DEVELOPMENT OF IMPROVED INTENSITY MEASURES AND IMPROVED SHAKEMAPS FOR LOSS ESTIMATION AND EMERGENCY RESPONSE

Eduardo Miranda, Marios Kyriakides and Qiang Fu

John A. Blume Earthquake Engineering Center
Stanford University

Abstract

An improved measure of ground motion intensity that is well correlated with structural and many kinds of nonstructural damage is presented. The proposed intensity measure is based on the peak interstory drift demand computed using a simplified continuous model that consists of a combination of a flexural beam and a shear beam. This new intensity measure accounts for the influence of higher modes and for concentrations of lateral deformation demands along the height of buildings. It is then proposed to compute this new intensity measure at all stations that recorded a seismic event in order to generate improved ShakeMaps for loss estimation and emergency response. The 2004 Parkfield event is used to illustrate both concepts.

Introduction

Interest in seismic hazard and the performance of structures in earthquakes has steadily increased in recent years. This interest has expanded from the seismology and earthquake engineering communities to national, state, county, city and local public officials, owners of critical facilities and utilities, emergency response organizations, insurance and other financial institutions, the media and the general public. In particular, there is a growing need for information of the intensity of earthquakes and their possible effects on structures within a few minutes of moderate and large magnitude events. Recent technological developments in instrumentation, storage, data transmission and increased computational power have allowed the generation and dissemination of valuable information of earthquakes in near real time. The most widely known example is ShakeMap which is computed and distributed within minutes of a seismic event and provides an instrumental measure of ground motion intensity.

Over the years various parameters have been proposed and used as measures of ground motion intensity. Of particular interest are parameters that are closely correlated to structural and nonstructural damage and that therefore can be used as demands parameters to identify whether damage is likely to occur and the severity of the damage. While it is possible to use response parameters of detailed nonlinear models of structures, these models require a great deal of information about the structure, require a significant amount of time to be developed, debugged and calibrated, require large computational power to run as well as many hours for interpreting their results, thus making the latter approach not practical for rapid assessment of large inventories buildings in urban areas. Therefore, there is a need for simplified ground motion intensity measures that require only a minimum amount of information, require only small amount of computation and yet their results are useful in identifying the capability of a ground motion to cause damage in structures.

The objective of this work is to summarize the results of an investigation whose main objectives were to develop improved ground motion intensity measures and improved ShakeMaps for loss estimation and emergency response, as well as to illustrate the computation and use of these new tools by using ground motions recorded during the 2004 Parkfield earthquake.

Commonly Used Intensity Measures

Existing approaches to characterize ground motion intensity for design, loss estimation and emergency response typically fall in one of the following categories:

(a) *Using peak ground motion parameters such as peak ground acceleration (PGA) or peak ground velocity (PGV)*. For example, in ShakeMap the instrumental intensity is computed from empirically-derived relations between Modified Mercalli Intensity (MMI) and PGA and PGV (Wald, et al. 1999a). When generating instrumental intensity ShakeMaps, the instrumentally-derived MMI is first computed from the PGA-MMI empirical relationship and if the instrumental intensity value determined from peak acceleration is equal or larger than VII, then the instrumental intensity derived from the the PGV-MMI empirical relationship is used (Wald, et al. 1999a).

(b) *Using response spectral ordinates at a few selected periods.* ShakeMap also produces information of 5% damped pseudo-acceleration spectral ordinates at 0.3s, 1.0s and 3.0s for earthquakes with magnitudes larger than 5.5. While originally envisioned as a pre-event planning tool and in a time when rapid data was not available, *HAZUS* has recently been enhanced to facilitate rapid post-event evaluation of damage and loss using ShakeMap data (Kircher, 2003). *Hazus* uses the peak response of linear single-degree-of-freedom systems with periods of vibration of 0.3s and 1.0s available in ShakeMap to estimate damage and losses to buildings and other types of structures.

(c) *Using peak ground motion parameters and response spectral ordinates at a few selected periods.* This approach is a combination of the two previously described approaches. This approach was recently incorporated in the ATC-54 (Guidelines for utilizing strong-motion and ShakeMap data in post-earthquake response) in which PGA, and spectral ordinates at 0.3s and 1.0s are used for post-earthquake evaluation of existing buildings by comparing the acceleration demand computed with ShakeMap acceleration data to the design lateral-force coefficient (Rojahn et al., 2003).

(d) *Using the linear spectral ordinate at the fundamental period of the building*. Current codes in the United States, such as the *International Building Code* (ICC 2000), define earthquake hazard in terms of acceleration spectral ordinates at the first mode period of vibration, *Sa(T1).* This approach was also used in the ATC-54 project for post-earthquake evaluation of existing buildings in order to estimate the peak roof displacement (Rojahn et al., 2003). The use of the pseudo acceleration spectral ordinate at the fundamental period of the building has also been extensively used to characterize the ground motion intensity in SAC (Cornell et al.

2002) and PEER (Deierlein, et al., 2003; Miranda and Aslani, 2003; Krawinkler and Miranda, 2004; Moehle and Deierlein, 2004).

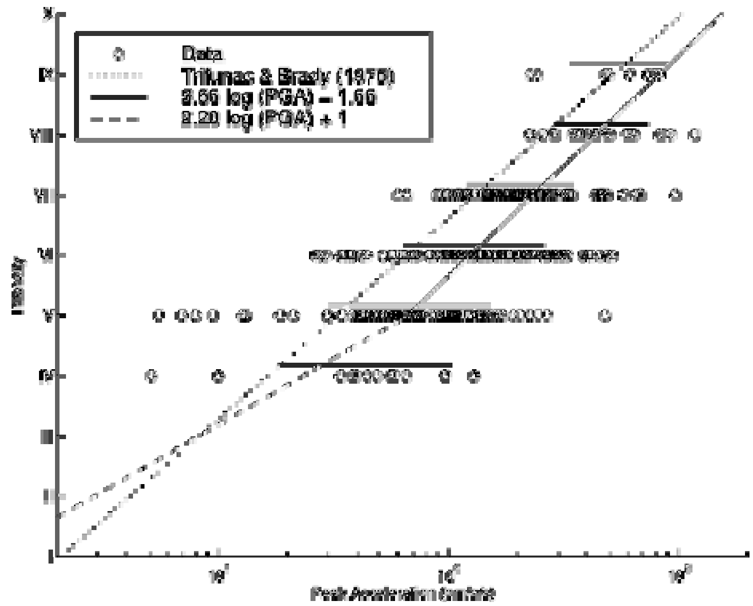

Figure 1. Empirical relationship between PGA and MMI used im ShakeMap (Wald et al. 1999b).

