Optimum Design of Eccentrically Braced Frames Using Endurance Time Method

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SUMMARY:
In this paper an optimization procedure is presented based on the concept of uniform distribution of deformation using a Shear Building Model (SBM). This SBM has an equivalent lateral stiffness in each story with a given Eccentrically Braced Frame (EBF). The SBM can facilitate non-linear time history analysis and be almost as accurate as a detailed model of EBF. This accuracy is controlled by performing different IDA and pushover analyses on EBFs and recording their equivalent SBMs responses like maximum roof displacement. Comparing these responses shows appropriate consistency between EBFs and the proposed SBMs. Many time history analyses are conducted on SBMs resulting in the uniform distribution of link rotation. The corresponding frames are then determined and verified for their similar behaviours. The optimization procedure is performed using seven earthquake records as well as Endurance Time method. The obtained results are then compared.

Keywords: Eccentrically Braced Frame; Shear-Building Model; IDA; Optimization; Endurance Time Method.

1. INTRODUCTION

Eccentrically Braced Frames (EBFs) are effective lateral load resisting systems for buildings in seismic regions. Their performances under seismic loading depend on the behavior of link beams usually designed for dissipating energy while other components remain elastic. Therefore, inelastic responses of EBFs are mostly affected by the behavior of their active links (Ricles and Popov, 1994). Popov et al. (1992), studied the seismic behavior of EBFs with improper strength distribution of links over the heights, based on the analytical and experimental evidences. Link deformation is concentrated at the lower levels as a result of incorrect proportioning of links. Consequently, excessive story drift may be developed at these levels and lead into the soft story mechanism. Popov et al. introduced a design procedure for the height-wise distribution of links strength to avoid such deficient behavior.

Karami Mohammadi et al. (2004), Moghaddam and Hajirasouliha (2005), and Moghaddam (2009) introduced an optimization method based on the uniform distribution of deformation (damage). They showed the effectiveness of their method through its application on different kind of buildings. In this method less computational effort is needed in comparison with other optimization methods. However, concerning sophisticated detailed models, the duration of nonlinear time history analyses increases and therefore the optimization process will be longer. As an alternative, analyzing Shear Building Models (SBMs) is simpler in this method and limited computational efforts are needed for complete optimization process.

Several researches have been conducted on the contribution of SBMs in nonlinear time history analyses. Lai et al. (1992) developed multi-rigid-body discrete model and examined its reliability and accuracy. Rahman and Grigoriu (1994) proposed a global hysteretic model and developed analytical relationship between the parameters of local and global hysteretic models for seismic analysis of multi-story buildings. Studying the results of numerical examples, they showed that global model can estimate accurately the seismic response and damage characteristics of structural systems. Moghadam and Hajirasouliha (2005) proposed a SBM having an equivalent lateral force-deformation behavior in each story with a given eccentrically braced frame.

In this paper an optimization procedure is presented based on the concept of uniform distribution of damage using a modified SBM. Accordingly, a new SBM is developed, taking advantage of the approach proposed by Moghaddam et al.. Several eccentrically braced frames are designed and their corresponding SBMs are determined in order to control the model's reliability. The maximum roof displacement and maximum interstory drift responses are derived from pushover and incremental dynamic analyses and compared. Many time history analyses are conducted to achieve an optimum distribution of link beam rotation in different stories of the studied models, thanks to the simplicity of SBMs. Based on the results obtained from previous stage, the corresponding frames are proportioned and examined to behave similarly as their corresponding optimized SBMs and have uniform distribution of link beam rotation in all stories. Finally, the entire procedure of optimization, applied on seven ground motion records, is repeated by the acceleration functions introduced in Endurance Time method and the obtained results are compared. Endurance Time (ET) method is a simple dynamic pushover analysis procedure that estimates the seismic performance of structure by subjecting it to a specific intensifying acceleration function introduced by Estekanchi et al. (2008).

2. SHEAR BUILDING MODELS

Based on Englekirk (1994) formulations, Richards (2010) assumed that the summation of horizontal displacements, caused by different frame component deformations, comprised the horizontal displacement of an EBF story. These deformations were: braces axial deformation (Δ_1), beam axial deformation (Δ_2), link shear deformation (Δ_3) and beam and link flexural deformation (Δ_4). Richards (2010) calculated yield displacement of each component, using AISC 2005 relations (Table 2.1). By the way, yield displacement of EBF story was obtained by summing all those components.

In this study a SBM is developed taking advantages of the concept proposed by Richards for computing story stiffness of an EBF. The elastic stiffness of each story of the building is modeled with four springs operated in series, Fig. 2.1. Each spring is representative of a component in EBFs. The properties of each spring, derived from Richards procedure, is tabulated in Table 2.1.

