MODAL STRENGTH REDUCTION FACTORS FOR THE SEISMIC DESIGN OF ECCENTRICALLY BRACED STEEL FRAMES

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1. ABSTRACT

A performance-based seismic design method for plane eccentrically braced steel frames (EBF) is proposed. The method is a force-based seismic design one utilizing not a single strength reduction factor as all modern codes do, but different such factors for each of the first significant modes of the frame. These modal strength reduction factors incorporate dynamic characteristics of the structure, different performance targets and different soil types. Thus, the proposed method can automatically satisfy deformation demands at all performance levels without requiring deformation checks at the end of the design process, as it is the case with code-based design methods. Empirical expressions for those modal strength reduction factors as functions of period, deformation/damage and soil types, which can be used directly in conjunction with the conventional elastic pseudo-acceleration design spectra with 5% damping for seismic design of steel EBFs, are provided. These expressions have been obtained through extensive parametric studies involving nonlinear dynamic analyses of 56 frames under 100 seismic motions. The method is illustrated by an example which demonstrates its advantages over code-based seismic design methods.

2. INTRODUCTION

According to force based seismic design, which is used by the vast majority of seismic design codes for buildings around the world, base shear results from the elastic design spectrum divided by a constant behavior factor, related to the material and type of a structure. The aforementioned factor essentially empirical and does not include the dynamic characteristics of a frame. For that reason in past decades, various researchers have proposed more elaborate values of behavior factor that are based on numerous parametric studies [1,2]. The present paper comes as a continuation of previous work by Papagiannopoulos and Beskos (2010) [3,4] on the development of a new seismic design method for plane moment resisting frames (MRF). It extends that method to eccentrically braced frames (EBF) made of steel. The basic concept of the method is related to the determination of an equivalent elastic structure that retains the mass and initial stiffness of the original nonlinear one, and has high amounts of viscous damping in order to balance the nonlinear energy of dissipation by the viscous one. By making use of appropriate equivalent modal damping ratios ξ_j dependent on deformation, a balance (equivalence) between the damping work and that of non-linearities is achieved. Thus, the non-linear structure becomes an equivalent linear structure with equivalent viscous modal damping ratios, which are computed with the aid of a transfer function expressed in the frequency domain. Through those equivalent damping ratios ξ_j for the first few modes of significance, one can easily obtain the corresponding modal strength reduction factors q_j for a wide range of natural periods. Finally, modal strength reduction factors can be easily used in conjunction with elastic acceleration spectra for the determination of the design base shear force of the frame.

3. DESIGN OF FRAMES CONSIDERED

This work deals with plane eccentrically braced frames (EBFs) of chevron (Fig. 1.a) and diagonal (Fig. 1.b) configuration. Frames of 2,3,6,9,12,15 and 17 storeys and two types of stiffness are considered. These frames have either long or intermediate seismic links of 1,5 m and 1,0 m respectively (segment x in Fig. 1). Thus, in total there are 2x2x2x7 = 56 different frames. Vertical load is 27,5 KN/m which corresponds to the G+0,3Q combination. Steel sections are HEB for columns, IPE for beams and CHS for braces. The grade of steel is S275. In EBFs seismic links are created to dissipate energy by the formation of plastic bending or plastic shear mechanism, while the other members are designed to remain elastic. According to the length and type of failure, seismic links are categorized by EC8 [5] (Table 1) where α is equal to the ratio of the smaller over the larger absolute value of the moment or shear force at both ends.

