Engineering Structures 154 (2018) 23–37

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Modal strength reduction factors for seismic design of steel moment resisting frames

Dimitris G. Loulelis[a], George A. Papagiannopoulos[a,*], Dimitri E. Beskos[b,a,c]

[a] Department of Civil Engineering, University of Patras, 26504 Patras, Greece
[b] Institute of Structural Engineering and Disaster Reduction, College of Civil Engineering, Tongji University, Shanghai 200092, PR China
[c] Office of Theoretical and Applied Mechanics, Academy of Athens, 11527 Athens, Greece

ARTICLE INFO

Keywords:
Equivalent modal damping
Modal strength reduction factor
Equivalent linear system
Transfer function
Steel MRF
Inter-storey drift ratio
Strength deterioration
Panel zone

ABSTRACT

Traditional seismic design of framed structures uses a single constant value for the strength reduction factor. In this work, explicit expressions for different values of strength reduction factors for the first significant modes of steel moment resisting frames including strength deterioration and panel zone effects are developed. These factors are functions of modal periods and deformation/damage and are defined for four performance levels in a performance-based seismic design framework. The above factors are obtained through extensive parametric studies involving 20 steel frames and 100 far-field ground motions and following a two steps procedure: First an equivalent linear structure with the same mass and elastic stiffness of the original non-linear one is constructed based on equivalent modal damping ratios, which substitute the non-linearities; then use of these equivalent modal damping ratios in conjunction with modal damping reduction factors leads to modal strength reduction factors. Using these factors, one can obtain the seismic design base shear of a frame through response spectrum analysis. Thus, a more accurate and rational seismic design method is established. Numerical examples are presented to illustrate the method and demonstrate its advantages against conventional seismic design methods.

1. Introduction

The most accurate way to estimate the seismic response of a structure for design purposes is the dynamic non-linear analysis in the time domain, taking into account material and geometric non-linearities and member and structure imperfections. Despite its accuracy, dynamic non-linear analysis remains an inconvenient way for seismic design of structures and presents some drawbacks, such as the need of a careful and detailed modelling of the real structure and excitation of that structure by a number of real or artificial accelerograms. In order to overcome these problems, various simplified methods for the seismic design of structures have been developed during the last 50 years. These simplified procedures can be separated into two major categories.

The first category involves elastic response spectra and the concept of the strength reduction factor. Current seismic codes like EC8 [1], use an inelastic design spectrum, which results from the elastic design spectrum by dividing its ordinates with the strength reduction factor q, which has the same constant value for all modes and depends only on the type of structure. During the last 15 years or so, more rational expressions of strength reduction factors in terms of dynamic (e.g., period) and deformational (e.g., ductility) characteristics of the structure have also been proposed (e.g., [2–5]). In particular, Miranda and Bertero [2] have proposed expressions for the strength reduction factor not only in terms of period and ductility but also of the soil class. The second category is based on equivalent linearization, where the real non-linear structure is substituted by an equivalent linear structure with equivalent mass, equivalent secant stiffness or period and an equivalent viscous damping (e.g., [6–10]).

Recently Papagiannopoulos and Beskos [11] presented a method for the seismic design of plane steel moment resisting frames based on the concept of equivalent linearization and involving equivalent modal damping ratios and an equivalent linear structure with the same mass and stiffness as the initial non-linear one. Explicit expressions for these equivalent modal damping ratios for the first significant modes in terms of period and deformation/damage have been derived with the aid of extensive parametric studies involving many frames and ground motions. Using these equivalent modal damping ratios in conjunction with an elastic acceleration spectrum with high values of damping, one can easily determine the seismic base shear of a structure for design purposes. At this point, one should mention that the concept of equivalent damping ratio has been extensively used in connection with the modeling of supplementary damping devices for seismic displacement

* Corresponding author.
E-mail address: gpapagia@upatras.gr (G.A. Papagiannopoulos).

http://dx.doi.org/10.1016/j.engstruct.2017.10.071
Received 28 August 2016; Received in revised form 27 October 2017; Accepted 30 October 2017
0141-0296/

control of structures. One can consult the resent comprehensive work of Akcelyan et al. [12] with comparisons among all the well known pertinent models for more details. These models are simplified single-degree-of freedom (SDOF) systems associated with an equivalent damping ratio and are completely different than the models used in [11] where (i) the equivalent linear system is a multi-degree-of-freedom (MDOF) one, (ii) the equivalence is only with respect to damping and not with respect to damping and stiffness and (iii) damping is modal, i.e., consists of damping ratios different for different modes.