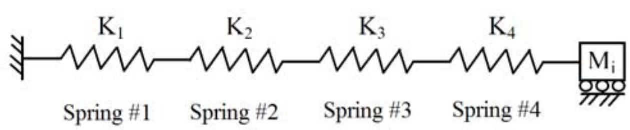

Figure 2.1. Schematic model of one story in shear building model of EBF

Correct anticipation of link beam behavior is the main concern in modeling inelastic behavior of EBFs. Different analytical models are proposed by Roder and Popov (1977), Ricles and Popov (1987, 1994) Ramadan and Ghobarah (1995) and Richards and Uang (2006). These models were used to describe the behavior of link beam during seismic excitations. The model of link element, proposed by Ramadan and Ghobarah (1995) and slightly modified by Richards and Uang (2006), appeared convenient in predicting maximum shear forces and deformation with respect to the experimental data of shear link (Okazaki et al., 2005 and Rozon et al., 2008). This element is an elastic beam element with lumped plasticity at each end (Fig. 2.2a). Three parallel bilinear springs are assigned between internal and external nodes at each end of the beam in order to achieve a multi-linear force-deformation relationship, Fig. 2.2b (Richards and Uang 2006).

Table 2.1. Components incorporated in elastic stiffness of EBFs, their corresponding yield displacements and yield strength

Spring1: representing the braces axial deformations		
Elastic stiffness	Yield displacement	Yield strength
$2E(\frac{A_d}{L_d})(\frac{a}{L_d})^2$	$if\ \lambda=\frac{KL_d}{r}\le 4.71\sqrt{\frac{E}{F_y}};\ 0.72\times(0.658^{\frac{F_y}{\frac{\pi^2E}{\lambda^2}}})F_y(\frac{L_d^2}{Ea})$ $if\ \lambda=\frac{KL_d}{r}> 4.71\sqrt{\frac{E}{F_y}};\ 0.72\times(0.658^{\frac{F_y}{\frac{\pi^2E}{\lambda^2}}})F_y(\frac{L_d^2}{Ea})$	$if\ \lambda=\frac{KL_d}{r}\le 4.71\sqrt{\frac{E}{F_y}};\ 1.44(0.658^{\frac{F_y}{\frac{\pi^2E}{\lambda^2}}})F_y(\frac{aA_d}{L_d})$ $if\ \lambda=\frac{KL_d}{r}> 4.71\sqrt{\frac{E}{F_y}};\ 1.26(\frac{\pi^2E}{\lambda^2})(\frac{aA_d}{L_d})$
Spring2: representing the beam axial deformation		
Elastic stiffness	Yield displacement	Yield strength
$\frac{2A_bE}{a}$	$if\ \lambda=\frac{KL}{r}\le 4.71\sqrt{\frac{E}{F_y}};\ (0.658^{\frac{F_y}{\frac{\pi^2E}{\lambda^2}}})F_y(\frac{a}{E})$ $if\ \lambda=\frac{KL}{r}> 4.71\sqrt{\frac{E}{F_y}};\ 0.877(\frac{\pi^2E}{\lambda^2})(\frac{a}{E})$	$if\ \lambda=\frac{KL}{r}\le 4.71\sqrt{\frac{E}{F_y}},\ 2(0.658^{\frac{F_y}{\frac{\pi^2E}{\lambda^2}}})F_yA_b$ $if\ \lambda=\frac{KL}{r}> 4.71\sqrt{\frac{E}{F_y}};\ 1.754(\frac{\pi^2E}{\lambda^2})A_b$
Spring3: representing the link beam shear deformation		
Elastic stiffness	Yield displacement	Yield strength
$G(\frac{L^2A_{bv}}{h^2e})$	$0.66\times F_y\times(\frac{he}{GL})$	$0.66F_y(\frac{LA_{bv}}{h})$
Spring4: representing the beam and link beam flexural deformation		
Elastic stiffness	Yield displacement	Yield strength
$\frac{12EIL}{h^2e^2}$	$\frac{SF_yhe}{6EI}$	$2\frac{SF_yL}{he}$

E: modulus of elasticity
G: shear modulus
F_y:material yield strength
L: span length
h: story height
L_d: diagonal length
a: distance between beam to column connection to brace to beam connection
e: link beam length

A_d: diagonal section area
r_d: minimum radius of gyration of diagonal section
A_b: beam section area
A_{bv}: beam shear section area
r_b: minimum radius of gyration of beam section
I: moment of inertia of the beam section
Z: plastic modulus of the beam section
k: effective length factor

