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Full Length Article

On the quantification of collapse margin of a retrofitted university building in Beirut using a probabilistic approach

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ABSTRACT

The scope of this study is to investigate the feasibility and performance of several retrofitting techniques on an existing building in Beirut Arab University (BAU). The implemented retrofitting techniques were adding RC shear walls (SW) and steel bracing systems. Simulation and analysis procedures were performed in a nonlinear platform. Models are designed based on ACI 318-14 and ANSI/AISC 360-10 for concrete and steel, respectively. Non-linear time history analysis (NL-THA), non-linear static analysis (NL-SA) and collapse margin ratio are carried out to evaluate the performance of existing and retrofitted structures. Incremental dynamic analysis (IDA) curves are then generated and used to develop the seismic fragility curves. Three different strong ground motions are used in the analyses by referring to the UBC 1997 requirement. The IDA curves are compared based on five performance levels; operational phase (OP), immediate occupancy (IO), damage control (DC), life safety (LS), and collapse prevention (CP). The fragility curves and the calculated CMRs indicated that the shear wall and steel bracing systems both provide good seismic improvement and are able to achieve strengthening solution targets for an existing building system; however, the performance of RC-SW system under seismic excitation was much better. To this, RC-SW is considered as the most appropriate technique for retrofitting the main building of Beirut Arab University.

1. Introduction

Lebanon is a Mediterranean country which is classified as a region of moderate to high seismicity and has experienced various seismic events in the past [1]. Since no two buildings are the same, the designer's main challenge is to assess and choose the best seismic retrofitting technique as well as the convenient solutions that are technically economical and socially suitable. Many approaches for seismic risk assessment exist all over the world. One of the effective approaches is the Seismic Priority Index (SPI) created by the National Research Council (NRC) of Canada which is now adopted by the Public Works and Government Services Canada (PWGSC). It is a part of a three-stage process including screening, assessment and retrofitting [2]. Thermou and Elnashai [3], studied almost all types of retrofitting and strengthening techniques that lead to a minimum seismic vulnerability. Their study confirmed the complexity of the selection process and level of intervention in any retrofitting project, where many factors come into consideration. Psychology, aesthetic, cost, importance, duration of work, disruption of use, compatibility with existing structural system, and sufficient capacity of foundation system are among the issues that dictate the choice of rehabilitation system.

Cheung et al. [4], presented a general overview of the innovative alternatives available in the field of seismic retrofitting. From their roles in the public works and government services in Canada, authors demonstrated the success and efficiency of new technologies such as; passive damping devices and advanced composite materials in Harry Steven Building and Port Alberini Federal Building in Canada. Besides, the innovation techniques when used in the seismic retrofitting may do not require heavy demolition, but they are costly to be used [5].

Researchers divided the intervention retrofitting methods into two main categories: Local and Global. In the local approach of retrofitting, many techniques could be applied such as the crack injection, shotcrete, steel plate adhesion, steel jacketing and the FRP external bonding. The local intervention targeted to increase the deformation capacity of poor structural and non-structural components so that these components will resist the imposed lateral forces without reaching their limit state. However, in the global

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approach of retrofitting, the structure is in fact retrofitted as a whole by adding lateral resisting elements such as; a) shear wall, b) bracing and c) introducing supplemental damping and base isolation [6]. Many researchers in the field of evaluating seismic improvements of RC-frame building and steel frame building utilized the non-linear analysis. Özel and Güneyisi [7], selected a case study to evaluate the seismic reliability of reinforced concrete building retrofitted by eccentric steel braces. Viswanath et al. [8], utilized the X- concentric steel bracing structural system that contributes to the structural stiffness of the four-storey RC building, as well it is applied on six steel frame structure by Xiong et al. [9] by proposing a performance-based plastic design method to achieve the target drift and the yield mechanism.

Faghihmaleki et al. [10], studied the seismic improvement of steel moment frame building by selecting three different structural improvement techniques. Silva et al. [11], proved experimentally that the concrete-filled steel tube columns made with rubberized concrete as a retrofitting techniques enhanced and achieved high seismic performance in comparison to steel frames only. Navaratnarajah Sathiparan [12], showed that the PP-band (Polypropylene band) as a retrofitted technique on the masonry structure has the capability to improve the seismic behavior with respect to drift, shear resistance and ductility, similarly via supplying GFRP jacketing technique which provided a satisfactory results in repairing masonry structures in terms of mechanical parameters [13].

Abou-Elfath et al. [14], studied the effect of shear walls and steel bracing when these elements are located at different locations in the building. Moazam et al. [15], proposed to estimate the seismic capacity of two old concrete arch bridges in Iran. The analyses were investigated by using non-linear dynamic analysis by simulating 22 far-field earthquake records. Azizan et al. [16], suggested studying the seismic performance of the Koyna dam which was classified as a concrete gravity dam. The assessment was performed by generating the incremental dynamic analysis under single and repeated earthquake excitations (seven ground motions) to identify the limit state of the dam. Azizi et al. [17], analyzed the vulnerability of the URM buildings in Barcelona using IDA curves. Sobhan et al. [18], investigated the dynamic buckling behavior of a steel cylindrical tank using nonlinear static pushover analysis (NSPO) and then compared with incremental dynamic analysis (IDA) to assess the accuracy of the result. Fanaie et al. [19], studied the dynamic properties of the concentric steel bracing system using incremental dynamic analysis via OpenSees software. Dong et al. [20], studied the seismic performance of moment resisting frame system supplied with viscous dampers by utilizing a set of nonlinear time history records. In addition to the aforementioned researchers in the nonlinear analysis, the fragility curves assessment approaches are also commonly used to assess the vulnerability of the structures that are exposed to earthquakes. Mai et al. [21], employed the lognormal shape or the fragility assessment for a three storey steel frame using a large number of synthetic ground motions to found that the accuracy of the curves depends on ground motion intensity. Yang and Li [22], analyzed the seismic resilient of Buckling restrained knee braced frame using incremental dynamic analysis and fragility curves, and they confirmed that the innovation of BRKBF has excellent seismic performance under different earthquake shaking intensities. Saruddin and Nazri [23], studied the development of fragility curves for the two prototype models in Malaysia which are concrete and steel moment resisting frames systems. Ahmadi et al. [24], developed seismic fragility curves for three-, six- and nine-storey of rectangular concrete filled steel tube (RCFT) structures to assess the risk posed by using composite members.

