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AN ENERGY SPECTRUM METHOD FOR SEISMIC EVALUATION OF STRUCTURES

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ABSTRACT

Seismic evaluation of structures generally involves determination of displacement demands from which story drifts, and component forces and deformations for specified hazard levels can be obtained for comparison with available capacities. A number of methods have been proposed by investigators in the past some of which are also used in current practice, such as MPA, FEMA 440, and Capacity Spectrum. Those methods generally involve non-linear pushover analyses. This paper presents adaptation of an energy based method that has been recently developed and successfully used by Goel et al., for design purposes, called Performance-Based Plastic Design (PBPD) method. In the PBPD method the design base shear for selected hazard level is determined by equating the work needed to push the structure monotonically up to a selected target drift to the corresponding energy demand of an equivalent SDOF oscillator. It turns out that the same work-energy equation can also be used to estimate seismic demands for existing structures. In this approach the skeleton force-displacement (capacity) curve of the structure is converted into energy-displacement plot (Ec) which is superimposed over the corresponding energy demand plot (Ed) for the specified hazard level to determine the expected peak displacement demand. The drift demands of two 20-story RC and steel moment frames as computed by the proposed energy spectrum method were in excellent agreement with those obtained from inelastic dynamic analyses using representative ground motion records.

Introduction

Static pushover method has been widely accepted as a useful tool for performance-based seismic design and evaluation of structures (FEMA, 2006). Since its introduction to the engineering community, the pushover analysis method has been a subject of extensive research and several new analysis approaches have been proposed. Recent notable modifications include adaptive load patterns and multiple modal analysis procedures. In most cases, the behavior of the structure is

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characterized by the capacity curve which is represented by a plot of the base shear versus the roof displacement. The capacity curve is used to establish an equivalent single degree of freedom system. The expected peak displacement demand can then be predicted using one of the methods such as the capacity spectrum approach, the modification factor approach, or the direct use of inelastic constant ductility spectra. The peak displacement can then be projected back to the roof displacement from which the story and member demands can be extracted.

After a brief description of the proposed energy spectrum method its application to a 20-story RC and a 20-story steel moment frame is presented. The results are compared with those obtained from time-history analyses. Evaluation of RC structures presents special challenge due to their complex and degrading ("pinched") hysteretic behavior. This aspect is taken care of by making appropriate modification in constructing the energy demand curve, Ed. The drift demand estimates given by the energy spectrum method (Ec = Ed) were generally quite close to those obtained from time-history analysis using representative ground motion records. This can be considered as a very good correlation between the results given by an approximate method with those from more precise time-history analysis.

Energy Balance Concept in Performance-Based Plastic Design

Determination of the design base shear for given hazard level is a key element in the PBPD method. It is calculated by equating the work needed to push the structure monotonically up to the target drift to that required by an equivalent elastic-plastic single degree of freedom (EP-SDOF) system to achieve the same state, Figure 1.

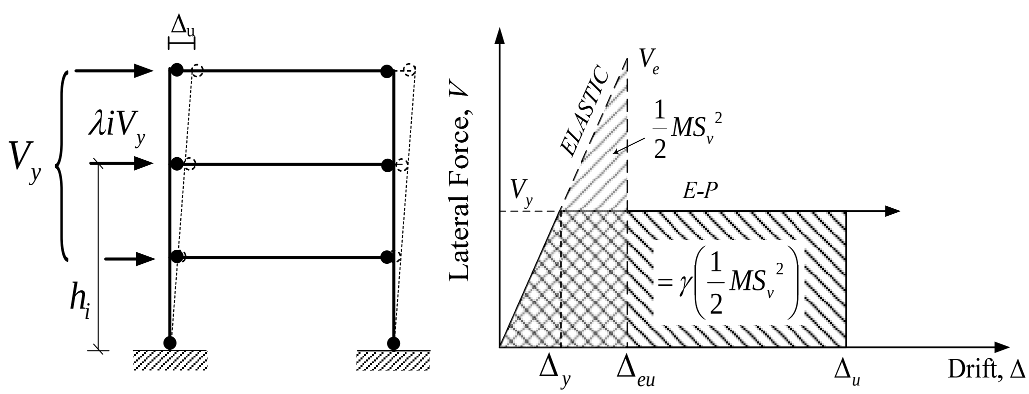

Figure 1. PBPD concept

Assuming an idealized E-P force-deformation behavior of the system, the work-energy equation can be written as:

$$(E_e + E_p) = \gamma\left(\frac{1}{2} M S_v^{\,2}\right) = \frac{1}{2}\gamma M\left(\frac{T}{2\pi} S_a g\right)^2 \tag{1}$$

where E_e and E_p are, respectively, the elastic and plastic components of the energy (work) needed to push the structure up to the target drift. S_v is the design pseudo-spectral velocity; S_a is the pseudo spectral acceleration; T is the natural period; and M is the total mass of the system. The energy modification factor, γ, depends on the structural ductility factor (μ_s) and the ductility reduction factor (R_μ), and can be obtained by the following relationship:

$$\gamma = \frac{2\mu_s - 1}{R_\mu^{\,2}} \tag{2}$$

Because of their simplicity, spectra proposed by Newmark and Hall (1982) were used to relate the ductility reduction factor, R_μ, and the structural ductility factor, μ_s, for EP-SDOF. Details can be found elsewhere (Goel and Chao, 2008).

Solution of the work-energy equation gives the required design as:

$$\frac{V_y}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma S_a^2}}{2} \tag{3}$$

where α is a dimensionless parameter given by,

$$\alpha = \left(h^* \cdot \frac{\theta_p 8\pi^2}{T^2 g} \right) \tag{4}$$

The term θ_p represents plastic component of the target drift ratio, and $h^* = \sum_{i=1}^{N} \left(\lambda_i h_i\right)$.

For systems that do not posses full EP type hysteretic property, such as steel braced frames with buckling type braces or RC frames, the following approach has been used which show good promise:

This approach is based on consideration of the effect of degrading hysteretic behavior on peak displacement. Investigators have studied the effect of degrading hysteretic behavior of SDOF systems on resulting peak displacements. The results show that the peak displacements are larger than those of systems with non-degrading hysteretic behavior in the short period range, but are about equal for longer periods. Approximate expressions have been proposed for modification factors to account for this effect, e.g., factor C_2 in FEMA 440 (2006), Figure 2. Thus, the target design drift for a given structural system with degrading hysteretic behavior can be divided by the C2 factor which would give design target drift for an equivalent non-degrading system. The design base shear can then be calculated by using this modified target drift and Equation (3).

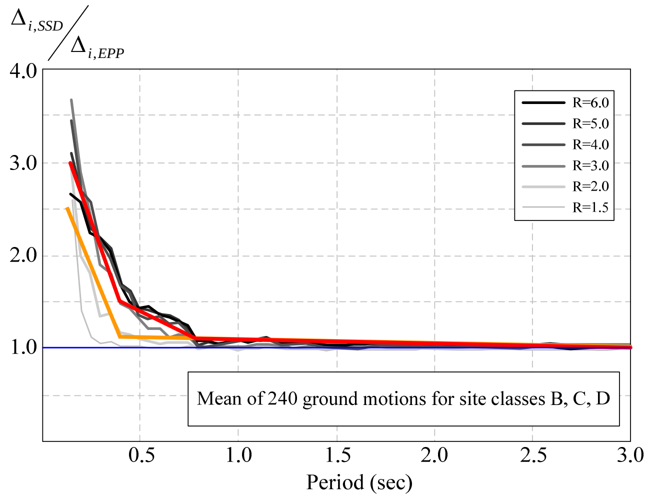

Figure 2. Mean displacement ratio of SSD to EPP models computed with ground motions recorded on site classes B, C, and D. (FEMA 440, 2006)

Seismic Evaluation Based on Energy Balance Concept

In the previous section the energy-based PBPD method was presented and discussed in the context of design of new structures for a target maximum drift. Therefore, with other terms being known, the design base shear is determined by solving the work-energy Equation (1). It turns out that the same energy equation can also be used for evaluation purposes, where the structure is defined, including its force-displacement characteristics, and the goal is to "predict" the expected maximum displacements for a given seismic hazard (Leelataviwat et al., 2007). Other response quantities, such as component forces and deformation demands, can then be easily calculated from the maximum reference displacement.

