SEISMIC EVALUATION AND RETROFIT OF AN ASYMMETRIC REINFORCED CONCRETE FLAT SLAB BUILDING

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Seismic performance evaluation of existing structures and retrofit strategies are considered as a complicated multi-criteria problem. This study presents a performance evaluation of a gravity load designed or designed to earlier codes six stories asymmetrical flat slab building. Building with flat slab system may lack the required lateral stiffness to withstand seismic lateral loads. Three different retrofit systems to increase the lateral strength and stiffness of the building are proposed and examined. These systems include the addition of external frames, using shear walls and introducing steel knee bracing. The 3-D nonlinear pushover analysis procedure is utilized in evaluating the seismic performance of the original building and the retrofitted one, capacity and demand spectrum curves are presented for each studied case. The results are compared with elastic time history analysis method. Three different real earthquake records with intensities suitable for zones I, III and IV appeared in the renewed (2003) version of the Egyptian code of loads are utilized. It is found that the applied 3-D nonlinear analysis can be considered as a powerful tool in evaluating the seismic performance of frame structures. The study also shows that how can every suggested retrofitting system upgrade the seismic behavior of the gravity load designed flat slab building. Cost analysis comparison study is carried out.

KEYWORDS: *seismic evaluation, nonlinear pushover analysis, flat slab building, retrofit systems, strength and stiffness*

1. INTRODUCTION

The seismic performance evaluation and upgrading of non-seismic designed building structures located in new seismic zones is considered as an innovative challenge for seismic engineers and researchers. This concept has become an urgent issue in Egypt after the potential damage observed for many buildings during 1992 and 1995 quakes to achieve the purpose of seismic hazard mitigation. The seismic risk analysis of buildings is important for identifying the seismic vulnerability of structural systems under the effect of seismic ground motions [1], [2]. A great task for seismic engineers

and researchers is to decide how to retrofit an existing structure to upgrade its seismic capacity and to what level of protection [3].

Flat slab building structure is widely used due to many advantages it posses over the moment resisting frames. It provides lower building heights, unobstructed space, architectural flexibility and easier frame work. However, due to lack of deep beams and/or shear walls, the resulted transverse stiffness will be low. This may lead to potential damage even when subjected to earthquakes with moderate intensity. The brittle punching failure due to transfer of shear forces and unbalanced moments between slabs and columns may cause serious problems. Flat slab systems are also susceptible to significant reduction in stiffness resulting from the cracking that occurs from construction loads, service gravity and lateral loads [4]. The importance of accurately evaluating the seismic performance of flat slab structures highly increases when the structure is asymmetric in plan due to the torsional effects; in this case, the 3-D analysis of the full structure is required. Due to the previous mentioned reasons, and due to the tremendous number of buildings that use this system, effective and economic retrofit systems should be provided for the weak buildings.

There are many retrofit systems developed for RC buildings. Essentially, there are two main retrofitting techniques, the first is considered as non-conventional method, which incorporates base isolation and energy dissipation systems. This technique aims to increase the structural ductility and hence reduce the earthquake demand. The practical applicability of this technique is somehow limited. The second one is the system of strengthening and stiffening which is considered the most common seismic performance improvement strategies adopted for buildings with inadequate lateral force resisting systems. Typical systems employed for stiffening and strengthening include column strengthening and the addition of new vertical elements as moment resisting frames, shear walls or braced frames. The philosophy here is to provide systems that are strong enough to resist the seismic forces and light enough to keep the structural elements from needing further reinforcement [5], [6]. Most of the existing methods need emptying the building during the retrofitting process, which creates serious problems due to the evacuation of the building during the retrofit process. Therefore, it is highly preferable that these systems could be installed quickly and eliminates the need to distribute the occupants of the existing structure [7].

Nonlinear time history analysis of a detailed analytical model may be the best decision for estimating the damage. However, there are many uncertainties due to the selection of specific input and with the analytical models representing the behavior of the structure. Pushover analysis monitors the progressive stiffness degradation of a structure as it is loaded into the post elastic range. The inelastic static pushover analysis is an effective option for estimating the strength capacity and highlighting potential weak areas in the structure. The method allows tracing the sequence of yielding and failure of the members and also capture the overall capacity curve of the structure. The static pushover procedure has been recommended as a tool for design and assessment purposes by many associations as the National Earthquake Hazard Reduction Program 'NEHRP' (FEMA 273) [8] guidelines for the seismic rehabilitation of existing buildings and the Seismic Evaluation and Retrofit of Concrete Buildings ATC-40) [9]. The technique has been used and evaluated as the main tool of analysis in several studies [10]- [14].

The seismic design provisions and analysis methods appeared in the 2003 version of the Egyptian Code of Loads (ECL 201) [15] are considered a significant step toward improving the seismic behavior of buildings constructed in Egypt. The concept of retrofitting and upgrading gravity load designed or designed according to earlier codes that do not guarantee seismic protections is considered important. However, the Egyptian Code of Practice for design and construction of concrete structures (ECCS 203) [16] or (ECL 201) [15] do not offer provisions about how to deal with such branch neither recommendations about the suitable approaches of evaluations and the acceptable performance limits.

The purpose of this study is to offer a seismic performance evaluation of a gravity load designed six stories asymmetric flat slab building with plan dimensions of 36.0 m x 30.0 m fewer than three different earthquake levels. Three different retrofitting techniques are suggested and evaluated, these techniques include the addition of external frames, using shear walls and introducing steel knee bracing. All these systems are external systems, which do not require the evacuation of the building. 3-D nonlinear pushover analysis is adopted to evaluate the performance of the existing and retrofitted structure. Moreover, an elastic time history analysis is carried out. The objectives of this investigation can be summarized as:

(i)   To examine the seismic performance of non-seismic flat slab building under three different earthquake levels.
(ii) Suggest three different retrofitting systems and compare there performances.
(iii) Apply the approach of nonlinear pushover analysis and compare it with the elastic time history analysis.
(iv) Present comparative cost analysis.

2. ORIGINAL BUILDING DESCRIPTION AND MODELING

The studied building is a six stories reinforced concrete office building. The plan measures 36.0 meter by 30 meter. A typical bay width is 6.0 m in both directions, the configuration of the building resulted in the L-shaped plan shown in **Fig. 1**. The building has six stories with height from the ground of 18.5 m, the typical story height is 3.0 m except the first story, which has a height of 3.5 m, and no basement is presented. The gravity load resisting system consists of 0.22 m thickness two-way flat slabs carrying the floor loads to interior columns and perimeter frames. The lateral load resisting system is only the relatively rigid slabs through frames and columns. The perimeter frames consist of beams and columns with rectangular sections, the interior columns have square sections. The dimensions and reinforcement of the interior columns vary with height; every successive two stories have the same dimensions and reinforcement, as shown in **Table 1**. The perimeter columns have fixed dimensions of 0.30 x 0.75 m over height and fixed reinforcement of 14 Ø 22.

The compressive strength of concrete used in the building is 22.50 MPa while the used steel is mild steel with yield strength of 240 MPa. The three dimensional nonlinear pushover and linear time history are constructed and analyzed using ETABS software package, nonlinear version 8 [17].
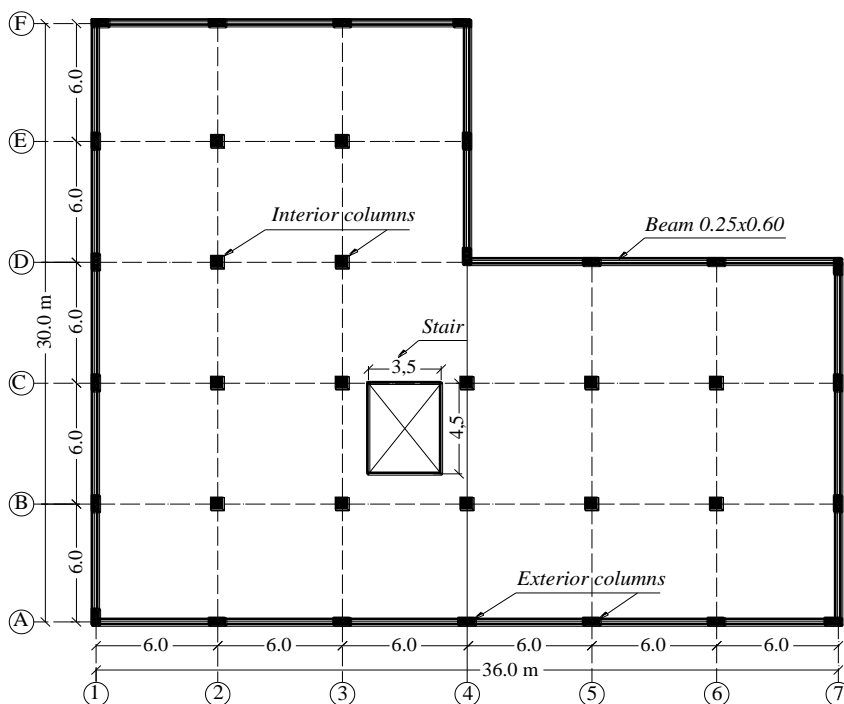
Fig. 1: Plan of the investigated building

Table 1: Variation of dimensions and reinforcement of interior columns over height

| Stories	1 & 2		3 & 4		5 & 6	
Interior columns	Dim.	Reinf.	Dim.	Reinf.	Dim.	Reinf.
	0.65 x 0.65	16 Ø 16	0.50 x 0.50	12 Ø 16	0.40 x 0.40	8 Ø 16

3. ACCELERATION RECORDS

Egypt is in low to medium seismicity region and subjected to frequent moderate earthquakes. The new edition of the (ECL 201) which was renewed in 2004, has divided Egypt to five different seismic zones. These zones are identified by their Peak Ground Accelerations (PGA) which range from 0.1g to 0.25g.