In order to assess the enhancement of structural performance using different seismic retrofitting techniques, it needs the performance-based seismic design (PBSD) requirements. The structural damage has been quantified in some guidelines provided by the Federal Emergency Management Agency (FEMA 273 & 356) considering various earthquake levels [25]. The level of damage should be defined by an engineering demand parameter (EDP), which usually the maximum Interstorey drift ratio (IDR) will be considered as an appropriate indicator. In the study performed by Xue et al., the max IDR values of 0.005, 0.010, 0.015, 0.020 and 0.025 were proposed for various performance levels of OP, IO, DC, LS and CP respectively. By contrast, other authors such as Uma et al. [27], have proposed different performance levels.

Hence, this study aims to assess the seismic improvement of an existing main building of the Beirut Arab University, (BAU) in Lebanon. The case study is a six-storey reinforced concrete structure which was designed in the late 1950s in Egypt as a school complex and was fully executed in 1958. The design of the building is with the absence of any seismic code design requirements. Two interventional retrofitting techniques are utilized: adding shear walls and steel bracing structural elements at the peripheral façade of the original building. Then, Incremental dynamic analysis (IDA) is performed using three ground motions of magnitude range between 6 and 7 Richter scale. After that, fragility curves shall be developed in the later stage considering the five performance limit states as suggested by Xue et al. [26], (OP), (IO), (DC), (LS) and (CP) at values 0.5%, 1%, 1.5%, 2% and 2.5% interstorey drift ratios, respectively for the assessment of the building before and after retrofitting.

2. Utilized methodology

In this work, the safety margin that is related to the collapse resistance of the structures under seismic ground motions is investigated. The incremental dynamic analysis (IDA) and pushover analysis (POA) procedures were adopted to assess the structural vulnerability of the existing building before and after intervention the retrofitting techniques to the structure, which are associated as a key tool to determine the safety margin of the structure. In addition to that, fragility curves as a probabilistic approach are developed according to five performance levels: operational performance (OP, %interstorey drift ratio = 0.5%), immediate occupancy (IO, %interstorey drift ratio = 1%), damage control (DC, %interstorey drift ratio = 1.5%), life safety (LS, %interstorey drift ratio = 2%) and collapse prevention (CP, %interstorey drift ratio = 2.5%). Eventually, a collapse margin ratio (CMR) is calculated in this study based on the fragility search method obtained from the IDA that is proposed as a new and efficient seismic indicator by referring to FEMA-P-695 process [28]. The flow chart in Fig. 1 describes and summarizes the methodology work for the case study.

2.1. Case study

2.1.1. Existing building (Model 1: Reference Model)

The main building of BAU is a six-storey Reinforced Concrete structure with an interstorey height equal to 3.5 m and with total building height equal to 21 m. The columns in this building are of 120×30 cm, 60×60 cm, and 30×60 cm dimensions range, with a ribbed slab of thickness 30 cm, having different drops and embedded beam dimensions. The compressive strength of concrete is 20 MPa while yielding stress of the reinforcing steel is 260 MPa.

A 3D model of the existing building is created to carry out the structural analysis. The building is analyzed under the effect of gravity loads of (5 kN/m2) as dead load, and (5 kN/m2) as live load, in addition to the seismic loadings. The outcomes of the assessment are used as control values to compare later with the modified retrofitted structural models. The reference model before retrofit-

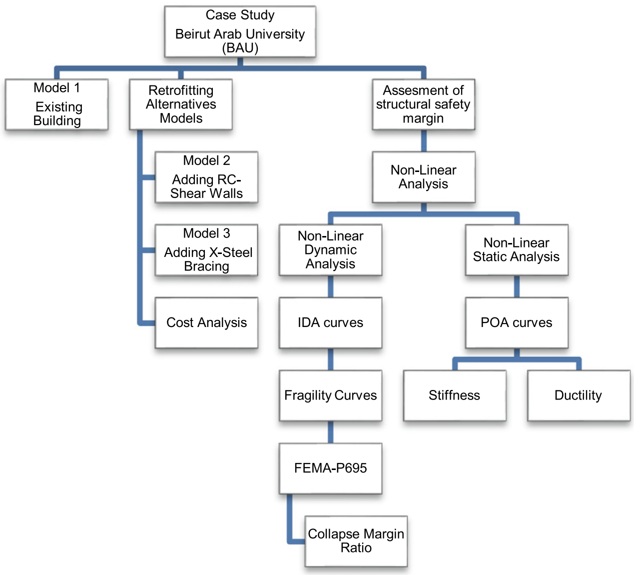

Fig. 1. Flow chart methodology.

ting is shown in Fig. 2, and the details of structural components are shown in Table 1.