The empirical relationship between PGA and MMI that is incorporated in the generation of ShakeMaps is shown in figure 1. As shown in this figure there is a significant scatter between PGA and MMI. For example, according to Wald et al. (1999b) 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 means that a PGA=0.3g could produce damages ranging from “very light damage” to “moderate to heavy damage”. Similarly, according to figure 1 a MMI of V could have been produced by peak ground acceleration ranging from values as small as 5 cm/s^2 to values as large as 450 cm/s^2 (a peak ground acceleration almost two orders of magnitude larger!). Analytical work, as well as observations of earthquakes effects, including records of strong ground motion, consistently indicate that PGA is not a reliable parameter on which to base evaluations of seismic risk (ATC 1982; Aptikaev, 1980; Borg, 1980; McCann et al. 1980; Kennedy et al. 1984). Although a slightly better correlation is obtained using peak ground velocity for higher intensity levels, the correlation remains relatively low. One of the fundamental problems of using peak ground motion parameters to characterize the intensity of ground motion is that they do not differentiate between the seismic intensity on structures with different dynamic characteristics (different periods of vibration).

Using spectral ordinates offers a significantly better approach to characterize the ground motion intensity of different types of structure. The main advantage of this approach is that it incorporates information about the frequency content of the ground motion, hence it is able to account for differences in seismic intensity for systems with different periods of vibration. However there are two main shortcomings with this approach: (1) cannot account for possible concentrations of deformation demands in certain stories; and (2) the response of higher modes is neglected.

Improved Ground Motion Intensity Measure

Structural damage and many types of damage to nonstructural components are primarily the result of lateral deformations that occur from the relative displacement between consecutive floors. In particular, many studies have concluded that the structural response parameter that is best correlated with seismic damage is the peak interstory drift ratio (Algan 1982; Sozen 1983; Qi and Moehle 1991; Gülkan and Sozen 1999), which is defined as the difference in lateral displacements in between two consecutive floors normalized by the interstory height. Therefore, parameters that provide direct estimates of interstory drift demands in buildings constitute more reliable measures of ground motion intensity.

In 1997, Iwan introduced a simple and direct measure of drift demand for earthquake ground motions called the *drift spectrum* (Iwan 1997). Like the response spectrum, the drift spectrum is based on a relatively simple linear model. However, the drift spectrum differs from the response spectrum in that it is based on a continuous shear beam rather than a single-degree-of-freedom (SDOF) system. The most important advantages of this new powerful tool are that it takes into account the fact that interstory drift demands are not uniformly distributed along the height of buildings and considers the contribution of higher modes. Therefore, the drift spectrum results on more accurate estimations of maximum interstory drift demands than does the response spectrum. In his study, Iwan strongly recommended the use of drift spectrum in structural design and concluded that the drift spectrum was particularly useful in estimating drift demands in buildings subjected to pulse-like ground motions.

For many buildings, the shear beam model can lead to reasonable estimations of interstory drift demands. This is particularly true in the case of moment-resisting frame buildings whose beams are significantly stiffer than the columns and where axial deformations in the columns are negligible. In such cases, modes of vibration will be relatively similar to those of a shear beam. However, in buildings with bracing or shear walls or for moment-resisting buildings where the lateral stiffness provided by the columns is significant relative to that provided by the beams or where axial deformations are significant, the use of a shear beam is not an adequate model.

In this study an improved ground motion intensity measure is proposed referred to as *generalized interstory drift spectrum*, which is based on a continuous model that consists of a combination of a flexural beam and a shear beam, rather than only a shear beam. By modifying one parameter this model can consider lateral deformations varying from those of a flexural beam to those of shear beam. Hence, Iwan drift spectrum is a particular case of the proposed intensity measure. Furthermore, it permits to account for a wide range of modes of deformation that represent more closely those of 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. Hence, providing a highly efficient computational tool which only requires a minimum amount of information in order to be used. It can be used for analysis of individual buildings or for large groups of building within urban areas.

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. 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.

While assuming the mass to remain constant along the height of buildings is reasonable for most buildings, 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, provided that no abrupt reductions exists. In the same study, it was similarly shown that reductions of mass along the height of the building also do not affect significantly the dynamic characteristics of the model. It should be noted that Miranda and Taghavi (2005) provided expressions to estimate the dynamic characteristic of non-uniform buildings, but concluded that, in many cases, using the dynamic characteristics of uniform models could provide reasonable approximations to the dynamic characteristics of non-uniform models.

As shown by Miranda and Akkar (2005), the response of a uniform shear-flexural model 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

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

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

and γ_i is the eigenvalue of the ith mode of vibration corresponding to the ith 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 by only one parameter, the lateral stiffness ratio α.

The contribution of the ith 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 ith mode of vibration, $\phi_i(x)$ is the amplitude of ith 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 ith 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 ith 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

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

Equation (10) indicates that the estimation of relative displacements along the height of the building require consideration of an infinite number of modes of vibration. However, Taghavi and Miranda (2005) and Reinoso and Miranda (2005) have shown that for most buildings considering three to six modes in each building direction is enough to capture the main aspects of the response of buildings subjected to earthquakes. Therefore, equation (10) can be reduced to

$$u_i(x,t) \approx \sum_{i=1}^{m} \Gamma_i \phi_i(x) D_i(t) \tag{11}$$

where m is the number of modes contributing significantly to the response.

Taghavi and Miranda (2005) compared acceleration 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 acceleration demands computed with the model to those 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) compared acceleration demands computed with the simplified continuous model to that recorded in five high rise buildings in California in various earthquakes. However, those studies did not compare displacement response.

As part of this study the ability of the simplified model to estimate displacement time histories and peak lateral displacements was evaluated. As an illustration Figures 2 to 5 show examples comparing relative displacement (relative to the base of the building) time histories of two instrumented buildings in California. It is noted that 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 that the damping ratio is the same for all modes included in the analysis, therefore the models were fully defined by using only 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 to 11 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 that the damping ratio is the same 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 lateral deformation response of the buildings.