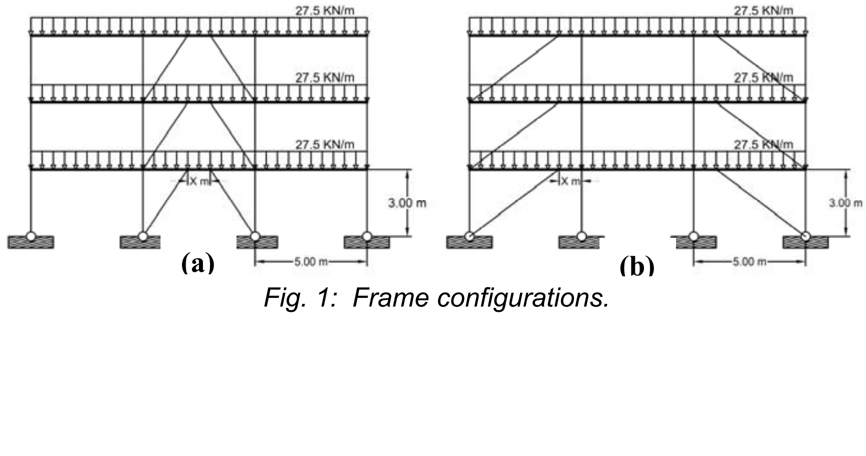

Fig. 1: Frame configurations.

Categorization	Equal moments at both ends	Non- Equal moments at both ends
Sort Links	$\mathrm{e} < \mathrm{e_s} = 1{,}6\mathrm{M_{p,link}}/\mathrm{V_{p,link}}$	$\mathrm{e} < \mathrm{e_s} = 0{,}8(1+\alpha)\mathrm{M_{p,link}}/\mathrm{V_{p,link}}$
Long Links	$\mathrm{e} > \mathrm{e_L} = 3{,}0\mathrm{M_{p,link}}/\mathrm{V_{p,link}}$	$\mathrm{e} > \mathrm{e_L} = 1{,}5(1+\alpha)\mathrm{M_{p,link}}/\mathrm{V_{p,link}}$
Intermediate Links	$\mathrm{e_s} < e < \mathrm{e_L}$	$\mathrm{e_s} < e < \mathrm{e_L}$

Table. 1: Categorization of seismic links.

The above mentioned frames are designed according to EC3and EC8 provisions for an elastic spectrum of soil type B and PGA equal to 0,24g. A behavior factor equal to 4 that corresponds to a medium ductility class design, has been chosen. Seismic design of frames was achieved through SAP2000 [6].

4. MODELING OF FRAMES CONSIDERED

Beam members are simulated on the basis of Giberson one component model, with the consideration of two rotational springs instead of the plastic hinges at both ends and without taking into account the interaction between moment capacity and axial force. Braces are also simulated with the before mentioned model with elastic hinges. Columns are simulated by using the steel beam-column model that takes under consideration the interaction of moment capacity and axial force. All members abide by a bilinear hysteretic rule with a bilinear factor of 0,03 for beams and columns and 0,012 for braces. Rigid links in a beam-column-brace connection are also considered upon the intersected members due to the influence of gusset plates [7,8]. Panel zone action is taken under consideration via the scissor model, only in nodes where their rotation is not suppressed by any gusset plate [9].

5. DETERMINATION OF MODAL BEHAVIOUR FACTORS

The determination of modal behaviour factors requires first the determination of modal damping ratios through extensive parametric analyses of the 56 frames considered. Nonlinear dynamic analyses of the frames performed with Ruaumoko2D [10] for 100 seismic excitations. With appropriate scaling of the ground motions, frames are able to reach certain damage levels in terms of interstorey drift ratio or maximum member ductility. According to SEAOC [11] those damage levels are directly related to various performance levels. The 100 seismic excitations are divided in four groups corresponding to soil types of EC8. After scaling of the seismic excitations, non-linear dynamic analysis of the same frames takes place once again for a given scale factor per seismic excitation and damage level, and for an incremental increasing of linear Rayleigh damping ratio of the system, until the corresponding transfer function which is expressed in frequency domain, meets certain smoothness criteria (Fig. 2).

The values of the transfer function modulus come from the ratio of max acceleration at the top of a frame, over the acceleration at its base in every time step. In case that a mode (peak of the transfer function) meets the smoothness criteria while the others not, then the transfer function modulus $|R_r(\omega)|$ and the corresponding resonant frequency ω_j are kept, and the increasing of the damping ratio of the system continues until all peaks of the modulus of the transfer function will become smooth. When the transfer function becomes completely smooth, one is able to utilize all couples of $(|R_j|,\omega_j)$ along with the corresponding participation factors Γ_j and the value of modal matrix φ_j that corresponds to the top of the frame, in order to solve a system of non-linear algebraic equations [3] for the values of modal damping ratios ξ_j.