In this investigation, set of three real earthquakes records of different intensities ranging between 0.097 to 0.215 are selected to cover a wide spectrum of frequency and meet the range of peak ground accelerations selected for Egypt. The three selected records are:

1- Aqaba 1995 (Aqb.), E-W component, peak ground acceleration = 0.097 g and peak velocity of 0.14 m/s. The duration of this quake is 60 second. This quake is considered as level I quake.
2- Victoria, Mexico 1980 (Mex.), peak ground acceleration = 0.15 g and peak velocity of 0.248. The time duration is 26.92 sec. This quake is considered as level II quake.

3- El Centro 1940 (ELC.), of peak ground acceleration = 0.215 g and peak velocity of 0.302 m/sec. Time duration is 40 sec. The quake is considered as level III quake.

These quakes are applied either in longitudinal or transverse directions. As the period of the investigated building is about 0.95 sec, which is within the effective period of the applied quakes, the actual response spectra of the selected quakes is normalized and used in the analysis, the dominant period in the elastic type 1 spectrum suggested by (ECL 201) is much less than the period of the investigated building.   **Fig. 2** shows the time history of the selected earthquakes while **Fig. 3** shows the acceleration response spectra for 5% damping.
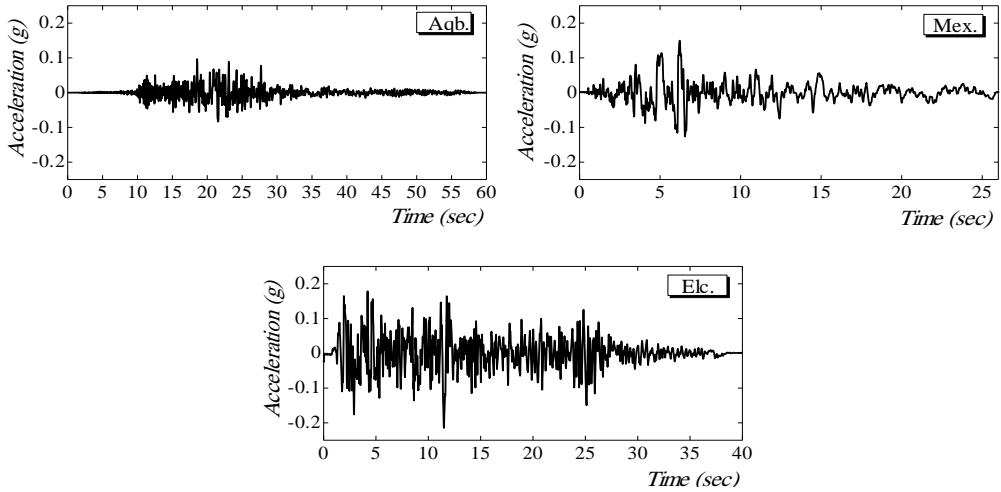
Fig. 2 : Time history of earthquake records
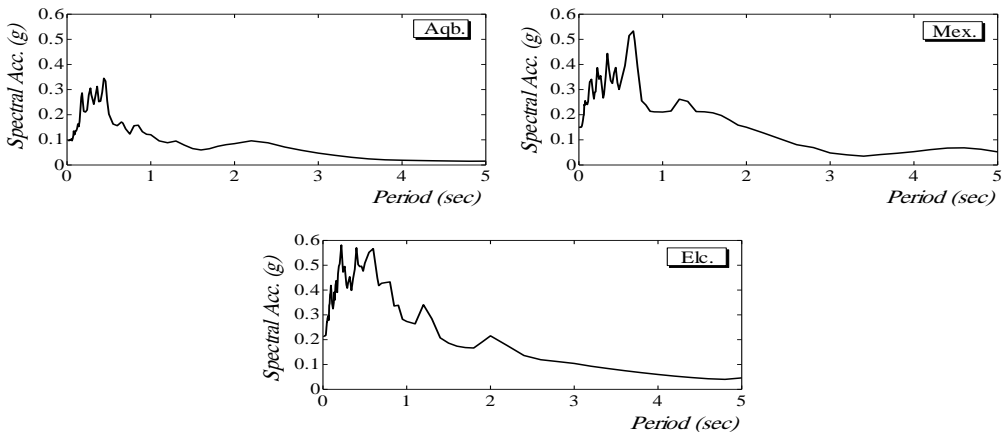

Fig. 3 : Acceleration response spectra

4- NONLINEAR PUSHOVER AND EVALUATION METHODOLOGY

The recent advent of performance based design has brought the nonlinear static pushover analysis procedure to the forefront. The pushover analysis procedure is performed by employing the capacity spectrum method. The structure is loaded first with vertical gravity loads, then pushed with incrementally increased static equivalent earthquake loads until the specified level of roof drift is reached. With the increase in the magnitude of loading, weak links and failure modes of the structure are found. The post elastic degradation of the flexural stiffness of a frame member begins when the material fibers furthest from the neutral axis of the cross section experience initial yielding. Under increasing moment, the degradation continues as plasticity spreads through the section depth and along the member length to form a fully developed plastic hinge, at which the flexural stiffness of the member section is exhausted.

The methodology of pushover analysis concentrates on the formulation of the inelastic capacity curve for the structure. This curve is a plot of the horizontal movement of the structure as it is pushed to one side. Initially the plot is a straight line as the structure moves linearly, as the parts of the structure yields the plot begins to curve. The generation of the capacity curve defines the capacity of the building uniquely and independently of any specific seismic demands, it replaces the base shear capacity of traditional procedures. When an earthquake displaces the building laterally, its response is represented by a point on the curve (performance point). This point defines a specific damage state for the building.

The methodology of the pushover performance analysis can be summarized in four steps as follows:

1- Idealizing the structure as a nonlinear model: A model of the entire structure is built from nonlinear representation of all of its elements and components.

2- Determining the capacity spectrum of the structure: The central focus of the simplified nonlinear procedure is the generation of the pushover or capacity curve This represents the lateral displacement as a function of the force applied to the structure. This process is independent of the method used to calculate the demand and provide valuable insight about the building. A schematic diagram displays the capacity curve of a building is shown in **Fig. 4**.
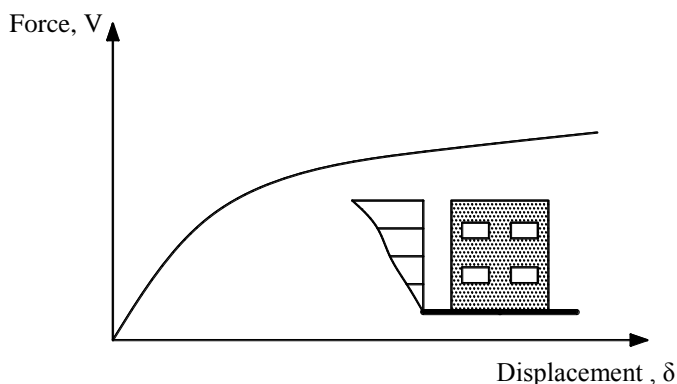

