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## Evaluating the Vulnerability Index of Reinforced Concrete Moment-Resisting Frames Retrofitted with Steel Braces

Ruhollah Shirdel Afousy

MS in Structural engineeri Javid Jiroft University of Civil Engineering

### ABSTRACT

*As a vast part of the structures, the reinforced concrete structures can provide very optimal buildings in case of being designed and implemented based on accurate calculations and ductility relations. These structures might be in need of reinforcement for various reasons, including designing and executive errors as well as procedural variations. In order to exactly evaluate such structures and determine the extent to which they may fail, the use of the failure indices that have been dealt with a lot during the recent decades can be a promising solution. The present study aims at studying the vulnerability indices of the reinforced concrete moment-resisting frames retrofitted with steel braces. The frames studied herein are in match with the criteria mentioned in the tenth chapter of the building regulations and 2800 guidelines with the buildings in which they are to be used featuring intermediate importance. In terms of the evaluating the vulnerability indices, the results indicated that the concrete moment-resisting frames are highly vulnerable when subjected to earthquake waves and they undergo a lot of displacement, movement and crushes. However, they would exhibit proper seismic behaviors if they are improved and retrofitted with steel braces. It has to be pointed out that a structure's performance status is specified in respect to its failure index.*

**Keywords:** *vulnerability index, improvement and retrofitting, concrete moment-resisting frames, steel braces*

### INTRODUCTION

Earthquake is one of the natural devastative phenomena and it has left behind a lot of financial and life losses only within the past half a century. As one of the earthquake-prone countries of the world, Iran has sustained damages as a result of this phenomenon and the consequences stemming thereof. The possibility of a severe earthquake's occurrence due to the natural geological conditions of Iran, on the one hand, and, also, the designing and construction of buildings by the use of a lot of concrete for the mere non-observance of the seismic loading in them, on the other hand, make it necessary to evaluate the quakes of the existing buildings as well as their improvement. In concrete frames, the lateral stiffness is usually secured in separate and in combination with the moment-resisting frames and shear walls. Researchers have suggested various methods for seismic strengthening of the frames with each of them possessing certain advantages and disadvantages. One of these methods is retrofitting new structural components like shear wall or steel brace to the structure. The use of steel brace can be taken into account for the reason that it imposes the least overweight to the structure and, in the meanwhile, it can be more simply implemented as compared to the other methods. The blending of the concrete moment-resisting frames retrofitted with metal brace can be useful in certain cases.

The reason for this can be justified by lower construction costs and lower need for system repair and replacement after the earthquake. Moreover, if the concrete structure is found in need of improvement after construction, use can be made of metal bracing system in comparison to shear wall due to the easy implementation and lower cost that make it an appropriate method for concrete improving. During the recent years, many researchers and institutes have performed research works regarding the improvement of the structures. One of the most well-known of these institutions is the Federal Emergence Management Agency (FEMA). FEMA-273 is the first journal of this agency which was published under the title of seismic improvement guidelines in September 1996 for investigation of the buildings' vulnerabilities. In addition, it was in 2000 that the standard draft named FEMA-356 was published for seismic improvement of the buildings.

In 1981, Higashi Vando and Kavamata Vahnoma performed certain studies about the use of the concentric and eccentric braces in the concrete frames and they reached satisfactory results. In 1987 and 1991, Bush and others used a braced steel frame system of a complex type in concrete frames and they reached a considerable increase in in-plane shear resistance. In 1988, Ohiti and colleagues and Segi Guchi and others performed similar investigations about the use of chevron braces. In all of these studies, metal brace has been embedded indirectly into the concrete frame in this way that the metal brace is seminally inserted inside a metal frame which is per se subsequently confined onto an attached concrete frame. In this case, the shear force is indirectly transferred through a metal frame between the metal brace and the concrete frame. This system is termed indirect bracing system which is also very costly and not cost-effective. In direct bracing system, metal brace is directly connected to the concrete frame. Considering the abovementioned materials, the present study aims at evaluating the vulnerability index of the reinforced concrete moment-resisting frames retrofitted with steel braces.