2.2. Retrofitting alternatives structural models

2.2.1. The first alternative (Model 2): Adding RC shear walls

As for the first proposed retrofitting scheme, one of the most commonly used technique is to provide additional elements with high lateral stiffness to reduce the structural responses and building vibration. Among these, adding RC shear walls can be determined as one of the best options. The most convenient way to implement new shear walls is by fully filling up the selected bays

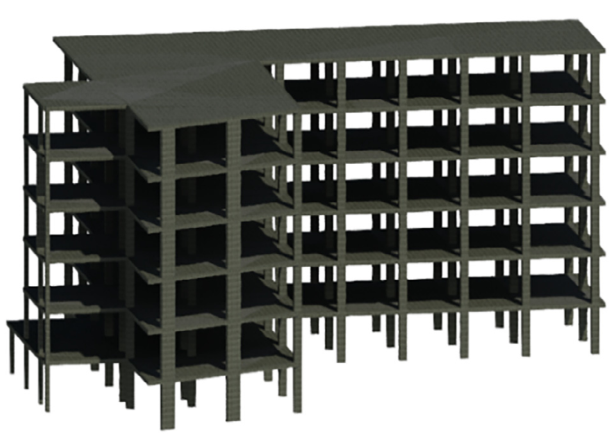

Fig. 2. Existing building before retrofitting, Model.

Table 1
Dimension and reinforcement design for beams and columns in BAU, case study.

Case Study	Reinforcement detailing of column and beam		
Column	60 cm × 60 cm	60 cm × 30 cm	120 cm × 30 cm
Main rebar	18 T 16	12 T 16	18 T 16
Shear link	T10 @ 300	T10 @ 300	T10 @ 300
Beam	Drop Beam: 25 cm × 60 cm		
	Top rebar: 2T12		Bottom rebar: 3T14

of the structure on the peripheral façade of the original building, as shown in Fig. 3. Similarly, to the previous step, a 3D model subjected to gravity and seismic loading is generated using FEA modeling software. The outcomes of the analysis are utilized and interpreted to get the most important parameters such as; inter-storey drifts and probability of damages. The thickness of the additional shear walls is equal to 25 cm. Concrete compressive strength is 30 MPa, while the yielding stress of reinforcing steel is 500 MPa In addition, the nonlinear modeling of the shear wall as a shell element is considered by assigning wall hinges (P-M) with confined boundary elements of 0.25Lw with a reinforcement ratio equals to 0.3%. Similarly, for the frame elements (Beam and Column) were modeled to have concentrated plastic hinges at the column and beam faces, where the beams have only M3 moment hinges, and the columns have an axial and biaxial moment (PMM) hinges

2.2.2. The second alternative (Model 3): Adding steel bracing

The second proposed retrofitting scheme to be assessed is to apply steel bracing. Steel bracing is usually provided to the peripheral bays and it is placed in the same position as the first alternative technique. Again, to assess its validity, a 3D model is created using FE modeling software. The same parameters used in the first alternative strengthening method are obtained for the second alternative method, as shown in Fig. 4. The steel bracing used are concentric hollow circular pipe (25 cm × 1 cm) diameter 25 cm and thickness 1 cm, of X-bracing type.

2.3. Non-linear dynamic analysis (NL-DA)

Incremental dynamic analysis (IDA) is applied to investigate the expected structural response, deteriorations, and financial losses under earthquakes with different intensities. The non-linear time history analysis (NL-THA) gives more realistic results about the performance of a particular type of structure under seismic excita-

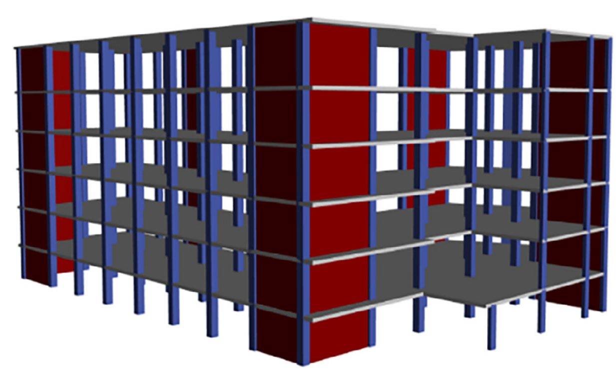

Fig. 3. Retrofitting by adding shear walls, Model 2.

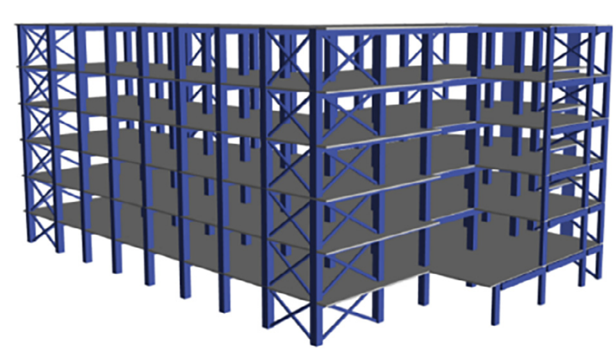

Fig. 4. Retrofitting by adding steel bracing, Model 3.

Table 2
Seismic ground motion details.

