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Extending the concept of energy-based pushover analysis to assess seismic demands of asymmetric-plan buildings

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ABSTRACT

The energy-based pushover analysis was developed in previous studies to address the issues regarding the distortion of capacity curve in conventional pushover procedures. Despite the conceptual superiority of an energy-based approach, its application is currently restricted to 2D structures. This study aims to extend the concept of this approach to asymmetric-plan buildings and bidirectional seismic excitation. For this purpose, a new energy-based multimode pushover analysis is developed. The overall procedure is quite similar to the well-known Modal Pushover Analysis (MPA). In contrast, however, the work done by lateral loads and torques here is used in preference to displacement of the roof center as an index to establish capacity curves. The efficiency of the proposed procedure is evaluated through seismic assessment of a set of one-way asymmetric (asymmetric around one axis) RC shear wall buildings. The results are compared with those of the MPA, ASCE41-13 pushover procedure, and the nonlinear response history analysis as a benchmark solution. Findings show that the proposed procedure can provide more accurate results than the MPA and ASCE41-13 procedures, in estimating the structural demands such as wall-hinge rotations and drift ratios.

1. Introduction

Rational estimation of seismic demands is an essential subject in the currently well-established “performance-based engineering design” approach. To accomplish this goal, the nonlinear response history analysis (NRHA) is recognized as the most accurate procedure. However, in addition to being time consuming, the results of the NRHA are highly sensitive to the methods of selecting and scaling ground-motion records. The other procedure widely used and also recommended by rehabilitation codes (e.g., [1]) is the nonlinear static procedure (NSP). The NSP has become popular because of its efficiency and capability to estimate seismic demands directly from the site-specific hazard spectrum. Nonetheless, the NSP methods stated in the codes suffer from some drawbacks. With reference to planar structures, two major of these drawbacks are: (i) excluding higher mode effects in analysis, and that (ii) the lateral load pattern in those are determined based on the initial characteristics of structure, ignoring the fact that these characteristics change constantly through pushover analysis. To overcome these limitations, various procedures have been proposed. Some of these procedures are based on adaptive load patterns (e.g., [2–5]). In adaptive NSPs, addressing the second major drawback mentioned above, the load pattern is updated in each step of analysis in accordance with the changes in characteristics of structure due to nonlinearity. Despite the theoretical superiority of the adaptive NSPs, Antoniou and Pinho [6] concluded that the adaptive force-based pushover procedures feature a relatively minor advantage over the nonadaptive procedures. A representative for nonadaptive NSPs (e.g., [7–11]) is Modal Pushover Analysis (MPA), introduced by Chopra and Goel in 2002 [11]. The MPA includes multiple runs of pushover analyses for an adequate number of modes. For each mode, the capacity curve—the resisting force of the equivalent single degree of freedom (ESDF) system versus the displacement of the ESDF system—is established by monitoring the roof displacement and the base shear through pushover analysis. Subsequently, the capacity curve is used to calculate the mode-corresponding target displacement.

In previous studies, it has been observed that taking the roof displacement as a reference point can lead to a distortion of capacity curve, typically as an outright reversal in higher modes [12]. Addressing this issue, Hernandez Montes et al. [13] proposed an alternative energy-based procedure to establish the capacity curve. The efficiency of this approach in estimating the peak roof displacement was studied by Tjhin et al. [14]. Their findings showed that, in most cases, the energy-based approach is more accurate than the conventional procedure. On this subject, some other studies have also

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suggested different applications of the strain energy. In an approach called the energy-balance concept, the absorbed energy itself is taken as the resistance force in capacity curve, and the target displacement is determined from the intersection of the capacity and demand curves, both in the energy phase [15,16]. In another study, Manoukas et al. [17] investigated a procedure in which the absorbed energy is used to calculate the base shear rather than displacement through pushover analysis.

Despite the fact that the energy-based approach has shown practical superiority and more accuracy, its application is currently limited to 2D structures. In fact, up to now, all the energy based pushover procedures have been developed for 2D and symmetric buildings.

