a hospital complex system is employed in order to assess the seismic behaviour of low-to-medium-rise RC Buildings retrofitted with steel braces and FRP composites (Mahini, 2015, Mahini and Ronagh, 2010, 2011) previously evaluated by Performance-Based Design (PBD) approach by Niroomandi et al. (2010) and Hadigeh et al. (2015). Although this type of design prevents loss of people life it cannot maintain functionality or limit damages using newly developed Resilience-Based Earthquake Design.

2 THE RESILIENCE

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} . For a single event, it can be defined by the following equation.

$$R = \int_{t_{OE}}^{t_{OE}+T_{LC}} Q(t)/T_{LC}dt \tag{1}$$

where

$$Q(t) = [1 - L(I, T_{RE})][H(t - t_{OE}) - H(t_{OE} + T_{RE}))]\{f_{\mathrm{Re}c}(t, t_{OE}, T_{RE})\} \tag{2}$$

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; $H\,()$ is the Heaviside step function, $f_{\mathrm{Rec}}(t, t_{0E}, T_{RE})$ is the recovery function and 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 graphically as the normalized shaded area underneath the functionality function of a system, Q (t). Q (t) is a non-stationary stochastic process and each ensemble is a piecewise continuous function as the one shown in Figure 1, where the functionality Q (t) is measured as a percentage function of time.

In this figure, L is the loss, or the drop of functionality, right after the extreme event, and R is the Robustness which 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 represented by:

Robustness (%) = 1-L (m_L, σ_L) (3)

where L is a random variable expressed as a function of the mean m_L and the standard deviation σL.

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.