In order to use the energy concept for evaluation purposes, the right hand side of Equation (1) can be viewed as energy demand for the given hazard, Ed, and the left hand side as energy capacity of the given structure, Ec. Both these quantities vary with displacement. The value of the desired maximum reference displacement can be obtained by either solving the work-energy equation analytically, or graphically by constructing the two energy curves as a function of the reference displacement and determining their point of intersection.

Figure 3 presents a graphical illustration of the evaluation process. Lateral force-displacement plot for the given structure is shown in Figure 3(a), where V represents the total force (base shear), and u_r the roof displacement, used as reference displacement. This plot can be obtained by a static pushover analysis by applying either an appropriately selected force or displacement pattern. It is common to plot total force versus roof displacement, but it can be done for any other floor or story level from which the force or displacement at other levels can be determined. The energy capacity curve, Ec-u_r, can be generated as a function of u_r, by calculating the work done by lateral forces up to the displacement at each level corresponding to

u_r, Figure 3(b). Next, the energy demand, Ed, can be calculated for varying values of u_r, and plotted as shown in Figure 3(c). The point of intersection of the two curves, where the energy demand and capacity become equal, gives the desired maximum roof displacement, as shown in Figure 3(d).

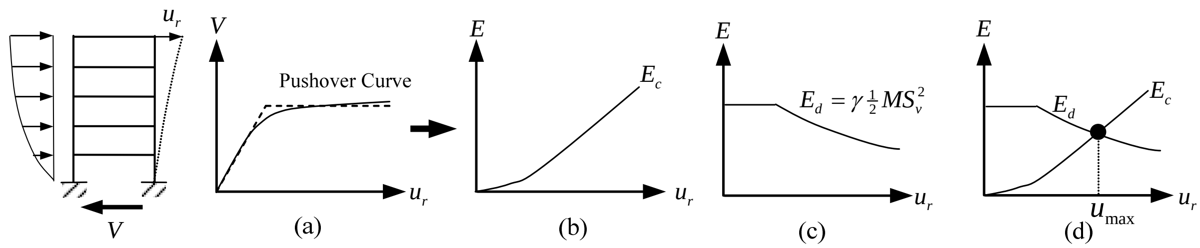

Figure 3. Proposed energy-based evaluation method for MDOF systems: (a) Push-over curve, (b) Energy-displacement capacity diagram, (c) Energy demand diagram, and (d) Determination of displacement demand

20-story RC Special Moment Frame

RC structures present special challenge due to their complex and degrading ("pinched") hysteretic behavior. While development of the PBPD method for RC structures is currently in progress, results from the study so far have been most promising. Two 20-story frames are briefly presented in this section. One is the baseline space frame from ATC 63 Project which was designed to comply with current building code provisions (FEMA P695, 2009; Haselton and Deierlein, 2007). The other frame was redesigned by the PBPD method. The design details can be found elsewhere (Goel et al., 2009). For response evaluation purposes the baseline code compliant frame and the PBPD frame were subjected to inelastic pushover and time-history analyses for selected earthquake record set from PEER-NGA as also used by Haselton and Deierlein (2007). PERFORM 3D computer program was used for the analyses.