Fig. 4 : Capacity curve of a structure

3- Determination of the demand spectrum and performance point: The elastic spectrum of the effective applied earthquake (5% damped) is determined and is reduced depending on the inelastic behavior of the structure to intersect the capacity curve to find a performance point (a_p, d_p). This spectrum is plotted in spectral ordinates (ADRS) format showing the spectral acceleration S_a as a function of spectral displacement S_d. The equal displacement point (a_o, d_o) is a good starting point for the iterative process. A schematic diagram illustrates this process is presented in **Fig. 5**.
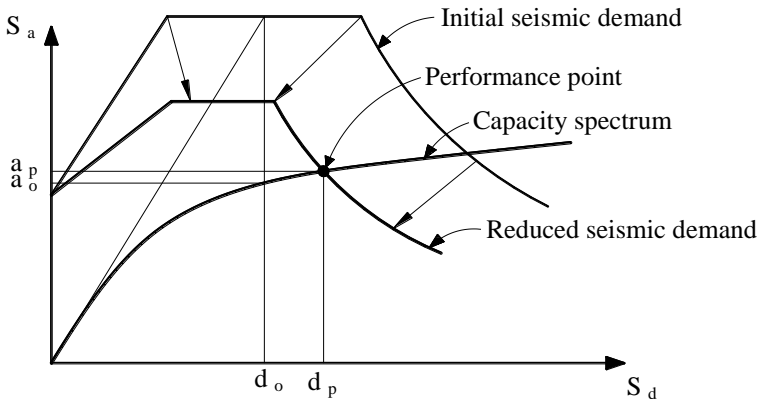
Fig. 5 : Determination of performance point

4- Specify the performance of the structure according to the applied criteria: Using the Performance Point or Target Displacement, the global response of the structure and individual component deformations are compared to limits in light of the specific performance goals of the structure. In this study, the ATC-40 [9] guidelines are used to define the force-deformation criteria for hinges used in the pushover analysis. As shown in **Fig. 6**, five points labeled A, B, C, D and E are used to define the force deflection behavior of the hinge and three points labeles IO, LS and CP are used to define the acceptance criteria of the hinge. (IO, LS and CP stand for Immediate Occupancy, Life Safety and Collapse Prevention, respectively). The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in the guidelines.
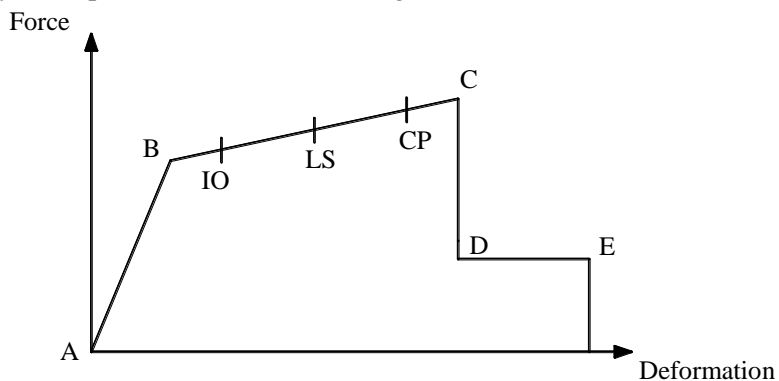

Fig. 6 : Force- deformation for pushover hinge

5 - RETROFITTING SYSTEMS

The basic safety objective requires that for the required performance level, the gravity resistance of the frame should be enhanced, the deformation in the frame columns should be limited and the vulnerability of the frame columns to shear failure should be reduced. The suggested and investigated retrofitting systems are based on adding new simple technique to constitute a primary system for seismic resistance while keeping the existing building as a secondary system mainly responsible for carrying the gravity loads. A number of alternatives are available to provide the needed strength and stiffness to satisfy the required performance characteristics. A review of some parameters as desired performance level, the existing architectural character of the building, the level of the disruption associated with each system and the access required to execute the required construction is carried out. Relying on this review, three retrofitting systems are suggested and evaluated in this study. A preliminary approach to design these systems using the nonlinear pushover analysis to find performance points within required deformation limit is carried out. The applied systems are briefly described as follows:

System I, Adding new exterior frames (St.): Two new R.C. exterior frames are added to the building, one is in the longitudinal direction adjacent to frame along axis A, between axes 2 and 6, the other is in the transverse direction adjacent to frame along axis 1, between axes A and F. The columns of the applied frames have section of 1.0 x 0.35 m and fixed reinforcement of 20 Ø 22. The new beams are with overall depth of 0.8 m and width of 0.3 m, the main reinforcement is 5 Ø 22 at regions of positive and negative moments. An elevation of the two frames is presented in **Fig. 7**.

(a) Frame in longitudinal direction   *(b) Frame in transverse direction*
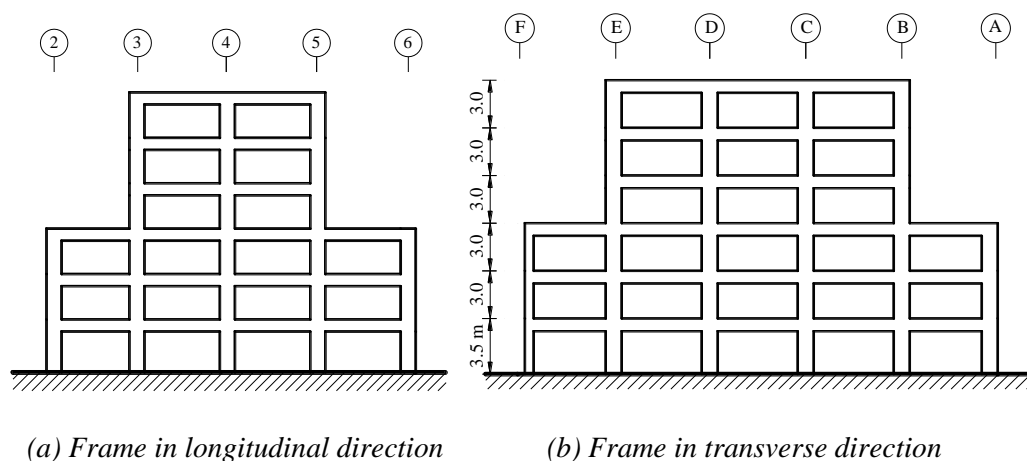
**Fig. 7** : Elevation of the two new exterior frames

System II, Applying external shear walls (Sw.): Two shear walls are externally applied to the frames already exist at the perimeter of the building. The first is in the longitudinal direction through axis F between axes 2 and 3, the second is in the transverse direction along axis 7 and between axes B and C. The length of each shear wall is 5.45 m, extends 0.10 m in the adjacent columns. Each shear wall has thickness of 0.25 m and goes over the whole height of the building as shown in **Fig. 8**.

(a) Frame in longitudinal direction   *(b) Frame in transverse direction*
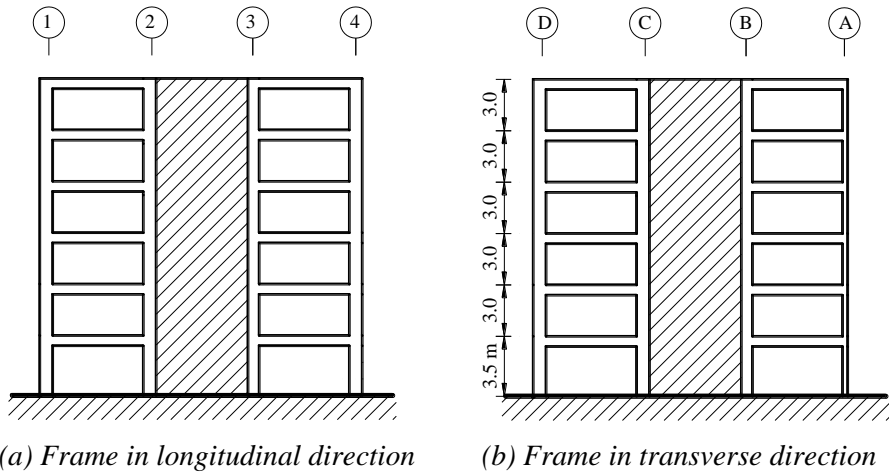
Fig. 8 : Elevation of the original frames with the new inserted shear walls

System III: Inserting external steel knee bracing (Br.) : In this system knee bracing members are applied to two originally existing exterior frames, one along axis A, between axes 2 and 6, the other is along axis 1 between axes B and F. The bracing members are tubular steel hollow sections of external dimensions of 0.25 x 0.25 m, the thickness is 0.01 m. The bracing members are strengthening the beams and columns of the two frames at the third points of each of them. The elevation of the two original strengthened frames using knee bracing are shown in **Fig. 9**.