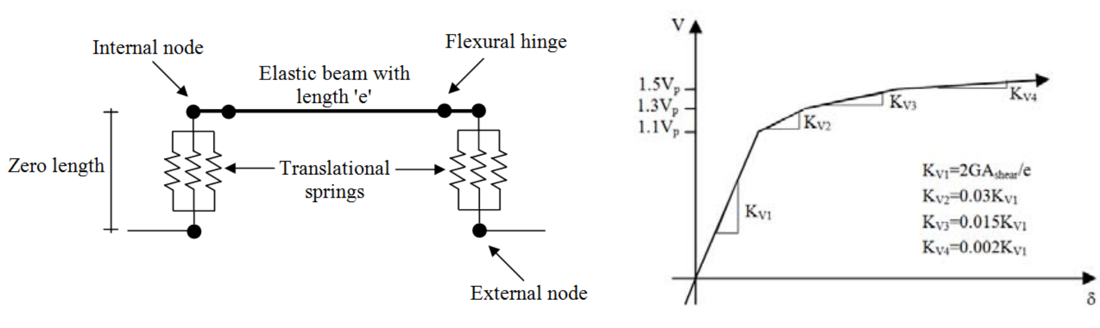

Figure 2.2. (a): EBF link element; (b): Combined behavior of parallel translational springs

According to the capacity design approach, the link is permitted to enter the plastic range; however, other members of the frame are inhibited from inelastic behavior. Therefore, the inelastic behavior of

SBM is that of spring #3, representative of link shear deformation; this spring is modeled with respect to the behavior proposed by Richards and Uang (2006), Fig. 2.2b.

The vertical axis of the curve, Fig. 2.2b, should be changed into story shear (V) instead of link beam shear (V_L) (V_L=V.L/h) in order to make the multi-linear force-deformation relationship usable for this study. The horizontal axis is also converted from vertical relative displacement of the ends of link beam (δ) into the horizontal displacement of story due to the link beam shear deformation (Δ_{bv}) ($\gamma_p=(\Delta_{bv}/h)(L/e)$). Therefore, the initial stiffness of Fig. 2.2b should be multiplied by h^2/L^2 to achieve the stiffness of link beam shear deformation, Table 2.1.

$$K = V / \Delta_{bv} = (h / L)V_L / (\delta.h / L) = (L^2 / h^2)(V_L / \delta) \tag{2.1}$$

3. VERIFYING THE SHEAR BUILDING RESPONSES

To validate the responses obtained from SBMs, 9 eccentrically braced frames with 3,5 and 7 stories and 1, 3 and 5 bays are designed using AISC Seismic Provisions (2005).

Table 3.1. Elements sections and fundamental periods of designed frames

Frames	Braces	Beams	Columns adjacent to braced span	Middle columns	Corner columns	Periods (sec.)
F3S1B	BOX140×140×7.1 (1-3)	IPE270 (1-3)	HE120 (1), HE100 (2-3)	-	-	0.284
F3S3B	BOX140×140×8 (1), BOX140×140×7.1 (2-3)	IPE300 (1), IPE270 (2-3)	HE200 (1), HE140 (2), HE100 (3)	-	HE100 (1-3)	0.465
F3S5B	BOX140×140×16(1), BOX140×140×12.5 (2), BOX140×140×7.1 (3)	IPE400 (1), IPE360 (2), IPE270 (3)	HE220 (1), HE160 (2), HE100 (3)	HE120 (1-2), HE100 (3)	HE100 (1-3)	0.46
F5S1B	BOX140×140×7.1 (1-5)	IPE270 (1-5)	HE200 (1), HE160 (2), HE140 (3), HE100 (4-5)	-	-	0.443
F5S3B	BOX140×140×16 (1-2), BOX140×140×12.5 (3), BOX140×140×10 (4), BOX140×140×7.1 (5)	IPE400 (1-2), IPE360 (3), IPE330 (4), IPE270 (5)	HE340 (1), HE260 (2), HE200 (3), HE140 (4), HE200 (5)	-	HE120 (1-2), HE100 (3-5)	0.533
F5S5B	BOX140×140×25 (1-2), BOX140×140×20 (3), BOX140×140×16 (4), BOX140×140×10 (5)	IPE500 (1-2), IPE450 (3), IPE400 (4), IPE330 (5)	HE550 (1), HE360 (2), HE240 (3), HE160 (4), HE100 (5)	HE160 (1), HE140 (2-3), HE120 (4), HE100 (5)	HE120 (1-2), HE100 (3-5)	0.555
F7S1B	BOX140×140×7.1 (1-7)	IPE270 (1-7)	HE240 (1), HE220 (2), HE180 (3), HE160 (4), HE120 (5-6), HE100 (7)	-	-	0.621
F7S3B	BOX140×140×20 (1-2), BOX140×140×16 (3-4), BOX140×140×12.5 (5), BOX140×140×8 (6), BOX140×140×7.1 (7)	IPE450 (1-2), IPE400 (3-4), IPE360 (5), IPE300 (6), IPE270 (7)	HE600 (1), HE500 (2), HE360 (3), HE260 (4), HE200 (5), HE140 (6), HE100 (7)	-	HE140 (1-2), HE120 (3-4), HE100 (5-7)	0.702
F7S5B	BOX160×160×30 (1), BOX160×160×25 (2-3), BOX140×140×25 (4), BOX140×140×20 (5), BOX140×140×16 (6), BOX140×140×10 (7)	IPE600 (1), IPE550 (2-3), IPE500 (4), IPE450 (5), IPE400 (6), IPE330 (7)	BOX400×400×25 (1), BOX360×360×25 (2), HE550 (3), HE340 (4), HE240 (5), HE160 (6), HE100 (7)	HE180 (1-2), HE160 (3), HE140 (4), HE120 (5-6), HE100 (7)	HE140 (1-2), HE120 (3-4), HE100 (5-7)	0.717