$$f_{\mathrm{Re}c}(t) = a\left(\frac{t - t_{OE}}{T_{RE}}\right) + b \tag{4}$$

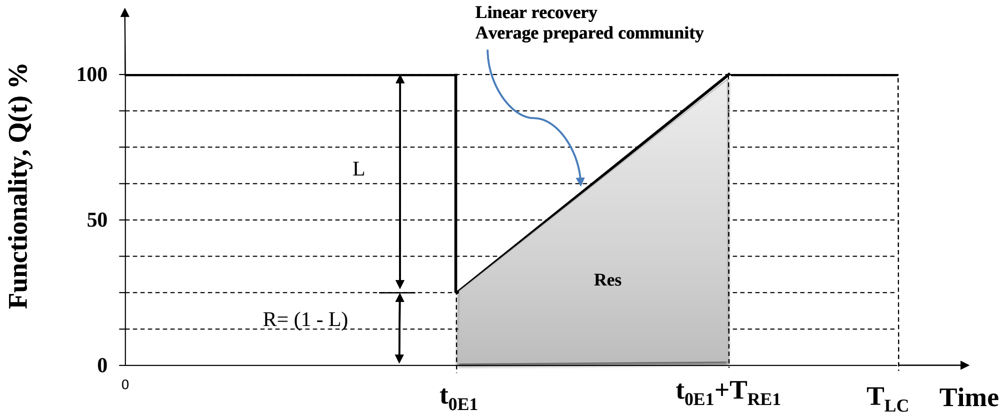


Figure 1. Resilience-Functionality curve: average prepared community

3 SEISMIC PERFORMANCE OF THE SELECTED LOW-MID-RISE RETROFITTED BUILDINGS

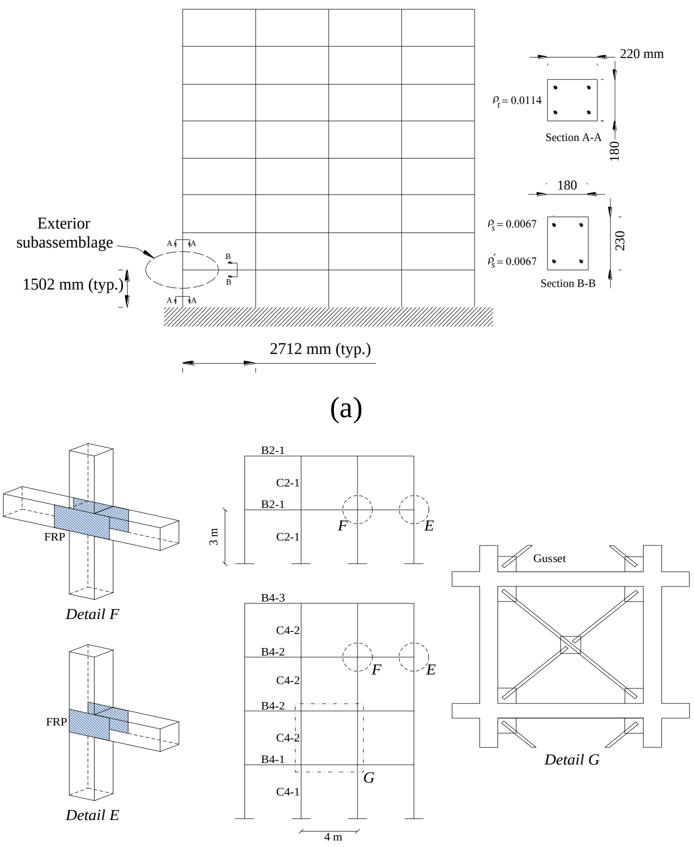
The selected frame was an eight storey four bay moment resisting frame. The frame was designed according to the Australian Concrete Code AS3600 (2001) as an Ordinary Moment Resisting Frame (OMRF). A 1/2.2 scale model of the frame (see Figure 2a) was then formed by the application of the similitude requirements of the Buckingham's theorem (Mahini, 2005). Based on the scale factors, the scale down frame was loaded, analysed and designed according to AS3600. Four N12 (ϕ 12mm) rebars were used for both the column vertical reinforcement and the beam longitudinal reinforcement. R6.5 bars (ϕ 6.5mm) were used for stirrups at a spacing of 150mm in both beam and column. A 30mm concrete cover was considered for the beam and column reinforcements which is about half of the corresponding covers in prototype. The tensile properties of various deformed N12 reinforcing steel bars and plain R6.5 (ϕ 6.5mm) stirrups and ties are obtained by testing them under monotonic loading in a Universal Testing Machine (UTM) using a mechanical extensometer of 20mm gage length. The average yield strengths of deformed N12 reinforcing steel bars and plain R6.5mm stirrups and ties, were 507 MPa and 382 MPa respectively and the modulus of elasticity of both reinforcements was 200GPa. Four N12 rebars were used for both the column vertical reinforcement and the beam longitudinal reinforcement. R6.5 bars were used for stirrups with a spacing of 150mm in both column and beam. To study the seismic behaviour of low-rise frames, a 2-storey and a 4-storey frame, retrofitted by FRP materials and by steel braces, two OMRFs were investigated (Figure 2c). The column and beam dimensions are presented in Table 1. The vertical gravity loads were calculated as D.L. = 21.6kN/m and L.L. = 13.7kN/m, and equivalent static earthquake lateral loads on the frames were derived by using the design response spectrum of Standard No. 2800 (2005).

In the case of steel brace retrofitting, braces were placed in the middle bay of the frames. For the FRP retrofitting technique, all of the joints (except the joints of the last floor) were retrofitted on their web by FRP sheets with overall 2mm thickness and a length of 350mm (Figure 2b). Further details of the FRP-retrofitting design can be found in Mahini (2005). For simplicity, the FRP retrofitting scheme is designed based on the critical joint at the first floor and is then kept identical for other levels in order to guarantee that the plastic hinge relocations are formed in the upper levels and prove the practicality of the proposed retrofitting system in a real-world application.

A nonlinear static procedure (NSP) represents the response of a structure undergoing dynamic loads. A pushover analysis, as a nonlinear static procedure, is employed as an alternative for static and nonlinear dynamic analysis because of its accuracy and simplicity. To study the influence of the retrofitting technique, a nonlinear pushover analysis was performed for both the original and the retrofitted frames. The moment-rotation relationship of the joints (obtained from ABAQUS) was then incorporated into the FE models of the frames and pushover analyses were carried out. The lateral load distribution is proportional to the product of the storey mass and the first mode shape of the structure.

