MODAL DAMPING RATIOS FOR SEISMIC DESIGN OF STEEL BUCKLING RESTRAINED BRACED FRAMES

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1. ABSTRACT

A seismic design method for plane steel buckling restrained braced steel frames is proposed. The method works with design acceleration spectra with high amounts of damping, modal damping ratios and modal synthesis to produce the design base shear of the frame. Explicit expressions in terms of period, deformation and damage for these modal damping ratios are derived for the first few modes controlling the response and for three performance levels. This is accomplished with the concept of the equivalent linear structure of the same mass and initial stiffness of the non – linear one under consideration and extensive parametric studies involving 20 steel buckling restrained braced frames under 100 ordinary far field ground motions for four different classes of soil. Non - linear dynamic analyses of those frames produce their response to the above ground motions properly scaled to drive the frames to the desired level of damage. The resulting response databank is used to derive the abovementioned modal damping ratios. The proposed method proves to be more accurate and efficient than that of current seismic design codes.

2. INTRODUCTION

In recent years there has been a great improvement in the analysis of structures subjected to earthquakes, following the development of experiments and analytical tools. The most precise way to estimate the seismic performance of a structure is the non – linear dynamic analysis in the time domain. However, this method cannot be used in practice since it demands the usage of sophisticated softwares and is time consuming. Furthermore, non - linear dynamic analysis depends solely on the type of earthquake excitation (magnitude, period) so it cannot give a certain prediction for the performance of structures in a future excitation. For these reasons, engineers have tried to develop simplified numerical tools to replace the nonlinear dynamic analysis in the design of structures. These methods are classified into two major categories: those based on inelastic response spectra reduction factors and those based on equivalent linearization.

Recently, Papagiannopoulos and Beskos [1], developed a new seismic design method based on equivalent linearization and the construction of an equivalent linear structure with the same mass and initial stiffness of the original non – linear structure, for plane steel MRF, utilizing equivalent modal damping ratios in conjunction with absolute acceleration elastic design spectra associated with a wide range of amounts of damping (5% - 100%), in order to provide the design base shear. The construction of the equivalent linear system is accomplished by defining equivalent modal damping ratios to balance the work of material and geometrical non – linearities. These modal damping ratios are computed in terms of period and deformation/damage with the aid of extensive parametric studies. This methodology has the advantages of rationality, high accuracy and simplicity in execution over current code – based methods.

Herein, the method developed by Papagiannopoulos and Beskos [1] is extended to a new innovative system, the buckling – restrained braced systems. Despite being a relatively new system, BRB have gained a high popularity especially, in countries with high seismicity, and have been under extensive investigation [2, 3]. BRB, were firstly introduced by Wada [4] in an effort to overcome the main disadvantage of the conventional braced systems which is the unsymmetrical behavior in tension and compression. They constitute a special structural type consisting of a ductile steel core (bar) encased in a concrete filled steel tube, coated with a low friction material [2] as shown in *Figure 1b*. The encasing and concrete prevent the steel core from buckling, while the coating of the steel core prevents the axial load to be transferred to the concrete. Extensive analytical and experimental studies conducted in [4] have shown that buckling restrained braces exhibit an enhanced energy dissipation capacity, excellent ductility and vertical load capacity without a reduction in strength and stiffness, which result from the symmetrical stable hysteretic response in both tension and compression of the brace, as shown in Figure 1a.

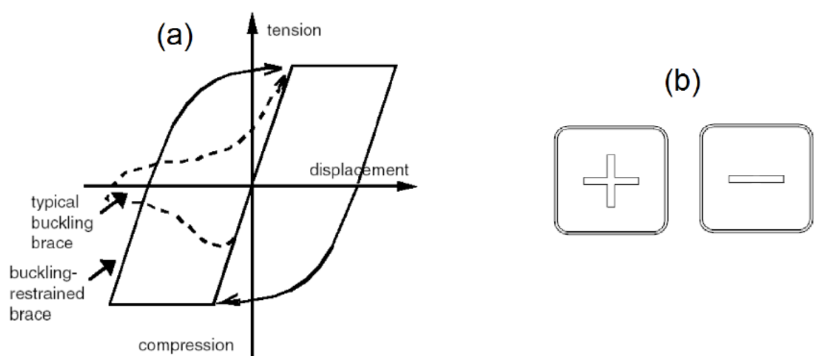

Figure 1 : (a) Comparison of the hysteretic behavior of a buckling restrained brace and a standard brace [1]; (b) Typical cross – sections of a buckling restrained brace.

