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Main research interests:
Performance-based design, Displacement-based design, Seismic design and performance of nonstructural components

Luis Esteva has made remarkable contributions to the field of Earthquake Engineering. His work in Probabilistic Seismic Hazard Analysis and in particular his pioneer 1967 paper are well known internationally. Here however, I would like to highlight some of his contributions in other areas of Earthquake Engineering. Of his many (hundreds) publications, I reproduce here two excerpts of a chapter Luis wrote in 1980 for a book edited by Emilio Rosenblueth. I first read this chapter when I was an undergraduate student at UNAM and over the years it has become one of my favorite papers.

While referring to seismic codes at the beginning of his chapter Luis wrote [*]:

> "Base shear coefficients and design response spectra are taken as measures of response parameters, as the latter are usually expressed in terms of accelerations and equivalent lateral forces acting on linear systems. But these variables are no more than indirect measures of system performance during earthquakes: they serve to control the values of more significant variables, such as lateral deflections of actual nonlinear systems, global and local ductilities, and safety margins with respect to instability failure (second-order effects). Because the relations of control variables to actual response are affected by the type and features of the structural system, better designs will be obtained if these relations are understood and accounted for, in contrast with blindly applying codified recommendations."

In the same chapter Luis also wrote the following:

> "Engineering design is rooted in society's need to optimize. The general goal of optimization can be expressed in terms of direct, particular objectives: seismic design aims at providing adequate safety levels with respect to collapse in the face of exceptionally intense earthquakes, as well as with respect to damage to adjacent constructions; it also seeks to protect structures against excessive material damage under the action of moderate intensity earthquakes, to ensure simplicity of the required repair, reconstruction or strengthening works in case damage takes place, and to provide protection against the accumulation of structural damage during series of earthquakes."

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[*] Esteva, L. (1980), "Design: General", chapter 3 in *Design of Earthquake Resistant Structures*, Ed. E. Rosenblueth, John Wiley & Sons, Inc., New York, pp. 54-99. Also available in Spanish through IMCYC.

"Achievement of the foregoing objectives requires much more than dimensioning structural members for given internal forces. It implies explicit consideration of those objectives and of the problems related with nonlinear structural response and with the behavior of materials, members, and connections when subjected to several cycles of high-load reversals. It implies as well identifying serviceability conditions and formulating acceptance criteria with respect to them."

"Establishment of design conditions follows cost-benefit studies, where the initial costs required to provide given safety levels and degrees of protection with respect to material losses are compared with the present value of the expected consequences of structural behavior. This is obtained by adding up the costs of failure and damage that may occur during given time intervals, multiplied by their corresponding probabilities and by actualization factors that convert monetary values at arbitrary instants in the future into equivalent values at the moment of making the initial investment."

"Let D be the cost of damage caused by an earthquake on a structure, which includes damage to the structure, its contents as well as all other consequences (such as loss of human lives and indirect effects) expressed in monetary terms, then the probability density function of D every time a significant event takes place is

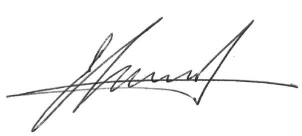
$$f_D(d) = \int \frac{\mathrm{d}Q(y)}{\mathrm{d}y} f_{D|Y}(d \mid y)\ \mathrm{d}y$$

where $Q(y)$ is the conditional cumulative probability distribution of the earthquake intensity given that a significant event has occurred, and $f_{D|Y}(d|y)$ the probability density function of D conditional to every possible value of earthquake intensity."

The first excerpt describes what many years later would become known as "displacement-based design" while the second excerpt published 25 years ago describes, of course, what we now know as "performance-based design".

From these paragraphs it is clear that Luis was back then, but continues to be today, at the forefront of Earthquake Engineering. For those who have not read this chapter, especially for those in the newer generations of earthquake engineering students, I strongly recommend reading it and learning more about the extraordinary humble man who wrote it.

I feel extremely honored to have been invited to participate in this symposium on his honor and more so to be counted as one of his friends. Hats off to you Luis!



Eduardo Miranda
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SIMPLIFIED ANALYSIS TOOLS FOR RAPID SEISMIC EVALUATION OF EXISTING BUILDINGS IN URBAN AREAS

ABSTRACT

Simplified analysis tools that make feasible rapid assessment of large inventories buildings in urban areas with a minimum amount of information about the buildings are presented. The simplified seismic analysis tools use a continuum model consisting of a flexural beam coupled with a shear beam. The model permits to obtain estimates of the seismic response of multi-story buildings with only three parameters, that is, only one parameter in addition to the two that are required to define a linear elastic single degree of freedom system. The simplified method is computationally very efficient and permits to obtain estimates of the response of a multi-story building in fractions of a second, hence providing an excellent tool to incorporate record-to-record variability and modeling uncertainty in probabilistic performance assessments of existing buildings. Results of lateral deformation time histories, floor acceleration time histories and floor response spectra computed with the simplified method have been compared to those measured during earthquakes in more than 80 instrumented buildings in California. Evaluation of the results indicates that the proposed analysis tool provides relatively good results of not only peak values of response parameters but in most cases also of time history results. Based on the simplified model two new type of spectra are presented. These spectra, referred to as generalized interstory drift spectrum and generalized building peak acceleration, have ordinates that provide the intensity of parameters closely correlated with structural and nonstructural damage.

Introduction

Seismic performance assessment of large inventories of buildings has traditionally been done by estimating performance through the use of empirical correlations between peak ground motions parameters such as peak ground acceleration (PGA) and peak ground velocity (PGV) with modified Mercalli intensity. In the United States the best known example of this approach is the “ShakeMap” (Wald, et al. 1999a). The Instrumental Intensity map used in ShakeMap is based on a combined regression of recorded peak acceleration and velocity amplitudes (Wald, et al. 1999c). The empirical relationships between PGA and PGV with MMI used in ShakeMap are based on a much larger data set than that used by Trifunac and Brady (1975) who developed one of the first relationships between peak ground motion parameters and MMI. Unfortunately, empirical correlations of peak ground acceleration and peak ground velocity with the Modified Mercalli Intensity are characterized by a very large scatter of data points. For example, Trifunac and Brady (1975) reported that, for a given intensity, the scatter was equal to about one order of magnitude in PGA or PGV. Despite using a significant larger data set, the empirical relationships obtained by Wald et al. (1999c) show approximately the same scatter. For example, peak ground acceleration ranging from 5 cm/s^2 to 450 cm/s^2 produced a MMI of V. Similarly, according to the data collected by Wald et al. (1999c), areas subjected to peak ground accelerations of 300 cm/s^2 (31%g) could be associated with Modified Mercalli Intensities of V, VI, VII or VIII. This range of MMI's corresponds to distinctly damage descriptions ranging from "very light damage" to "moderate to heavy damage", making performance predictions not very reliable. Although correlation is improved when using peak ground velocity, the correlation remains relatively low.