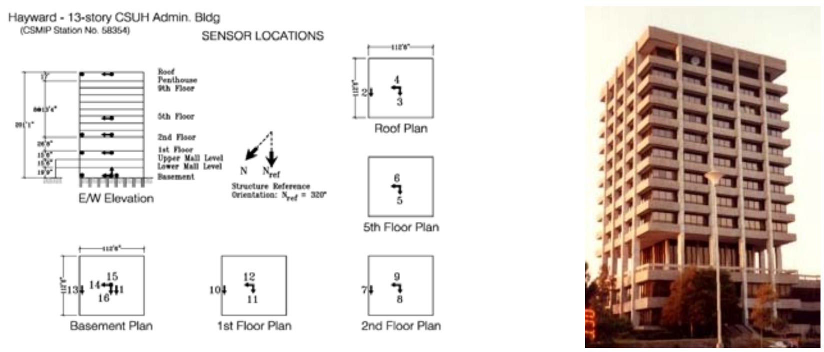

Fig 2. Sensor location and photograph of a 13-story RC building in California (CSMIP station 58354).

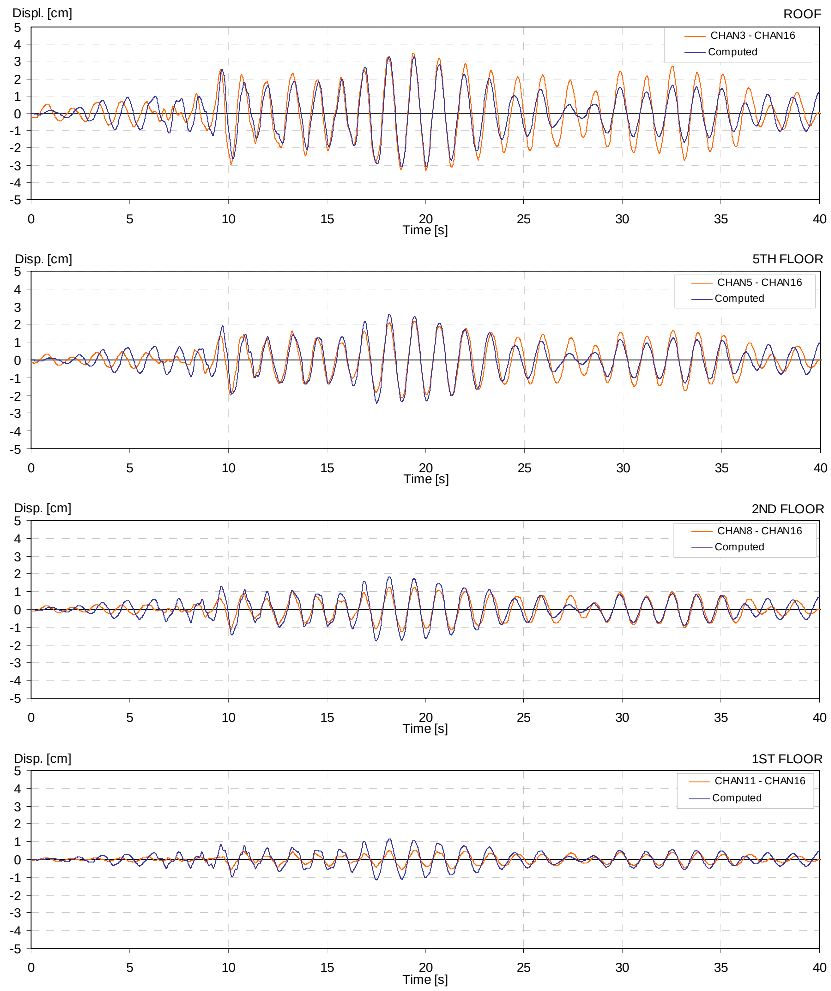

Fig. 3. Comparison of computed and recorded relative displacements in the NS components of the 13-story building in Hayward California during the 1989 Loma Prieta earthquake.

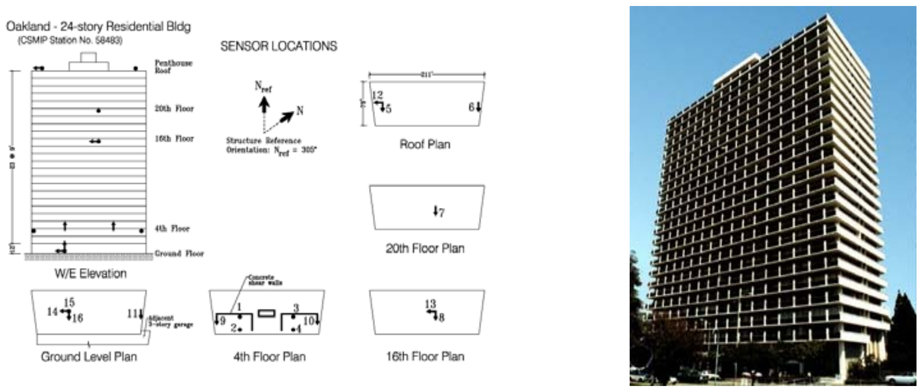

Fig. 4. Sensor location and photo of a 24-story RC building in California (CSMIP station 58483).