3. THEORETICAL BACKGROUND

The theoretical background of the present paper is presented herein for reasons of completeness:

It has been proven in [1] that for a linear MDOF system, the modulus of the roof – to - basement frequency response transfer function $|R(\omega)|$ is given by

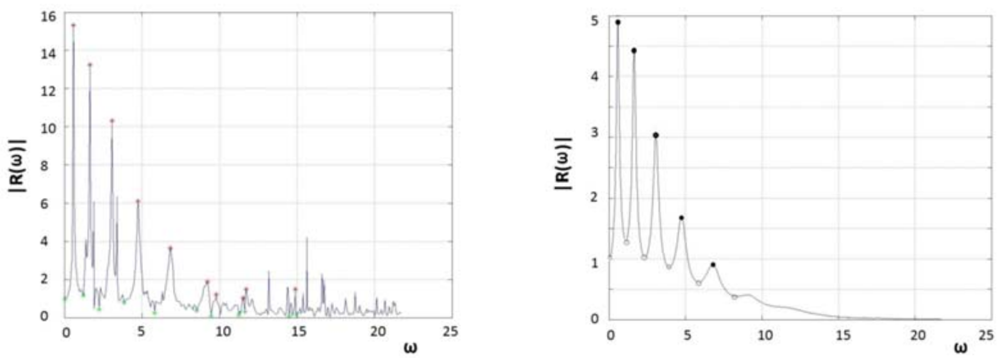

Figure 2 : Distorted and smooth shape of the transfer function |R(ω)|, of a non – linear and an equivalent linear MDOF structure respectively.

$$|R(\omega=\omega_k)|^2 = 1 + 2\sum_{j=1}^{N} \frac{\phi_{rj}\Gamma_j\omega_k^2(\omega_j^2-\omega_k^2)}{(\omega_j^2-\omega_k^2)^2+(2\xi_j\omega_j\omega_k)^2} + \sum_{j=1}^{N} \frac{\phi_{rj}^2\Gamma_j^2\omega_k^4}{(\omega_j^2-\omega_k^2)^2+(2\xi_j\omega_j\omega_k)^2} + 2\sum_{j\neq m, m>j}^{N} \frac{\phi_{rj}\Gamma_j\phi_{rm}\Gamma_m\omega_k^4\left[(\omega_j^2-\omega_k^2)(\omega_m^2-\omega_k^2)+4\xi_j\xi_m\omega_j\omega_m\omega_k^2\right]}{\left[(\omega_j^2-\omega_k^2)^2+(2\xi_j\omega_j\omega_k)^2\right]\left[(\omega_m^2-\omega_k^2)^2(2\xi_m\omega_m\omega_k)^2\right]} \quad (1)$$

where φ_{rj}, is the modal shape for the j – mode in the top floor r, ω_j and ξ_j are the undamped natural frequency and modal damping ratio of the j_{th} mode, Γ_j is the corresponding participation factor and $\bar{R}(\omega) = \bar{U}_r(\omega) / \bar{\ddot{u}}_g(\omega)$ with $\ddot{U}_r(\omega)$ being the absolute acceleration of the roof equal to $\bar{\ddot{u}}_r(\omega) + \bar{\ddot{u}}_g(\omega)$, $\bar{\ddot{u}}_g(\omega)$ being the earthquake motion in the frequency domain and overbars denoting Fourier transformation.