No.	Event	Station	Year	Magnitude	PGA (g)
1	Imperial Valley-02	ElCentroArray#9	1940	6.95	0.281 g
2	Kobe, Japan	Shin-Osaka	1995	6.90	0.233 g
3	Duzce, Turkey	Lamont 1059	1999	7.14	0.136 g

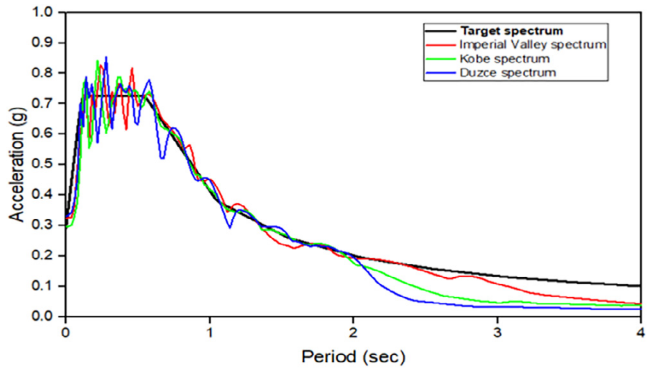

Fig. 5. Scaling ground motions with the target response spectrum.

tions. According to Nazri [29], the most commonly used parameter is the peak ground acceleration (PGA). The IDA curves can be developed based on the relationship between interstorey drift ratio (IDR), and the intensity of ground motion (PGA). As proposed by several seismic codes that recommend a minimum of three or seven sets of ground motions (ATC, 1996; UBC, 1997; NEHRP, 2005). Thus, in this paper, three sets of strong ground motions for each model were used which have been selected from the Pacific Earthquake Engineering Research Centre (PEER) NGA website. The ground motions details are given in Table 2. After that, using SiesmoMatch software the selected ground motion records were scaled according to the target response spectrum in order to match the characteristic with the soil type as shown in Fig. 5.

2.4. Fragility curve

Fragility curves are used analytically to estimate the risk of the seismic effect on the structural models, which are considered as useful tools to predict the probability of damage to any structural systems. The fragility curves can be used as a method in retrofitting decisions. In order to measure the performance of the proposed structure against the lateral loads, the drifts will be used to observe the critical damages that will lead to structural collapse. Then, the %drift can be calculated by dividing maximum roof displacement with the total height of the building (21 m), as shown in Eq. (1).

$$\%\ \text{Drift} = \frac{\text{Roof displacement}}{\text{Building height}} \times 100 \tag{1}$$

Numerous seismic parameters are responsible to develop fragility curves since PGA parameter was used in the incremental dynamic analysis (IDA), it also used in developing vulnerability curves. The performance levels that specify the damage state of the three models are: OP, IO, DC, LS, and CP with vertical gridline at % drift values 0.5%, 1%,1.5%, 2%, and 2.5% respectively as mentioned in Xue et al. [26]. Then two main parameters are needed to develop the fragility curves, mean (μ) and standard deviation (σ). Many equations were used to develop fragility curves, however, the Equation below has already been used by Ibrahim, El-Shami [30].

$$P[D/PGA] = \Phi\left(\frac{\ln(PGA) - \mu}{\sigma}\right) \tag{2}$$

where: Φ is the standard normal cumulative distribution function, μ and σ are the mean value and standard deviation of logarithm PGA, and D is the damage state.

2.5. Non-linear static analysis (NL-SA)

In the earthquake engineering field, the non-linear static analysis method gets to be profoundly requested in assessing the structural performance and its behavior when subjected to a serious seismic tremor. Because of its simplicity, it becomes an active engineering tool for estimating the structural safety against earthquake struck-induced collapse. The non-linear static analysis refers to the pushover analysis that is a well-known curve identified as "Capacity Curve". The aim of this method is to create the capacity curve in order to decide the stiffness, and ductility of the reference and retrofitted models. This tool predicts the base shear forces and deformations for investigating the lateral seismic response behavior of a current or newly designed structure, as it is a beneficial tool in the field of retrofitting estimation based on the performance seismic design as presented in the seismic regulation and guidelines of FEMA-356. The following equations are provided to compute the stiffness and the ductility of the structures.

$$K,\ \text{Stiffness} = \frac{\text{Yielding Force, Vy}}{\text{Yielding Displacement},\ \Delta y} \tag{3}$$

$$\mu,\ \text{Ductility} = \frac{\text{Ultimate Displacement},\ \text{Du}}{\text{Yielding Displacement},\ \text{Dy}} \tag{4}$$

2.6. Collapse margin ratio (CMR)

One of the best collapse indicators which has been developed in the last decade is the CMR, initially proposed in FEMA P695. This indicator will characterize the collapse safety of the structure by integrating the median spectral acceleration and MCE spectral acceleration in the fundamental period of the structure related to the site classifications in Lebanon- Beirut as shown in Table 3. The median collapse intensity is defined when half of the structure has the form of life-threatening collapse, or the probability of damages reaches ($P_{collapse}$ = 50%). Determining the collapse margin ratio can be significantly influenced due to several uncertainties such as: (1) record to record uncertainty related to the variation in the frequency of the seismic records (RTR), (2) design requirement uncertainty (DR), (3) test data uncertainty (TD), and modeling uncertainty (MD), which required to adjust the collapse margin ratio (ACMR) by multiplying with a spectral shape factor (SSF) that's if function of ductility, μ and fundamental period, T of the structure.

In this study, the peak ground acceleration is used as an earthquake intensity measurement instead of spectral acceleration due to its simplicity and it is the most commonly used intensity. With the use of series seismic ground motion records, and from the collapse data obtained from IDA and fragility results the (R_{CM}) can be calculated. In Fig. 6, Ic is the earthquake intensity corresponding to the 50% probability of structural collapse, and I_{MCE} used to be 0.25 g as the MCE intensity in PGA.