The mass source of frame is assumed to be the dead load plus 20% of the live load, according to Standard No. 2800 (2005).

To evaluate the seismic load, it was assumed that the frames are located in a zone with a high seismic hazard (DBA = 0.35 g). The seismic reduction factor was initially assumed to be R=4 and 75% of the lateral load (0.75 V) was applied to the design of each RC frame. However after adding the X-steel brace to the RC frames 100% of the lateral load was applied (V). Therefore, the steel braces were designed to withstand a 25% share of the lateral load. In case of FRP retrofitting, frames were designed with 75% V and retrofitted with FRP sheets. Details of the bracing system are presented in Hadigeh et al. (2015). The compressive strength of the concrete and the yield stress of the steel reinforcements were assumed to be 40N/mm^2 and 340N/mm^2, respectively. For all sections, the minimum and maximum values of the steel reinforcements were checked against the ABA concrete code (2005).



(b) (c) (d)

Figure 2. Studied frames, (a) geometry of eight-storey frames (b) FRP strengthening details, (c) geometry of two and four-storey frames, and (d) steel bracing system (Hadigeh et al., 2015).

To evaluate the earthquake performance of the frames before and after retrofitting the performance points of two frames are calculated by the Capacity Spectrum Method (CSM) of ATC-40 (1996). To determine the performance point of the eight-storey frames, the response spectrum of Standard 2800 (2005) is used for a region with a low risk of an earthquake corresponding to DBA=0.2 g. According to procedure A from ATC-40 a computer programme was created to reach a solution for the approximation and find the performance point of the two frames.

Table 1 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)

Figure 3 shows the capacity and the seismic demand spectra in an acceleration-displacement response spectra (ADRS) format for the original and retrofitted 8-, 4- and 2-storey frames. The original 8-storey frame cannot satisfy the performance level required by the design earthquake and it has no performance point in relation to the design earthquake demand. However, the FRP retrofitted 8-storey frame has a performance point with coordinates (12.27, 0.126 g). Both the FRP retrofitted and the steel braced 2-storey frames have performance points with coordinates (3.79, 0.38 g) and (1.00, 0.70 g), respectively, while the 4-storey X-braced frame has performance point coordinates of (0.92, 0.66 g) (see Table 2). Although no beam hinging occurred in the low-rise building frames after FRP retrofitting, the plastic hinging improved in the 4-storey building, which was reclassified from Collapse (C) to the acceptance criteria of Immediate Occupancy (IO). This trend was also observed for a 2-story frame.

The roof displacement at the performance point for the FRP retrofitted 8-storey frame is 156.9mm (Figure 3). According to FEMA 356 (2000) the limitation of displacement for the life safety (LS) performance level of reinforced concrete frames is 1% to 2% of the height of the frame (120.2mm to 240.3mm). Therefore, the retrofitted frames meet the performance objectives of life safety. According to these results, 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 2800 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, the FRP retrofitting of the joints upgraded the frame to satisfy the LS performance level of Standard 2800.

Table 2 presents the performance points of the frames. According to this table, the FRP-retrofitting of the 4-storey frame failed to upgrade the frame to satisfy the life-safety performance demand of the selected Standard 2800 earthquake, indicating insufficient thickness for the 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.