Several studies have shown that rather than estimating seismic performance directly from peak ground motion parameters, a better estimation of earthquake damage can be obtained by first obtaining an estimate of the building seismic response and then obtaining an estimate of damage from peak structural response parameters. An improved performance assessment can be obtained if single-degree-of-freedom (SDOF) systems are used as analytical models to estimate building response. This approach, which was pioneered by Luis Esteva and Emilio Rosenblueth in the late 60's, are now widely used all over the world. Tools for seismic risks of urban areas in the U.S. (e.g. HAZUS) and in Europe (e.g., EU-Risk) used this approach in which building damage is estimated from response spectral ordinates (peak responses of SDOF systems).

Although SDOF systems provide a much better a much better basis for estimating possible damage in buildings than peak ground acceleration or peak ground velocity, they still have a number of important disadvantages. Among others, SDOF systems cannot account for the contribution of higher modes, which are particularly important for predicting acceleration demands in buildings. Furthermore, even if displacements response spectrum ordinates are used, they only provide a measure of the overall lateral deformation in the building and do not take into account concentrations in lateral deformations in certain stories that usually occur in buildings.

It is well know that structural damage and many kinds of nonstructural damage in buildings are the result of lateral deformations. In particular, several studies have concluded that the structural response parameter that is best correlated with seismic damage is the peak interstory drift ratio, which is defined as the difference in lateral displacements in between two consecutive floors normalized by the interstory height. Similarly, other studies have shown that damage to contents and many types of nonstructural components is primarily related to peak floor accelerations and to floor spectral ordinates. Hence, much better performance estimates can be achieved by first computing peak interstory drift demands and peak floor acceleration demands. However, conventional analysis techniques (e.g., finite element models) require a great deal of time to generate building models and to run the analyses which makes the process extremely time consuming. Furthermore, the information that is required to build these improved building models is usually not available to the engineers who are interested in assessing the seismic performance of large inventories of buildings.

The objective of this paper is to present new analysis tools for rapid assessment of building response. The new analysis tools are based on a continuous model that consists of a combination of a flexural beam and a shear beam. By modifying a single parameter this model can consider lateral deformations varying from those of a flexural beam all the way to those of shear beam. Hence, it permits to account for a wide range of modes of lateral deformation that represent more closely those occurring in multistory buildings. Mode shapes, modal participation factors and period ratios required to compute the response of the model are all computed with closed-form solutions and are a function of only one parameter. This provides a highly efficient computational tool which at the same time only requires a minimum amount of information about the building whose seismic response is being assessed, making it then particularly valuable when evaluating large inventories of buildings. New types of spectra and response maps based on this building model are also presented.

Simplified Building Model

The simplified model consists of a linear elastic continuum model. Continuum models have been proposed before for approximating the response of buildings to wind or seismic forces. For a review of previously-proposed models the reader is referred to Miranda and Taghavi (2005) and Miranda and Akkar (2005). The proposed continuum model consists of a combination of a flexural cantilever beam and a shear cantilever beam deforming in bending and shear configurations, respectively (Figure 1). It is assumed that along the entire length of the model, both beams undergo identical lateral deformations. Furthermore, mass and lateral stiffness are assumed to remain constant along the height of the building.

As shown by Miranda and Akkar (2005), the response of uniform shear-flexural model shown in Figure 1 when subjected to an horizontal acceleration at the base $\ddot{u}_g(t)$ is given by the following partial differential equation:

$$\frac{\rho}{EI}\frac{\partial^2 u(x,t)}{\partial t^2}+\frac{c}{EI}\frac{\partial u(x,t)}{\partial t}+\frac{1}{H^4}\frac{\partial^4 u(x,t)}{\partial x^4}-\frac{\alpha^2}{H^4}\frac{\partial^2 u(x,t)}{\partial x^2}=-\frac{\rho}{EI}\frac{\partial^2 u_g(t)}{\partial t^2} \tag{1}$$

where ρ is the mass per unit length in the model, H is the total height of the building, $u(x,t)$ is the lateral displacement at non-dimensional height $x=z/H$ (varying between zero at the base of the building and one at roof level) at time t, c is the damping coefficient per unit length, EI is the flexural rigidity of the flexural beam and α is the lateral stiffness ratio defined as

$$\alpha = H\sqrt{\frac{GA}{EI}} \tag{2}$$

where GA is the shear rigidity of the shear beam. The lateral stiffness ratio, α is a dimensionless parameter that controls the degree of participation of overall flexural and overall shear deformations in the continuous model and thus, it controls the lateral deflected shape of the model. A value of α equal to zero represents a pure flexural model (Euler-Bernoulli beam) and a value of $\alpha \rightarrow \infty$ corresponds to a pure shear model. Intermediate values of α correspond to multistory buildings that combine overall shear and flexural lateral deformations.