Table 3
Summary mapped values of Lebanon zone in seismic design parameters.

Seismic Design Category	Ca	Cv	Ss	S1	Fv	Fa	Maximum Considered Earthquake Intensity I_{MCE}
C	0.29	0.40	1.20	0.40	1.40	1.0	**0.25 g**

*Ca: acceleration coefficient, Cv: velocity acceleration, Ss: Spectral response acceleration (short period), S1: Spectral response acceleration (1-second), Fa: Site coefficient taking into account Ss parameter, and Fv: Site coefficient taking into account S1 parameter.

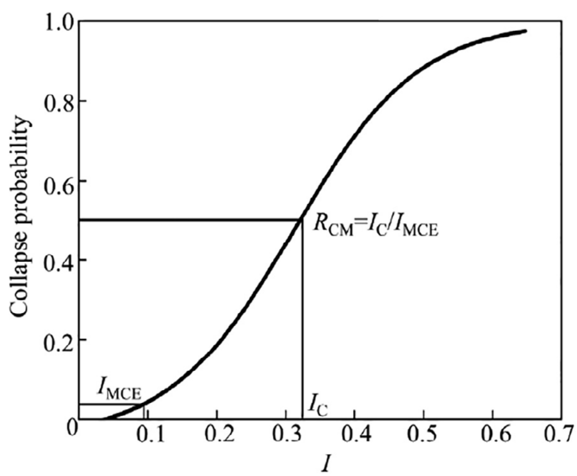

Fig. 6. Collapse fragility curve of a structure.

3. Result and discussion

3.1. Incremental dynamic analysis (IDA)

The IDA curves are developed for three different structural models taking into consideration the original case and the two alternatives retrofitting techniques as discussed previously. The Intensity measure (PGA) of the scaled time history ground motions and the damage measure (Interstorey drift %), were plotted in IDA curves to provide an overview of the seismic performance of structures subjected to earthquakes until the collapse point reached as shown in Fig. 7. Based on each ground motion (GM1, GM2, and GM3), the non-linear time history analysis (NL-THA) is performed using a NL software. The intensity measure based on peak ground acceleration (PGA) increasingly scaled by 0.1 g until it reaches 1.3 g. When PGA reaches 1.3 g the analysis stopped due to the dynamic instability of the structures. To assess the structural performance before and after retrofitting, five performance levels were used. The vertical gridlines at drifts of 0.5%,1%,1.5%, 2%, and 2.5% respectively represent OP, IO, DC, LS, and CP. In addition to these performance levels, two cases of ground motion intensities are considered to be used in the assessment. Case 1: Weak ground motion intensity of 0.2 g PGA and Case 2: Strong ground motion intensity of 1.0 g.

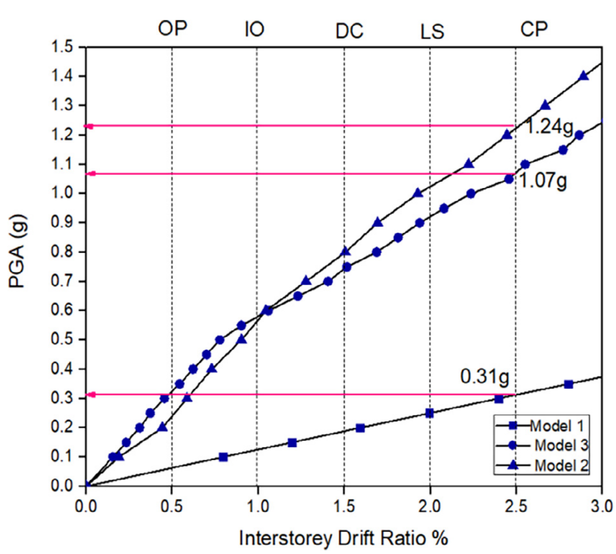

Fig. 7. IDA curve of the structure.

For earthquakes with low intensity with PGA of 0.2 g, which are rated as relatively weak ground motions, the ex30303isting structure (Model 1) experienced interstorey drift ratio equals to 1.6% which exceeded the damage control performance limit 1.5%, and near to the collapse limit margin. This means that the structure will suffer significant damage in its structural components if it is exposed to a high-intensity PGA. However, in Model 2 and in case of adding shear walls as a retrofitting technique, the value of interstorey drift ratio equals to 0.31% which is less than 0.5% of the operational phase. In this case, the structure is maintained in the operational performance level, which means no structural damage is observed and the structure is in continuous service with negligible structural damage. Similar to the first retrofitting technique, but instead of adding shear walls to the existing building, the steel bracing is used in Model 3 as a second alternative. It is observed that the recorded value of interstorey drift ratio equals to 0.44% which is also within the operational performance range when subjected to the same intensity of 0.2 g.

Based on the mean results, the interstorey drift value in Model 1 will be reduced when the shear walls and steel bracing are added to the existing structure by 81% and 73% respectively.

For the category of high intensity and stronger earthquakes with PGA of 1.0 g, the value of interstorey drift ratio of Model 1 totally exceeded 2.5% drift limit of the collapse prevention performance level. Thus, it is anticipated that the structure will suffer from substantial structural and non-structural damages. However, in the two retrofitting techniques of Model 2 and Model 3, the values of the interstorey drift ratios are 1.9% and 2.2% respectively. This means that the retrofitting techniques and their structural elements performed satisfactorily under high-intensity ground motions and still remain in the range of life safety zone. Meanwhile, in order to reach the CP level, Model 2 and Model 3 requires 1.24 g and 1.07 g respectively, whereas Model 1 requires 0.31 g. Among this result, Model 2 has the best performance, and the mean difference between Model 2 and Model 1 is 75%, whereas 14% with Model 3.