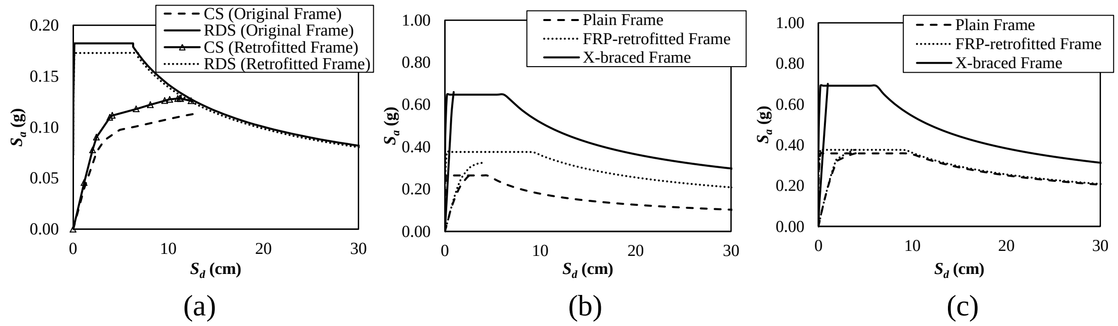


Figure 3. Performance point of the original and retrofitted (a) 8-storey (CS=Capacity Spectrum, RDS=Reduced Demand Spectrum), (b) 4-storey and (c) and 2-storey frames (Hadigeh et al., 2015).

Table 2. Performance points of the original and retrofitted frames

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

4 **SEISMIC RESILIENCE- REFERENCE CASE STUDY: HOSPITAL BUILDING** (CIMERALLO ET AL., 2010)

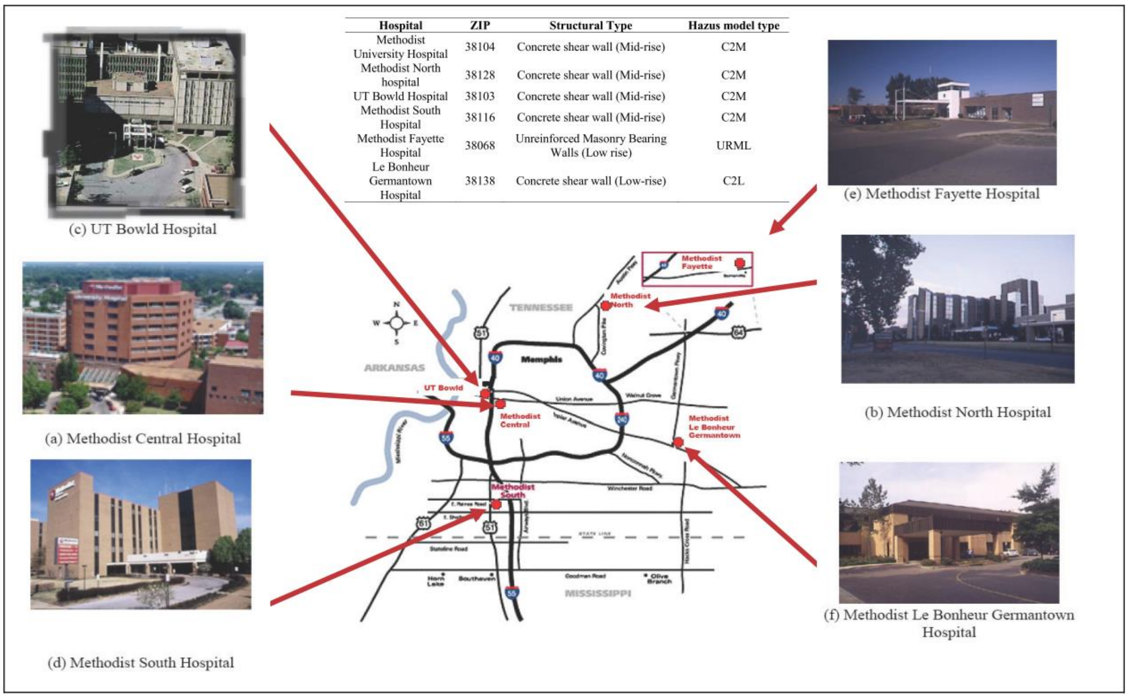
To illustrate the proposed framework equation (Eq. 1) a complex of six hospitals located in Memphis, Tennessee has been selected (Figure 4). It consists of a regional loss estimation study aimed at the estimation of economic losses of several buildings within a geographical region like a city. Figure 4 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.