The mode shapes of the simplified model are given by (Miranda and Taghavi, 2005):

$$\phi_i(x)=\sin(\gamma_i x)-\gamma_i\beta_i^{-1}\sinh(x\beta_i)-\eta_i\cos(\gamma_i x)+\eta_i\cosh(\beta_i x) \tag{3}$$

where β_i and η_i are nondimensional parameters for the ith mode of vibration which are given by

$$\beta_i=\sqrt{\alpha^2+\gamma_i^2} \tag{4}$$

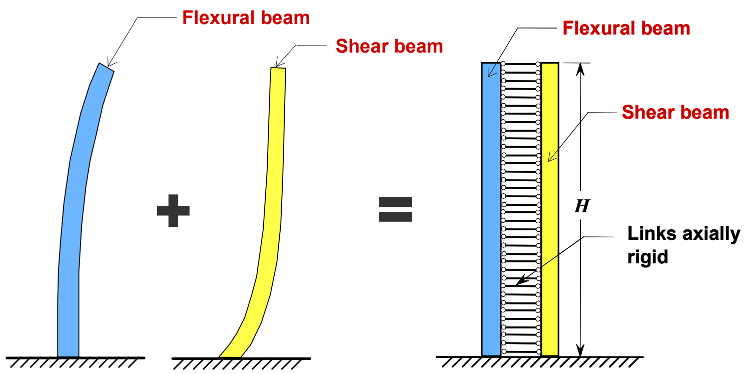

Figure 1. Simplified continuum model to estimate the seismic response of buildings.

$$\eta_i = \frac{\gamma_i^2 \sin(\gamma_i) + \gamma_i \beta_i \sinh(\beta_i)}{\gamma_i^2 \cos(\gamma_i) + \beta_i^2 \cosh(\beta_i)} \tag{5}$$

and γ_i is the eigenvalue of the *i*th mode of vibration corresponding to the *i*th root of the following characteristic equation:

$$2 + \left[2 + \frac{\alpha^4}{\gamma_i^2 \beta_i^2}\right] \cos(\gamma_i)\cosh(\beta_i) + \left[\frac{\alpha^2}{\gamma_i \beta_i}\right] \sin(\gamma_i)\sinh(\beta_i) = 0 \tag{6}$$

Periods of vibration corresponding to higher modes can be computed as a function of the fundamental period of vibration of the building T_1 by using period ratios computed as

$$\frac{T_i}{T_1} = \frac{\beta_1 \gamma_1}{\beta_i \gamma_i} \tag{7}$$

Since the masses are assumed to remain constant, the modal participation factors Γ_i can be computed with the following equation:

$$\Gamma_i = \frac{\int_0^1 \phi_i(x)dx}{\int_0^1 \phi_i^2(x)dx} \tag{8}$$

Integrals shown in equation (8) can be solved in closed-form solution. Readers interested in these closed-form solutions are referred to Miranda and Akkar (2005). As shown by these equations, mode shapes and modal participation factors, which control the spatial distribution of seismic demands, are fully defined only on a single parameter, the lateral stiffness ratio α.

While assuming the mass to remain constant along the height of buildings is reasonable in most cases, assuming that the lateral stiffness remains constant along the height of the building is perhaps only a reasonable assumption for one to three-story buildings. However, Miranda and Taghavi (2004) have shown that the product of modal shapes and modal participation factors as well as period ratios are relatively robust and are not significantly affected by reductions in lateral stiffness. In the same study, it was similarly shown that reduction in masses along the height of the building also do not affect significantly the dynamic characteristics of the model. Although Miranda and Taghavi (2005) provided expressions to compute the dynamic characteristic of non-uniform buildings, they concluded that in many cases, using the dynamic characteristics of uniform models could provide reasonable approximations to the dynamic characteristics of non-uniform models.

Relative Displacements

The contribution of the *i*th mode of vibration to the lateral displacement (relative to the ground) at non-dimensional height $x=z/H$ at time t is given by

$$u_i(x,t) = \Gamma_i \phi_i(x) D_i(t) \tag{9}$$

where Γ_i is the modal participation factor of the *i*th mode of vibration, $\phi_i(x)$ is the amplitude of *i*th mode at nondimensional height x, and $D_i(t)$ is the relative displacement response of a SDOF system, with period T_i and modal damping ratio ξ_i corresponding to those of the *i*th mode of vibration, subjected to ground acceleration $\ddot{u}_g(t)$. The product $\Gamma_i \phi_i(x)$ controls the spatial variation of the contribution of the *i*th mode to the total response, while $D_i(t)$ controls its time variation. Assuming that the structure remains elastic and that it has classical damping, the displacement at non-dimensional height $x=z/H$ at time t is given by

$$u_i(x,t) = \sum_{i=1}^{m} \Gamma_i \phi_i(x) D_i(t) \tag{10}$$

where m is the number of modes contributing significantly to the response. Taghavi and Miranda (2005) have shown that for most buildings with 30 or less stories only three modes are necessary for each direction. More recently, Reinoso and Miranda (2005) have shown that for high rise buildings and response parameters strongly influenced by higher modes (such as floor accelerations) including five or six modes may be necessary.

Taghavi and Miranda (2005) compared the response computed with the simplified model to that computed with detailed finite-element models of a ten-story steel moment resisting frame building and a twelve-story reinforced concrete building whose properties were available in the literature. Additionally, they compared the response computed with the model to that recorded in four instrumented buildings in California that have been subjected to earthquakes. In all cases, it was shown that the simplified model provided very good results. More recently, Reinoso and Miranda (2005) validated the model by comparing the response computed with the simplified continuous model to that recorded in five high rise buildings in California in various earthquakes.

The simplified method of analysis is currently being evaluated by comparing the seismic response recorded in a large number of instrumented buildings in California to that computed with the model. Figures 2 to 5 show examples comparing relative displacement (relative to the base of the building) time histories of five instrumented buildings in California. All of these analyses have been conducted assuming that the lateral stiffness and mass of the continuous system remains constant along the height of the building, and models were fully defined only using three parameters, namely the fundamental period of vibration of the building, a damping ratio that characterizes the damping in the model and the lateral stiffness ratio. In equations 9 and 10 one could use different damping ratios for computing $D_i(t)$ for each mode. However, for simplicity and in order to keep the number of parameters to a minimum, here it has been assumed the same damping ratio for all modes. Furthermore, the base of the model has been assumed as fixed and torsional deformations have been neglected. As shown in these figures, despite the important simplifications, the model is capable of capturing relatively well the peak and the most important features of the response of the buildings.