Therefore, based on the results obtained, it can be deduced that the installation of RC shear walls and steel bracing as additional structural elements provides structural enhancement and improvement. While the effect of retrofitting using shear walls has been more evident. Moreover, the analyses demonstrated that with the increment of PGA up to 1.0 g, the strengthening of the building achieved by control the interstorey drift ratios.

3.2. Fragility curves

The fragility estimations of simulated models are depicted in Figs. 8–10. The results indicated that under weak ground motions (PGA = 0.2 g), the possibilities of reaching IO performance level are 100%, 0%, and 0% for Model 1, Model 2, and Model 3 respectively. By considering the damage control (DC) performance level, the probability of reaching or exceeding this performance level is

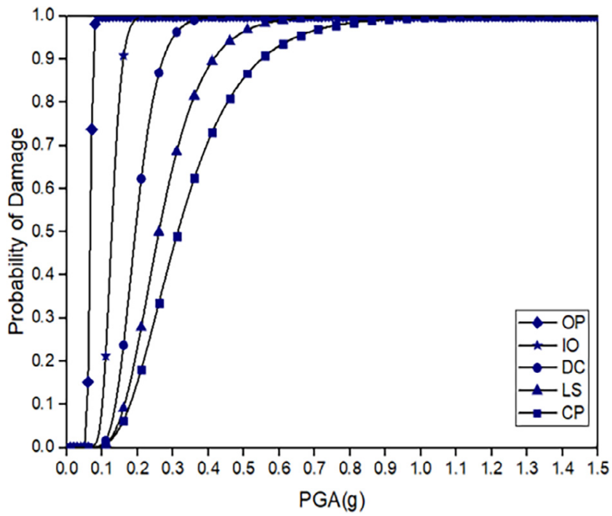

Fig. 8. Existing building fragility curves.

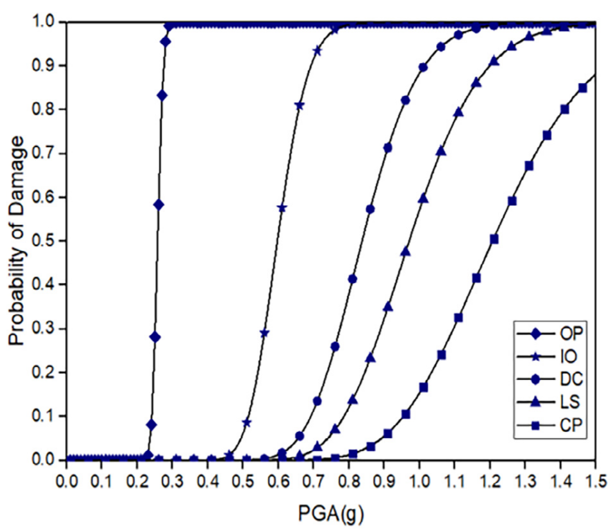

Fig. 9. Existing building fragility curves.

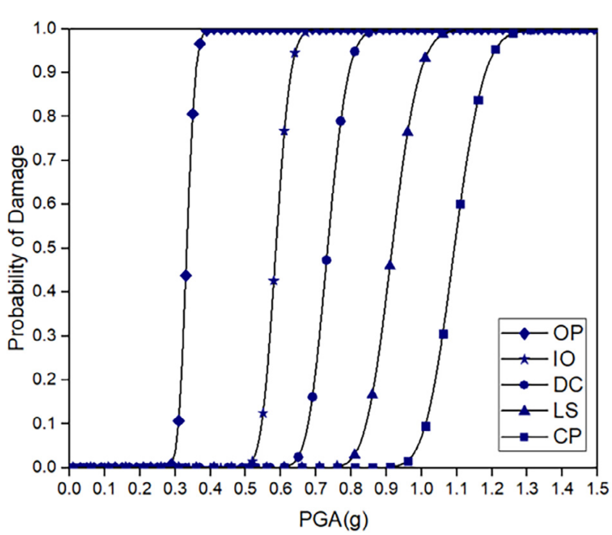

Fig. 10. Existing building fragility curves.

55% for the existing building Model 1 before retrofitting, while for Model 2 and Model 3 after retrofitting with shear walls and steel bracing, the probability of exceeding the (DC) performance level is still found to be null, 0%, and 0% respectively.

When the structural system is exposed to strong ground motion with PGA = 1.0 g, the probability of reaching or exceeding the immediate occupancy level is 100%, 100%, and 100% for the Model 1, Model 2, and Model 3 respectively. Similarly, by taking into account the damage control (DC) performance level, the probability of reaching or exceeding the DC stage is approximately 100% for the existing building of Model 1, while for Model 2 and Model 3 it is found to be 88%, and 100% respectively. In addition, under the high-intensity ground motions with PGA = 1.0 g, the probability of reaching or exceeding the collapse prevention (CP) level for the shear walls and steel bracing retrofitting techniques are 7% and 15%, respectively. These observations are opposite to the behavior of the existing building, where the building experience a total collapse because the probability of damage has approximately reached its maximum value of 100%.

The result shows that the structural and non-structural elements of Model 1 are suffering from extensive damages. Thus, it can be concluded that both retrofitting techniques with shear walls and steel bracing systems, show improvement over the existing building. Additionally, these retrofitting techniques provide good enhancement in seismic performance. In fact, adding shear walls to the main building could provide the best enhancement in different performance levels.