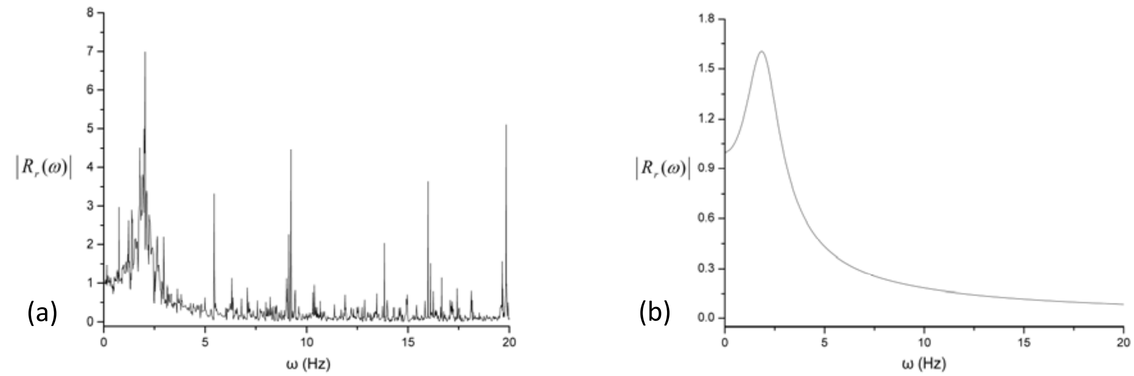

Fig. 2: (a) non-linear transfer function for damping ratio of a system equal to ξ=12% *(b) linearized transfer function for damping ratio of a system equal to ξ=41,1%*

$$
\begin{aligned}
|R_r(\omega)|^2 = 1 &+ 2\sum_{j=1}^{N} \frac{\varphi_{rj}\cdot\Gamma_{rj}\cdot\omega^2\cdot(\omega_j^2-\omega^2)}{(\omega_j^2-\omega^2)^2+(2\cdot\xi_j\cdot\omega_j\cdot\omega)^2} \\
&+ \sum_{j=1}^{N} \frac{\left[(\varphi_{rj}\cdot\Gamma_{rj}\cdot\omega^2)^2\cdot(\omega_j^2-\omega^2)^2+(2\cdot\xi_j\cdot\omega_j\cdot\omega)^2\right]}{\left[(\omega_j^2-\omega^2)^2+(2\cdot\xi_j\cdot\omega_j\cdot\omega)^2\right]^2} \\
&+ 2\sum_{j\neq k,k>j}^{N} \frac{\varphi_{rj}\Gamma_{rj}\varphi_{rk}\Gamma_{rk}\omega^4\cdot\left[(\omega_j^2-\omega^2)(\omega_k^2-\omega^2)+4\xi_j\xi_k\omega_j\omega_k\omega^2\right]}{\left[(\omega_j^2-\omega^2)^2+(2\xi_j\omega_j\omega)^2\right]\cdot\left[(\omega_k^2-\omega^2)^2+(2\xi_k\omega_k\omega)^2\right]}
\end{aligned} \tag{1}
$$

By exploiting the lower bounds of ξ_j values a relationship between modal damping ratio and natural period T for the first four significant modes is established. Finally, the corresponding modal behavior factors q_j can be calculated by dividing the ordinates of the elastic mean absolute spectrum of the excitations (ξ_{el}=5%) over the mean absolute spectrum that corresponds to ξ_j for a given natural period T_j.

$$
q_j = \frac{V_{el,j}}{V_{y,j}} = \frac{M_j^*\cdot S_{a,j}(T_{acc,j},\xi_{5\%})}{M_j^*\cdot S_{a,j}(T_{acc,j},\xi_{eq,j})} = \frac{S_{a,j}(T_{acc,j},\xi_{5\%})}{S_{a,j}(T_{acc,j},\xi_{eq,j})} \tag{2}
$$

Table 2 provides explicit expressions for modal behaviour factors. For the case that a couple of mode and damage limit is not included in Table (2), or for out of range natural period values, we consider this particular mode *j* as overdamped, and hence the corresponding modal behavior factor can be calculated from Eq. 2 for ξ_j=100%.