The pushover curves for the two frames in Figure 4 show that, even though the design base shear for the baseline code compliant frame is smaller than that of the PBPD frame by 41%, the ultimate strength of the two are almost equal. That is mainly due to the fact that the design of the baseline frame was governed by drift which required major revision of the member sizes after having been designed for required strength. That iteration step is not needed in the PBPD method. Calculated values of R_{max} for the baseline and PBPD frames according to the recommended equation in FEMA P440A (2009) are 5.3 and 10.8, respectively. That reflects much enhanced margin against dynamic instability (collapse) of the PBPD frame over that of the baseline frame. Figure 5 shows the deformed shape and location of plastic hinges of the two frames at 2.5% roof drift. Formation of plastic hinges in the columns and story mechanism in the lower part of the baseline frame can be clearly seen. In contrast, there are no unintended plastic hinges in the columns of the PBPD frame, resulting in more favorable deformed shape and yield pattern as intended in the design process.

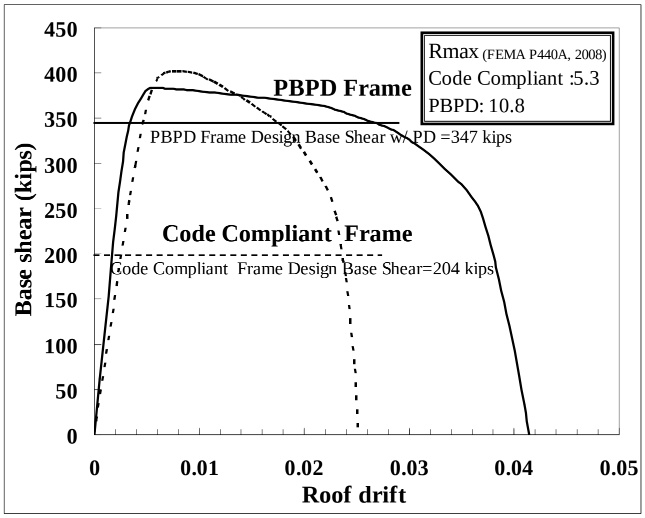

Figure 4. Pushover curves for PBPD and code compliant RC moment frames

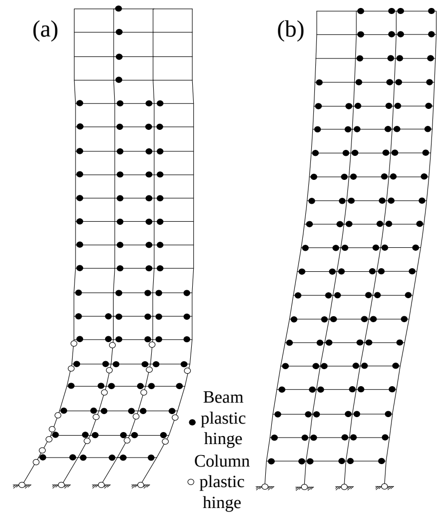

Figure 5. PH locations at 2.5% roof drift (a) Code compliant, (b) PBPD frame

In terms of energy spectrum method for evaluation purpose, the energy capacity and demand curves of these two frames are shown in Figure 6. For each frame, the capacity curve was obtained by calculating the work done by the applied forces in the pushover analysis. The energy capacity corresponding to each roof drift was calculated by numerically integrating the lateral load-deflection values at the floor levels. The energy demand curve was obtained by using the total mass of the frame. The peak roof drift demand was determined from the intersection point of the corresponding demand and capacity curves.