#### Study Method:

In this study, the selected 2D concrete frames have been designed for 5-storey, 8-storey and 12-storey heights (each storey 3 meters in height) buildings with fixed one-span plans. The span lengths of 5-storey, 8-storey and 12-storey buildings are 3m, 5m and 6m, respectively. The lateral load-bearing system of these frames includes strong moment-resisting frames with intermediate ductility. Each of the storeys is 3 meters in height.

ACI-318-99 guidelines have been applied herein for designing the elements of the models' reinforced concrete. Furthermore, the rules in the sixth chapter of Iran's national regulations have been exercised in regard of the loading procedures. The fifth and ninth chapters of Iran's national regulations, as well, have been utilized for the specifications and characteristics of the constructional materials such as concrete and steel.

The selected members' cross-sections with rectangular dimensions have been prepared for designing the beam elements for f-storey, 8-storey and 12-storey frames. The cross-sections of the columns have been selected in square shapes. Table (1) gives the names of the beam and column cross-sections along with the geometric specifications and applied reinforcements.

Table 1: details of the specifications of the concrete cross-sections for the designing of the beam and column members

Cross-section's name	Cross section's height ( $b_f$ ) in centimeters	Cross section's width ( $t_f$ ) in centimeters	Reinforcement
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B35x35	35	35	-
B35x40	35	35	-
B40x40	40	40	-
C30x30-8φ14	30	30	8φ14
C30x30-8φ16	30	30	8φ16
C35x35-8φ16	35	35	8φ16
C35x35-8φ18	35	35	8φ18
C35x35-8φ20	35	35	8φ20
C40x40-8φ20	40	40	8φ20
C45x45-8φ20	45	45	8φ20
C45x45-12φ20	45	45	8φ20
C45x45-16φ20	45	45	16φ20
C50x50-16φ20	50	50	16φ20
C50x50-20φ20	50	50	20φ20
C55x55-20φ20	55	55	20φ20
C55x55-24φ20	55	55	24φ20
C60x60-24φ20	60	60	24φ20
C60x60-24φ22	60	60	24φ22
C65x65-24φ22	65	65	24φ22
C65x65-24φ25	65	65	24φ25



In order to determine the stiffness matrix of the entire structure in SAP 2000 finite element Software, the beams are considered without the existence of their reinforcements and only with their net concrete cross-sections; as for the columns, their concrete cross-sections along with their steel bars are taken into account. Based on the designing procedures of Iran's reinforced concrete buildings guidelines (ninth chapter of Iran's national building regulations), the bending stiffness is set at 35% about the strong axis, torsional stiffness of the beams at 15% and bending hardness at 70% around both axes of the columns for the reinforced concrete moment-resisting systems in line with the determination of the entire structure's total stiffness.

In this study, the extensive dead gravitational load (including the weight of the floor slab and its paving works plus partitioning) is 150 kilograms per every square meter with the live weight being 300kg/m<sup>2</sup>. It was concluded herein that all of the beams and all of the columns can be controlled by bending.

The amounts needed for the determination of a, b and c modeling coefficients as well as the acceptance scales are all based on journal 360.

Table 2: acceptance amounts-scales in the beams

Positive and negative moment areas						
Joint type	Scales of accepting the amount of plastic rotation			Modeling parameters		
	IO	LS	CP	a	b	c

Moment-resisting (bending)	0.005	0.0	0.02	0.02	0.04	0.2
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Table 3: scales for columns' acceptance

Positive and negative moment areas						
Joint type	Scales of accepting the amount of plastic rotation			Modeling parameters		
	IO	LS	CP	a	b	c
Moment-resisting (bending)	0.003	0.01	0.012	0.012	0.02	0.2

Cross-bracing systems wherein both of the braces intersect one another longitudinally meet each other in the middle feature an effective length equal to 0.5 m of the total length of the brace in the in-plane buckling state and an effective length equal to 0.67 m of the total length of the brace in the out-plane buckling state. As for the V-shaped, inverted V-shaped and diagonal (inclined) braces, these cross-bracing systems feature an effective length equal to 0.8 m of the total brace's length in the in-plane buckling state and an effective length equal to the brace's total length in the out-plane buckling state for both of the braces. The total length of the brace also includes the sheets attached to the ending section, as well, for all the states.