6. NUMERICAL EXAMPLE

An 8-storey chevron EBF with long links, is designed for the Life Safety (LS) performance level by making use of both EC8 (assuming q equal to 4) and the proposed method. The elastic spectrum is for soil type B and characterized by a PGA equal to 0.36g. For the needs of the proposed design method, modal reduction factors can be obtained from Table. (2) and Eq. (2). By assuming initial sections for which T_1=1,19sec, T_2=0,42sec, T_3=0,24sec, T_4=0,16sec, one can calculate q_1=3,36, q_2=4,1 q_3=3,4, q_4=3,0 and then insert these values in SAP2000 [6] in the form of a modified spectrum Fig. (3). Using modal synthesis one can determine the base shear equal to 425.01KN and from there dimension the frame as shown in Table 3.

Frame type	mode	Damage (IDR/μ)	Modal behavior factor	Range of natural period
Chevron EBF, Intermediate Seismic Links	1^{η}	0,0085/2,3	$q_1 = -0{,}26T_1 + 1{,}65$	$0{,}20 < T_1 < 1{,}90$
		0,013/3,6	$q_1 = -0{,}24{T_1}^2 + 0{,}52T_1 + 1{,}99$	$0{,}20 < T_1 < 1{,}90$
		0,022/6,2	$q_1 = -0{,}74{T_1}^2 + 1{,}65T_1 + 2.70$	$0{,}20 < T_1 < 1{,}90$
	2^{η}	0,0085/2,3	$q_2 = 1{,}10{T_2}^2 - 1{,}26T_2 + 1.37$	$0{,}20 < T_2 < 0{,}64$
		0,013/3,6	$q_2 = 0{,}07T_2 + 1{,}22$	$0{,}32 < T_2 < 0{,}64$
	3^{η}	0,0085/2,3	$q_3 = 1{,}10{T_3}^2 - 1{,}26T_3 + 1.37$	$0{,}13 < T_3 < 0{,}34$
	4^{η}	0,0085/2,3	$q_3 = -0{,}26T_3 + 1{,}65$	$0{,}13 < T_4 < 0{,}34$
Diagonal EBF Intermediate Seismic Links	1^{η}	0,0085/2,3	$q_1 = -0{,}15T_1 + 1{,}48$	$0{,}20 < T_1 < 1{,}70$
		0,013/3,6	$q_1 = -1.1T_1^3 + 3{,}1{T_1}^2 - 2{,}72T_1 + 2{,}65$	$0{,}20 < T_1 < 1{,}70$
		0,022/6,2	$q_1 = -1{,}4{T_1}^2 + 2{,}74T_1 + 2{,}13$	$0{,}20 < T_1 < 1{,}70$
	2^{η}	0,0085/2,3	$q_2 = 0{,}73T_2 + 1{,}43$	$0{,}26 < T_2 < 0{,}55$
		0,013/3,6	$q_2 = 0{,}73T_2 + 1{,}66$	$0{,}33 < T_2 < 0{,}55$
	3^{η}	0,0085/2,3	$q_3 = -1{,}2T_3 + 1{,}43$	$0{,}20 < T_3 < 0{,}29$
Chevron EBF, Long Seismic Links	1^{η}	0,0085/2,3	$q_1 = -0{,}28T_1 + 1{,}53$	$0{,}30 < T_1 < 1{,}70$
		0,013/3,6	$q_1 = -0{,}49{T_1}^2 + 0{,}42T_1 + 2{,}14$	$0{,}30 < T_1 < 1{,}70$
		0,022/6,2	$q_1 = 1{,}86{T_1}^3 - 7{,}08{T_1}^2 + 7{,}39T_1 + 1{,}46$	$0{,}30 < T_1 < 1{,}70$
	2^{η}	0,0085/2,3	$q_2 = -0{,}26T_2 + 1{,}16$	$0{,}26 < T_2 < 0{,}60$
		0,013/3,6	$q_2 = 81{,}8{T_2}^3 - 106{,}6{T_2}^2 + 43{,}9T_2 - 4{,}34$	$0{,}34 < T_2 < 0{,}60$
	3^{η}	0,0085/2,3	$q_3 = -0{,}42T_3 + 1.13$	$0{,}14 < T_3 < 0{,}35$
		0,013/3,6	$q_3 = 479{,}1{T_3}^3 - 392{,}4{T_3}^2 + 103{,}8T_3 - 7{,}56$	$0{,}24 < T_3 < 0{,}35$
Diagonal EBF Long Seismic Links	1^{η}	0,0085/2,3	$q_1 = -0{,}08T_1 + 1{,}59$	$0{,}22 < T_1 < 1{,}50$
		0,013/3,6	$q_1 = -2{,}34{T_1}^2 + 3{,}78T_1 + 1{,}3$	$0{,}22 < T_1 < 1{,}50$
		0,022/6,2	$q_1 = -3{,}28{T_1}^3 + 5{,}65{T_1}^2 - 1{,}13T_1 + 2{,}93$	$0{,}22 < T_1 < 1{,}50$
	2^{η}	0,0085/2,3	$q_2 = 8{,}2{T_2}^2 - 7{,}33T_2 + 2{,}74$	$0{,}25 < T_2 < 0{,}48$
		0,013/3,6	$q_2 = -0{,}85T_2 + 1{,}84$	$0{,}35 < T_2 < 0{,}48$
	3^{η}	0,0085/2,3	$q_3 = -1{,}2T_3 + 1{,}43$	$0{,}19 < T_3 < 0{,}26$
		0,013/3,6	$q_3 = 0{,}8T_3 + 1{,}19$	$0{,}22 < T_3 < 0{,}26$