Utilizing the equivalent modal damping ratios and the corresponding modal damping reduction factors, Papagiannopoulos and Beskos [13] were able to derive explicit expressions for modal strength reduction factors to be used in the framework of a response spectrum analysis for design purposes. Thus, for the first time, strength reduction factors with different values for different modes were constructed in [12] to be used in spectrum analysis, instead of factors with a single value for all modes, as described in the existing literature.

The above two seismic design methods in [11,13] are characterized by some limitations, which restrict their practical applicability. These limitations are the following: (i) the frames are assumed to be simple elastoplastic with hardening but without consideration of strength deterioration and panel zone effects; (ii) the seismic motions used were not enough, did not cover all soil types and were restricted to near-fault ones; (iii) the response acceleration spectra have to be absolute and not pseudo-acceleration ones as it is the case in practice.

In this work, an effort has been made to considerably improve the method of [13] utilizing modal strength reduction factors and make it more realistic and truly applicable to practice. Thus, the present work utilizes the concepts and ideas in [11,13] and develops a performance-based seismic design method for plane steel moment resisting frames based on modal strength reduction factors and spectrum analysis and characterized by its ability to take into account strength deterioration and panel zone effects and almost all soil types. The modal strength reduction factors developed here are based (i) on a large motion data consisting of far-field motions associated with current codes and (ii) are expressed on the basis of pseudoaccelerations and hence appropriate for direct use in current code spectra.

The paper is organized as follows: In Section 2 the theoretical background of the proposed method and a brief description of the method are presented. Section 3 presents the steel moment frames used in this work. Section 4 presents the seismic ground motions databank used to conduct the non-linear analyses. Section 5 describes the details for the analytical models that simulate the seismic behavior of the frames used in this study. Section 6 presents the conversion of the absolute modal reduction factors into the pseudo modal-reduction factors and provides explicit expressions for them in terms of period and deformation/damage for four levels of seismic performance. Section 7 presents realistic design examples based on the proposed method and compares their results against those obtained by EC8 [1] and the dynamic non-linear analysis. Finally, in Section 8 of this paper the conclusions of the present work are summarized.

2. Theoretical background

This section briefly describes the theoretical background of the present paper based on the works of [11,13] and presented here for reasons of completeness. More specifically, the theoretical aspects of the computation of equivalent modal damping ratios and modal strength reduction factors for a structure subjected to seismic motion are briefly presented.

The governing equation of motion for a plane linear N-degree of freedom building system, subjected to an earthquake acceleration is given by

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = -[M]\{I\}\ddot{u}_g \tag{1}$$

where [M], [C] and [K], are the mass, damping and stiffness matrices, respectively, $\{u\}$ is the displacement vector, $\ddot{u}_g$ is the ground acceleration, $\{I\}$ is the unit vector and overdots denote time differentiation. Assuming that $\{u(t)\} = [\Phi]\{q(t)\}$, where $[\Phi]$ is the modal matrix and $\{q\}$ the generalized displacement vector, the above system uncouples into a set of N second-order differential equations of the form

$$\ddot{q}_j(t) + 2\xi_j\omega_j\dot{q}_j(t) + \omega_j^2 q_j(t) = -\Gamma_j\ddot{u}_g(t) \tag{2}$$

where ω_j and ξ_j are the undamped natural frequency and modal damping ratio of the j_{th} mode and Γ_j is the corresponding participation factor. Eq. (2) can be transformed in the frequency domain and provide $\bar{q}_j(\omega)$ in the form