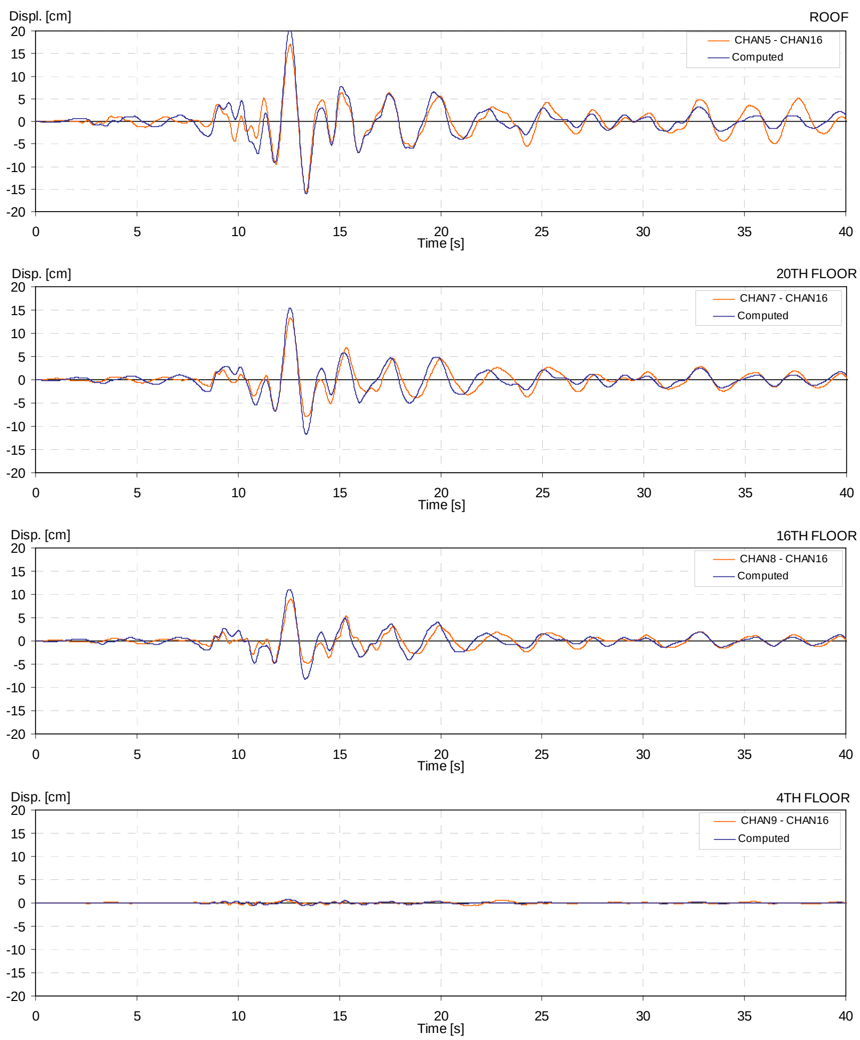

Fig. 5. Comparison of computed and recorded relative displacements in the NS components of the 24-story building in Oakland California during the 1989 Loma Prieta earthquake.

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

$$IDR(j,t) = \frac{1}{h_j} \sum_{i=1}^{n} \Gamma_i \left[\phi_i \, (x_{j+1}) - \phi_i \, (x_j) \right] D_i(t) \tag{12}$$

where h_j is the floor to floor height of the jth story, n is the number of modes in the building, and $\phi(x_{j+1})$ and $\phi(x_j)$ are the mode shape values corresponding to the jth+1 and jth 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

$$IDR(j,t) \approx \theta(x,t) = \frac{1}{H} \sum_{i=1}^{\infty} \Gamma_i \phi_i^{'}(x) D_i(t) \tag{13}$$

where H is the total building height above ground, $\theta(x,t)$ 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:

$$\phi_i^{'}(x) = \gamma_i \cos(\gamma_i x) - \gamma_i \cosh(\beta_i x) + \eta_i \gamma_i \sin(\gamma_i x) + \eta_i \beta_i \sinh(\beta_i x) \tag{14}$$

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

$$IDR_{\max} \equiv \max_{\forall t,x} \left| \theta(x,t) \right| \tag{15}$$

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 buildings. In particular, the GIDS provides a rapid estimation of peak interstory drift demand in buildings with different periods of vibration.

As mentioned before, 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 further reduced from four to three.

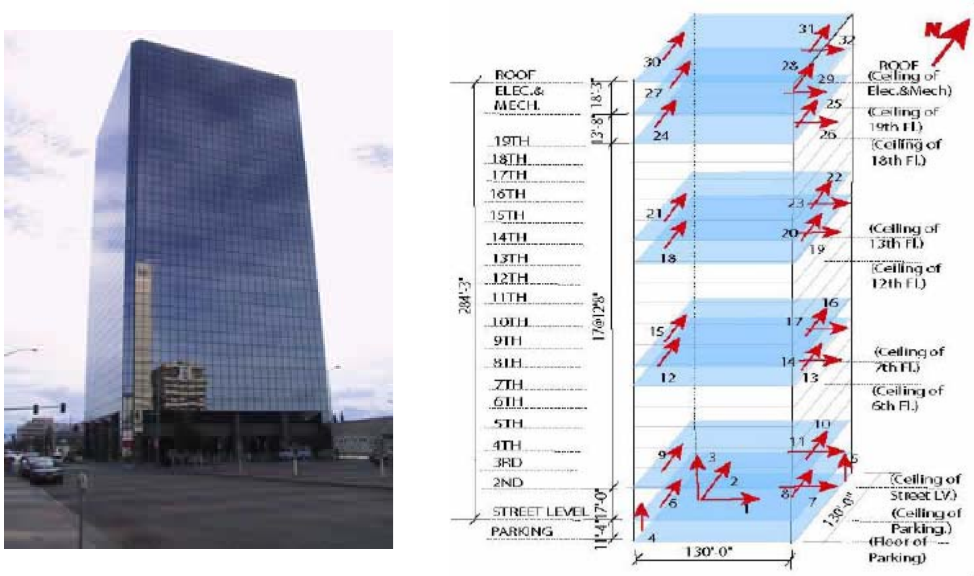

Fig. 6. Photograph and instrumentation layout of the Atwood Bldg. in Anchorage Alaska.

Validation of the simplified contious model for capturing interstory drifts is more complicated because most instrumented buildings only have accelerometers installed at 3 or 4 locations along the height. However, the United States Geological Survery has recently deployed dense instrumentation arrays in a few buildings. Figure 6 show the photograph and instrumentation layout of the Atwood building which is located in Anchorage, Alaska. As shown in this figure sensor are located at consecutive floors in the first and second floors, 7th and 8th floors, 13th and 14th floors, and 19th, 20th and 21st floors, which allows the computation of interstory drifts at the 1st, 8th, 14th, 20th and 21st floors.

On December 15th, 2003 a magnitude 3.7 earthquake occurred only at 18 km from the building. Although the event was very small, still it triggered the instruments and very high quality recording were obtained. Figure 7 shows a comparison of interstory drift time histories computed with the simplified model and those computed through double integration and substraction of acceleration time histories recorded in consecutive floors in the building. It can be seen that with exception of the first floor where the match is not good in the rest of the stories the results of the simplified model are remarkably good, especially if one considers the simplicity of the model.