Tabas and Manjil are the earthquakes selected for analyzing the seismic damages imposed onto the concrete frames. The following table shows the specifications of the two selected earthquakes.

Table 4: the characteristics of the applied

Accelerogram	Station recorded the earthquake	PGA (m/s <sup>2</sup> )		Site class	Distance from the fault (km)
		x-direction	y-direction		
Manjil, Iran	BHRC 99999 Abbar	0.515 g	0.496 g	D	40.43
Tabas, DAY	Dayhook	0.328 g	0.406 g	D	27.0

#### Determination of the standard designs' spectrum in BSE-1 risk levels for Tehran:

The amounts of the standard designs' spectrum ( $S_a$ ) are obtained through multiplying the building's reflection coefficient (B) by the design's base acceleration (A). Corresponding to the relations in Iran's 2800 earthquake guidelines, 4<sup>th</sup> edition, for type III lands, these amounts are obtained as explained below:

B is the variable defined as the structures' coefficient of reflection; it is the way a structure responds to the ground motions and it is specified according to the following relations in which 1B is the coefficient of the spectrum's shape and N is the spectrum's modification coefficient:

$$B1 = \begin{cases} S0 + (S - S_0 + 1) \left(\frac{T}{T_0}\right) & 0 < T < T_0 \\ S + 1 & T_0 < T < T_s \\ (S + 1) \times \left(\frac{T_s}{T}\right) & T > T_s \end{cases}$$

$$N = \begin{cases} 1 & T > T_s \\ \frac{0.7}{4 - T_s} (T > T_s) + 1 & T_s < T < 4sec \\ 1.7 & T > 4sec \end{cases}$$

(1)

$$B = N \times B1$$

T is the structure's main period for the reinforced concrete moment-resisting frame retrofitted with convergent braces and it is computed based on the experimental relations in Iran's 2800 earthquake guidelines as given in relations (2) and (3). It should be noted that H is the structure's height calculated from the base level.

(2)

$$T = 0.05(H)^{0.9}$$

(3)

$$T = 0.05(H)^{0.75}$$

$$T_{12st-Original} = 0.05 \times (36)^{0.9} = 1.258$$

$$T_{12st-Rehabilitated} = 0.05 \times (36)^{0.75} = 0.735$$

$$T_{8st-Original} = 0.05 \times (24)^{0.9} = 0.873$$

$$T_{8st-Rehabilitated} = 0.05 \times (24)^{0.75} = 0.542$$

$$T_{5st-Original} = 0.05 \times (15)^{0.9} = 0.572$$

$$T_{5st-Rehabilitated} = 0.05 \times (15)^{0.75} = 0.381$$

Assuming type III as the land type on which the studied buildings are going to be built, seismic parameters denoted S, S0, T0 and TS are respectively equal to 1.75, 1.1, 150 and 0.7.

This way, the diagram of the standard designs' spectrum (Sa) would be as shown beneath for the designed base acceleration intensity (A=0.35) for BSE-1 risk level for Tehran.

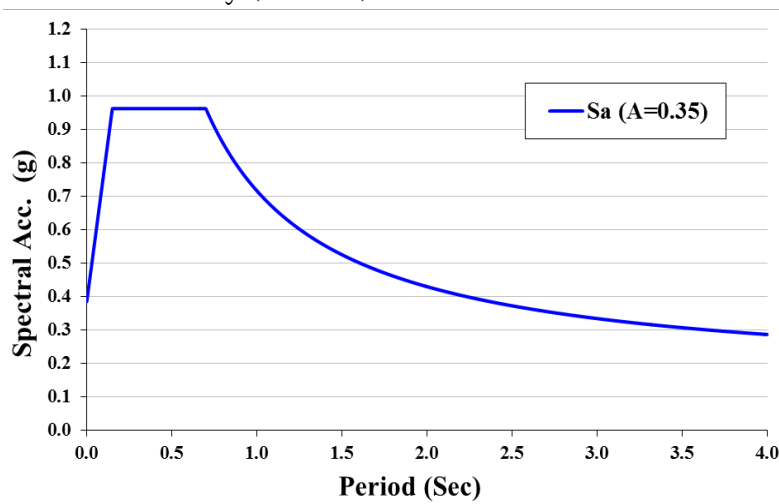


Figure (1): standard designs' spectrum for earthquakes' BSE-1 risk level



In this stage and in order to improve the status of the diagrams, the scale coefficients are changed through trial and error and, in the end, the final scale coefficients are attained. The following table presents the abovementioned earthquakes' scales for the frames' spectrum of standard designs.