Four seismic rehabilitation alternative schemes are considered for each structural type as per FEMA 276: 1) no action; 2) rehabilitation to LS level; 3) rehabilitation to IO level; 4) construction of a new building which are the target performance levels for rehabilitation against an earthquake.

Fragility curves for each rehabilitation alternative, as per defined in FEMA 276, are obtained directly correlating to the HAZUS code levels. Therefore, the HAZUS code levels are assigned to the rehabilitation levels mentioned above with reasonable assumptions. For example, the "No Action" option, corresponds to the low code level. Fragility curves are developed for structural damage and non-structural damage of drift sensitive and accelerations sensitive components using the HAZUS approach. Figure 5 shows fragility curves of structural damage for concrete shear walls mid-rise building type (C2M) versus return period.

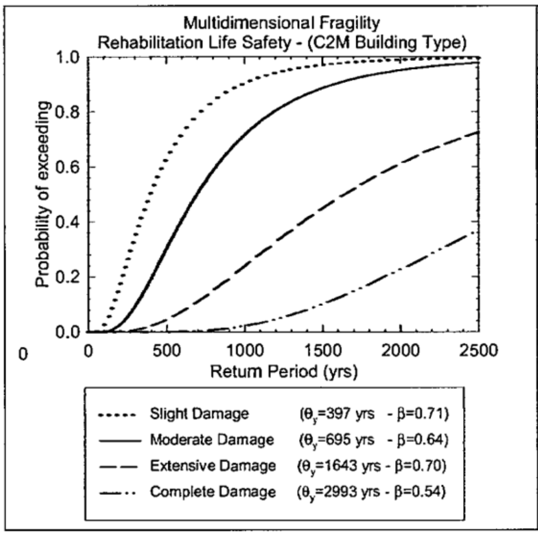
The time control period T_{LC} for a decision analysis is chosen based on the decision maker's interest. Generally, the longer time period of the building the better justification for system rehabilitation. On the other hand a decision maker may prefers to retrofit structures when the rehabilitation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

A comparison of structural damage distributions for C2M type structures for two time control periods T_{LC}=30 yrs and T_{LC} =50 yrs is shown in Figure 6. As expected the probability of having no damage increases for the shorter time periods.



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

Figure 4. System definition (Cimerallo et al., 2010)



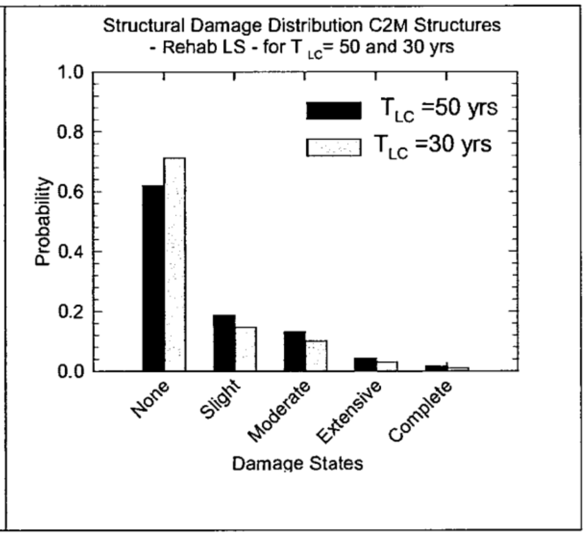


Figure 5. Multidimensional fragility curves for C2M type structure -Rehabilitation to Life Safety (Cimerallo et al., 2010)

Figure 6. Structural damage distribution for different rehabilitation strategies (T,c=30 yrs) for C2M type structure - Rehabilitation to Life Safety (Cimerallo et al., 2010)

In order to avoid the seismic record the input value was normalized using the four different hazard levels. After normlization the value were determined for different rehabilitation strategies (Figure 7). In order to improve the disaster resilience of the hospital system, four different seismic retrofit schemes were considered for this reference case study: (a) Moment resisting frames (MRF); (b) Buckling restrained braces (UB); (c) Shear walls (SW) and (d) Weakening and Damping (Cimerallo et al., 2010).