In the case of asymmetric buildings, there is an additional source of errors in NSPs due to the coupled torsional-lateral displacement of the buildings. To take these torsional effects into account, several NSPs have been proposed (e.g., [18–24]). However, the results of relevant evaluations in the literature [25] still suggest limited success in estimating the structural demands in the stiff and flexible side of asymmetric buildings. In the MPA procedure for asymmetric structures [19,26], by analogy with the procedure for symmetric buildings, x- or y-translational component of the roof center displacement is taken as an index to determine the displacement of the corresponding ESDF system. Compared to 2D structures, in this case, the chosen component of the roof displacement is representative for three times of degree of freedoms (DOFs); therefore, the resulting capacity curve is more prone to distortion as a result of disproportional displacement. According to the literature, there are limited relevant studies to evaluate the suitability of this underlying assumption [26,27]. Moreover, all the related papers have investigated steel-frame buildings as case studies.

In this paper, an energy-based multimode pushover analysis (E-MPA) is developed, which accounts for asymmetric-plan buildings and bidirectional seismic excitation. The steps of the proposed procedure are quite similar to those of the MPA procedure. The major difference is the concept based on which the capacity curve is established. In contrast to the MPA procedure, herein, the work done by lateral loads and torques through pushover analysis (or the absorbed energy) is considered as an index to compute the displacement of the corresponding ESDF system. By this approach, we intend to achieve more accurate results, and to eliminate the abnormalities of capacity curve observed in the MPA procedure (i.e. outright reversals in higher rotational and translational modes).

The theoretical background and the major assumptions are primarily explained in this paper. The stepwise instruction of the E-MPA is also presented. A set of symmetric and one-way asymmetric RC shear wall buildings are defined as case studies, in which the asymmetric models include both the stiffness and strength eccentricities. The efficiency of the E-MPA is evaluated through seismic assessment of the building models. The obtained results are compared with those of the nonlinear response history analysis (NRHA) as a benchmark solution. In addition, the MPA and ASCE41-13 pushover procedures are performed and their accuracy in estimating the seismic demands is discussed in comparison with E-MPA.

2. The proposed E-MPA procedure

2.1. Theoretical background

For a given N story three-dimensional building (assumed to have rigid diagrams in the plan and fixed base), each floor displacement is described by three DOFs defined at each floor mass center (CM). The displacement vector is expressed as:

$$\boldsymbol{u} = \langle \boldsymbol{u}_x, \boldsymbol{u}_y, \boldsymbol{\theta} \rangle^T \tag{1}$$

In which $\boldsymbol{u}_x$, $\boldsymbol{u}_y$, and $\boldsymbol{\theta}$ are $1 \times N$ vectors, representing the x, y, and rotational component of floor center displacement. For such a building, the governing equations of motion due to the horizontal components of a ground motion are:

$$m\ddot{\boldsymbol{u}}(t) + c\dot{\boldsymbol{u}}(t) + \boldsymbol{f}_s(t) = -\boldsymbol{m}\boldsymbol{l}_x\ddot{u}_{gx}(t) - \boldsymbol{m}\boldsymbol{l}_y\ddot{u}_{gy}(t) \tag{2}$$

The right side of the equation represents the "effective earthquake forces"; $\boldsymbol{m}$ is a diagonal mass matrix of order $3N$, including the floors' x and y-lateral masses, and the torsional moment of inertia; $\boldsymbol{c}$ is designated as a classical damping matrix; $\boldsymbol{f}_s$ stands for the resisting force vector; and

$$\boldsymbol{l}_x = \langle \boldsymbol{1}, \boldsymbol{0}, \boldsymbol{0} \rangle^T \tag{3}$$

$$\boldsymbol{l}_y = \langle \boldsymbol{0}, \boldsymbol{1}, \boldsymbol{0} \rangle^T \tag{4}$$

where $\boldsymbol{1}$ and $\boldsymbol{0}$ are $1 \times N$ vectors with all elements equal to unity and zero, respectively.