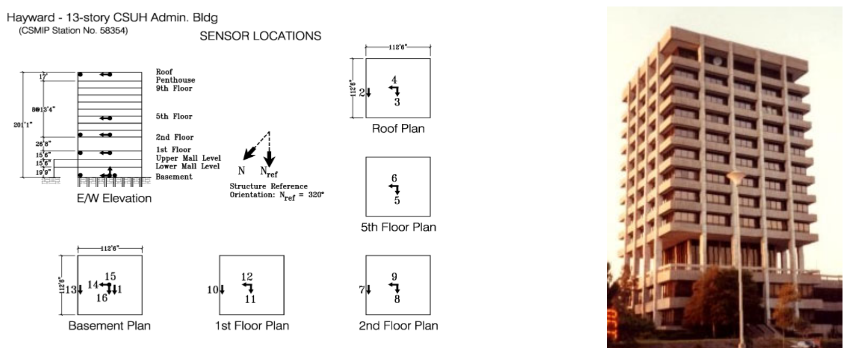

Figure 2. Sensor location and photograph of a 13-story RC building in Hayward California.

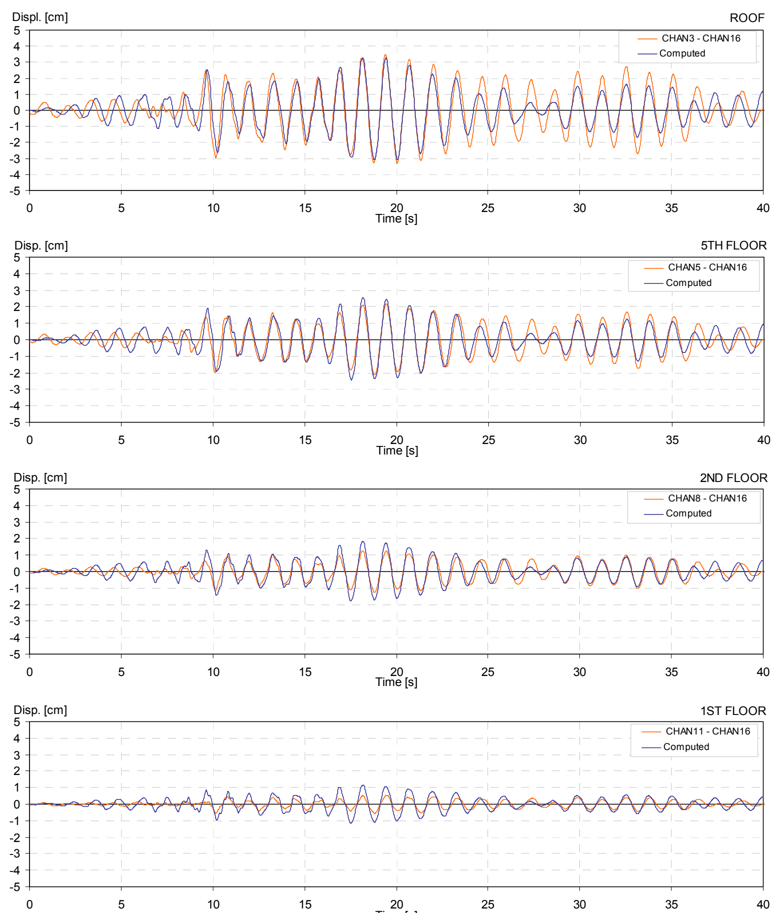

Figure 3. Comparison of computed and recorded relative displacements in the NS components of the 13-story building in Hayward California during the 1989 Loma Prieta earthquake.

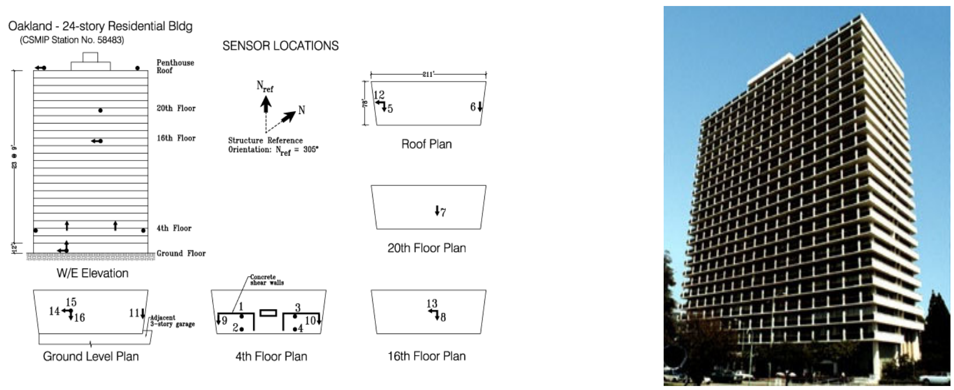

Figure 4. Sensor location and photograph of a 24-story RC building in Oakland California.