(a) Frame in longitudinal direction   *(b) Frame in transverse direction*
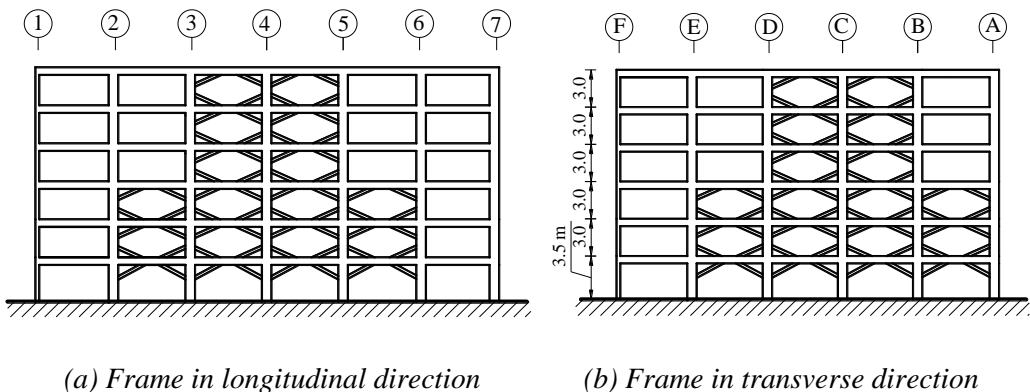
Fig. 9 : Elevation of the original frames with the new inserted knee bracing

6 – NONLINEAR PUSHOVER ANALYSIS RESULTS

Load displacement and modal analysis results are combined to generate the required ADRS. A five percent damped elastic demand response spectrum for each of the three applied earthquakes are generated and applied to the capacity spectrum of the original building in longitudinal and transverse directions as shown in **Fig. 10**. It can be observed that the lateral capacity of the original building is small that the maximum

value of spectral acceleration exhibited by the building does not exceed 0.11. The original building fails to intersect the elastic spectra for Mex. or Elc. quakes while it can hardly intersect the elastic spectra of Aqb. quake at the almost the end of the response. The effective inelastic damping of the applied earthquakes are calculated using the reduction factors for both acceleration and velocity then applied to original building in longitudinal and transverse direction, observe **Fig. 11**. The effective inelastic damping ratios calculated for Aqb. and Mex. quakes reaches up to 20% and 26%, respectively. This inelastic response enables the original building to meet the inelastic response of the two quakes (for Mex. quake the performance point is almost at the end of performance) in performance point coordinates of (0.041, 0.09 g) and (0.073, 0.10 g), respectively for the longitudinal direction. These values are (0.038, 0.08 g) and (0.079, 0.094 g) for the transverse direction. It can be seen that the original structure is not capable of achieving any performance level under Elc. quake.
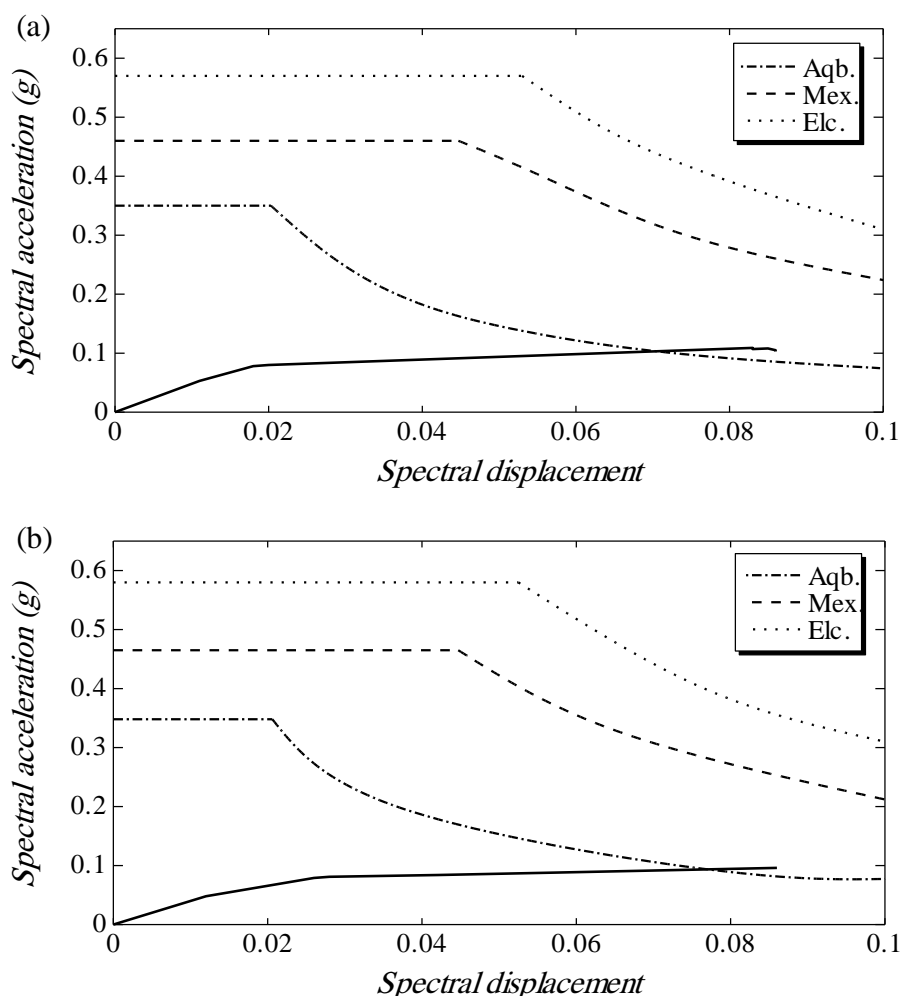
Fig. 10 : Five percent damped elastic spectrum: (a) longitudinal dir., (b) transverse dir.
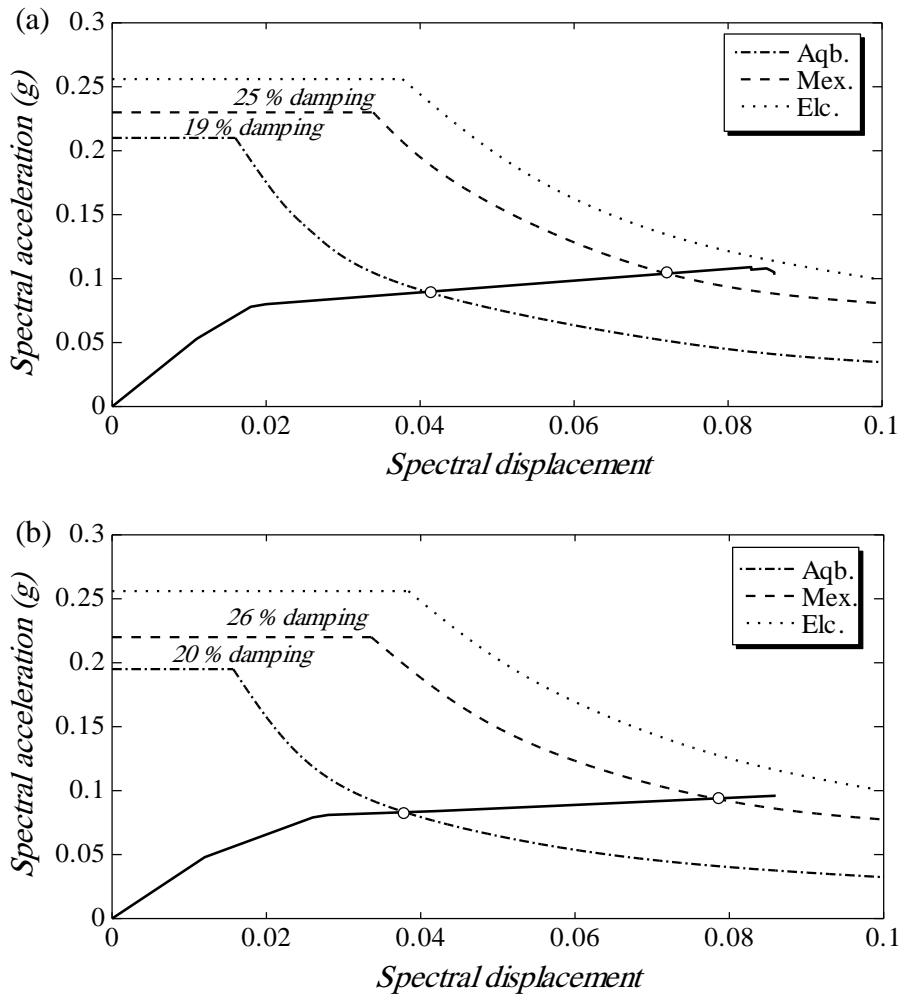
Fig. 11 : Effective inelastic response spectrum: (a) longitudinal dir., (b) transverse dir.

      The discussed results about lateral capacity spectrum of the original building compared to the elastic and inelastic spectrum demand of the applied earthquakes emphasizes that the a seismic retrofitting program is required. The suggested retrofitting systems should increase the strength and stiffness of the original building to prevent collapse under quakes close in intensity and dominant period to Elc. quake and enhance its behavior under the two other quakes.