The "effective earthquake forces" can be decomposed into a summation of modal effective forces as:

$$-\boldsymbol{m}\boldsymbol{l}_x\ddot{u}_{gx}(t) - \boldsymbol{m}\boldsymbol{l}_y\ddot{u}_{gy}(t) = -\left(\sum_{n=1}^{3N} \boldsymbol{s}_{nx}\right) \times \ddot{u}_{gx}(t) - \left(\sum_{n=1}^{3N} \boldsymbol{s}_{ny}\right) \times \ddot{u}_{gy}(t) \tag{5}$$

in which

$$\boldsymbol{s}_{nx} = \Gamma_{nx}\boldsymbol{m}\boldsymbol{\varphi}_n = \Gamma_{nx}\boldsymbol{m}\left\langle \boldsymbol{\varphi}_{xn}, \boldsymbol{\varphi}_{yn}, \boldsymbol{\varphi}_{\theta n} \right\rangle^T \tag{6}$$

$$\boldsymbol{s}_{ny} = \Gamma_{ny}\boldsymbol{m}\boldsymbol{\varphi}_n = \Gamma_{ny}\boldsymbol{m}\left\langle \boldsymbol{\varphi}_{xn}, \boldsymbol{\varphi}_{yn}, \boldsymbol{\varphi}_{\theta n} \right\rangle^T \tag{7}$$

$\boldsymbol{\varphi}_n$ is the nth elastic mode, including three sub-vectors of $\boldsymbol{\varphi}_{xn}$, $\boldsymbol{\varphi}_{yn}$, and $\boldsymbol{\varphi}_{\theta n}$, and

$$\Gamma_{nx} = L_{nx}/M_n \tag{8}$$

$$\Gamma_{ny} = L_{ny}/M_n \tag{9}$$

where

$$L_{nx} = \boldsymbol{\varphi}_n^T \boldsymbol{m}\boldsymbol{l}_x \tag{10}$$

$$L_{ny} = \boldsymbol{\varphi}_n^T \boldsymbol{m}\boldsymbol{l}_y \tag{11}$$

$$M_n = \boldsymbol{\varphi}_n^T \boldsymbol{m}\boldsymbol{\varphi}_n \tag{12}$$

To develop an approximate procedure for inelastic systems, two major assumptions are made in this section. Firstly, the total response of the system to the "effective earthquake forces" is assumed to be equal to the summation of responses due to individual terms of the modal effective forces [Eq. (5)]. In other words, it is presumed that the superposition principle is also valid through an inelastic range of behavior:

$$r(t) = \sum_{n=1}^{3N} r_{nx}(t) + \sum_{n=1}^{3N} r_{ny}(t) \tag{13}$$

In the above-mentioned equation, $r(t)$ is the total response, and $r_{nx}(t)$ and $r_{ny}(t)$are the responses resulted from the following equations:

$$\boldsymbol{m}\ddot{\boldsymbol{u}}_{nx}(t) + \boldsymbol{c}\dot{\boldsymbol{u}}_{nx}(t) + \boldsymbol{f}_{snx}(t) = -\boldsymbol{s}_{nx}\ddot{u}_{gx}(t) \tag{14}$$

$$\boldsymbol{m}\ddot{\boldsymbol{u}}_{ny}(t) + \boldsymbol{c}\dot{\boldsymbol{u}}_{ny}(t) + \boldsymbol{f}_{sny}(t) = -\boldsymbol{s}_{ny}\ddot{u}_{gy}(t) \tag{15}$$

in which the x and y subscripts indicate the associated component of the ground motion.

It should be noted that only the displacement-type responses (i.e. displacement, drift, hinge rotation, etc.) can be determined from Eq. (13). The reason is that forces computed in this way may exceed the specified member capacity. Instead, the member forces can be recomputed from the member deformations accordingly. Information about this is available in ref. [28]. In this paper, only the displacement-type responses is concerned.