Frame name: F (Frame) – Number of stories – S (Story) – Number of bays – B (Bays)
In element description for example (1-3) means from story 1 to 3

It is assumed that the frames are located on stiff soil in a high risk seismic zone. The gravity loads on beams consist of combined 6 ton/m dead and 1.5 ton/m live loads. The story height, the span length and the link length are considered as 3, 5 and 0.5 meters, respectively. The element sections and fundamental periods of the frames are listed in the Table 3.1. Finite Element Models (FEM) of 9 mentioned frames are provided in OpenSees. The equivalent SBM of each frame is also created using the formulation presented in Table 2.1 and Fig. 2.2a. The periods obtained from FEMs and SBMs are exactly the same.

Pushover and IDA analyses are conducted on FEMs and SBMs in order to control the reliability of introduced SBM in estimating maximum responses; the responses are then compared.

3.1. Pushover Analyses

Appropriate consistency is seen between pushover analysis results of FEMs and their corresponding SBMs applied for estimating the responses of roof displacement and maximum interstory displacement, Fig. 3.1. According to the figure, the proposed SBM can almost exactly estimate the critical displacement responses of the frame through static pushover analysis.

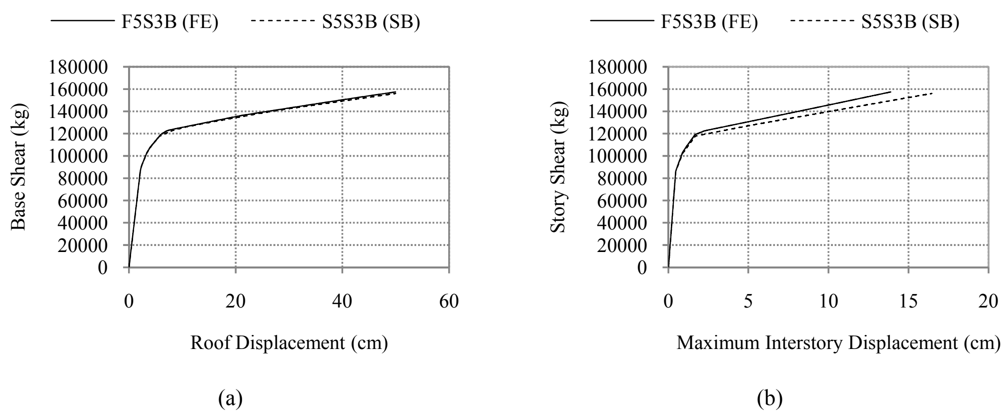

Figure 3.1. Comparing the responses obtained from pushover analysis of models F5S3B and its equivalent shear building (S5S3B): a) roof displacement; b) maximum interstory displacement

3.2. IDA Analyses

IDA analyses consist of a set of nonlinear dynamic analyses under a suite of ground motion records, scaled to several intensity levels. One of the most common intensity measures is 5%-damped first-mode spectral acceleration $S_a(T_1,5\%)$, used in this study to scale earthquake records (Vamvatsikos, 2002). The properties of 19 far-field records, used in this research, are given in Table 3.2.