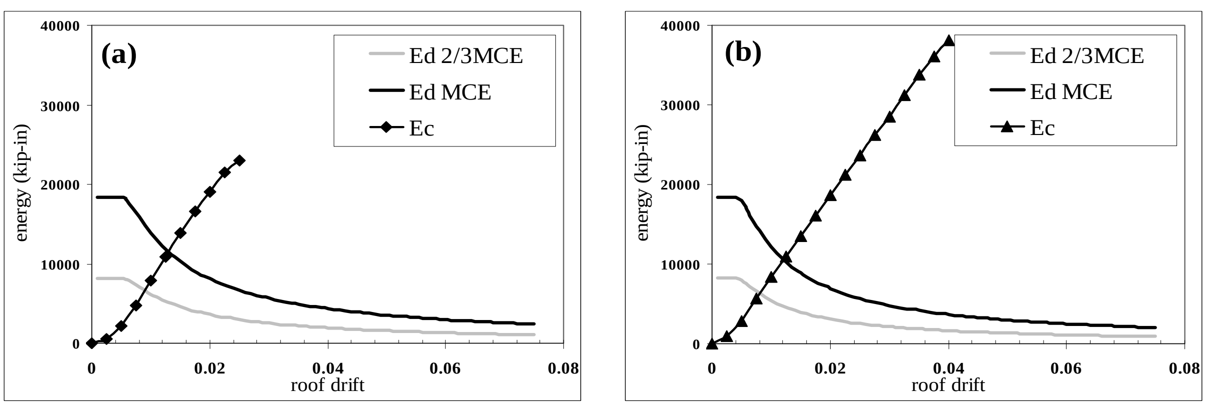

Figure 6. Energy spectrum evaluation methods for (a) Code compliant, (b) PBPD frame

Figure 7 shows comparison of maximum interstory drifts of the two frames as calculated by the energy spectrum method with those obtained from the time-history analyses using appropriately scaled ground motion records representative of 2/3 MCE and MCE hazard levels. It is worth noting that the interstory drifts predicted by the energy spectrum method are in excellent agreement with those obtained from the dynamic analyses for both frames, but more so for the PBPD frame. The results also show that the mean maximum interstory drifts of the PBPD frame are well within the corresponding target values, i.e., 2% for 2/3 MCE and 3% for MCE. Moreover, the story drifts of the PBPD frame are more evenly distributed over the height as compared with those of the baseline frame where undesirable "softness" in the lower stories is evident, which is caused mainly by plastic hinges in the columns. It can also be said that the effect of higher modes and soft stories is much more prominent for the code compliant frame than for the PBPD frame.