3.3. Pushover analysis (POA)

From the pushover curve illustrated in Fig. 11, the reference building system has the lowest stiffness value in the elastic phase among the three analysis models, which is 58110kN/m. The bracing system model has a higher stiffness value of 140048kN/m which is approximately 2.5 times higher than the original building, while the shear wall system model offers the highest stiffness value of 513661kN/m, which is 4 times higher than the bracing system and 9 times higher than the original model.

It is noted that the bracing system enhanced the stiffness by 141% and the shear wall system enhanced the stiffness by 784%. The elastic stiffness of the three structural systems is designated by the letter K and is the slope of the elastic part of the pushover curves as shown below.

- K Existing = $Vy/\Delta y = 58110\ kN/m$
- K Bracing System = $Vy/\Delta y = 140048\ kN/m$
- K Shear Wall System = $Vy/\Delta y = 513661\ kN/m$

In terms of ductility, the shear wall system shows the best results, whereas the frame system (existing system) does not provide any significant ductility compared to that of the shear wall system. Even though the two retrofitting techniques provide an improvement and ductility enhancement, for the bracing system it is a negligible enhancement, while for the shear wall system it shows the best enhancement, which is almost 4 times the original ductility. The steel bracing provides a 27% increment of the maximum base shear force compared to the current structural model, while shear wall system provides an increase in the base shear force, approximately 124% higher than that of the original model.

The ductility capacity ratio, as a parameter μc, shows the ductility enhancement for each structural system as shown below:

- For the existing building: $\mu_1 = Du/Dy = 1.71$
- For the shear wall system: $\mu_2 = Du/Dy = 3.91$
- For the steel bracing system: $\mu_3 = Du/Dy = 1.95$

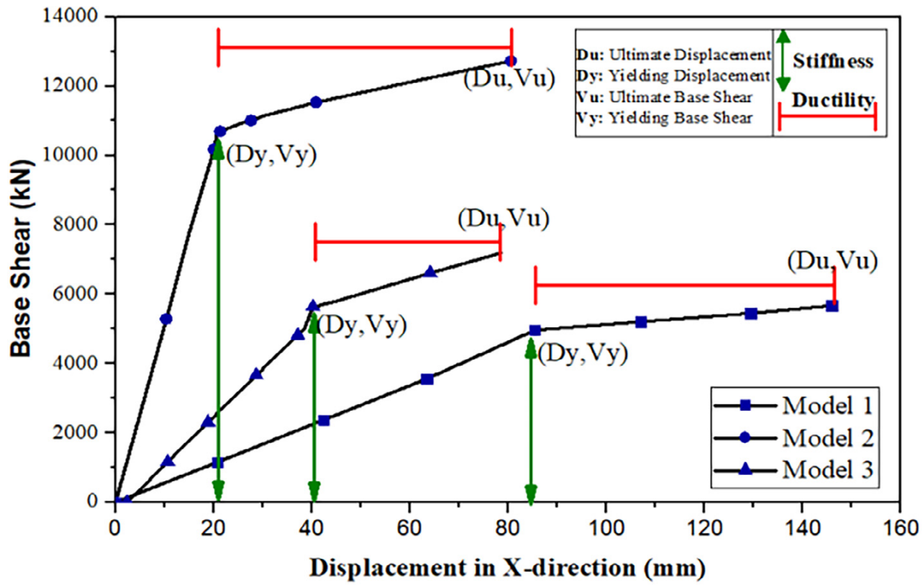

Fig. 11. Capacity curves for Model 1, Model 2, and Model 3.

Thus, from the non-linear static analysis, it is observed that the shear wall system gives the best results for all the criteria tested and investigated in this paper.

3.4. Collapse margin ratio (CMR)

The collapse fragility curves of the reference building and the two retrofitted models, with taken (PGA) as intensity measure are shown in Fig. 12. According to the collapse fragility curves, the values of CMR and ACMR could be determined as listed in Table 4. Based on the results obtained from IDA, POA and the fragility assessments, the collapse margin ratios are determined to show that the existing building has the possibility to reach fully half threatening damages is 79% at a seismic intensity (Ic = 0.31 g), PGA. However, for the alternative retrofitted models are far away from having a half significant damages by 80% for model 2 retrofitted with shear walls, and 77% for model 3 retrofitted with concentric steel bracing at seismic intensities of 1.21 g, and 1.1 g, respectively.

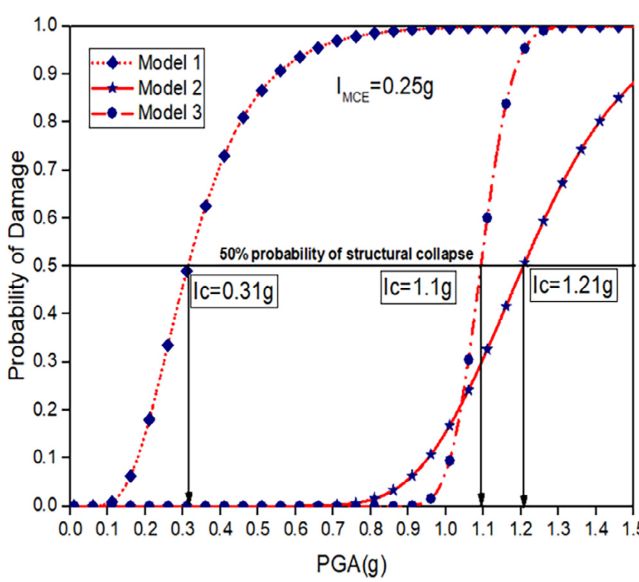

Fig. 12. Collapse fragility curves of the 3 models.