Table 3.2. Ground motion properties

	Earthquake	PGA (g)	R (km)		Earthquake	PGA (g)	R (km)
1	Parkfield (1966) / TMB205	0.36	16.1	11	Morgan Hill (1984) / G06090	0.29	11.8
2	San Fernando (1971) / ORR021	0.32	24.9	12	Morgan Hill (1984) / AND250	0.42	10
3	Whittier Narrows (1987) / 116360	0.40	22.5	13	Northridge (1994) / GLP177	0.36	25.4
4	Whittier Narrows (1987)/ ALH180	0.33	13.2	14	Northridge (1994) / ORR360	0.51	22.6
5	Whittier Narrows (1987)/ OBR360	0.40	13.9	15	Northridge (1994) / MU2125	0.44	20.8
6	Landers (1992) / SP000	0.17	23.2	16	Parkfield (1966) / TMB295	0.27	16.1
7	Loma Prieta (1989) / AND270	0.24	21.4	17	Tabas (1978) / DAY-LN	0.33	17
8	Loma Prieta (1989) / GIL067	0.36	11.6	18	Imperial Valley (1979)/ SHP000	0.29	11.1
9	Loma Prieta (1989) / STG000	0.51	13	19	Kobe (1995) / NIS090	0.50	11.1
10	Loma Prieta (1989) / LOB000	0.45	17.9				

A set of IDA analysis is performed on both FEMs and SBMs and their responses are recorded for maximum roof displacements, maximum interstory drifts and base shears. The average responses of 19 records are computed for comparison purposes. The average maximum roof displacement is plotted versus spectral acceleration in Fig. 3.2 for F5S3B, S5S3B, F7S5B and S7S5B models. According to the figure, maximum roof displacements of FEMs show proper consistency with that of their equivalent SBMs. In fact, this figure shows the capability of SBMs in predicting maximum roof displacement of EBFs, with acceptable accuracy, in comprehensive nonlinear dynamic analysis. The responses' accuracies are to some extend lower in the 7 story frames comparing to those of the 3 and 5 story ones. This is because the effect of the axial deformation of columns is not considered in creating SBMs.

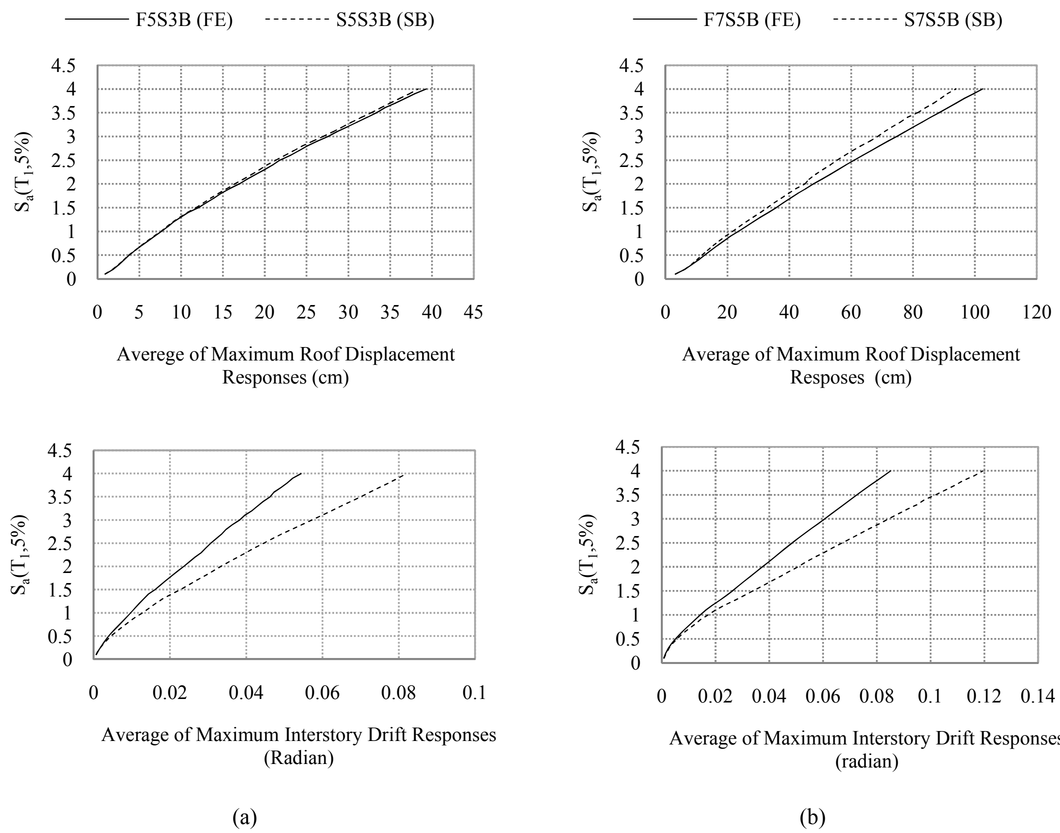

Figure 3.2. Comparing the averages of maximum roof displacement responses and maximum interstory drift in IDA analyses of the models: (a) F5S3B, S5S3B; (b) F7S5B and S7S5B; under 19 selected records