The shape of the transfer function modulus versus ω curve indicates if a system is linear or not since the level of jaggedness (distortion) of the transfer function corresponds to the level of non – linearity induced in the structure by the seismic excitation. A distorted and a smooth shape of the transfer function modulus corresponding to a non – linear and its equivalent linear structure can be clearly seen in *Figure 2* for the case of a 10 storey steel frame under the Cape Mendocino ground motion (25/4/1992). The values of ϕ_{rj}, ω_j and Γ_j are obtained from modal analysis, while the values of $|R(\omega = \omega_k)|$ are computed as the peaks (maxima) of the function $|R(\omega)|$ versus ω corresponding to the N resonant frequencies appearing in the transfer function. By progressively increasing the viscous damping of the non – linear structure till the work of viscous dissipation becomes equal to that of inelastic dissipation, the structure becomes an equivalent linear and its transfer function modulus attains a smooth pattern with visible peaks. At that moment, Eq. (1) is applicable.

Eq. (1), on the assumption that ϕ_{rj}, ω_j and Γ_j are known, represents a set of N nonlinear algebraic equations that is solved to provide the equivalent modal damping ratios ξ_k of the equivalent linear system. Since the structural response is practically obtained by appropriate superposition of the first few significant modes, N is equal to that number.

It is very important to note that these equivalent modal damping ratios are damage dependent and correspond to specific performance levels of damage, such as the interstorey drift ratio (IDR) or the member axial ductility. Since all non – linear structures exhibit an inherent viscous damping varying from 2% - 5% depending on the material of the structure, from the equivalent modal damping a 2% – 5% is subtracted. The viscous damping that remains corresponds solely to the material and geometrical non – linearities of the real structures.

In addition to that, one should stress that these equivalent damping ratios ξ_k, are associated with absolute acceleration response spectra and not pseudo – acceleration ones due to the presence of high amounts of damping, which result in significant damping forces that cannot be ignored. Indeed from the dynamic equilibrium of forces, the inertia (or seismic) force at the k_{th} mode is equal to the sum of linear elastic restoring and damping forces. i.e.,

$$m\ddot{u}_k^t(t) = -m[\omega_k^2 u(t) + 2\xi\omega_k \dot{u}(t)] \tag{2}$$

When damping and period are small (ξ < 10%, T < 0.15sec), the maximum acceleration can be approximately assumed to occur for vanishing velocity and hence it is only $\omega^2_k\, u\ (t)$. This leads to the pseudo – acceleration spectrum. However, when damping and period are large (ξ > 10%, T > 0.15 sec), as it is the present equivalent linear system, the maximum acceleration is equal to the maximum value of $\omega^2_k\, u\ (t) + 2\xi\omega_k\, u\ (t)$. This leads to the absolute acceleration spectrum. In *Figure 3* the absolute acceleration spectra for high amounts of damping and soil class B and C ground motions are presented for the IO performance level.

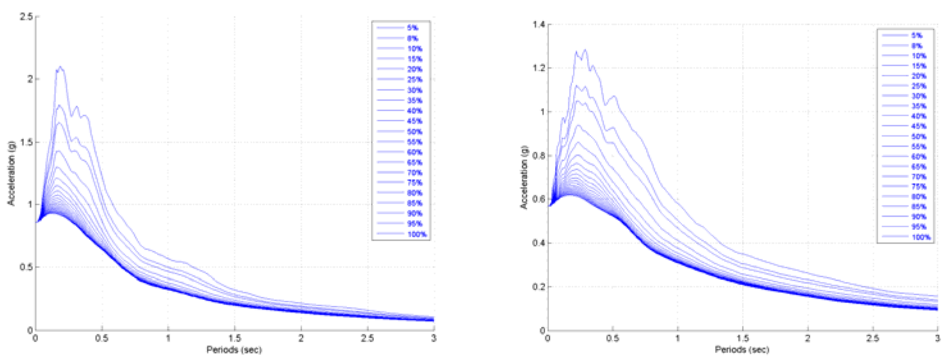

Figure 3 Absolute Acceleration Spectra for different values of damping (5% - 100%)

4. FRAME MODELLING AND ANALYSIS

The plane, steel BRB frames considered here are orthogonal and regular in elevation, with storey height equal to 3.0 m and bay width equal to 5.0 m. The number of bays is 3, while the number of stories varies with the aim of covering a wide range of periods. The frames have been designed according to EC3 [6] and EC8 [7] provisions. A design ground acceleration $a_g = 0.24g$ and a strength reduction factor $q = 6$ have been selected. The seismic loading combination consists of the gravity load G + 0.3Q on beams plus the laterally applied earthquake load, where G = 25.4 kN/m is the dead load and Q = 7 kN/m is the live load. The design of the frames is performed with the well - known software SAP2000 [8].