$$\bar{q}_j(\omega) = \frac{-\Gamma_j\bar{\ddot{u}}_g(\omega)}{(\omega_j^2-\omega^2) + i(2\xi_j\omega_j\omega)} \tag{3}$$

where overbars indicate transformed quantities and ω is the frequency. The transfer function $R(\omega)$, is defined in the frequency domain as the ratio of the absolute roof acceleration of the building over the acceleration at its base, i.e.,

$$R(\omega) = \frac{\bar{\ddot{U}}_r(\omega)}{\bar{\ddot{u}}_g(\omega)} \tag{4}$$

where $\bar{\ddot{U}}_r(\omega)$ is equal to $\bar{\ddot{u}}_r(\omega) + \bar{\ddot{u}}_g(\omega)$. By evaluating $R(\omega)$ when $\omega = \omega_k$, Eq. (4) can be recast in the form

$$R(\omega = \omega_k) = 1 + \sum_j \frac{\phi_{rj}\Gamma_j\omega_k^2}{(\omega_j^2-\omega_k^2) + i(2\xi_j\omega_j\omega_k)} \tag{5}$$

where ϕ_{rj}, is the modal shape for the j-mode in the top floor r, and $i = \sqrt{-1}$. Complex arithmetic can be avoided by squaring the absolute value of Eq. (5), which leads in the expression

$$\begin{aligned} |R(\omega = \omega_k)|^2 = 1 &+ 2\sum_{j=1}^{N} \frac{\phi_{rj}\Gamma_j\omega_k^2(\omega_j^2-\omega_k^2)}{(\omega_j^2-\omega_k^2)^2 + (2\xi_j\omega_j\omega_k)^2} \\ &+ \sum_{j=1}^{N} \frac{\phi_{rj}^2\Gamma_j^2\omega_k^4(\omega_j^2-\omega_k^2)^2 + 4\xi_j^2\omega_j^2\omega_k^2}{[(\omega_j^2-\omega_k^2)^2 + (2\xi_j\omega_j\omega_k)^2]^2} \\ &+ 2\sum_{j\neq m,m>j}^{N} \frac{\phi_{rj}\phi_{rm}\Gamma_m\omega_k^4[(\omega_j^2-\omega_k^2)(\omega_m^2-\omega_k^2) + 4\xi_j\xi_m\omega_j\omega_m\omega_k^2]}{[(\omega_j^2-\omega_k^2)^2 + (2\xi_j\omega_j\omega_k)^2][(\omega_m^2-\omega_k^2)^2(2\xi_m\omega_m\omega_k)^2]} \end{aligned} \tag{6}$$

The above equation, on the assumption that ϕ_{rj},ω_j,Γ_j and $R(\omega)$ are known, constitutes a system of N non-linear algebraic equations to be solved for the modal damping ratios ξ_k of the linear structure. This system is solved by the Levenberg-Marquardt algorithm in Matlab [14]. The values of ϕ_{rj},ω_j and Γ_j are obtained from modal analysis, while $|R(\omega = \omega_k)|$ are computed as the peaks (maxima) of the function $|R(\omega)|$ versus ω corresponding to the resonant frequencies. Since the structural response is practically obtained by appropriate superposition of the first few significant modes, the system of Eq. (6) is solved for only those first few modes to provide the corresponding ξ_k.

When the structure is linear, its $|R(\omega)|$ versus ω curve is smooth with visible peaks for the first few modes. However, when the structure is non-linear, that curve has a distorted shape with no visible peaks, especially for higher modes. Increasing progressively its damping by using the Rayleigh type viscous damping [11], the non-linear structure results in having a smoother and smoother $|R(\omega)|$ versus ω curve, until for some value of damping that curve becomes completely smooth with clearly visible peaks. A mode that does not exhibit a peak in the transfer function is considered to be overdamped. This smoothness judgement, is accomplished with the aid of a special computer program based on certain smoothness mathematical criteria developed in Matlab [14,15]. At that value of damping, the originally non-linear structure has become an equivalent linear for which Eq. (6) for determination of the equivalent damping ratios ξ_k is applicable. Fig. 2 clearly shows the above described procedure for the case of the 10 storey steel frame of