The capability of SBMs in estimating roof displacement is not exactly the same as in maximum interstory drift. Fig. 3.2 shows average maximum interstory drifts for both FEMs and SBMs. According to the figure, interstory drift responses of SBMs have good agreement with those of FEMs in lower intensity measures ($S_a(T_1,5\%)<0.5g$); however, they are overestimated by SBMs in higher intensity measures ($S_a(T_1,5\%)>0.5g$). The average error of SBMs in estimating mean maximum interstory drift ($\check{D}$), obtained from IDA analysis, is $(\check{D}_{FE}-\check{D}_{SB})/\check{D}_{FE}$ that reaches its maximum level (41%) for $S_a(T_1,5\%)=4g$, shown in Fig. 3.3. This error is less than 10% in lower intensity measures ($S_a(T_1,5\%)<0.4g$). Despite many simple assumptions, included in constructing SBMs, ignoring columns axial deformation may be considered as another reason for their inaccuracies. Besides the simplicities of proposed SBMs, the conservative results and great speed of IDA analysis make them convenient tools for time history and IDA analysis. Table 3.2 shows the elapsed time for 760 IDA analyses performed on FE and SBMs. It can be easily deduced from Table 3.2 that 760 IDA analyses conducted on 3 and 7 story SBMs take 4.4% and 1.5% of the time needed for performing them on FEMs, respectively.

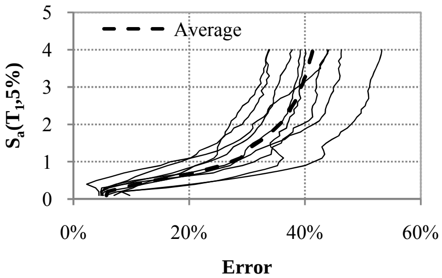

Figure 3.3. The error of SBMs in estimating the mean maximum interstory drift

Table 3.2. time needed for performing 760 IDA analyses

Number of story	FE model (hr.)	SB model (hr.)
3	8	0.35
5	20	0.43
7	34	0.5

4. EFFECTIVE SEISMIC DESIGN USING THE DEVELOPED SHEAR BUILDING MODEL

Based on the analytical and experimental evidences, Popov et al. (1992) concluded that improper design of EBFs can usually lead to excessive story drifts at the lower levels and cause their link beams to dissipate energy while other link beams remain elastic. Such deficient behavior is due to the incorrect proportioning of links over the frame's height. Therefore, they elaborated a design procedure resulting in a more uniform distribution of strength through the frame's height. In this method the ratio (α) of link shear yield capacity to the story shear is constant for all stories. This ratio is calculated by $(L/h)(V_p/V_{story})=\alpha \geq 1$, where, L and h are the span length and story height respectively; V_p is the nominal shear yield capacity of the shear link; V_{story} is the nominal shear yield capacity of story shear; α is link strength index. The nominal capacity is calculated as 0.6 times the product of steel yield stress and the web area.

In this study it is attempted to distribute uniformly the maximum rotation of the link beam using the uniform damage theory. As numerous time history analyses are needed for obtaining a uniform pattern of maximum link beam rotation, they are conducted on SBMs. In the next step, taking the results of optimized SBMs, the FEMs of corresponding EBFs are proportioned and examined whether FEMs and SBMs behave similarly.

The following optimization algorithm is adopted by MATLAB and OpenSees collaborating programs:

1. Arbitrary properties are selected for beams, braces and columns (link beams have the same properties as beams). It is assumed that the beams sections are selected from IPE sections and the braces from TUBO sections. Therefore, all formulas, presented in Table 2.1, can be expressed in terms of beam shear or brace sectional areas. For example the relation between beam moment of inertia and the beam shear area is shown in Fig. 4.1. Then the properties of springs constituting SBM are calculated and the corresponding SBM is modeled in OpenSees.

2. The natural period of modeled SBM is determined and the record selected for time history analysis is automatically scaled by MATLAB in such a way that its acceleration spectrum matches with the design spectra at the frame period.