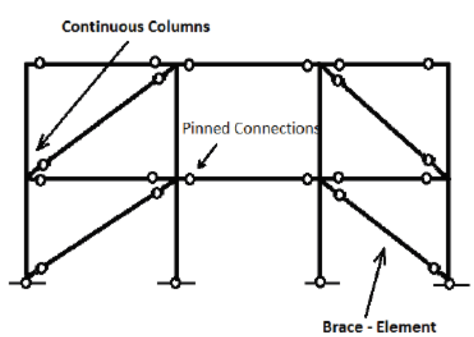

Figure 4 Geometrical configuration of a typical plane steel BRB frame

5. EQUIVALENT MODAL DAMPING RATIOS

For every pair of frame and accelerogram, the structural seismic response consisting of the equivalent modal damping ratios and the corresponding periods for the whole range of seismic intensity of every motion is recorded (*Figure 5*). These equivalent modal damping ratios are period, deformation and damage dependent. The deformation of the structure is taken into account in terms of the allowable interstorey drift ratio (IDR) while that of damage in terms of the axial ductility of the diagonals. The three performance levels considered here are associated with the following maximum IDR values: IO with 1.1%, LS with 2% and CP with 4% [9].

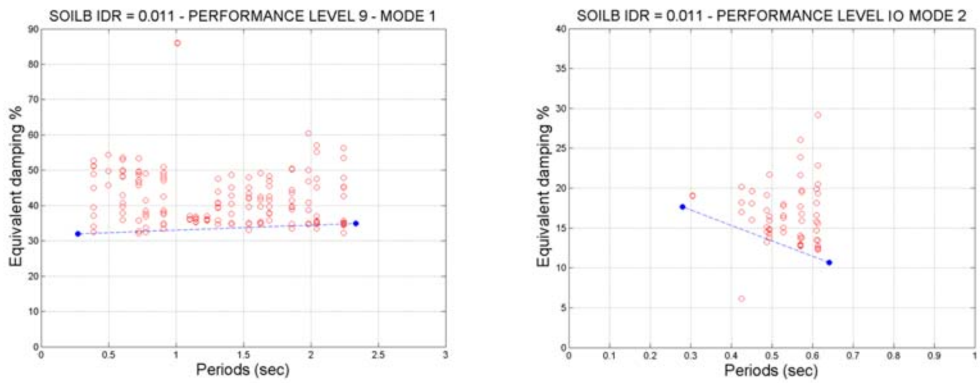

Figure 5 Modal damping ratios for Soil B. Modes 1 and 2.

The dotted line in *Figure 5* indicates the lower bound of damping ratio, for which an equivalent system is formed. Equations of these lines representing the relationship between modal damping ratios versus period are recorded and are given in the following *Table 1*. These equations are used for the design of structures in conjunction with the associated absolute response spectra and provide the maximum base shear required in order to design a structure that will perform within the predefined performance levels.