Table 5: the accelerograms' scale coefficients in respect to the standard designs' spectrum for earthquakes in LS performance level

State	Frame	Manjil	Tabas DAY
Base	12-storey	2.30	2.00
	8-storey	2.25	1.25
	5-storey	1.025	1.15
Improvement	12-storey	1.75	1.20
	8-storey	1.02	1.15
	5-storey	1.00	0.77

### Preliminary Nonlinear Analysis:

Using SAP2000 finite element software, the models' pushover analysis was conducted. The increase in the forces' amounts is conducted by the application itself and automatically. The determinant scale for the analysis termination is the structure's roof displacement meaning that the target spot's change is determined initially for the roof and, then, it is with the increase in the forces' amounts that the structure's analysis is advanced to the ending stage.

### Designing Convergent Braces for Improving the Concrete Frames' Seismic Resistance:

5-storey, 8-storey and 12-storey buildings' concrete frames have been designed for seismic improvement only for the gravitational loads. Now, selecting the steel convergent brace system for improving the concrete frames' seismic resistance, the distributed shear forces should be obtained in elevations. This shear force is going to be entirely tolerated by the steel braces. Next, the process of determining the designed shear force in regard of the steel braces would be explicated in accordance with Iran's 2800 earthquake guidelines, 4<sup>th</sup> edition.

### Step One: Determining the Earthquake Coefficient (C):

In this section of the research, the static earthquake coefficient needed for the seismic loading of the convergent braced frames should be obtained in regard of the fourth edition of 2800 Standards (1):

III →  $T_0 = 0.15$ ;  $T_s = 0.7$ ;  $S = 1.5$  ;  $S = 1.10$  soil type

I = 1.0 importance factor

The design's base acceleration according to the construction place of Tehran:

$A = 0.35$

The specific behavior coefficient of the convergent braced frame:

$R_u = 5.5$

Magnification coefficient of the structure's lateral displacement:

$C_d = 5.0$

Structure's over-resistance coefficient:

$\Omega_0 = 2.0$

Calculation of the structure's period:

$T_{5st} = 0.05 \times H^{(0.75)} = 0.05 \times (15)^{(0.75)} = 0.381 \text{ sec}$

$$T_{8\text{ st}} = 0.05 \times H^{(0.75)} = 0.05 \times (24)^{(0.75)} = 0.542 \text{ sec}$$

$$T_{12\text{ st}} = 0.05 \times H^{(0.75)} = 0.05 \times (36)^{(0.75)} = 0.735 \text{ sec}$$

#### Earthquake's Reflection Coefficient:

##### 5-Storey Frame:

$$B_1 = 1 + S = 1 + 1.75 = 2.75$$

$$N = 1.0$$

$$B_X = N \times B_1 = 1.0 \times 2.75 = 2.75$$

##### 8-Storey Frame:

$$B_1 = 1 + S = 1 + 1.75 = 2.75$$

$$N = 1.0$$

$$B_X = N \times B_1 = 1.0 \times 2.75 = 2.75$$

##### 12-Storey Frame:

$$B_1 = (S + 1) \times \left(\frac{T_s}{T}\right) = (1.75 + 1) \times \left(\frac{0.7}{0.735}\right) = 2.619$$

$$N = \frac{0.7}{4 - T_s} (T - T_s) + 1 = \frac{0.7}{4 - 0.7} (0.735 - 0.7) + 1 = 1.007$$

$$B_X = N \times B_1 = 1.007 \times 2.619 = 2.637$$

#### Earthquake Coefficient:

$$\rho = 1.0$$

$$C_{5\text{ st}} = \rho \frac{ABI}{R_u} = \frac{0.35 \times 2.75 \times 1}{5.5} = 0.175$$

$$C_{8\text{ st}} = \rho \frac{ABI}{R_u} = \frac{0.35 \times 2.75 \times 1}{5.5} = 0.175$$

$$C_{12\text{ st}} = \rho \frac{ABI}{R_u} = \frac{0.35 \times 2.75 \times 1}{5.5} = 0.1678$$