The disaster resilience value is calculated according to Eq. (1). 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 Figure 7. 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 investments and resilience are not linearly related

When the functionality Q (t) is very high for it to be improved by a small amount it is required to invest a very large amount compared with the case when the function Q (t) of the system is low. Although this is obvious the procedure presented herein can be used by decision makers.

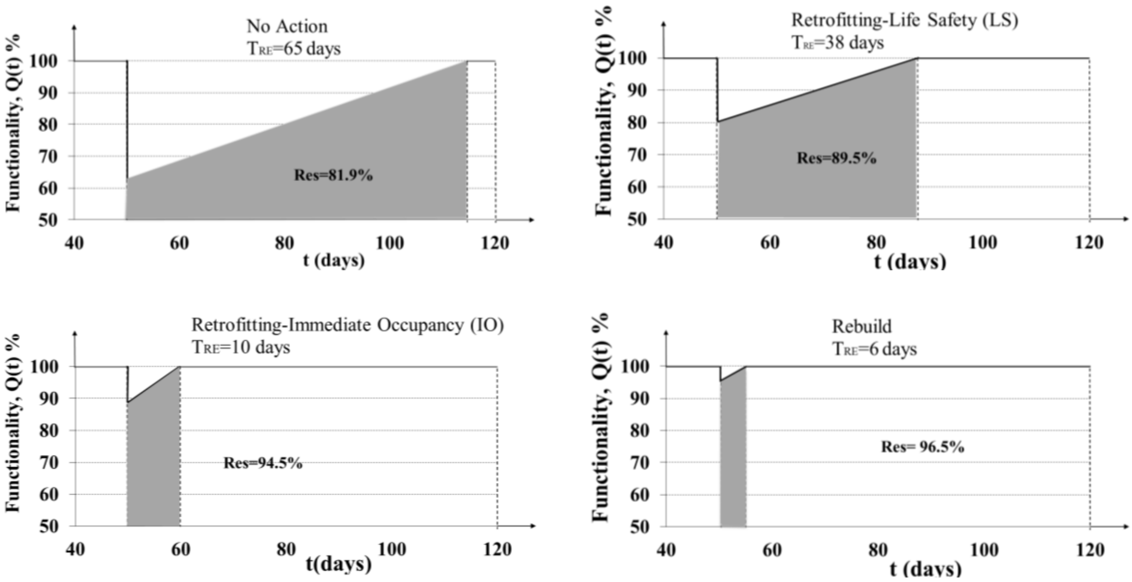


Figure 7. Resilience for different retrofitting strategies, adapted from Cimellaro (2008).

5 **ENHANCEMENT OF SEISMIC RESILIENCE OF RETROFITTED OMRFS** (THIS STUDY)

Table 3 shows the values of resilience for the two different retrofit techniques (FRP and X-braces) and for different low-to-medium-rise RC frames (see Figure 2).

This 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.

Although in term of resilience the difference seems small, the loss term of ductility (complementary to resilience) clearly shows the advantage of the FRP scheme. This retrofit technique produces a reduction of displacements and maintains the ductility, regardless of the number of stories. The maintenance of ductility is important for RC OMRFs) as it is not desirable in the seismic performance of existing frames with OMRFs.

The steel bracing considerably enhanced 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.

Table 3 Recovery time and resilience of RC frames for rehabilitation strategies (T_{LC} = 65 days).

Rehabilitation Alternatives	Type of frame / Performance level	2-storey	4-storey	8-storey	Recovery Time T_{RE}, [days]	Resilience Res [%]
Plain	No Action	●	●	●	65	81.9
	Life Safety (LS)	NA	NA	NA	-	-
	Immediate Occupancy (IO)	NA	NA	NA	-	-
	Rebuild	●	●	●	6	96.5
FRP-Retrofitted	Life Safety (LS)	-	-	●	38	89.5
	Immediate Occupancy (IO)	●	●	-	10	94.5
X-braced steel	Life Safety (LS)	●	●	●	38	89.5
	Immediate Occupancy (IO)	-	-	-	-	-