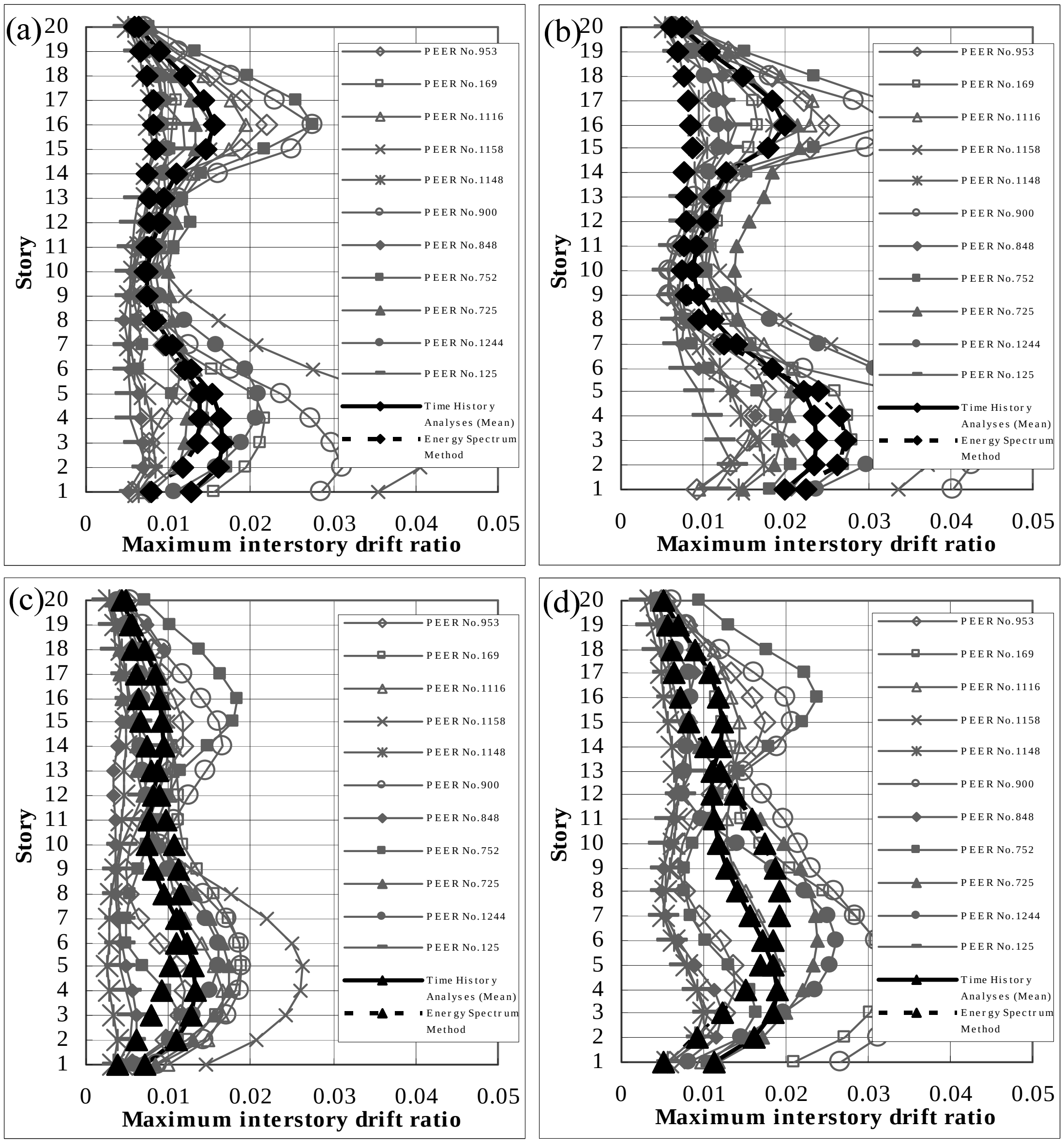

Figure 7. Comparison of maximum interstory drifts by the energy spectrum method and time-history analyses for a) code compliant frame for 2/3 MCE, b) code compliant frame for MCE, c) PBPD frame for 2/3 MCE, d) PBPD frame for MCE hazard levels.

20-story SAC Steel Moment Frame

The SAC LA20-story steel moment frame, which has been the subject of many previous studies (Gupta and Krawinkler, 1999, Lee and Goel, 2001), is used herein. The design procedure for the PBPD frame can be found in Goel and Chao (2008). The performances of the original SAC frame and the PBPD frame, were evaluated by inelastic pushover as well as time-history analyses. Nonlinear dynamic analyses were carried out by using 10/50 and 2/50 SAC ground motions (Somerville et al., 1997). PERFORM 3D software was used for modeling and analysis.

Figures 8 and 9 show the pushover curves and plastic hinge formation, respectively, for the two frames. The energy capacity and demand curves of these two frames, as used in energy spectrum method, are shown in Figure 10. For each frame, the energy capacity and demand curves were obtained by following the procedure as described earlier for the RC frame. Total mass of the frame was used for the energy demand curve. The peak roof drift demand was determined from the intersection point of the corresponding demand and capacity curves.