#### Step Two: Determination of the Base Shear (V):

Seismic masses of the concrete frames as obtained from SAP2000 Software:

$$W_{5\text{ st}} = 7765 \text{ kg}$$

$$W_{8\text{ st}} = 21360 \text{ kg}$$

$$W_{12\text{ st}} = 37560 \text{ kg}$$

$$V_{5\text{ st}} = C_{5\text{ st}} \times W_{5\text{ st}} = 0.175 \times 7765 \text{ kg} = 1359 \text{ kg}$$

$$V_{8\text{ st}} = C_{8\text{ st}} \times W_{8\text{ st}} = 0.175 \times 21360 \text{ kg} = 3738 \text{ kg}$$

$$V_{12\text{ st}} = C_{12\text{ st}} \times W_{12\text{ st}} = 0.1678 \times 37560 \text{ kg} = 5967 \text{ kg}$$

#### Step Three: distribution of the base shear force in frames' elevations (V<sub>i</sub>):

The lateral load patterns can be observed in the following table:

Table 6: lateral load patterns

Storeys	$\frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k}$		
	5-storey building's frame	8-storey building's frame	12-storey building's frame
ST-12			0.1621
ST-11			0.1471
ST-10			0.1322
ST-9			0.1175
ST-8		0.2242	0.1030



ST-7		0.1957	0.0887
ST-6		0.1672	0.0747
ST-5	0.3333	0.1388	0.0609
ST-4	0.2667	0.1105	0.0475
ST-3	0.2000	0.0824	0.0344
ST-2	0.1333	0.0545	0.0219
ST-1	0.0667	0.0268	0.0101

Now, the seismic force exerted onto every level is as shown in the table below:

Table 7: seismic force exerted onto every level

Storeys	$F_x = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \times V(kg)$		
	5-storey building's frame	8-storey building's frame	12-storey building's frame
ST-12			967.2
ST-11			877.5
ST-10			788.8
ST-9			701.2
ST-8		838.2	614.7
ST-7		731.4	529.4
ST-6		624.9	445.6
ST-5	453.0	518.7	363.4
ST-4	362.4	413.0	283.2
ST-3	271.8	307.9	205.3
ST-2	181.2	203.5	130.5
ST-1	90.6	100.3	60.1

In this section, the shear force of every storey obtained from cumulative summing of the seismic forces exerted onto the above floors (previous table) is obtained as given in the following table:

Table 8: every storey's shear force

Storeys	$V_i = \sum_{j=i}^n F_j(kg)$		
	5-storey building's frame	8-storey building's frame	12-storey building's frame
ST-12			967.2
ST-11			1844.7
ST-10			2633.6
ST-9			3334.7
ST-8		838.2	3949.4
ST-7		1569.6	4478.8
ST-6		2194.5	4924.5

ST-5	453.0	2713.2	5287.9
ST-4	815.4	3126.2	5571.1
ST-3	1087.2	3434.2	5776.4
ST-2	1268.4	3637.7	5906.9
ST-1	1359.0	3738.0	5967.0

#### Step Four: Determination of Every Storey's Brace Cross-Section

The following table gives the least calculated buckling capacity ( $P_b$ ) for the frames on various levels:

Table 9: the least calculated buckling capacity

Storeys	$(P_{bi} = \frac{1.25R_y V_i}{\cos \alpha} (kg))$		
	5-storey building's frame	8-storey building's frame	12-storey building's frame
ST-12			1622.0
ST-11			3093.7
ST-10			4416.6
ST-9			5592.5
ST-8		1466.3	6623.4
ST-7		2745.7	7511.2
ST-6		3838.7	8258.6
ST-5	961.0	4746.1	8868.1
ST-4	1729.7	5468.7	9343.0
ST-3	2306.3	6007.3	9687.3
ST-2	2690.7	6363.4	9906.1
ST-1	2882.9	6538.8	10007.0