      3-D nonlinear pushover analysis is applied to the retrofitted building with a procedure similar to that applied to the original building. As the primary elements of the retrofitted building are combinations of the existing and new elements, the structural behavior type is selected as type B [9]. The classical capacity curves represented by base shear and lateral displacement for the original and retrofitted building are obtained in both longitudinal and transverse directions, they are presented in **Fig. 12**. The mechanism of the retrofitting systems can be clearly observed from this figure, the suggested systems can highly increase the lateral strength of the original

building. The percentage increase in strength for the building with different retrofitting systems ranges between 90 to 99% in the longitudinal direction and between 88% to 98% in the transverse direction, relative to the strength of the original building. These percentage ratios give a glance that the capacity base shear curves of the retrofitted building with different systems have similar trends. The observed stiffness of the retrofitted building also increased but with ratios much less than those of the strength.
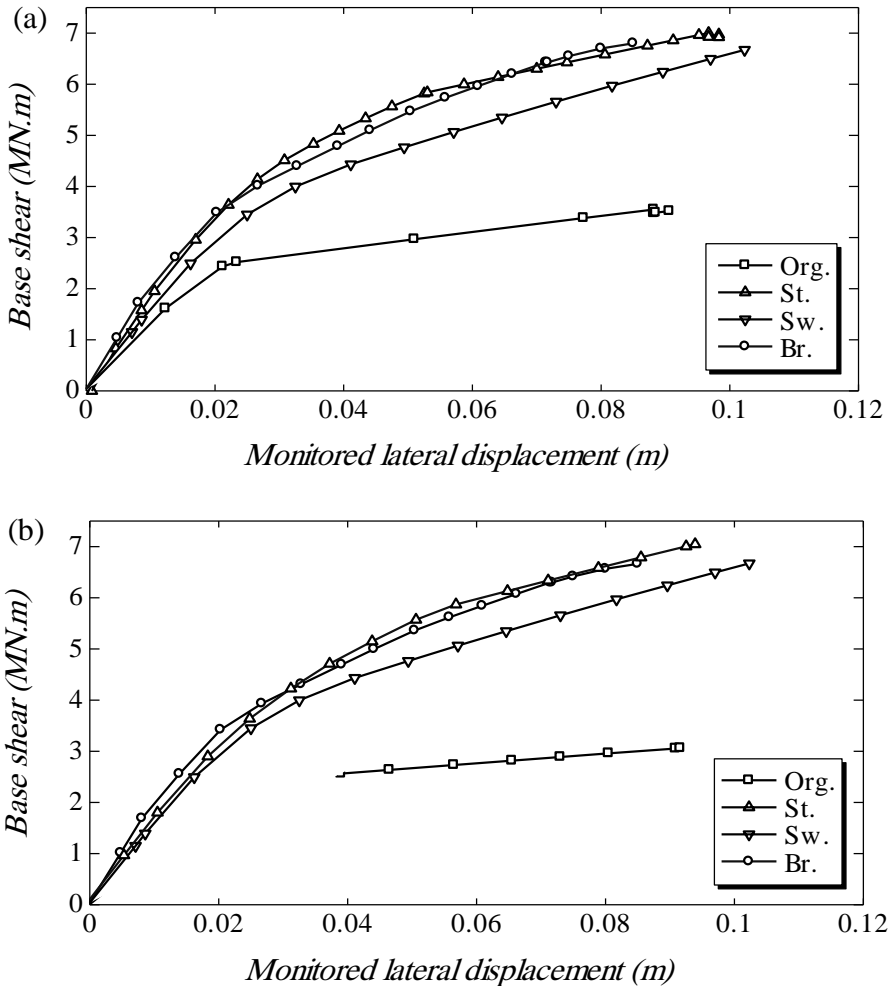
Fig. 12 : Base shear versus lateral displacement (a) longitudinal dir., (b) transverse dir.

The capacity spectrum curves defined by spectrum displacement and spectrum acceleration (ADRS) are calculated and plotted for the building with different retrofitting systems relying on the pushover analysis. The performance points resulting from the intersection between nonlinear capacity spectrum and reduced effective earthquake spectrum are illustrated in **Figs. 13 to 15**. It can be realized that all suggested retrofitting systems succeed in highly increasing the spectrum acceleration

associated with the original building, this increase is considered as a direct result of increasing the lateral stiffness of the building. The ratios of maximum increase in spectrum acceleration is not less than 120 % relative to the original building, this ratio increases in some cases to reach up to 267 %. The highest ratios are observed for Sw., Br. and St. systems, respectively, the ratios of percentage increase in acceleration are shown in **Table 2**. As the suggested retrofitting systems are applied to increase the stiffness and strength of the original building rather than increasing its ductility, the percentage increase in spectrum displacement for the retrofitted building have small values relative to the spectrum acceleration. These ratios range between 9.67 % and 25.43 % as shown in **Table 3**. The increase in capacity spectrum acceleration of the retrofitted building enables the capacity spectrum of the building from safely intersecting the reduced demand spectrum of the different applied quakes.
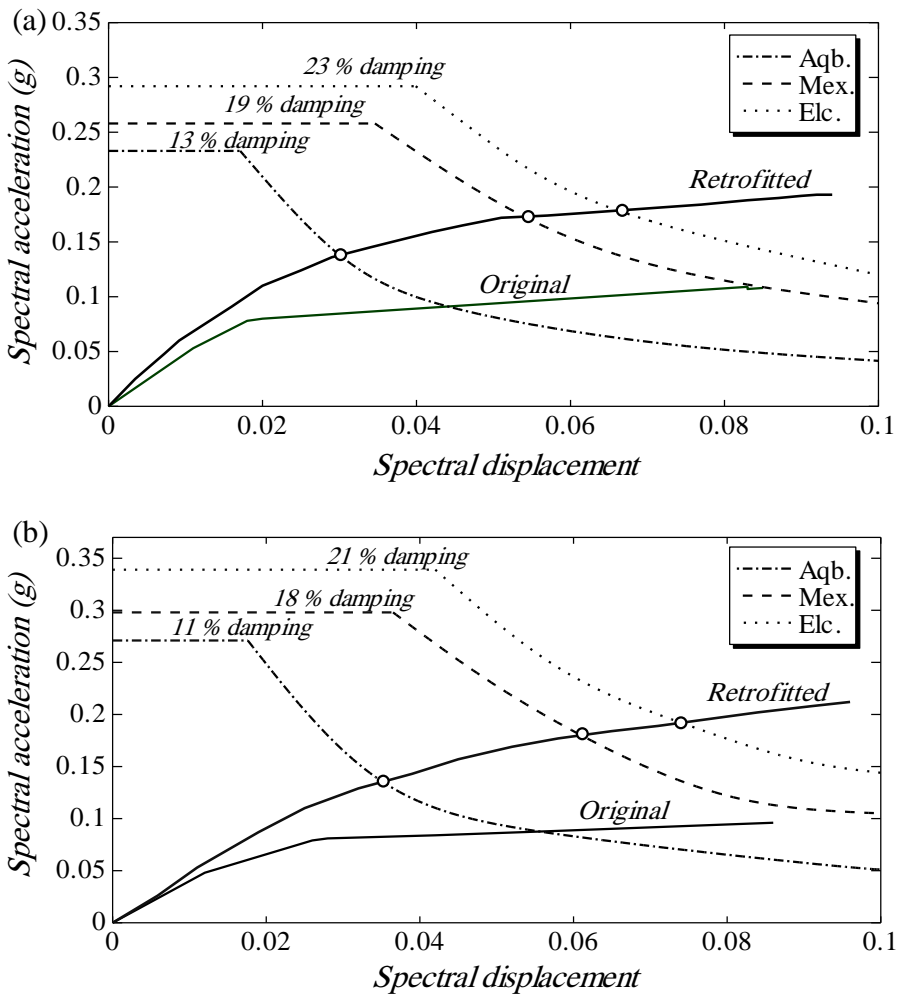
Fig. 13 : Response spectra of St. system (a) longitudinal dir., (b) transverse dir.