Another building that offers a unique opportunity to validate the simplified continuous model is the Millikan library of the campus of the California Institute of Technology (CalTech). The instrumentation in this building was recently improved and now sensors are available on all floor levels. A photograph of the building and sketches of its plan and elevation are show in figure 8.

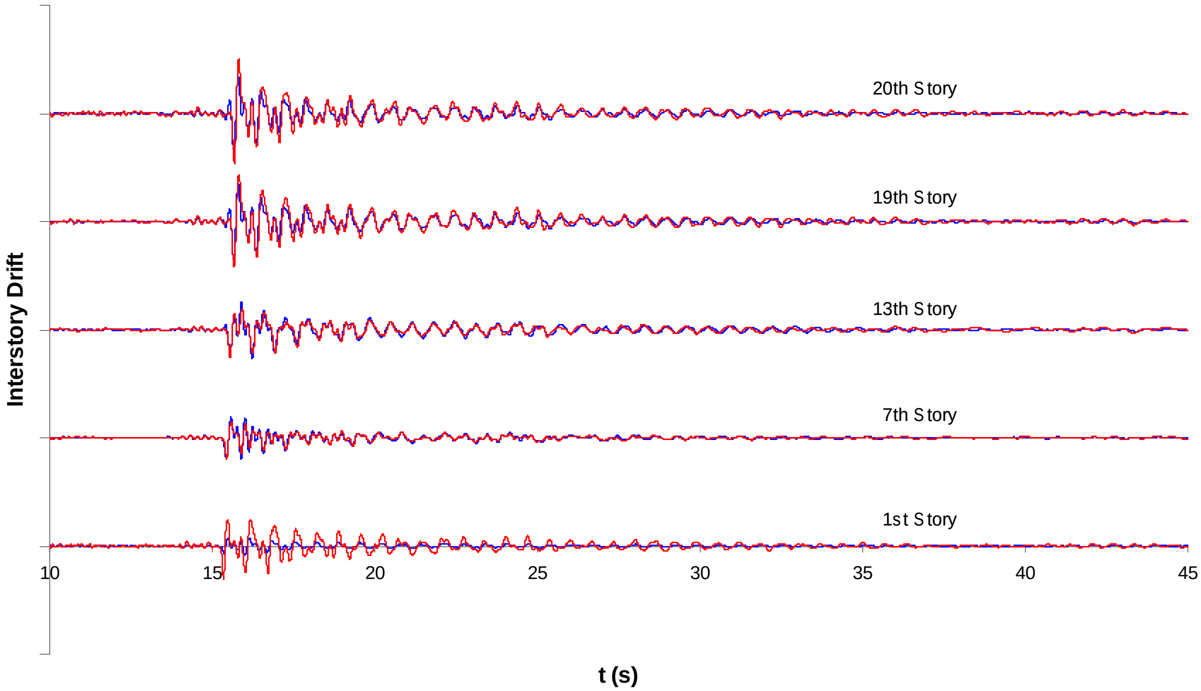

Fig. 7. Comparison of 'recorded' and computed interstory drift time histories in the Atwood Bldg. in Anchorage Alaska.

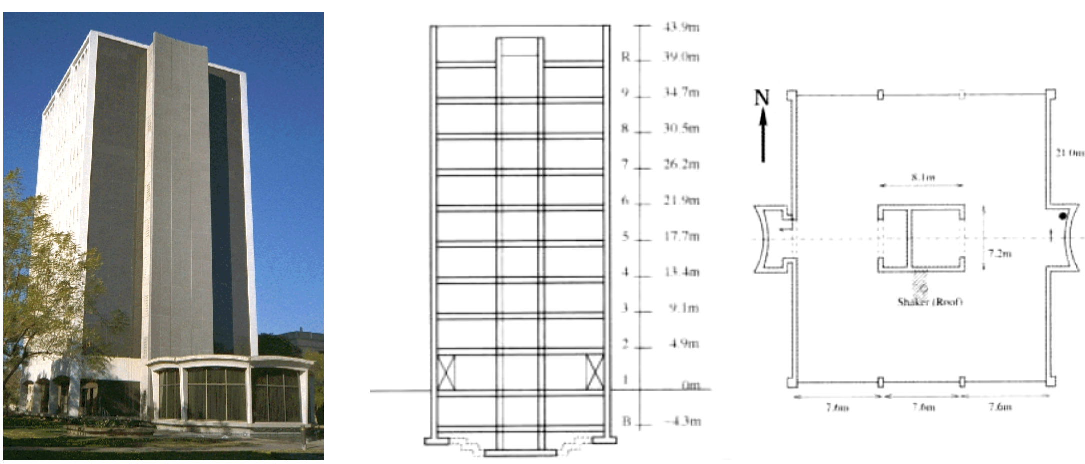

Fig.8. Photograph and instrumentation layout of the Millikan library on Pasadena, CA.

On September 4th, 2002 the Yorba Linda earthquake occurred within the Los Angeles metropolitan region. The event had a magnitude on 4.6. Although the event was small it triggered all instruments and high quality records were obtained. Figure 9 compares interstory drifts computed with the simplified continuous model and those computed from recorded accelerograms. Again it can be seen that the results are very good, especially if one considers the level of difficulty.