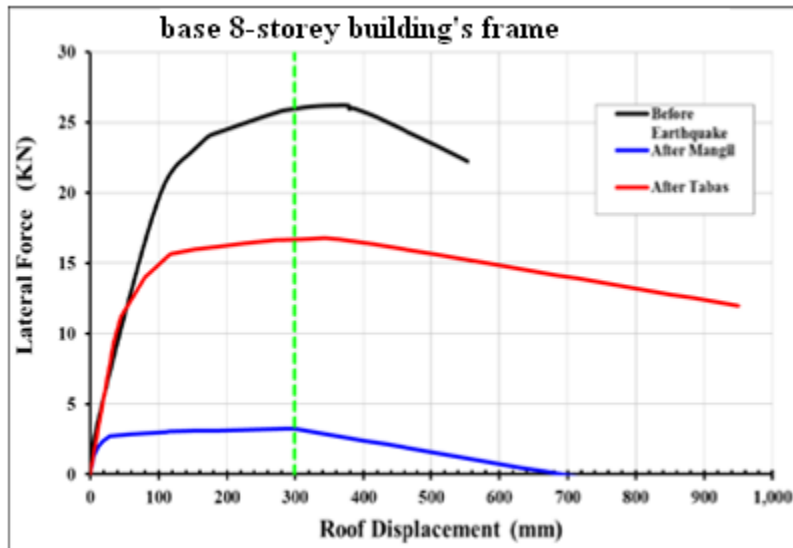
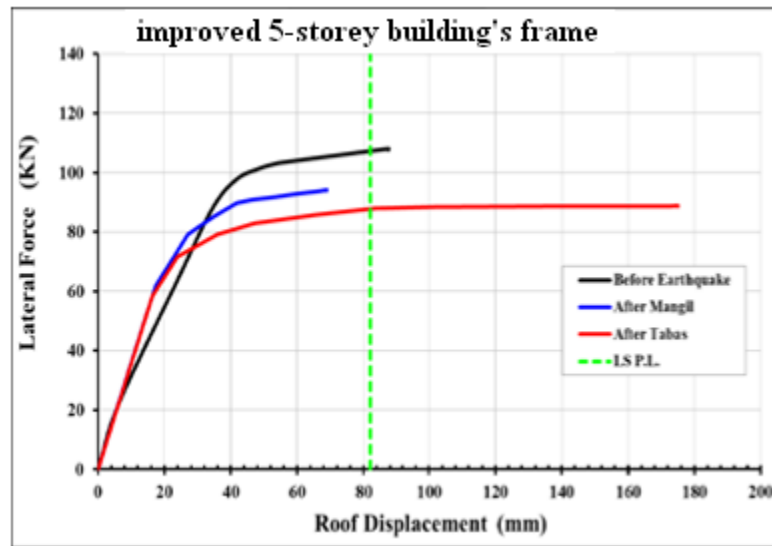
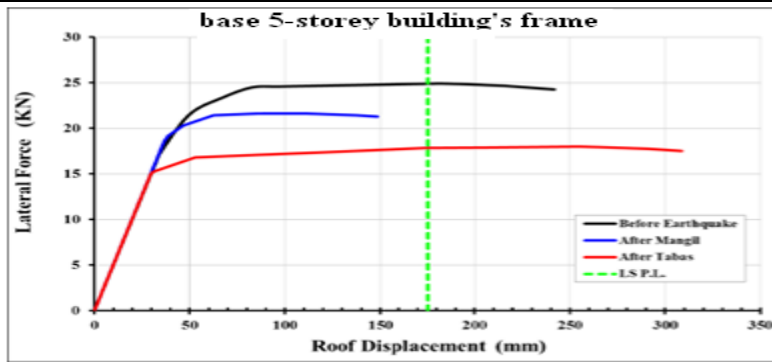
Now, the method existing in paragraph 4-4-2-10 in tenth chapter, page 48, is applied for calculating the nominal load-resisting rate of the singular braces with equal two-winged angle bar cross-section. Due to the fact that the lengths of the braces are different in 3-storey, 8-storey and 12-storey buildings' frames, the effective length of the equal two-winged angle bar cross-section can resist a given amount of nominal load in all the three frames of the 5-storey building subject to buckling.