3. Time history analyses are conducted on the SBM and the story displacements are recorded.

4. First, the story drifts are computed in order to calculate the link beam rotation with SBMs. Then, the link beam rotations are computed, using the relationship presented in AISC 2005, $\gamma_p=\delta_p.L/(e.h)$. Where, γ_p is link rotation and δ_p is story drift. Now, beam shear areas are modified according to the formula introduced by Moghaddam and Hajirasouliha (2005), $[(A_{shear})_i]_{n+1}=[(A_{shear})_i]_n[\gamma_{pi}/\gamma_{po}]^{\alpha}$. Where, γ_{pi} is the maximum link beam rotation and $[(A_{shear})_i]_n$ is the link beam shear area of the element i in step

n; $[(A_{shear})_i]_{n+1}$ is the maximum link shear area of the same element in step n+1; α is the convergence coefficient (0.1); γ_{po} is the allowable limit for link beam rotation in specified performance level.
In this equation the unexploited and extra material are allowed to move from strong region to weak one and therefore an optimum distribution of material and strength is achieved.

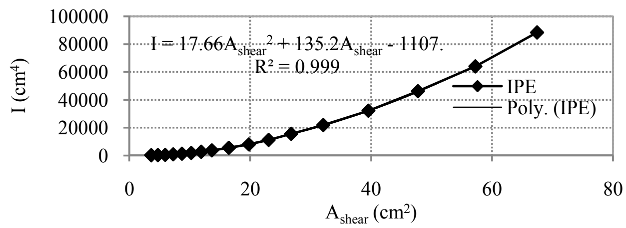

Figure 4.1. The relation between the beam moment of inertia and the beam shear area for IPE sections

5. SBM with the properties derived from step 4 is analyzed and its period is determined.

6. Step 2 to 5 are repeated until the uniform distribution of link rotations of all stories is achieved.

The results of optimizing 3, 5 and 7 story SBMs are illustrated in Fig. 4.2 for life safety limit state (γ_{po}=0.11). In all SBMs, the optimization process is conducted for each 7 first records of Table 3.1 and the average maximum link beam rotation is calculated for each story. As can be deduced from Fig. 4.2, using the theory of uniform deformation is an effective approach for meeting uniform maximum link beam rotation in all stories.

In the next step, the equivalent FEM is created using the average of seven link beam shear areas resulted in each story from optimizing SBMs. The relevant time history analyses are conducted on FEM applying the above mentioned records. The average maximum link beam rotation is calculated for each story of this model and illustrated in Fig. 4.2. According to this, the average maximum link beam rotation, resulted from 7 time history analyses of the equivalent FEM, is approximately uniform and mainly restricted to the specified limit for life safety performance level. Crossing the limit state in some stories is in a small range and can be ignored.

Here, the results deduced from this study are compared with those derived from Popov et al. method. For this purpose, an FEM is constructed having the same primary properties of the SBM with which the optimization procedure is begun. Using CQC method (Der Kiureghian, 1981), story shears and link beam shears are computed by ($V_{link}=V_{story}.h/L$) and link beam shear areas by ($V_p/V_{link}=\alpha=1$). A new FEM with the calculated properties is modeled on which the same 7 time history analyses are conducted. The average maximum link beam rotations, deduced from 7 time history analyses, is also computed and shown in Fig. 4.2. This figure shows that the procedure, adopted in this study, offers more uniform distribution of link beam rotations and higher consistency with a specified performance level, comparing to Popov et al. method.

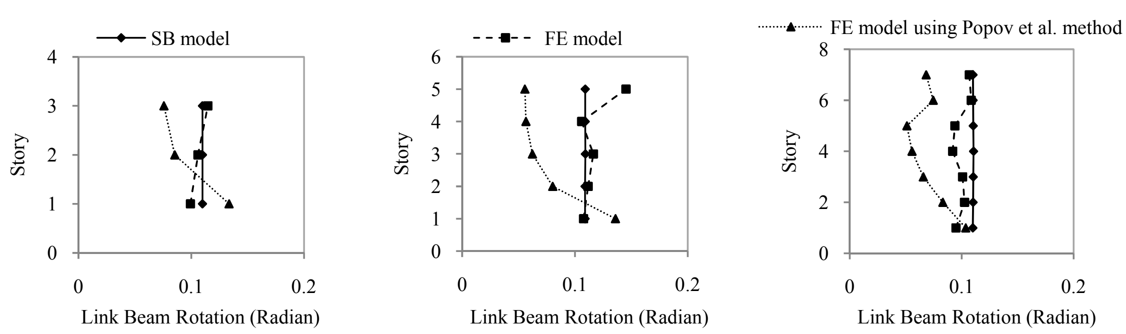

Figure 4.2. The average maximum link beam rotations over the frame's height, analyzed subjected to 7 records

5. OPTIMIZATION OF EBFS FE AND SB MODELS BY ENDURANCE TIME METHOD

Endurance time method is a simple dynamic pushover procedure that predicts the seismic performance of structure by analyzing its resilience when subjected to predesigned intensifying dynamic excitations. Major structural responses, such as displacements, drift ratios, stresses or other appropriate engineering demand parameters are monitored up to the desired limiting point where the structure collapses or failure criteria are met (Estekanchi et al., 2008).