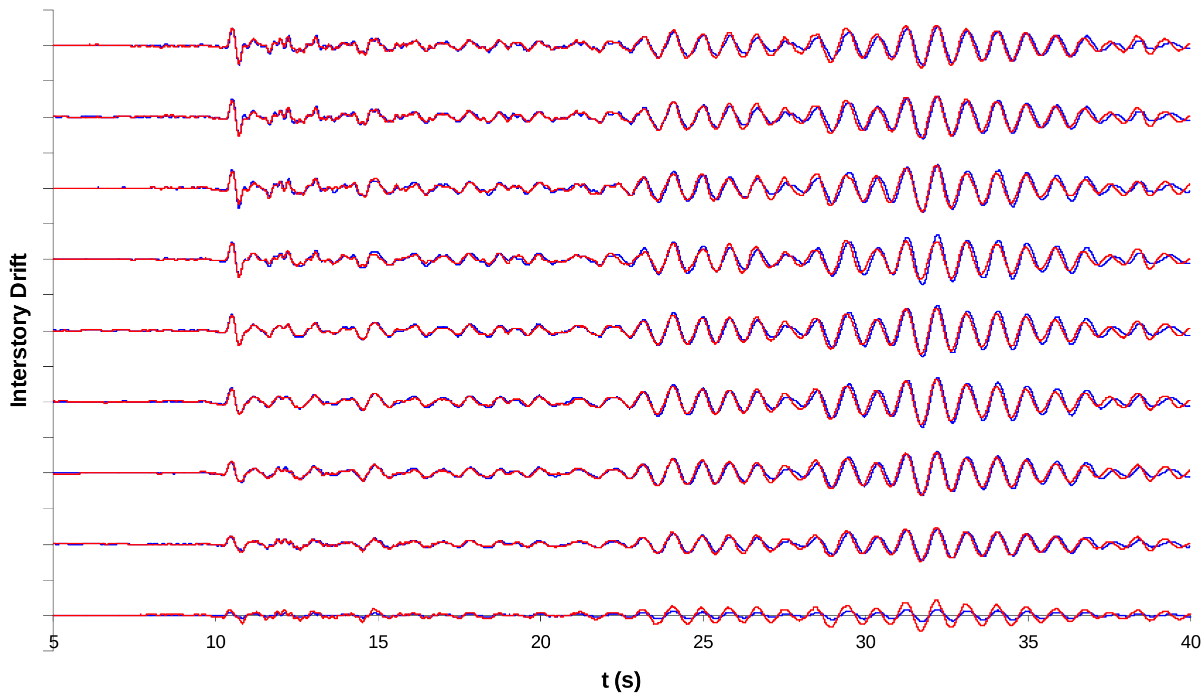

Fig. 9. Comparison of 'recorded' and computed interstory drift time histories in the EW component of the Millikan library on Pasadena California..

Improved ShakeMaps

Improved maps were computed and plotted for the 2004 Parkfield event. The area that was considered in shown in Fig. 10, where the trace of the San Andreas fault running in the NW-SE direction. The peak interstory drift was computed with all ground motions that recorded this event in the facility of the fault. This is equivalent to placing instrumented simplified models at all recording stations prior to the earthquake. This is shown schematically in Figure 11.

Improved Shake Maps provide direct estimates of interstory drift ratios that might have occurred in the area if buildings with a wide range of periods of vibration would have been there. Improved maps were computed for north-south and east-west components. Additionally, improved Shake Maps were also computed and generated for fault-normal and fault parallel directions.

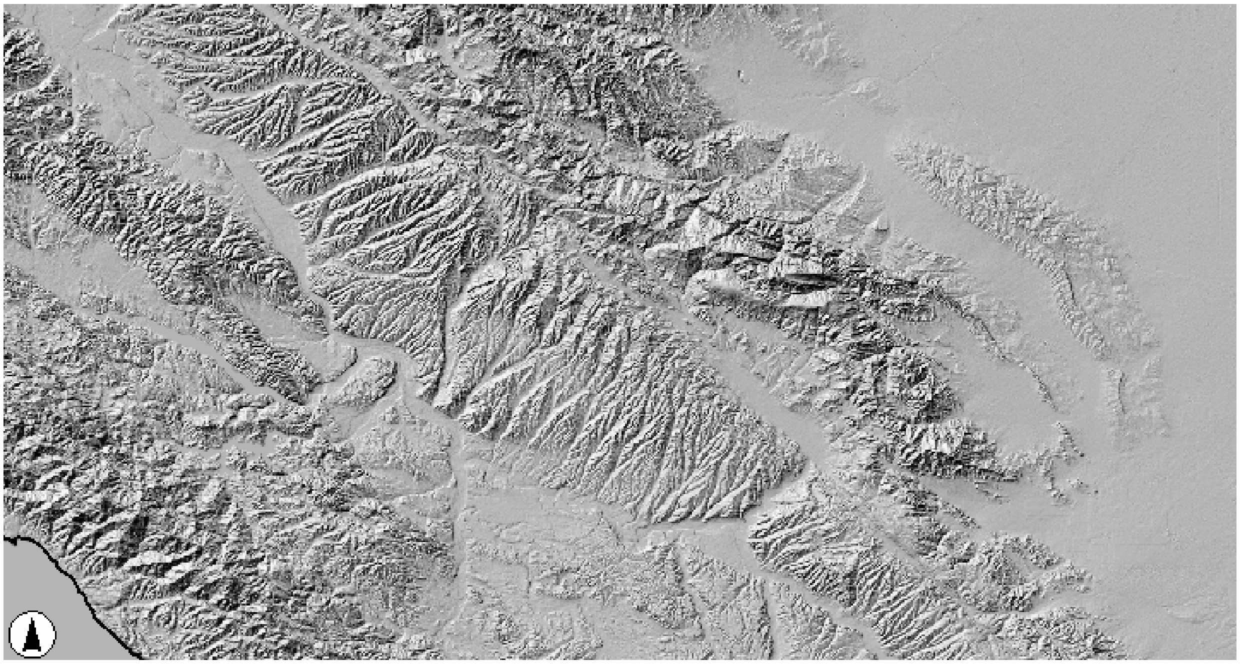

Fig. 10 Region in the vicinity of the 2004 Parkfield earthquake that was used in this study.