6 CONCLUSION

Seismic resilience defines information from the technical fields of earthquake engineering, social science and economics. Resilience-based design integrates information from these different fields into a unique function leading to results that are unbiased by uninformed intuitions or preconceived notions of risk. This paper aims to provide a quantitative definition of seismic resilience versus the more conventional DBDM in OMRFs retrofitted with FRPs or steel bracing techniques. In this rational way an analytical function is used that may fit both technical and organizational issues. A regional complex of six hospitals built in low to medium-rise concrete frames has been used as a reference to illustrate the applicability of the framework and to assess the seismic resilience of the selected retrofitted OMRFS versus the DBDM assessment. It is shown that FRP retrofitting improves resilience of 15% from 89.5% to 94.5%. However, it should be mentioned that the assumptions made herein are only representative for the cases presented.

It is shown that FRP retrofit is more effective in terms of improving performance and ductility in low-rise RC buildings as the measured resilience shows an enhanced value compared to the un-retrofitted structure.

REFERENCES:

ABA. 2005. Iranian Concrete Code. *Iran Management and Planning Organization.*

AS1170.4.1993. Structural design Actions Part 4: Earthquake Actions in Australia. *Standards Australia, Sydney, Australia.*

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

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

Biondini. F., Camnasio. E. and Titi. A. 2015. Seismic resilience of concrete structures under corrosion. *Earthquake Engng Struct. Dyn. Published online in Wiley Online Library (wileyonlinelibrary.com).* DOI: 10.1002/eqe.2591.

Chang. S. and Shinozuka M. 2004. Measuring Improvements in the disaster Resilience of Communities. *EER/ Spectra Journal*. 20, (3). 739-755.

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.

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, A volume in Woodhead Publishing Series in Civil and Structural Engineering Edited by: S. Tesfamariam and K. Goda*, ISBN: 978-0-85709-268-7.

Cimerallo. G.P., Reinhorn. A.M. and Bruneau. M. 2010. Framework for analytical quantification of disaster resilience. *Engineering Structures* 32. 3639–3649.

Ghobarah. A. 2001. Performance-based design in earthquake engineering: state of development. *Engineering Structures*. 23. 878–884.

Hadigheh. S. A., Mahini. S.S. and Maheri. M.R. 2014. Seismic Behaviour of FRP-Retrofitted Reinforced Concrete Frames. *Journal of Earthquake Engineering*, 18. 1171–1197. (ISSN: 1363-2469 print / 1559-808X online. DOI: 10.1080/13632469.2014.926301).

Lumantarna. E., Lam. N., Wilson. J. and Griffith. M. 2010. Inelastic displacement demand of strength-degraded structures. *Journal of Earthquake Engineering*. 14. 487–511.

Niroomandi. A., Maheri. A., Maheri. M. R. and Mahini. S. S. 2010. Seismic performance of ordinary RC frames retrofitted at joints by FRP sheets. *Engineering Structures*. 32(8). 2326-36.

Mahini. S.S. 2015. Damage and Seismic Performance Assessment of FRP-Retrofitted Multi-Storey RC Buildings. *Electronic Journal of Structural Engineering*. 14(1). 49-61.

Mahini. S.S. 2005. Rehabilitation of Exterior RC Beam-Column Joints using CFRP Sheets. *PhD thesis submitted to the Division of Civil Engineering of the University of Queensland*, Australia.

Mahini. S. S. and Ronagh. H. R. 2010. Strength and ductility of FRP web-bonded RC beams for the assessment of retrofitted beam–column joints. *Composite Structures.* 92(6). 1325-32.

Mahini. S. S. and Ronagh. H. R. 2011. Web-bonded FRPs for relocation of plastic hinges away from the column face in exterior RC joints. *Composite Structures*. 93. 2460–2472.

Standard No. 2800. 2005. *Iranian code of practice for seismic resistant design of buildings.* Third edition, Building and Housing Research Centre. Tehran. Iran.