The performance points associated with the original building are successfully shifted towards highly enhancing the seismic behavior of the building. This enhanced performance is considered as a direct result of applying the suggested retrofitting systems which highly increase the strength of the building and hence the capacity spectral acceleration. The effectiveness of this shift increases as the spectral displacement of the retrofitted building itself increased. A notable point that can be realized is that the effective damping of the reduced spectrum of the earthquakes applied to the original structure is reduced for the same quakes applied to the retrofitted building. The calculated inelastic effective damping for Mex. quake does not exceed 19%, 15% and 17% for St., Sw. and Br. systems while for the original structure this values was up to 26%. The reduction in the effective damping results from the corresponding reduction in the hysteretic plastic behavior of the building, the values of the effective damping are illustrated in **Figs. 13** to **15**.
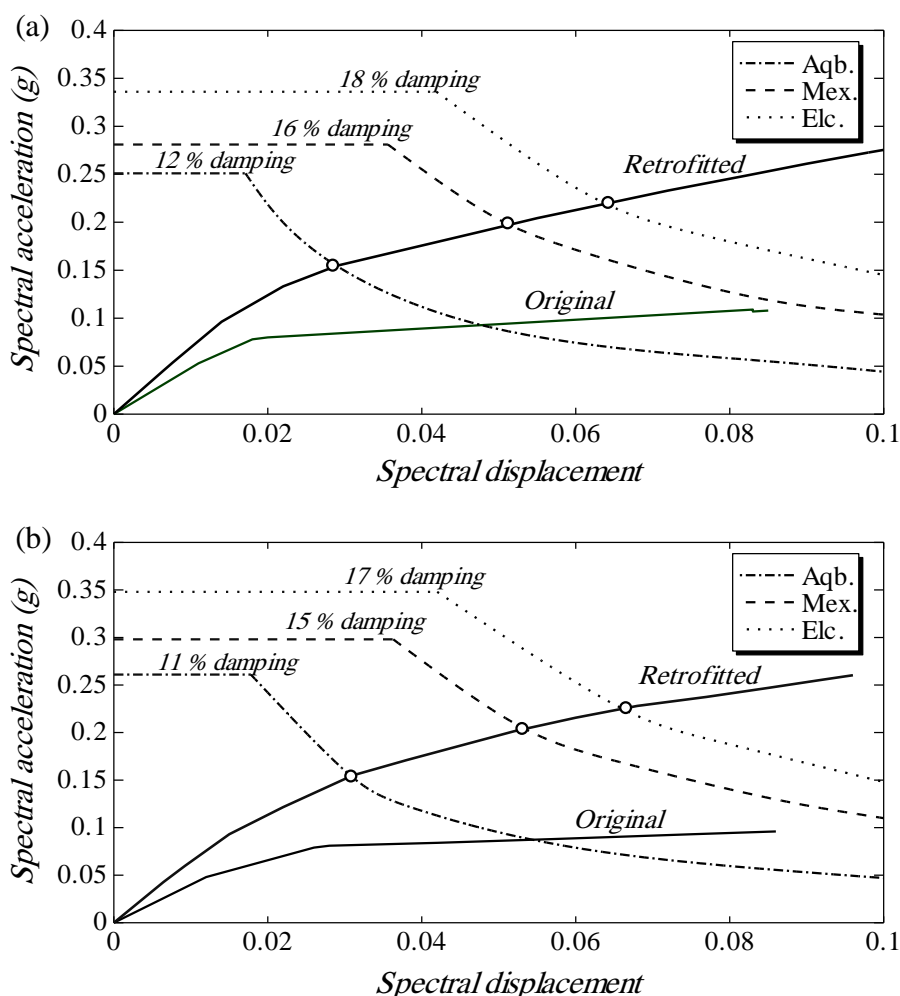
Fig. 14 : Response spectra of Sw. system (a) longitudinal dir., (b) transverse dir.
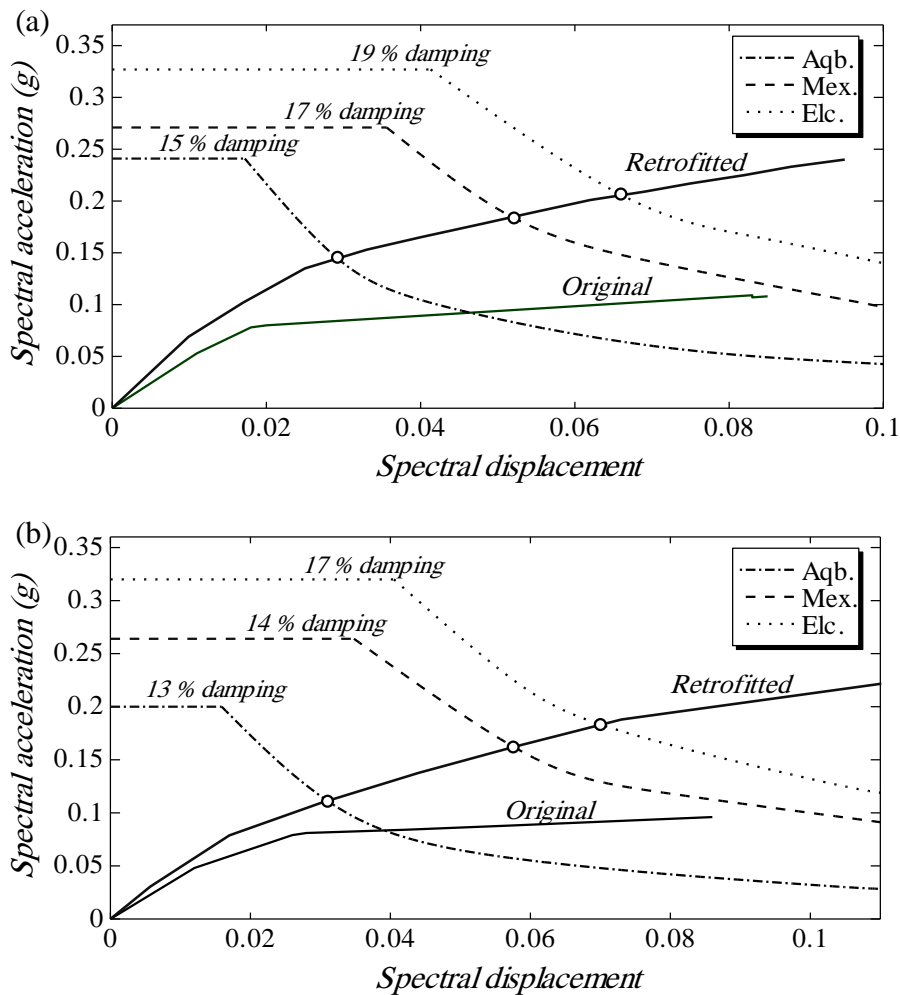
Fig. 15 : Response spectra of Br. system (a) longitudinal dir., (b) transverse dir.

Table 2 : Percentage increase in spectral acceleration.

System	St.	Sw.	Br.
Longitudinal dir.	127.06 %	267.06 %	182.35 %
Transverse dir.	120.83 %	185.41 %	133.33 %

Table 3 : Percentage increase in spectral displacement.

System	St.	Sw.	Br.
Longitudinal dir.	9.67 %	18.27 %	10.53 %
Transverse dir.	10.41 %	11.45 %	25.43 %

Based on the pushover analysis the sequence of hinging of beams and columns is studied for both original and retrofitted building under the different applied earthquake levels, the number and type of hinges associated with the rehabilitation building at the performance points are shown in **Tables 4** and **5**.

 It can be observed that under Aqb. (level I quake), the retrofitted building with any of the suggested systems can shift the performance of the original building from life safety (LS) performance limit to immediate occupancy (IO) performance limit. For this level of performance the maximum number of elements with plastic rotations up to 0.005 does not exceed 11% of the total number of elements.

Affected by Mex. (level II quake), applied in the longitudinal direction, it can be observed that only one element of the retrofitted building exhibits performance criteria of collapse preventions (CP), as the case of St. system. The behavior of all elements of the other two systems do not exceed the life safety (LS) criteria. The behavior in the transverse direction shows that the elements of the building with shear wall systems are within the life safety criteria while only three elements of St. system and four elements of Br. systems exhibited plastic hinges higher than Life safety (LS) and less than the collapse prevention (CP) limit. Note the for the existing building 24 elements exhibited collapse prevention (CP) behavior.

The retrofitted building with all retrofitting systems could eliminate completely the collapse that occurred to the 26 elements of the original building under Elc. (level III quake) applied in the longitudinal direction and 17 elements under the same quake applied in the transverse direction. All the elements of the retrofitted building did not exceed the collapse prevention (CP) criteria. The maximum number of elements in limits ranges between LS and CP is 19 elements.

Table 4 : Number and limits of plastic hinges in longitudinal direction.

System	Quake	B-IO	IO-LS	LS-CP	CP-C	CD	D-E
Org.	Aqb.	42	14				
	Mex.	64	1	26			
	Elc.	74	2				26
St.	Aqb.	65					
	Mex.	107	21	1			
	Elc.	109	14	19			
Sw.	Aqb.	62					
	Mex.	85	12				
	Elc.	111	10	8			
Br.	Aqb.	42					
	Mex.	35	25				
	Elc.	61	11	16			

Table 5 : Number and limits of plastic hinges in transverse direction.