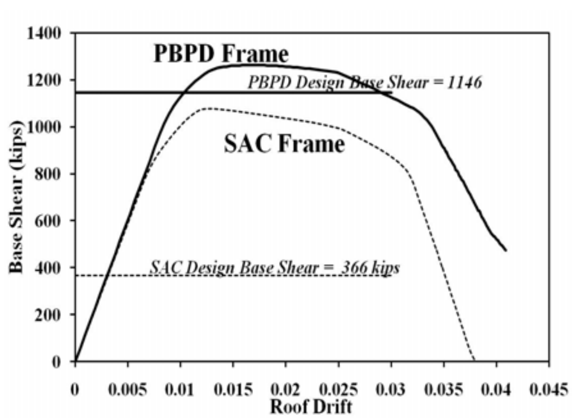

Figure 8. Pushover curves for PBPD and SAC frames

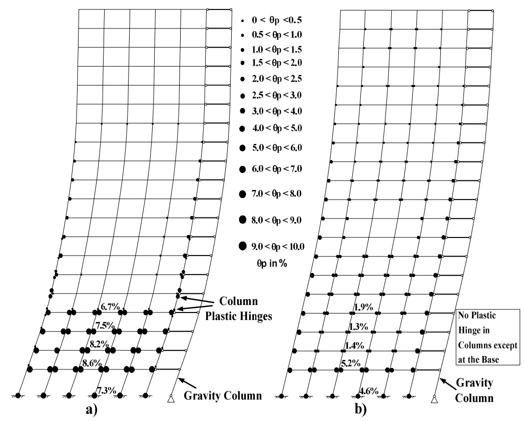

Figure 9. PH locations at 3.5% roof drift (a) SAC, (b) PBPD frame

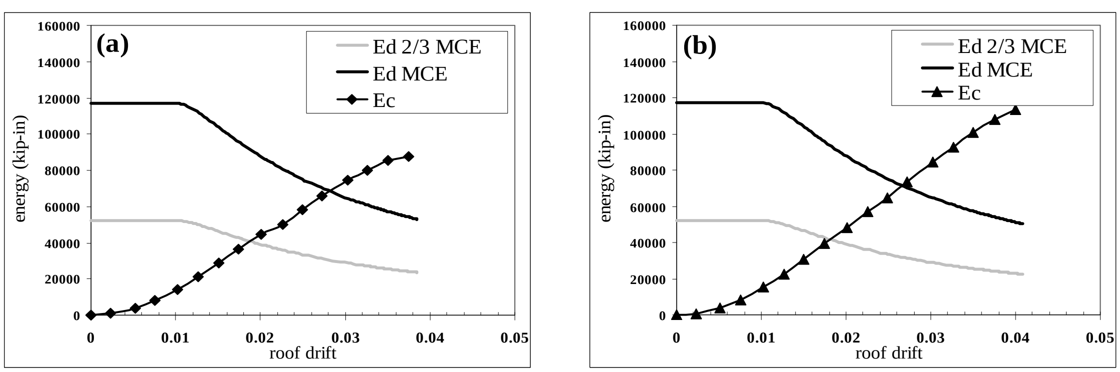

Figure 10. Energy spectrum evaluation methods for (a) SAC, (b) PBPD frame

Figure 11 shows comparison of maximum interstory drifts of the two frames as calculated by the energy spectrum method with those obtained from the time-history analyses using SAC ground motion records representative of 2/3 MCE and MCE hazard levels. Energy spectrum method results for story drifts match better with the envelopes of the story drifts at lower and middle stories rather than with the mean values. It appears that the deflected shape of the frames under pushover analysis may be the key to explain this observation. Lower and middle stories are experiencing larger drifts whereas the upper stories are not getting involved that much under the pushover lateral loads. This becomes more prominent at larger roof drifts. The results also show that the mean of maximum interstory drifts of the PBPD frame are well within the corresponding target values, i.e., 2% for 2/3 MCE and 3% for MCE. Moreover, the story drifts of the PBPD frame are somewhat more uniformly distributed along the height as compared with those of the SAC frame where undesirable “softness” in the lower stories is evident, which is caused mainly by plastic hinging in the columns.