Mode	**IDR = 0.5%** $\mu = 0$	**IDR = 1.1%** $\mu = 4$	**IDR = 2.0%** $\mu = 9$
1st	$\xi_1 = 6.0$ for $0.25 < T < 2.35$	$\xi_1 = 32$ for $0.25 < T < 2.35$	$\xi_1 = 93 - 1{,}5(T - 0{,}55)$ for $0.25 < T < 2.35$
2nd	$\xi_2 = 5 - 6(T - 0.09)$ for $0.09 < T < 0.64$	$\xi_2 = 17{,}6\,T - 19(T - 0{,}28)$, for $0.25 < T < 0.64$	
3rd	$\xi_3 = 4{,}55 - 2(T - 0.09)$ for $0.07 < T < 0.33$		
4th	$\xi_4 = 4$ for $0.10 < T < 0.22$		

Table 1 Design Equations for Modal Damping Ratios vs various values of IDR and μ (SOIL B)

6. APPLICATION EXAMPLE

A steel plane buckling restrained framed structure consisting of ten stories and three bays is examined in this example. The bay length is 5.0 m, while the storey height is 3.0 m. The design of the frame is performed with the use of equivalent modal damping ratios in conjunction with the mean absolute acceleration spectrum with high values of damping and for soil class B and IO performance level. HEB profiles are used for the columns, while IPE profiles are used for the beams. The braces have a rectangular core cross - section as shown *Figure 1b*. In the seismic design, the beams are subjected to the vertical load G + 0.3Q equal to 27.5 kN/m. The steel grade of the braces and the beams is assumed equal to 235 MPa, while that of columns equal to 355 MPA. The sections of the beams are determined from the vertical loading combination (1.35G + 1.5Q), while the braces from the seismic loading combination and the restriction on θ and $P - \Delta$. The columns satisfy the capacity design

For the assumed beam, column and brace sections the first four significant periods and the corresponding modal damping ratios according to *Table 1* are: $T_1 = 1.1 sec$ with $\xi_1 = 32\%$, $T_2 sec$ with $\xi_2 = 16.1\%$, $T_3 = 0.2 sec$ with $\xi_3 = 100\%$ and $T_4 = 0.14 sec$ with $\xi_4 = 100\%$. The proposed method provides a base shear equal to 853.10kN and the resulting sections are: 1*st* storey: HEB400 - 81.12 *cm*2; 2*nd* storey: HEB400 - 67.26 *cm*2, 3*rd* storey: HEB360 - 67.26 *cm*2; 4*th* story: HEB340 - 67.26 *cm*2; 5*th* storey: HEB320 - 53.50 *cm*2; 6*th* HEB300 - 53.50 *cm*2; 7*th* storey: HEB260 - 53.50 *cm*2; 8*th* storey: HEB260 - 38.96 *cm*2; 9*th* storey: HEB240 - 37.30 *cm*2; 10*th* storey: HEB220 - 30.92 *cm*2. For these sections, the first four periods of the frame are: $T_1 = 0.88 sec$, $T_2 = 0.28 sec$, $T_3 = 0.15 sec$, $T_4 = 0.13 sec$ and the corresponding equivalent modal damping ratios are: $\xi_1 = 32\%$, $\xi_2 = 17.6\%$, $\xi_3 = 100\%$ and $\xi_4 = 100\%$. The frame is analysed and designed again and is found that these sections satisfy all the checks according to EC3 [6] and EC8 [7] provisions. Thus, convergence is achieved and the analysis is finalized. The above analysis and design of the frame is performed with the aid of SAP2000 [8] software.

The frame is also designed according to the EC8 [7] provisions involving a strength reduction factor q = to 6 and a response spectrum of type 1 with soil class B and peak ground acceleration 0.24 g. From the analysis and design of the frame, the following sections are obtained: 1*st* storey: HEB400 - 32.46 *cm*2; 2*nd* storey: HEB360 - 31.00 *cm*2; 3*rd* storey: HEB340 - 31.00 *cm*2; 4^{th}-7*th* stories: HEB300 - 31.00 *cm*2; 8*th* storey: HEB260 - 25.20 *cm*2 and 9^{th}-10*th* stories: HEB240 - 22.21 *cm*2. The maximum IDR = 0.55 %, which is significantly smaller than the permitted target value of 1.1 %. The base shear was found to be equal to 779 kN. The design of the frame by the proposed method and the method of EC8 [7] are now compared with respect to their IDR's on the basis of non - linear dynamic analyses involving 10 artificial accelerograms compatible with the response spectrum of EC8 [7] that corresponds to the IO performance level. This is obtained from the LS spectrum of EC8 [7] by multiplication of its ordinates by 0.5. The mean value of the peak interstorey drifts of the 10 non - linear time histories analyses of the frame design according to the proposed method is found to be 0.83%, which is smaller than the permitted 1.1%. The corresponding nonlinear analyses result for the frame designed with the EC8 [7] provisions is equal to 1.2% which is higher than the target value of 1.1%. This indicates that despite the fact that the EC8 design gives a peak IDR = 0.55%, the real IDR is significantly higher. This indicates that EC8, because of the employment of the equal displacement rule, underestimates the deformation and the error is from the unsafe side.