System	Quake	B-IO	IO-LS	LS-CP	CP-C	CD	D-E
Org.	Aqb.	46	14				
	Mex.	53	1	24			
	Elc.	60	1	6		1	17
St.	Aqb.	47					
	Mex.	87	17	3			
	Elc.	101	10	18			
Sw.	Aqb.	44					
	Mex.	82	13				
	Elc.	100	11	4			
Br.	`	50					
	Mex.	80	16	4			
	Elc.	105	7	13			

The distribution of plastic hinges and their limits for the original building before and after the retrofit is monitored. For brevity, only examples of plastic hinge distribution and their limits under Elc. quake, applied in the longitudinal direction, for frame on axis 1 as shown in **Fig. 16 (a)**, **(b)**, **(d)** and **(e)**, or the strengthening frame adjacent to it as shown in **Fig. 16 (c)** are plotted. It can be realized that the damage of the original building is concentrated in the columns of the first floor that under the Elc. earthquake, almost all these columns suffer collapse. For these columns, the values of plastic hinge ratios exceed 0.035 and the performance limit is (D-E). The plastic hinges also developed in the columns of the second floor and third floors but with performance level (B-IO). The distribution of plastic hinge for beams are concentrated in the beams located in the first three floors, these beams exhibited plastic hinges of limits ranging between (B-IO) to (IO-LS).

The columns of the retrofitted building exhibited similar behavior for the different retrofitting systems. The maximum limit of performance is observed for the columns of the first floor, for these columns, the plastic hinges are with limits range between (B-IO) and (LS-CP). The distribution of plastic hinges spreads to the columns of the second, third and fourth floors but with plastic hinge does not exceed (B-IO) limit. The plastic hinge distribution of beams for Sw. and Br. systems are similar, for those two systems, the beams still exhibit some plasticity distributed over the second , third, fourth and fifth floor, the maximum performance limit observed for these beams does not exceed (B-IO) limit. As the St. retrofitting system is the only retrofitting system that presents new beams to the original building, the observed plastic hinge distribution is different. For this system, the plastic hinges distributed only between the beams of the strengthening frame, the original marginal beams did not display any plasticity.
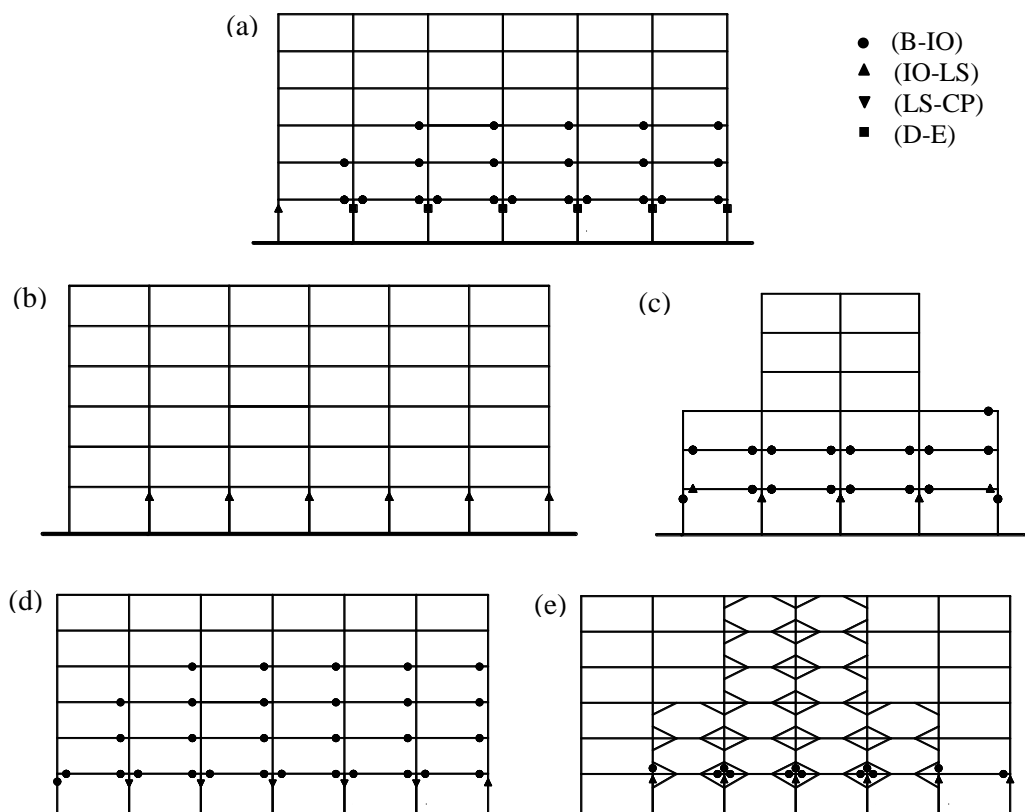
Fig. 16 Examples of plastic hinge distribution; (a) Original building, (b) St. sys. (frame on axis 1), (c) St. sys. (strengthening frame) (d) Sw. sys. (e) Br. sys.

7. TIME HISTORY COMPARISONS

3-D linear time history analysis is carried out under the three selected earthquakes. For brevity the results of base shear versus horizontal displacement under level III quake only are presented in **Fig. 17**. It can be concluded that the values of maximum displacement obtained from the time history can match the values of performance points calculated from nonlinear pushover analysis in the range of 3% to 30%. As the time history does not consider the nonlinearity of the elements the obtained values of base shear (Bs. system) is much higher than that obtained for the pushover analysis with percentage of difference up to 140%. From the time history analysis its clear that the retrofitted frame can reduce the displacement associated with the existing building with a percentage ratio up to 20% in the longitudinal direction and up to 36% in the transverse direction. The behavior of base shear versus displacement is close to be linear behavior except the case of retrofitted building with shear wall which exhibits hysteretic behavior. It can be concluded that the linear time history analysis can nearly estimate the lateral deformation of the building but it can not accurately evaluate the behavior of the existing or retrofitted building as it can not give data about the development of the plastic hinges in the different elements.
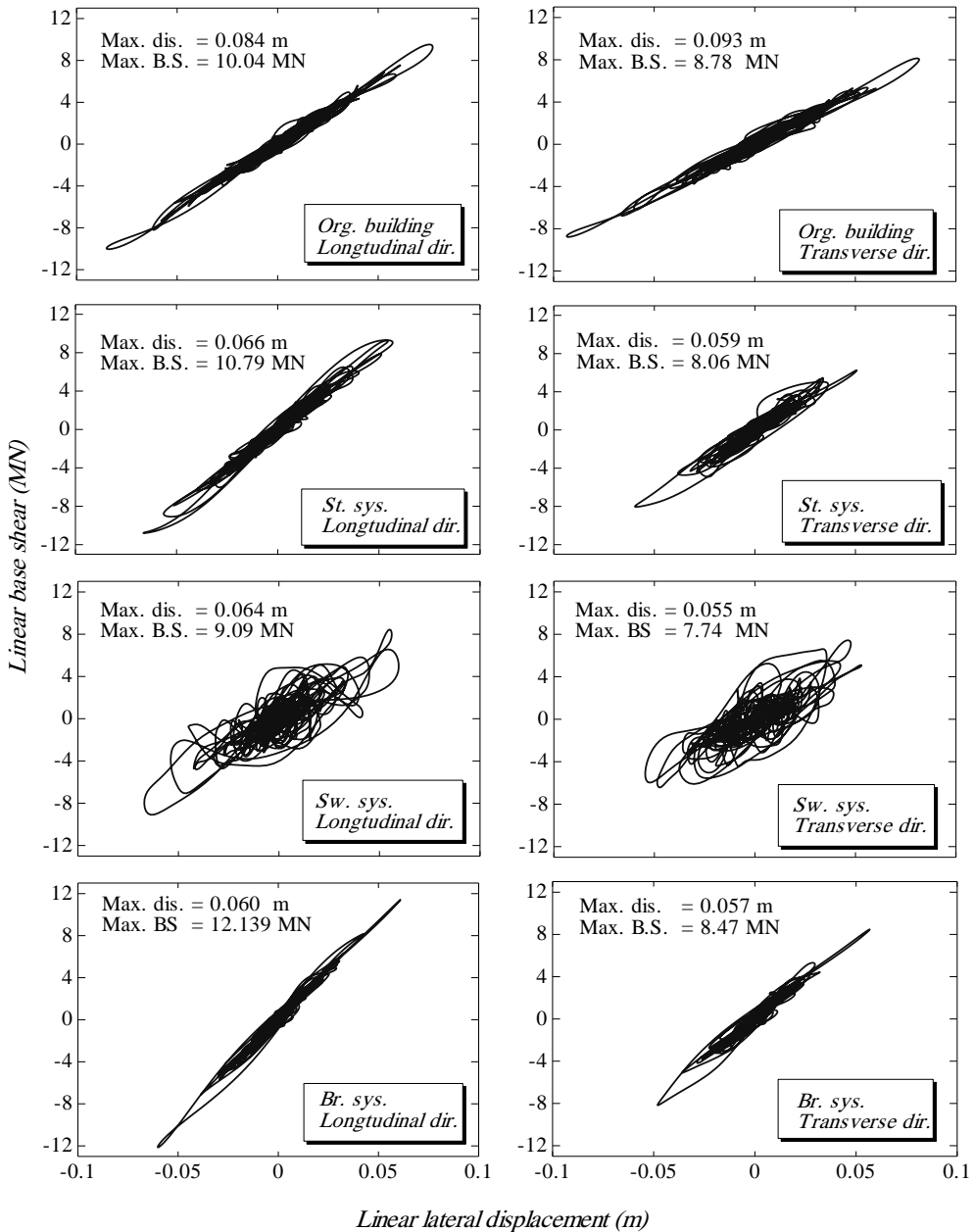
Fig. 17 : Base shear & lateral displacement for the different systems

8. COST ANALYSIS

A cost model study is carried out to report on the applicability of using the structural performance levels for the seismic retrofit designs. The objective of the cost analysis is to estimate approximately the building retrofit costs and provide a comparison between

the cost of retrofit of the different suggested retrofitting systems. The following assumptions are made considering the prices for year 2007 in Egypt.