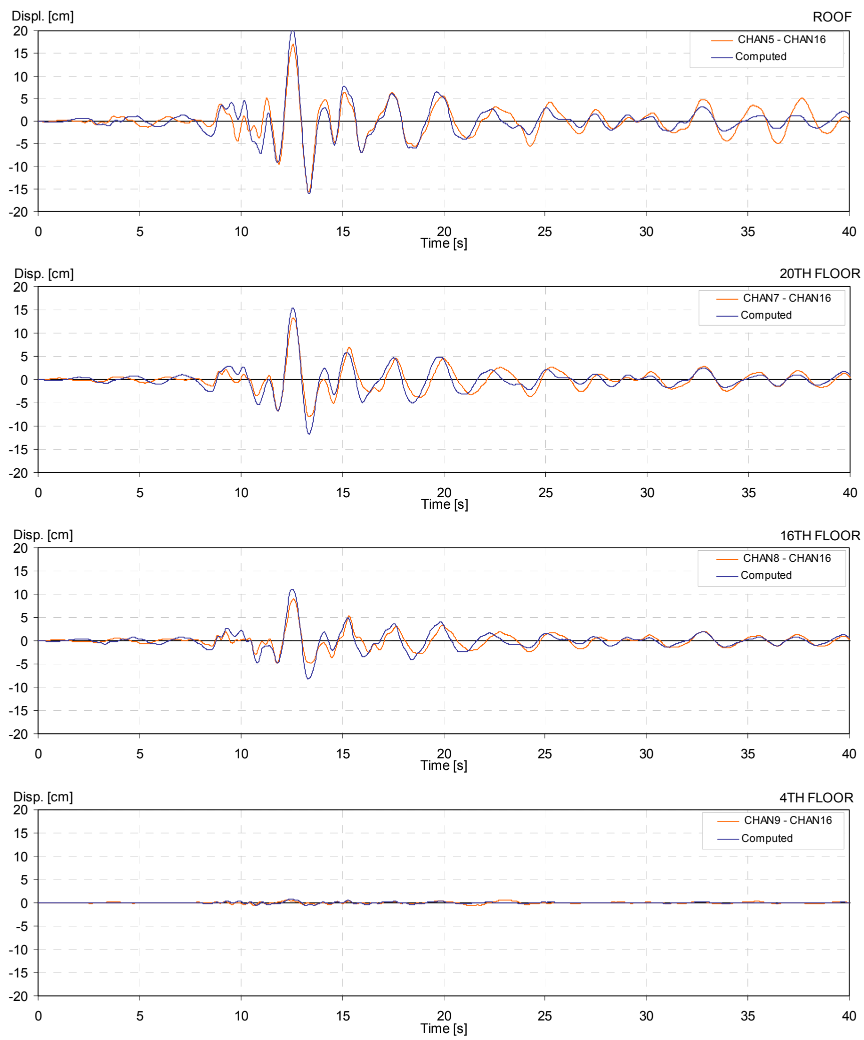

Figure 5. Comparison of computed and recorded relative displacements in the NS components of the 24-story building in Oakland California during the 1989 Loma Prieta earthquake.

Interstory Drift Ratios

The interstory drift ratio at the *j*th story can be computed as

$$IDR(j,t) = \frac{1}{h_j}\sum_{i=1}^{m}\Gamma_i\left[\phi_i\ (x_{j+1}) - \phi_i\ (x_j)\right]D_i(t) \tag{11}$$

where h_j is the floor to floor height of the jth story, and $\phi(x_{j+1})$ and $\phi(x_j)$ are the mode shape values corresponding to the *j*th+1 and *j*th floor computed with equation (3), respectively. If the interstory height is assumed to remain constant along the height of the building, it can be shown that for buildings with 6 or more stories a relatively good estimation of the interstory drift at non-dimensional height $x=z/H$ at time t can be computed with

$$IDR(j,t) \approx \theta(x,t) = \frac{1}{H}\sum_{i=1}^{\infty}\Gamma_i\phi_i^{'}(x)D_i(t) \tag{12}$$

where H is the total building height above ground, □(x,t) is the rotation in the simplified model at height x at time t, and $\phi_i^{'}(x)$ is the first derivative of the ith mode shape $\phi_i(x)$ with respect to non-dimensional height x. The derivative of the mode shapes with respect to non-dimensional height x is obtained by taking the derivative of Eq. (3) with respect to x as follows:

$$\phi_i^{'}(x) = \gamma_i\ \cos(\gamma_i x) - \gamma_i\ \cosh(\beta_i x) + \eta_i\gamma_i\ \sin(\gamma_i x) + \eta_i\beta_i\ \sinh(\beta_i x) \tag{13}$$

Generalized Interstory Drift Spectrum

Motivated by the relatively good results produced by the model, Miranda and Akkar (2005) developed a new type of spectra that, unlike conventional response spectrum, the ordinates provide a direct estimation of peak interstory drifts that are likely to occur in buildings. The new spectrum, referred to as the *generalized interstory drift spectrum* (GIDS) can be considered as an extension of Iwan's drift spectrum (Iwan, 1997). However, unlike Iwan's drift spectrum which is only applicable to buildings that can be modeled as shear beams, the GIDS is capable of considering a wide range of buildings ranging from those that can be modeled as flexural beams all the way to those that can be modeled as shear beams. Besides being able to consider a wide range of types of deformation the GISD has other advantages. For example, the GISD is based on modal analysis techniques that are familiar to structural engineers while Iwan's spectrum is based on wave propagation techniques that are typically not known to structural engineers. Furthermore, the GIDS uses a classical damping which as shown in figures 3 and 5 is capable of reproducing the recorded response of buildings and avoids the problems that are encountered when using the damping model used in the conventional drift spectrum (the reader is referred to Kim and Collins, 2002 or Miranda and Akkar, 2005 for a description of these problems).

The ordinates of the *generalized interstory drift spectrum* (GIDS) are defined as the maximum peak interstory drift demand over the height of the building and are computed as

$$IDR_{\max} \equiv \max_{\forall t,x}\left|\theta(x,t)\right| \tag{14}$$

The generalized interstory drift spectrum is a plot of the fundamental period of the building in the abscissas versus IDR_{max} in the ordinates. Similarly to the response spectrum, the GIDS provides seismic demands for a family of systems with different periods of vibration. However, instead of having ordinates of maximum relative displacement, maximum relative velocity or maximum acceleration of SDOF systems, the GIDS provides a measure of peak interstory drift demands, which is a demand parameter that is better correlated with damage. In particular, the GIDS provides a rapid estimation of peak interstory drift demand in buildings with different periods of vibration.

If the same damping ratio is used for the m contributing modes, then the model is fully defined by using only four parameters: (1) the fundamental period of vibration of the building, T_1; (2) a modal damping ratio that represents the damping ratio in the building, ξ; (3) the lateral stiffness ratio α; and (4) the building height, H. Since the derivative of the modes, modal participation factors and period ratios can be computed in closed-form solution, the GIDS is computationally very efficient, requiring just a few seconds in most personal computers. If empirical relations between building height and fundamental period are used, the number of parameters is then reduced from four to three.