Table. 2: modal behavior factor for EBFs and soil type B.

	EC8			Proposed Method		
	HEB	IPE	CHS	HEB	IPE	CHS
1st	360	330	168.3x4	400	360	193.7x4.5
2nd	360	300	152.4x4	400	330	168.3x4
3rd	320	300	152.4x4	360	300	152.4x4
4rth	320	300	139.7x4	360	300	139.7x4
5th	280	300	139.7x4	320	300	139.7x4
6th	280	300	127x4	280	300	127x4
7th	260	270	114.3x3.6	260	270	114.3x3.6
8th	260	270	108x3.6	260	270	108x3.6

Table. 3: Section per storey level.

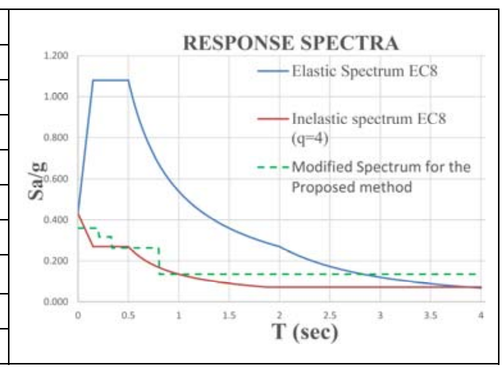

Fig. 3: Design response spectra.

		EC8				Proposed Method			
	Motion	V_y (KN)	IDR_u	Θu_{link} ($rad \cdot 10^{-3}$)	μ	V_y (KN)	IDR_u	Θu_{link} ($rad \cdot 10^{-3}$)	μ
Nonlinear dynamic analysis	1	479,02	0,0106	25,63	3,23	536,74	0,0117	15,26	2,92
	2	494,71	0,0097	22,74	2,85	566,80	0,0112	15,74	2,87
	3	489,10	0,0131	32,94	4,15	557,40	0,0091	19,50	2,68
	4	493,70	0,0147	34,90	4,58	591,47	0,0099	20,21	2,86
	5	468,30	0,0130	31,23	3,96	453,18	0,0120	17,18	3,04
	6	500,65	0,0097	22,59	2,86	473,46	0,0100	13,29	2,59
	7	503,16	0,0117	28,12	3,54	622,50	0,0117	19,71	3,19
	8	505,96	0,0099	23,16	2,90	515,81	0,0118	17,03	3,01
	9	478,31	0,0138	33,54	4,23	427,20	0,0111	20,93	2,88
	10	495,51	0,0134	32,26	4,07	591,78	0,0101	24,16	3,33
	Av/ge	**490,90**	**0,0120**	**28,71**	**3,63**	**533,63**	**0,0109**	**18,30**	**2.94**
Seismic design		**376,96**	**0,0133**			**425,01**			

Table. 4: Results of the numerical example.