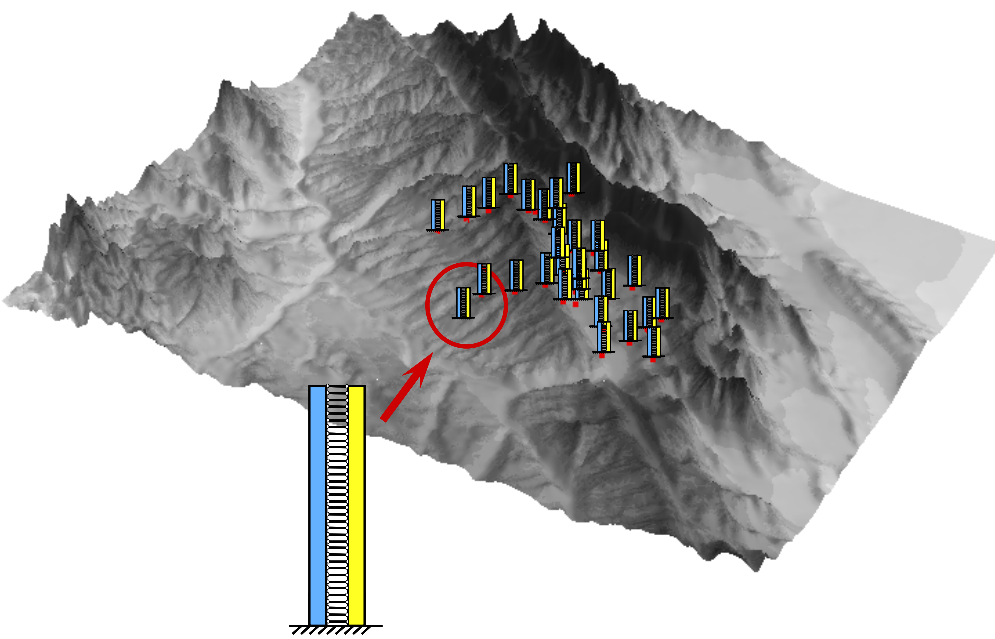

Fig. 11 Ground motion stations for which continuous models were used to compute and display improved shake Maps.

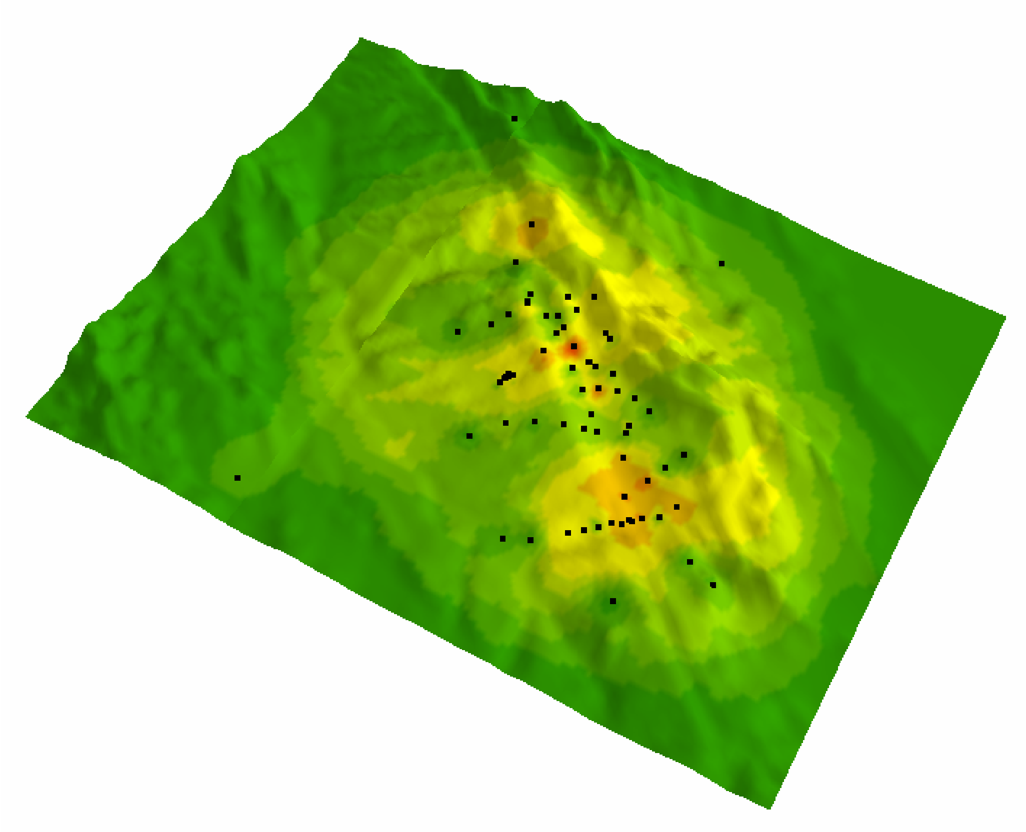

Fig. 12 Interstory drift map in the fault normal direction.