In this study the results obtained by ET method are evaluated as well. In this regard, all 6 steps, described in the previous section, are repeated using the 3 acceleration functions defined for ET method instead of 7 ground motion records. The average maximum link beam rotation, obtained from each time history analysis, is plotted for each story of 3, 5 and 7 story frame, Fig. 5.1.

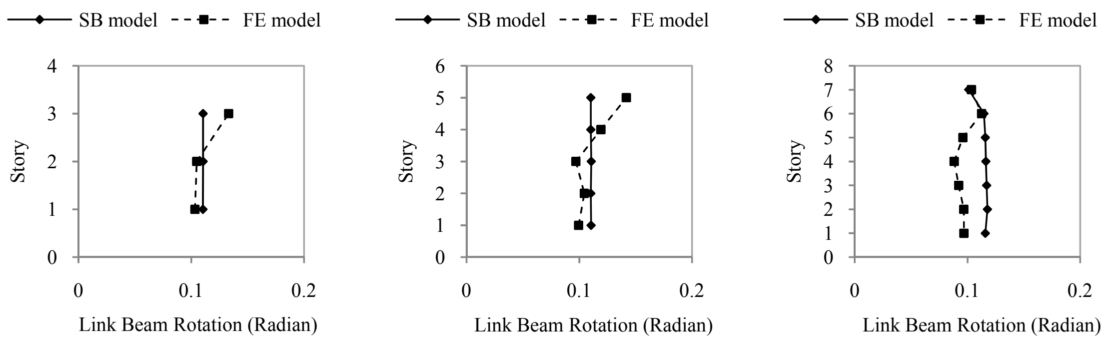

Figure 5.1. The average of maximum link beam rotations over the height of frames analyzed by 3 acc. functions

The optimization procedure is conducted using 7 ground motion records and the 3 acceleration functions in order to compare the obtained results. For this purpose, the average optimized link beam shear areas ($A_{shear\text{-}avearge}$), deduced from these 2 kinds of analysis (using 7 ground motion records, and the 3 acceleration functions) are computed and presented in Table 5.1. Concerning the averages of link beam shear areas, the results obtained from ET method for the 5 and 7 story frames are similar showing the varieties of 3% to 12% comparing to those obtained from 7 time history analyses. However, in the 3 story frame the responses have less consistency showing the differences of 30%.

Table 5.1. The average link beam shear area resulted from time history analysis conducted on 7 ground motion records and the 3 acceleration functions

	Story No.	$A_{shear\text{-}avearge}$ – 7re. (cm^2)	$A_{shear\text{-}avearge}$ – 3ac. (cm^2)		Story No.	$A_{shear\text{-}avearge}$ – 7re. (cm^2)	$A_{shear\text{-}avearge}$ – 3ac. (cm^2)
3story frame	1	31.41	39.63	7story frame	1	53.96	52.51
	2	22.30	29.05		2	44.17	42.60
	3	13.02	16.52		3	36.64	35.03
5story frame	1	47.46	42.88		4	30.17	28.43
	2	37.67	34.70		5	23.74	22.63
	3	29.53	27.57		6	17.41	16.74
	4	21.71	20.10		7	11.15	9.84
	5	12.51	11.34				

6. CONCLUSION

Here, an effective EBF design procedure is developed using a simple yet effective shear building model. The responses of the complex finite element models and their corresponding shear building models are compared in pushover analysis. Accordingly, the proposed shear building model is accurate enough in predicting base shear versus roof displacement behavior comparing with the corresponding results of pushover analysis of finite element model.

This shear building model can be a proper alternative of detailed (finite element) model of EBF for estimating maximum roof displacement especially in time consuming non-linear analysis (IDA). The interstory drift responses of shear building models are in good agreement with those of finite element models in low intensity earthquakes but they are overestimated in higher intensity ones.

The required strength and stiffness of link beams are determined, using SB models, through a simple optimization process in order to achieve a uniform height-wise distribution of maximum links' rotation during the design earthquake. The optimized shear building model and its corresponding finite element model behave consistently. Moreover, the link beams of optimized model tolerate approximately uniform rotations. The results of optimization procedure, deduced from Endurance Time method, are in good agreement with those of time history analyses regarding 5 and 7 story frames. In case of 3 story frame this consistency decreases.

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