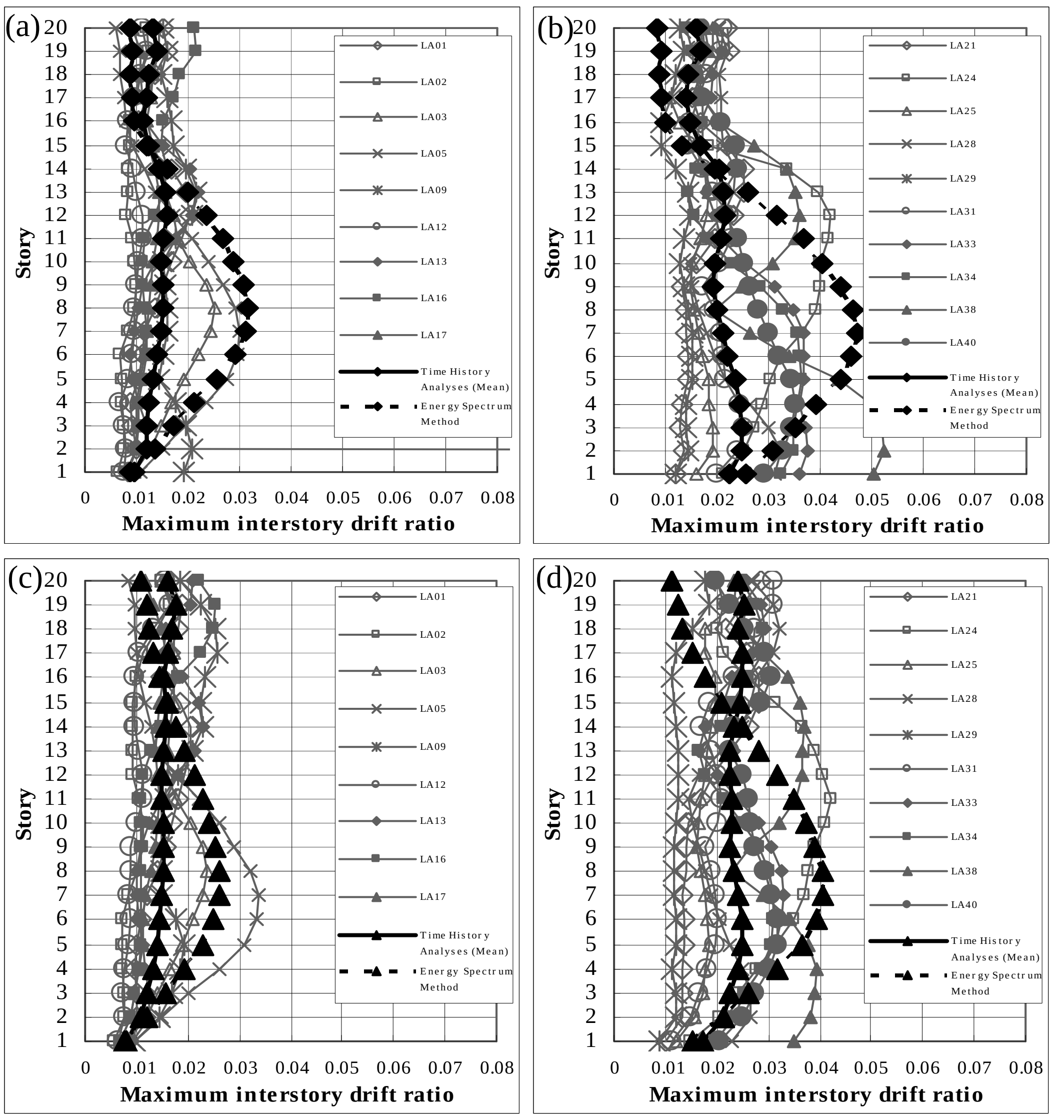

Figure 11. Comparison of maximum interstory drifts by the energy spectrum method and time-history analyses for a) SAC frame under 2/3 MCE, b) SAC frame under MCE, c) PBPD frame under 2/3 MCE, d) PBPD frame under MCE ground motions.

Conclusions

The basic work-energy equation used in the PBPD method for determination of design base shear for new structures can also be used for seismic evaluation purposes where the goal is to determine expected displacement demand for a given structure and earthquake hazard. The results of 20-story steel and RC moment frames as presented in this paper showed excellent agreement with those obtained from more elaborate inelastic time-history analyses.

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