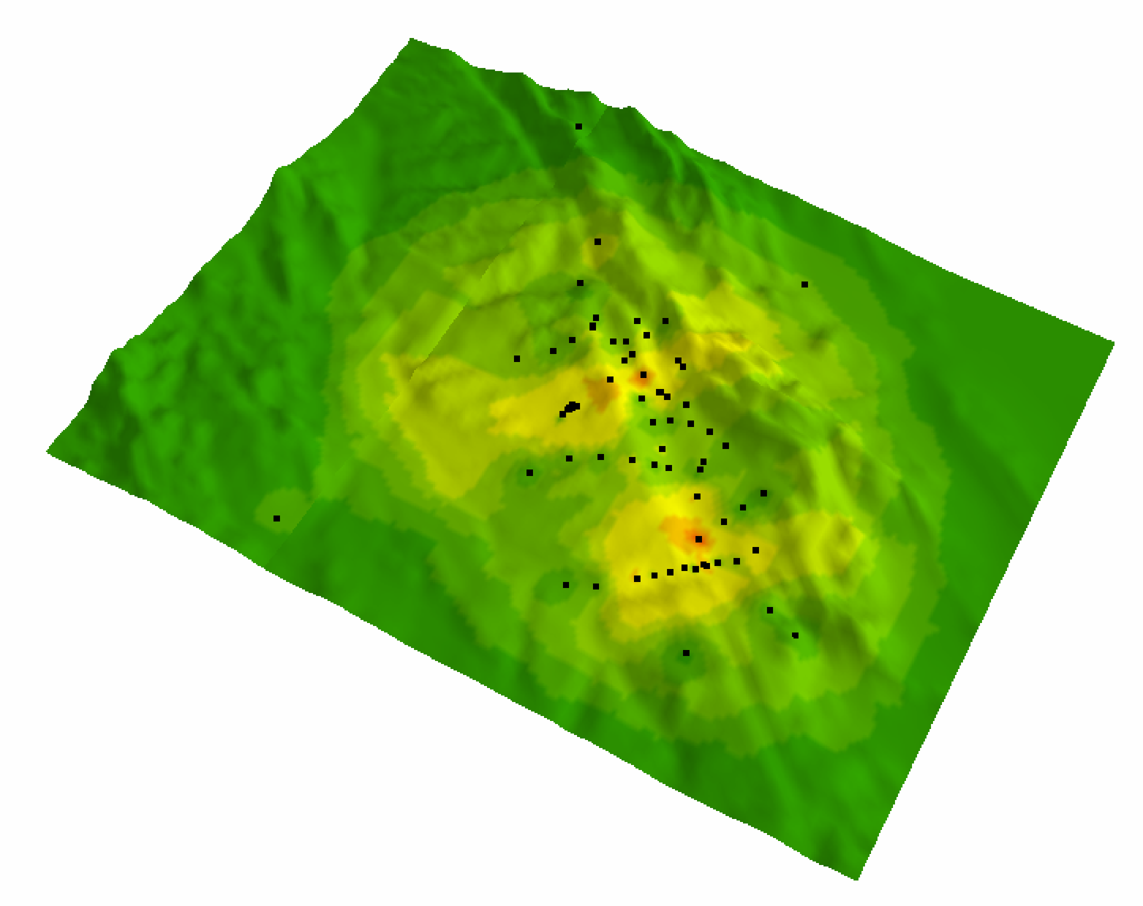

Fig. 13. Interstory drift map in the fault parallel direction.

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 of the buildings being analyzed 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. It should be noted that the proposed analytical tool has not been developed as replacement of more refined and accurate models. In particular, the proposed tool is not aimed at providing accurate estimates of interstory drift demands for buildings experiencing strong nonlinearities. However, together with information of drifts at which inelastic behavior is initiated, this tool is useful in identifying when yielding and possible damage is likely to occur and provides information on whether large interstory drifts are likely to occur in a structure.

The simplified building model, the generalized interstory drift spectra and the improved Shake Maps 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 for studying possible damage in future events; (5) For rapid loss estimation of large inventories of building, and (6) To provide emergency managers and city officials early performance estimates within minutes of a seismic event.

Acknowledgements

This work was supported by the California Strong Motion Instrumentation Program of the California Geological Survey. Their support is gratefully acknowledged. Special thanks are given to Anthony Shakal, Moh Huang and members of the advisory committee for their comments.

References

Algan, B.B. (1982). “Drift and damage considerations in earthquake resistant design of reinforced concrete buildings,” *PhD thesis*, University of Illinois, Urbana-Champaign, Ill.

Chopra, A.K., and Chintanapakdee, C. (2001). “Drift spectrum vs. modal analysis of structural response to near-fault ground motions,” *Earthquake Spectra*, EERI, 1**7**(2), 221-234.

Cornell, C.A., Jalayer, F. Hamburger, R.O. and Foutch, D.A. (2002). “The Probabilistic Basis for the 2000 SAC/FEMA Steel Moment Frame Guidelines,” *Journal of Structural Engineering*, Vol. 128, No. 4, pp. 526-533.

Deierlein, G.G., Krawinkler, H. and Cornell, C.A. (2003). “A framework for performance-based earthquake engineering.” *Proceedings of 2003 Pacific Conference on Earthquake Engineering.*

Iwan, W.D. (1997). “The drift spectrum: a measure of demand for earthquake ground motions,” *J. Struct. Engrg.,* ASCE, 123(4), 397-404.

Kircher, C.A. (2003). “Near-Real-Time Loss Estimation Using HAZUS and ShakeMap Data,” *Proceedings SMIP03 Seminar on Utilization of Strong-Motion Data*., May 22, 2003, Oakland, California, pp. 59-66.

Krawinkler, H., and Miranda, E. (2004). “Performance-based earthquake engineering,” Chapter 9 of *Earthquake engineering: from engineering seismology to performance-based engineering*, Y. Bozorgnia and V.V. Bertero, Editors, CRC Press.

McCann, M.W., Sauter, F. and Shah, H. C. (1980), “A technical note on PGA-intensity relations with applications to damage estimation”, *Bulletin of the Seismological Society of America*, Vol. 70, No. 2, pp. 631-637.

McJunkin, R. and Shakal, A. (1983) "Parkfiled strong-motion array," *California Geology*, February 1983, pp. 61-67.

Miranda, E. and Aslani, H. (2003). "Probabilistic response assessment for building-specific loss estimation." *Report PEER 2003/03,* Pacific Earthquake Engineering Research (PEER) Center, Richmond, California.

Miranda, E., and Taghavi, S. (2005). "Approximate floor acceleration demands in multi-story buildings. I: Formulation." *J. Struct. Eng.*, 131(2), 203–211.

Miranda, E., and Akkar, S. (2005). "Generalized interstory drift demand spectrum," accepted for publication in *J. Struct. Eng.*, 132, No. 6, pp. 840-852.

Moehle, J.P., and Deierlein, G.G. (2004). "A framework for performance-based earthquake engineering" *Proceedings of 13th World Conference on Earthquake Engineering*, Paper #679, Vancouver, Canada.

Qi, X., and Moehle, J. P. (1991). ''Displacement design approach for reinforced concrete structures subjected to earthquakes.'' *Rep. No. EERC/UCB-91/02*, Earthquake Engineering Research Center, University of California, Berkeley, Calif.

Rojahn, C., C.D. Comartin and S.A. King (2003). "Guidelines for Utilizing Strong-Motion and ShakeMap Data in Post-Earthquake Response (ATC-54)," *Proceedings SMIP03 Seminar on Utilization of Strong-Motion Data*., May 22, 2003, Oakland, California, pp. 47-58.

Taghavi, S., and Miranda, E. (2005). "Approximate floor acceleration demands in multi-story buildings. II: Applications." *J. Struct. Eng.*, 131(2), 212–220.

Wald, D.J., V. Quitoriano, T. Heaton, C.F. Scrivner and C. Worden (1999a), "TriNet ShakeMaps: rapid generation of peak ground motion and intensity maps for earthquakes in Southern California," *Earthquake Spectra*, Vol. 15, No. 3, pp. 537-555.

Wald, D.J. et al. (1999b), "Relationships between peak ground acceleration, peak ground velocity, and modified Mercalli intensity in California," *Earthquake Spectra*, Vol. 15, No. 3, pp. 557-564.