1- The price of cubic meter of R.C including constituent, additives, frame work and labors is 2400 Egyptian pound.
2- The cost of one cubic meter of steel required for bracing including fabrication and erection is 10000 Egyptian pound, 20% of the section material is added to accommodate the connections.
3- The prices of finishes are not included.

The computed quantities and prices required for each system are shown in **Table 6**, it can be seen that most expensive system is applying external strengthening frames, for this system the cost is about 408720 EP. The cost of the Sw. and Br. retrofitting systems are much less than the St. retrofitting system, the cost for these two systems are calculated to be 35.7% and 21.8 %, respectively of the St. retrofitting system. The St. retrofitting system requires strengthening of the foundation while the Sw. retrofitting system requires the erection of new foundations under the new added shear walls. The Br. retrofitting system is the less expensive retrofitting system, the cost for this alternative is about 89000 EP, and from the architecture point of view, it presents the minor disturbance to the building and does not need any work to be carried out to the foundation.

Table 6 : Comparison between material quantities and cost of retrofitting systems

	St. system	Sw. system	Br. system
RC (m^3)	170.30	60.74	-
Steel sec. (ton)	-	-	8.90
Total cost	408720	145776	89000
%	100%	35.7%	21.80%

9. CONCLUSIONS

An analytical seismic performance evaluation of an asymmetric flat slab building designed for gravity loads only is carried out. Three different retrofitting systems, all of them are located on the perimeter of the building or adjacent to it, are suggested and evaluated using 3-D nonlinear pushover analysis. However, comparison with linear time history analysis is carried out. Three real earthquake records with different intensities close to the intensities proposed by ECL 201 are applied. The following conclusions may be drawn out.

1) The original flat slab building is susceptible to the applied seismic loads, the building fail to meet the requirements of the inelastic spectrum of Elc. (level III quake), most of the columns of the first floor almost collapsed due to the application of this quake. The original building could meet the inelastic requirements of Mex. (level II quake), at almost the end of the response, high plasticity is observed for many members under this quake.

2) All the suggested retrofitting systems succeed in highly increasing the capacity base shear of the building and hence increasing the spectrum acceleration, the percentage increase in spectrum acceleration ranged between 120 % and 267 %. As a direct result, the collapse of the members of the original building under level III quake is completely eliminated. The performance of the building under the rest two quakes is highly enhanced.

3) The linear time history analysis can only estimate nearly the lateral displacement of the building while the obtained values of base shear are highly overestimated. It is found that the estimating only the lateral drift is not sufficient in evaluating the performance.

4) The 3- D nonlinear pushover analysis proved to be a powerful tool in reasonably evaluating the seismic performance of original building, suggesting the suitable retrofitting systems and determining the locations, sequence and limit of plastic hinges.

5) The suggested bracing system has the advantage that it requires minimum time in erection, does not disturb the residents of the building and does not need any work to be carried to the foundation.

6) From the carried out cost analysis, it is found that the St. system is the most expensive system, the cost of applying Sw. or Br. systems are much less, the required cost for these two systems are 35.7 % and   21.8 %, respectively of the cost of St. system.

7) Finally, provisions about the procedures and accepted performance limits of gravity load designed buildings or buildings designed according to earlier codes need to be presented by the Egyptian Codes.

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تقييم السلوك الزلزالي وتأهيل مبني خرساني غير متماثل منشأ بنظام البلاطات اللاكمرية

نتيجة للزلازل التي تعرضت لها مصر في الآونة الأخيرة فقد أصبح تقييم السلوك الزلزالي واقتراح وتقييم نظم تأهيل للمباني القائمة بالفعل والمصممة فقط طبقا للأحمال الرأسية أو طبقا لكودات قديمة من التحديات الأساسية التي تواجه مهندسي الزلازل والباحثين. وقد استأثر نظام البلاطات اللاكمرية بانتشار واسع نتيجة للعديد من المميزات التي يقدمها, ولكن نظرا لنقص الكمرات العميقة و/أو حوائط القص يعتبر هذا النظام هشا عند تعرضه لأحمال الزلازل مما يتطلب تقييم سلوكه الزلزالي بدقه واقتراح ودراسة أنظمة تأهيل تضمن سلامته.

ويقدم هذا البحث دراسة تحليلية لتقييم سلوك مبني غير متماثل في المستوي الأفقي مكون من ستة أدوار, النظام الإنشائي لهذا المبني عبارة عن بلاطات لاكمرية مرتكزة علي أعمدة داخلية وخارجية. وقد تم تصميم المبني ليتحمل فقط الأحمال الرأسية. تم اقتراح ودراسة ثلاثة أنظمة لتأهيل المبني الأصلي مقدمة جميعها في أماكن علي المحيط الخارجي أو ملاصقة له في الاتجاه الطولي والعرضي. هذه الأنظمة عبارة عن إنشاء إطارين خارجيين (St. system) أو إنشاء حائطين للقص (Sw. system) كما تم اقتراح ودراسة تقديم نظام تدعيم (Br. system) من النوع الركبة (knee). تم إجراء التقييم باستخدام

الحل اللاخطي الثلاثي الأبعاد مستخدما طريقة الدفع (pushover analysis) كما تم مقارنة النتائج التي تم الحصول عليها باستخدام هذه الطريقة مع نتائج طريقة السجل الزمني الخطي. تم استخدام ثلاثة زلازل حقيقية ذات شدة منسوبة لعجلة الزلزال تتراوح بين 0.097 و 0.215 لتتناسب مع الشدة الزلزالية المقترحة من الكود المصري للأحمال,التحديث الأول, الصادر عام 2004 لثلاثة من المناطق المختلفة, ونظرا لأن الفترة الطبيعية للمبني محل الدراسة تقع في الحدود المؤثرة لهذه الزلازل فقد تم استخدام طيف التجاوب الحقيقي لهذه الزلازل بعد عمل نموذج لها.

وقد بينت النتائج أن المبني , قبل التأهيل, لم يتمكن من إيجاد نقطة أداء تحت تأثير الزلزال ذو المستوي الثالث وبناءا عليه فقد عانت معظم أعمدة الدور الأرضي لهذا المبني من الانهيار الكلي أما تحت تأثير الزلزالين ذوي المستوي الأول والثاني فقد ظهرت مفصلات لا مرنة (plastic hinge) كثيرة وذات حدود متفاوتة. وباستخدام أنظمة التأهيل المقترحة وجد أن عناصر المبني لم تعاني من أي انهيار وتم تحسين السلوك الزلزالي للمبني نتيجة زيادة طيف التجاوب للعجلة بنسب تتراوح بين 120% و 267 %. كما بينت الدراسة أن طريقة الدفع اللا خطية تعتبر طريقة فعالة في تقييم الأداء الزلزالي للمبني وتتبع نقاط المفصلات اللامرنة وحدودها في الأعضاء المختلفة علي عكس طريقة السجل الزمني الخطي للزلزال التي نتج عنها قيم مرتفعة (over estimated) للقص عند القاعدة بنسب وصلت الي 140 % مقارنة بطريقة الدفع اللاخطية. وبدراسة تكلفة نظم التأهيل المقترحة وجد أن أكثرها تكلفة هو نظام التأهيل الذي يتضمن إنشاء إطارين خارجيين (St. system) وبينت الدراسة أن تكلفة النظامين الآخرين, إنشاء حائطين للقص (Sw. system) وتقديم نظام للتدعيم (Br. system) هي علي التوالي 35.7 % و 21.8%. يتميز نظام التدعيم المقترح (Br. system) ذو التكلفة الأقل بضألة تأثيره علي الشكل المعماري بالإضافة إلي أنه لا يحتاج أي أعمال خاصة بالأساسات.