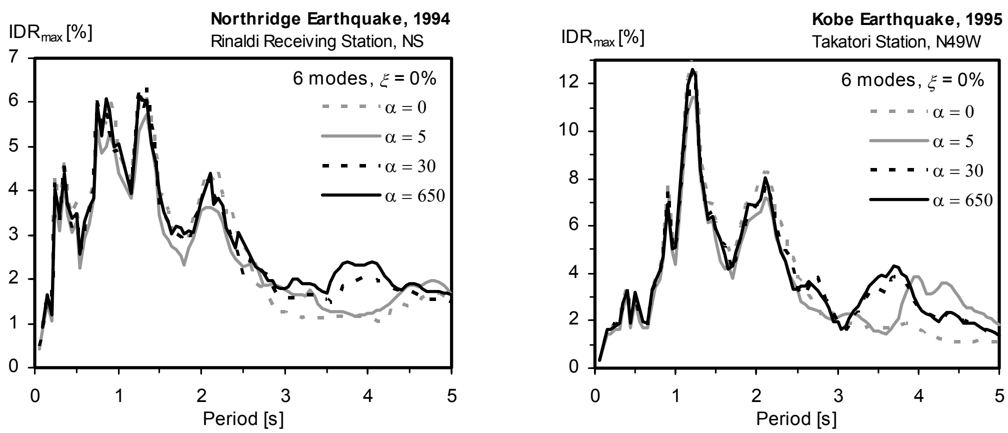

Figure 6. Influence of α on generalized interstory drift spectra (after Miranda and Akkar, 2005).

Figure 6 presents generalized interstory drift spectra for four different values of α values computed for the NS component of the Rinaldi Receiving Station from the 1994 Northridge earthquake and the N49W component of the Takatori Station from the 1995 Hyogo-ken-Nambu (Kobe) earthquake. Results presented in this figure were computed using Eqs. (13) and (14) considering the first six modes of vibration (i.e., m=6). For a given fundamental period of vibration, the total height of the model needed in Eq. (13) was computed using the relationship used for steel moment-resisting frames in the 1997 UBC code (ICBO, 1997), namely, T_1=0.0853$H^{0.75}$, where H is in meters. It can be seen that the influence of α is relatively small for fundamental periods of vibration smaller than 1.5s. However, for longer fundamental periods of vibration the differences become larger. In particular, it can be seen that interstory drift demands can be larger or smaller than those computed with a model corresponding approximately to a shear beam (α=650) indicating that Iwan's drift spectrum may underestimate or overestimate drift demands for buildings that cannot be modeled as shear beams.

Floor Accelerations Demands

While structural damage and many kinds of nonstructural damage are primarily caused by interstory drift demands, Miranda and Taghavi (2005) have shown that damage to building contents, ceilings, light fixtures, piping and many other types of nonstructural components is primarily related to peak floor accelerations and to floor spectral ordinates. The simplified model shown in figure 1 can also be used to estimate floor acceleration demands in buildings. Unlike lateral displacements or interstory drifts that are often dominated by the fundamental mode of vibration, floor accelerations are typically strongly influenced by higher mode response even more building of moderate height.

The total floor acceleration at non-dimensional height *x=z/H* can be computed as

$$\ddot{u}^t(x,t) \cong \ddot{u}_g(t) + \sum_{i=1}^{m} \Gamma_i \, \phi_i(x) \ddot{D}_i(t) \tag{15}$$

where $\ddot{D}_i(t)$ is the relative acceleration of a SDOF system with a period of vibration equal to that of the *i*th-mode of vibration of the structure. In the proposed method, the damping ratio in all modes is assumed to be the same and equal to a damping ratio that approximately characterizes the damping in the structure. It should be noted that Eq. (15) would be exact for the building shown in figure 1 only if the actual modes shapes, frequencies of vibration and modal participation factors of the building are used, and only if the summation included an infinite number of modes. Therefore, the main sources of error in the proposed method when applied to real buildings responding elastically are: (a) truncation error (e.g., only considering the first three modes of vibration); (b) use of approximate mode shapes and approximate modal participation factors; and (c) simplified representation of the damping in the structure.

Equation (15) can be also be written in terms of, more familiar, absolute modal acceleration times histories as follows:

$$\ddot{u}^t(x,t) \cong \left[1 - \sum_{i=1}^{m} \Gamma_i \, \phi_i(x)\right] \ddot{u}_g(t) + \sum_{i=1}^{m} \Gamma_i \, \phi_i(x) \ddot{D}_i^t(t) \tag{16}$$

where $\ddot{D}_i^t(t)$ is the absolute acceleration time history of a SDOF system with a period of vibration equal to that of the *i*th-mode of vibration of the structure.

Peak floor accelerations computed with equation (15) for four buildings are shown in figure 7. In this figure peak floor accelerations at instrumented floors have been normalized by the peak acceleration recorded at the base of the building and are indicated in the figure by red squares. Also shown in the figure are the normalized peak floor acceleration demands computed with the proposed simplified method. It can be seen that again, the proposed method is capable reproducing relatively well the variation of acceleration demands along the height of the buildings. Furthermore, it can be seen that the distribution of acceleration demands may differ considerably from the linear variation that is commonly assumed in many building codes. Taghavi and Miranda (2003) have shown that, unlike present U.S. recommendations which for the design of nonstructural components assume that the variation of floor acceleration demands is period-independent, this variation is strongly influenced by the fundamental period of vibration of the structure and also, but to a lesser degree, by the lateral stiffness ratio.

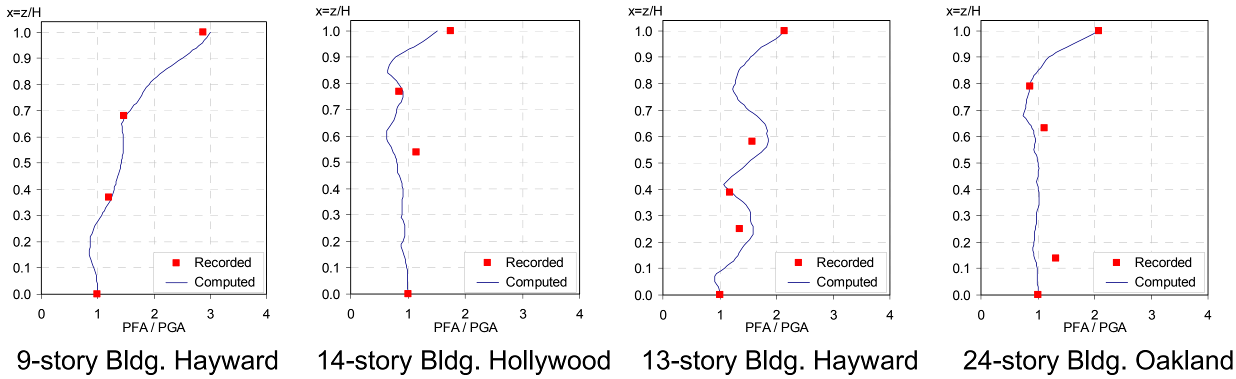

Figure 7. Comparison of computed and recorded variation of peak floor acceleration demands in four reinforced-concrete buildings located in California.