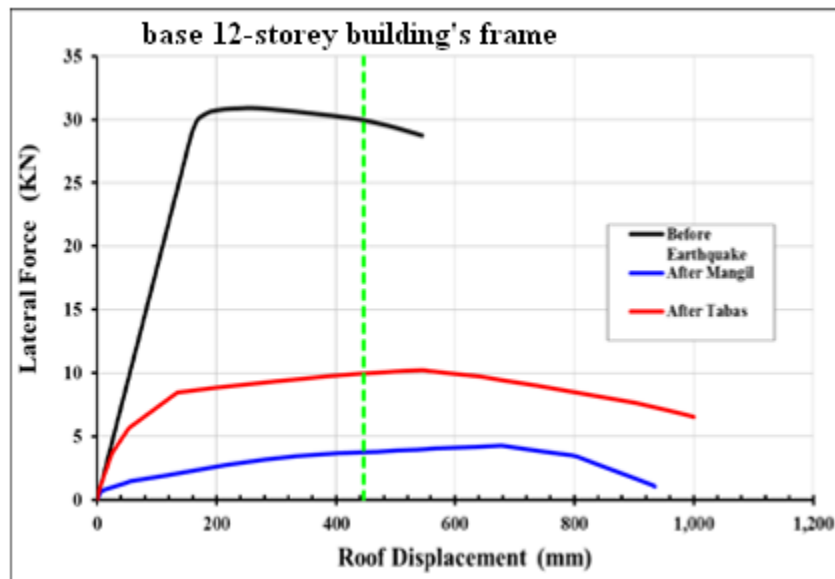
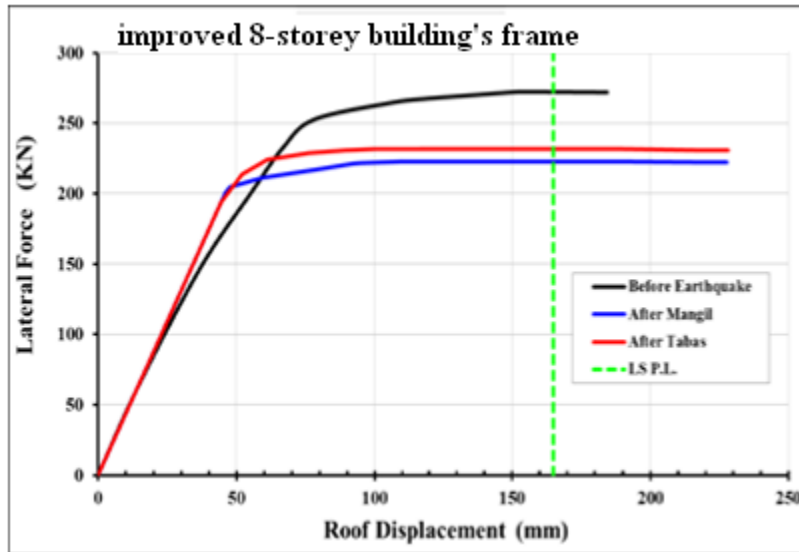
#### Findings:

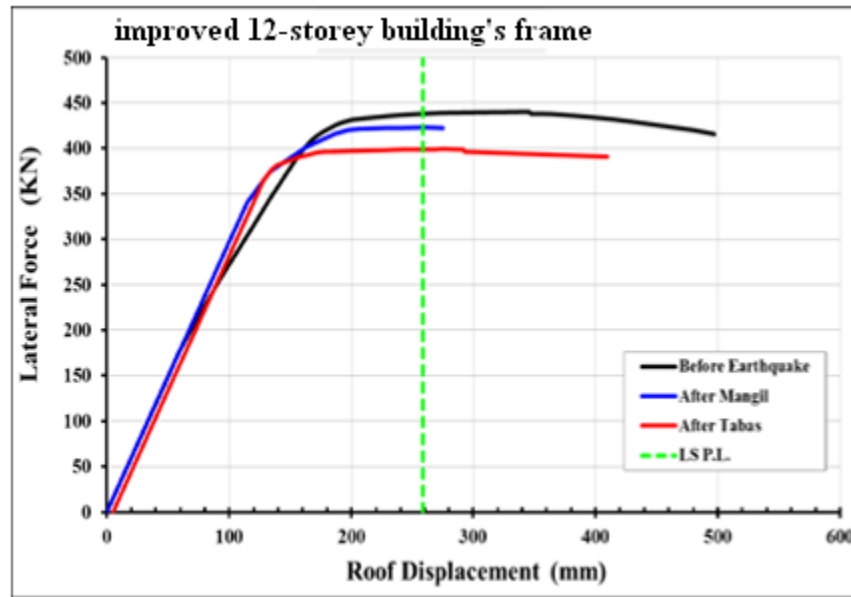
##### Seismic Damage Evaluation:

In the following figures, the capacity curves have been displayed for the displacements of the roof under and after the exertion of Tabas and Manjil earthquakes' base shear forces on the frames applied in 5-, 8- and 12-storey buildings in both base and improved states. As expected, the frames' force-resisting capacity is gradually decreased following the exertion of the earthquakes' seismic forces onto them as compared to the state without the exertion of them.









The analytical results of the frames' seismic damages have been demonstrated in the following figure based on the  $D \times g$  damage index. As it is clear, the amount of the damage index is close to unity for the base buildings in BSE-1 earthquake level and this is reflective of the overturn or collapse possibilities; but, the damage index is below unity and within an acceptable level for the buildings improved as proposed herein due to the increase in the stiffness, ductility and lateral load-bearing capacity of the base buildings.

Moreover, the following table presents the final resistance rates before and after the exertion of Tabas and Manjil earthquakes' seismic forces onto the frames in both base and improved states. Table 10: the final resistance rates before and after the exertion of Manjil and Tabas seismic forces

5-storey building's frame									
Base					Improved				
F <sub>0</sub> (KN)	F <sub>final</sub> (KN)		D × g F <sub>0</sub> (KN)		F <sub>final</sub> (KN)	F <sub>final</sub> (KN)		D × g	
	Manjil	Tabas	Manjil	Tabas		Manjil	Tabas	Manjil	Tabas
24.901	21.619	17.366	0.132	0.303	107.656	93.95	88.553	0.127	0.177

Table 11: the final resistance rates before and after the exertion of Manjil and Tabas seismic forces

8-storey building's frame									
Base					Improved				
F <sub>0</sub> (KN)	F <sub>final</sub> (KN)		D × g F <sub>0</sub> (KN)		F <sub>final</sub> (KN)	F <sub>final</sub> (KN)		D × g	
	Manjil	Tabas	Manjil	Tabas		Manjil	Tabas	Manjil	Tabas
26.194	3.213	16.715	0.877	0.362	271.96	221.95	230.57	0.184	0.152

Table 12: the final resistance rates before and after the exertion of Manjil and Tabas seismic forces

12-storey building's frame									
Base					Improved				
F <sub>0</sub> (KN)	F <sub>final</sub> (KN)		D×g F <sub>0</sub> (KN)		F <sub>final</sub> (KN)	F <sub>final</sub> (KN)			D×g
	Manjil	Tabas	Manjil	Tabas		Manjil	Tabas	Manjil	
30.774	4.192	10.299	0.864	0.665	438.062	422.663	384.47	0.035	0.122

### Conclusion:

Considering the models and their outputs as well as the modeling hypotheses for the models proposed herein, although the use of the analytical methods is a proper solution for the investigation of the structures' performances, it is by the calculation of the failure index that the structures' failures and statuses can be detected and identified. The results of the 5-storey building's frame indicated that the vulnerability index has been reduced about 58% for the improved buildings in contrast to the base ones. The results obtained for the 8- storey building's frame showed that the vulnerability index has been reduced for the improved building by about 80% in comparison to the base building. The results obtained for the 12- storey building's frame signified that the vulnerability index has been reduced by about 82% for the improved building as compared to the base one.

Considering the outputs and the diagrams for the 5-storey building, the structures have been transformed from DC State (limited failure) to IO state (limited usability). As for the 8-storey building, the results indicated that the structure has been transformed from CP state (collapse threshold) in the base form to IO state (incessant usability) in improved form. In the 12-storey building, as well, the structure has been found transformed from CP state (collapse threshold) in base form to IO state (incessant usability) in improved form.

It is suggested that further investigations should make use of improvement instruments like dampers and other kinds of bracing that have not been applied in this research paper.

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