7. CONCLUSIONS

According to the preceding developments, the following conclusions can be stated:

1. Empirical formulae for equivalent modal damping ratios as functions of period and soil class are constructed for three performance levels defined on the basis of deformation and damage. This is accomplished with the aid of extensive parametric studies involving 20 frames under 100 seismic motions. The design base shear of the system can be derived through response spectrum analysis and modal synthesis, using the aforementioned expressions for the modal damping ratios

2. In contrast to the EC8 design, which requires two steps (strength and deformation checking), the proposed method requires only one (strength checking) because the employed modal damping ratios are deformation dependent and hence the second step is automatically satisfied. However, absolute acceleration spectra for high amounts of damping are required.
3. The comparison between the EC8 seismic design and the proposed seismic design methodology indicates that the latter provides more accurate results and offers a safer design. Apart from that, the concept of using different strength reduction factors leads to a more rational approach than the conventional one, which utilizes one single value for the strength reduction factor.

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ΙΔΙΟΜΟΡΦΙΚΟΙ ΛΟΓΟΙ ΑΠΟΣΒΕΣΗΣ ΓΙΑ ΤΟΝ ΑΝΤΙΣΕΙΣΜΙΚΟ ΣΧΕΔΙΑΣΜΟ ΜΕΤΑΛΛΙΚΩΝ ΠΛΑΙΣΙΩΝ ΜΕ ΣΥΝΔΕΣΜΟΥΣ ΕΞΑΣΦΑΛΙΣΜΕΝΟΥΣ ΣΕ ΛΥΓΙΣΜΟ

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Περίληψη

Στην παρούσα εργασία προτείνεται μία νέα μέθοδος αντισεισμικού σχεδιασμού επίπεδων μεταλλικών πλαισίων με συνδέσμους δυσκαμψίας εξασφαλισμένους σε λυγισμό. Η μέθοδος χρησιμοποιεί φάσματα σχεδιασμού με υψηλές τιμές απόσβεσης, ιδιομορφικούς λόγους ιξώδους απόσβεσης και επαλληλία ιδιομορφών προκειμένου να υπολογιστεί η τέμνουσα βάσης του πλαισίου. Για τους ιδιομορφικούς λόγους απόσβεσης προτείνονται ακριβείς εκφράσεις συναρτήσει της περιόδου, παραμόρφωσης και βλάβης για τις πρώτες ιδιομορφές και για τρία επίπεδα επιτελεστικότητας. Αυτό επιτυγχάνεται με τη δημιουργία μίας ισοδύναμης γραμμικής κατασκευής, ίδιας μάζας και αρχικής δυσκαμψίας με τη μη –γραμμική κατασκευή καθώς και εκτεταμένες παραμετρικές αναλύσεις οι οποίες περιλαμβάνουν 20 μεταλλικά πλαίσια υποκείμενα σε 100 σεισμικές καταγραφές οι οποίες αντιστοιχούν σε 4 διαφορετικές εδαφικές κατηγορίες. Καταγράφεται η απόκριση των παραπάνω πλαισίων σε κατάλληλα κλιμακούμενα επιταχυνσιογραφήματα προκειμένου να υπολογιστούν οι ιδιομορφικοί λόγοι απόσβεσης. Η προτεινόμενη μέθοδος αποδεικνύεται πιο ακριβής και αποτελεσματική από αυτή των υπάρχοντων αντισεισμικών κωδίκων.