While capturing peak floor accelerations with simplified models and a limited number of parameters in building is challenging, the estimation of floor acceleration time histories and ordinates of floor spectra are even more challenging as they depend on an accurate estimation of frequencies of vibration of higher modes. In the proposed method the same parameter is used to control the spatial variation of the motion within the building and period ratios, therefore it is interesting to explore the level of accuracy that can be obtained with the proposed simplified analysis method when estimating these response parameters that are strongly influenced by higher modes. Figures 17 to 20 show comparison of recorded floor acceleration time histories with those computed with the approximate method and comparison of 5% damped floor spectra computed with recorded and approximate acceleration time histories. As shown in these figures the model is able to capture quite well these response parameters as well.

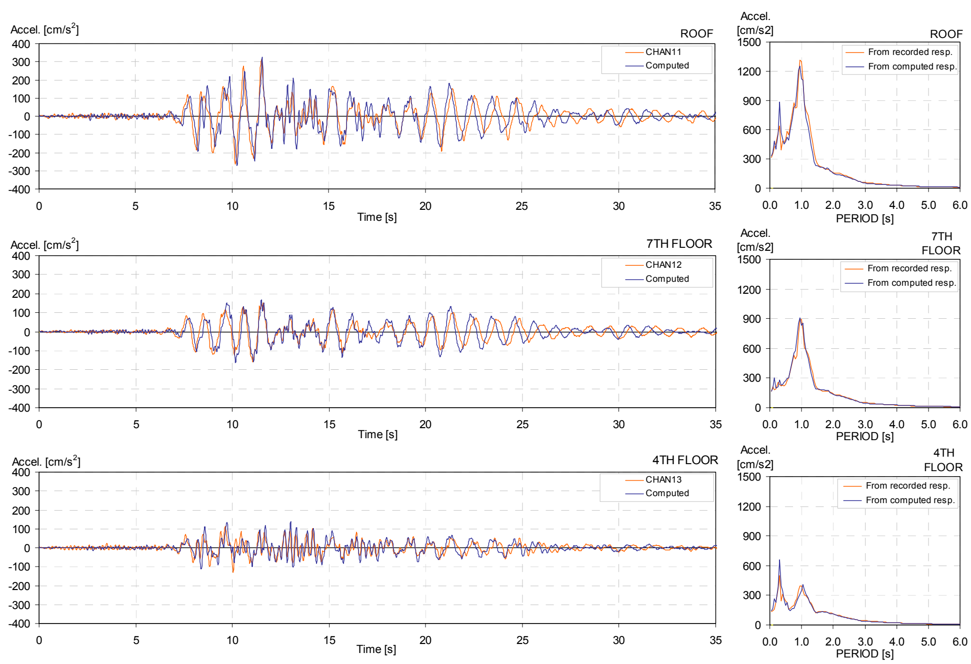

Figure 8. Comparison of computed and recorded acceleration demands in the EW components of a 9-story building in San Bruno California.

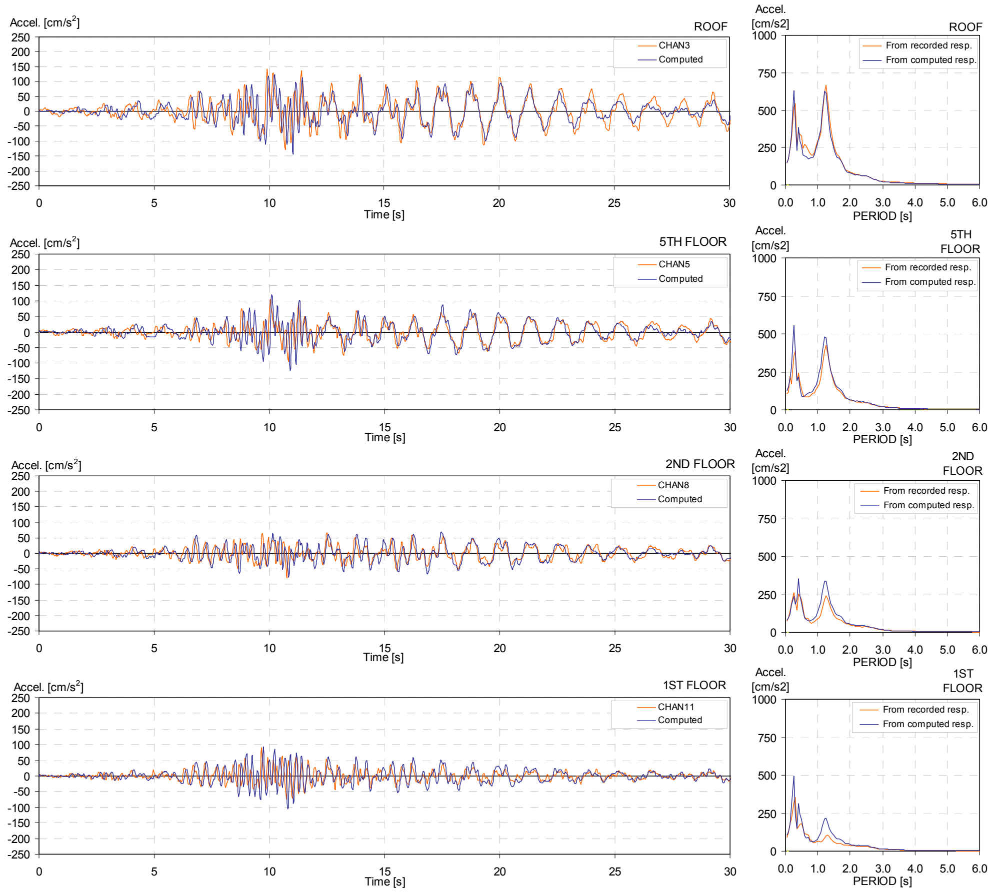

Figure 9. Comparison of computed and recorded acceleration demands in the NS components of a 13-story building in Hayward California.