Thus, the usage of CMR as a seismic indicator can an important tool in the seismic assessment of the structures which assure the results generated from the IDA and POA analyses. Meanwhile, the analyses show that the shear wall and steel bracing systems demonstrated better building performances compared to the original structure, such that these systems can resist the targeted performance or the MCE of 0.25 g PGA based on Lebanese seismic zone.

3.5. Cost analysis

In order to choose the most suitable and feasible retrofitting technique for any execution or construction process, a comparison is made between the two proposed retrofitting techniques. Tables 5 through Table 8.

It is deduced from this study that the shear wall system is more expensive than the steel bracing system by 15%. However, it is noted that in terms of time execution of the steel bracing scheme, only the outer bays need to be worked on and installed with the steel bracings. The steel bracing scheme is also more practical than adding shear walls, whereby extensive labor and cost are needed including preparing shear wall formworks (poly-wood) or shuttering, preparing steel bars, and concrete casting before the installation of the shear walls.

In the steel bracing scheme, the bracing members need to be attached to the concrete frame at four points with an appropriate welding technique or bolting, as shown in Fig. 13. This scheme

Table 4
Collapse Margins of the three structural models.

Models	I_{MCE} (g)	I_C (g)	μ	T (sec)	SSF	CMR	ACMR
Model 1	0.25	0.31	1.71	2.031	1.13	1.24	1.401
Model 2	0.25	1.21	3.91	0.576	1.10	4.84	5.324
Model 3	0.25	1.10	1.95	1.781	1.15	4.40	5.060

Table 5
Cost Estimation for Steel bracing system material.

Cost estimation for Steel Bracing System (USD)	
Cross Sectional Area for Bracing (m^2)	0.007536
Mass (kg)	399.4683
Rate/kg (USD)	2.5
Total Number of Braces	84
Total Cost (USD)	**USD 83,888**

Table 6
Cost Estimation for Shear Wall system material.

Cost estimation for Shear Wall System concrete material (USD)	
Walls thickness (cm)	25
Walls Length (m)	36.6
Walls Height (m)	21
Rate/m^2 (USD)	100
Total Cost (USD)	**USD 76,860**

Table 7
Cost Estimation for the reinforcement of the shear wall system.

Cost estimation for Shear Wall System steel reinforcement (USD)			
Reinforcing Bars	**T16**	**T14**	**T12**
Kg/m	1.58	1.21	0.89
Rebar Length	12 m		
Number of bars	1421	90	1765
Total Mass (ton)	26.6	1	18.63
Rate/ton (USD)	USD 483		
Total Cost (USD)	**USD 22,330**		

Table 8
Total cost evaluation for the two-retrofitting system.

Total Cost Evaluation for the two-retrofitting system (USD)	
Steel Bracing	Shear Walls System
83,888	99,190

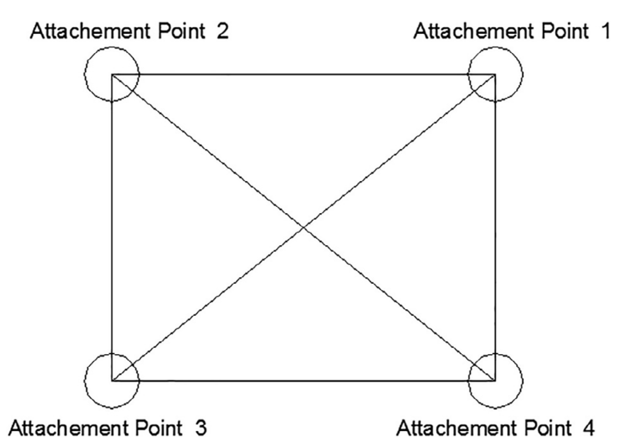

Fig. 13. Steel bracing attached to the concrete element at point 1, 2, 3, and 4.

requires less work to be done in less time with less labor to install steel bracings. In the cost estimation, only the cost of the structural members is considered, while the cost for connections, welding or bolting is not included.

Hence, even though shear wall system is costly compared to bracing system, its seismic performance is much better based on the discussions in the previous analyses and therefore, a suitable choice for seismic improvement of an existing building.

4. Conclusion

The Beirut Arab University main building has been selected as a case study. Based on the study carried out, the following conclusions have been drawn:

1. The incremental dynamic analysis (IDA) for the three structural systems are compared based on the observations of the IDA curves. The existing structural system is adequate to resist gravity loadings assigned to the building, however, is foreseen unable to resist any potential earthquake that could hit Beirut in the future. The analyses show that the shear wall and steel bracing systems demonstrated better building performances compared to the original structure, such that these systems are able to resist the targeted performance of 0.25g PGA of the Lebanese seismic zone. The observation of fragility curves results shows that the shear wall and steel bracing systems provide good seismic improvement techniques which are able to achieve strengthening solution targets for an existing building system.
2. From the pushover analysis (POA), the capacity curves demonstrated that the shear wall system gives better results. In terms of ductility and stiffness, the steel bracing system shows better enhancement whereby the ductility increases by four times and the building is two times stiffer than the original model, while the shear wall system provides enhanced ductility by nine times more than the original model and the building is four times stiffer.
3. Based on the feasibility study, the shear wall system is costly compared to bracing system, and its seismic performance is much better based on the discussions in the previous analyses and therefore, a suitable choice for seismic improvement of an existing building

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