Table 3 also shows the dimensioning results of the EC8 method. Finally, Table 4 provides results of nonlinear dynamic analyses for both frames of Table 3 under 10 ground motions for soil type B and compatible with the elastic spectrum of EC8. Base shear values obtained from non-linear dynamic analyses, correspond to the appearance of the first plastic hinge (first yield) and are directly comparable to the base shear of seismic design. According to SEAOC, maximum IDR=2,2% and maximum member ductility μ=6,2, constitute the limits of LS. Additionally, according to EC8, the maximum long link rotation Θu_{link}=0,02 rad.

7. COMPARISON - CONCLUSIONS

Maximum IDR (0,022) and member ductility (6,2) values are not exceeded. In case of EC8, IDR of seismic design deviates 10,8% from the corresponding mean value of dynamic analyses from the side of safety. In both cases, base shear values of seismic design are lower than those of nonlinear analyses by 25%-30% and is from the side of safety. The major discrepancy between design methods is related to max link rotation, where in case of EC8 the limit of 0,02 rad is exceeded by 43,5% by the mean value of dynamic analyses. On the other hand, at the expense of small weight increases, the proposed method automatically leads to a link rotation that attains reasonable values within the permitted limits.

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ΙΔΙΟΜΟΡΦΙΚΟΙ ΣΥΝΤΕΛΕΣΤΕΣ ΣΥΜΠΕΡΙΦΟΡΑΣ ΓΙΑ ΤΟ ΣΧΕΔΙΑΣΜΟ ΠΛΑΙΣΙΩΝ ΑΠΟ ΧΑΛΥΒΑ ΜΕ ΕΚΚΕΝΤΡΑ ΣΥΝΔΕΔΕΜΕΝΟΥΣ ΔΙΑΓΩΝΙΟΥΣ

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ΠΕΡΙΛΗΨΗ

Στην παρούσα εργασία προτείνεται μέθοδος αντισεισμικού σχεδιασμού με βάση την επιτελεστικότητα για επίπεδα πλαίσια από χάλυβα με έκκεντρα συνδεδεμένους διαγώνιους (EBFs). Η προτεινόμενη μέθοδος βασίζεται στον αντισεισμικό σχεδιασμό με βάση τις δυνάμεις, και κάνει χρήση διαφορετικών συντελεστών συμπεριφοράς για κάθε μια από τις πρώτες σημαντικές ιδιομορφές, σε αντίθεση με τους σύγχρονους κανονισμούς που χρησιμοποιούν ένα καθολικό συντελεστή. Οι εν λόγω συντελεστές συμπεριφοράς εμπεριέχουν τα δυναμικά χαρακτηριστικά της κατασκευής, για διάφορες στάθμες επιτελεστικότητας και για διαφορετικούς τύπους εδάφους. Επομένως η προτεινόμενη μέθοδος μπορεί αυτομάτως να ικανοποιήσει τις απαιτήσεις παραμόρφωσης για κάθε επίπεδο επιτελεστικότητας, χωρίς να χρειάζεται έλεγχος μετακινήσεων μετά το πέρας της ανάλυσης. Παρέχονται εμπειρικές εκφράσεις των συντελεστών συμπεριφοράς συναρτήσει της ιδιοπεριόδου, της παραμόρφωσης/βλάβης και του τύπου εδάφους, ώστε να μπορούν να συνδυαστούν απευθείας με το συμβατικό ελαστικό φάσμα σχεδιασμού ψευδοεπιταχύνσεων με 5% απόσβεση για το σχεδιασμό. Αυτές οι εκφράσεις προέκυψαν ύστερα από εκτενή παραμετρική διαδικασία που περιλαμβάνει μη-γραμμικές δυναμικές αναλύσεις 56 πλαισίων για 100 σεισμικές καταγραφές.