Rapid Damage Assessment

Many studies have indicated that interstory drift ratios are correlated very well with structural damage in buildings. Because of its many computational advantages, the generalized interstory drift spectrum described in the previous sections, can be used in rapid damage assessment at urban areas with wide range of structural systems for a major earthquake event. This can be accomplished by analyzing continuous models corresponding to different structural systems at instrumented locations. A typical example is presented for the Los Angeles Metropolitan area in Figure 10. This three dimensional contour map was prepared by using the ground motions recorded at 91 strong motion stations that were triggered during the 1994 Northridge earthquake in the city of Los Angeles and San Fernando Valley.

Ground motions were downloaded from the website of COSMOS Virtual Data Center (http://db.cosmos-eq.org) and all of them are either free-field records or ground motions recorded at the ground floor of one-story buildings. Figure 10 shows interstory drift demands for buildings with fundamental periods T_1=1.0s corresponding to mid-rise moment-resisting reinforced concrete frame buildings in this region. The value of α was taken as 12.5 and the mean fundamental period empirical relationship by Chopra and Goel (2000) was used to compute the building height as a function of the fundamental period of vibration. Mean interstory drift ratios of both horizontal components are computed at each recording station using 5 percent damping ratio.

The map presented in Figure 10 indicates that for the given distribution of strong motion stations, frame buildings with fundamental periods of approximately 1.0s located in the northern portion of the San Fernando Valley were subjected to large interstory drift demands and therefore more susceptible to serious structural/nonstructural damage compared to those of stiffer or more flexible buildings in the same region. Another important observation from this map is the consistency of computed interstory drift demands with the reported rupture direction.

It should be noted that even though the elaboration of the map shown in figure 10 involves the computation of the seismic response of both directions of building models located at more than 90 recording stations, they can be computed in personal computers within a few minutes after an earthquake provided that ground motions are sent to a central location using telemetry. These maps can provide a valuable tool in rapid damage assessment as well as for planning purposes using various ground motion scenarios.

Instead of placing an identical structure at each location where a ground motion recording instrument is located, it is possible to use the ground motions recorded in the city to conduct an analysis of large number of existing structures within the city, hence extending the concept of structural building analysis to structural "city" analysis.

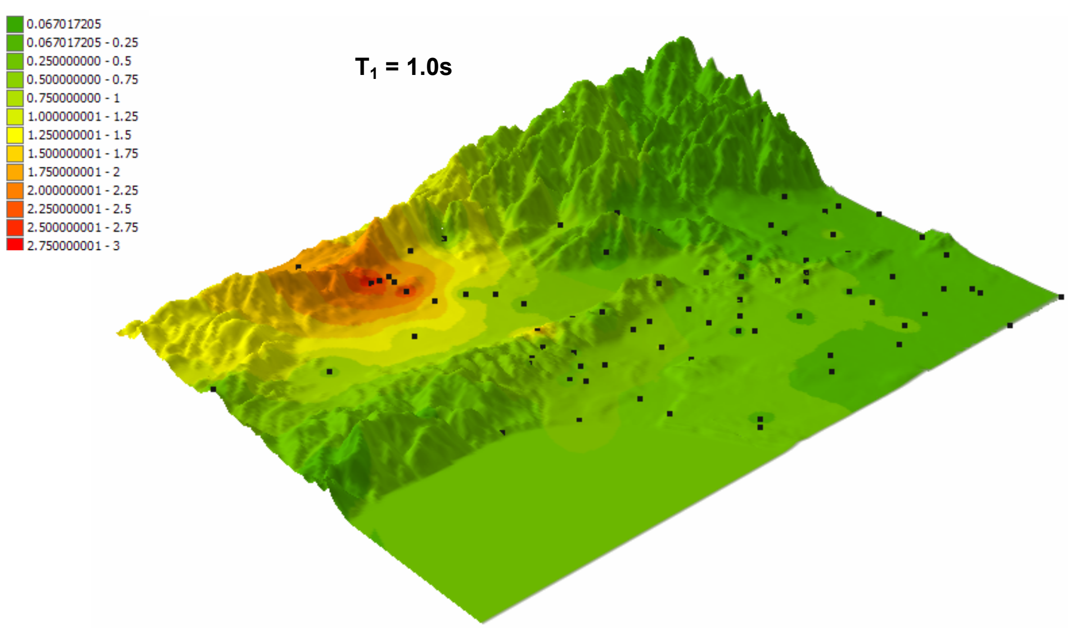

Figure 10. Maximum IDR_{max} contour map for moment-resisting buildings with fundamental periods of vibration of 1.0s computed using 91 recording stations deployed in the Los Angeles metropolitan area and triggered during the 1994 Northridge earthquake.

Summary and Conclusions

New analytical tools for rapid building seismic response estimation aimed at rapid seismic performance assessment of large inventories buildings in urban areas have been presented. The simplified seismic analysis tools make use of continuum models consisting of a flexural beam coupled with a shear beam. Unlike sophisticated analysis models that require a significant amount of information and are computationally very demanding, the proposed analytical tool is fully defined by only three or four parameters. That is, only one or two parameters in addition to those required to define a linear elastic single degree of freedom system. Seismic response computation using the proposed analytical tool takes only fractions of a second in most personal computers, hence allows for the rapid assessment of hundreds of buildings, within few minutes after an earthquake. A typical case study from the 1994 Northridge earthquake was presented to demonstrate how the proposed procedure can be used as an efficient tool to serve for such a purpose.

It should be noted that the proposed analytical tools have not been developed as replacement of more refined and accurate models. The simplified models can be particularly helpful for the following applications:

1. Screening tool to identify buildings that are likely candidates for more detailed analyses.
2. Screening tool to identify buildings and urban regions that are more likely to be damaged in future earthquakes
3. As a tool to conduct parametric studies to identify structural parameters or ground motion parameters that increase seismic demands on buildings
4. Planning tool for emergency managers and city officials by using motions from previous ground motions or synthetic ground motions from possible future events.
5. For loss estimation of large inventories of building by insurance or reinsurance companies.
6. To provide early performance estimates within minutes of a seismic event.

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