SMARTER STRUCTURES: REAL-TIME LOSS ESTIMATION FOR INSTRUMENTED BUILDINGS

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ABSTRACT

A technique is developed to estimate damage, loss, and post-earthquake operability for instrumented buildings shortly after the cessation of strong motion. The analysis modifies PEER's second-generation performance-based earthquake engineering (PBEE-2) technique to account for the additional information that strong-motion instruments provide. In the modification, Bayesian updating is used to update the prior probability distribution of the facility's structural characteristics. The outcome of the analysis is the estimated likely locations of physical damage (important for concealed structural damage); the probability distribution of repair cost (valuable for quickly arranging for financing); and a rapid pre-inspection estimate of safety and operability.

Introduction

This paper addresses how performance-based earthquake engineering (PBEE) methods can be modified to incorporate accelerometer data from an instrumented building to produce a rapid estimate of repair cost, safety, and probable locations of (potentially concealed) physical damage. Full details of this study are provided in Porter et al. (2004).

The PBEE approaches addressed here generally use building-specific knowledge of the facility's structural system, the fragility of its structural and nonstructural building components, and standard construction contracting principles to estimate repair cost, human losses, and repair duration ("dollars, deaths, and downtime"). Relevant literature includes Czarnecki (1973), Kustu et al. (1982), Kircher et al. (1997), Beck et al. (1999), Porter (2000, 2003), and others. The Pacific Earthquake Engineering Research (PEER) center has recently developed consensus around a second-generation PBEE method; see http://peer.berkeley.edu.

Recent PBEE methods share four stages. The first is the hazard analysis, in which one or more hazard levels are selected, and for each, one or more structural analyses are performed to determine member forces, member deformations, and other structural response parameters. Given the structural response to which each component is subjected, one uses fragility functions to calculate physical damage. Given physical damage, one estimates repair cost using

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construction-contracting principles. Some methodologies allow for the calculation of repair duration, post-earthquake operability, and deaths and injuries. Uncertainties are propagated using Monte Carlo techniques, quadrature, first-order approximations, or other methods.

Sensor information has played little role in PBEE. A notable exception is Celebi *et al.* (2004), who recently combined sensor information with FEMA 273 (FEMA 1997). They illustrate the methodology with a 24-story steel-frame building that has been instrumented to compute interstory drift ratios at a few story levels with sensors at adjacent floors. These interstory drift ratios are then compared with drift limits associated with the FEMA-273 performance levels: operational, immediately occupiable, life-safety, and collapse-prevention. When a drift limit is exceeded, the associated performance level is assumed to be exceeded.

Methodology

Structural analysis. The present methodology uses additional probabilistic information about the structural system, component damageability and repair cost, to estimate building performance. The key to using accelerometer data within PBEE – and a principal novelty of the methodology described here – lies largely in the use of Bayesian updating to modify the probability distribution of the uncertain structural characteristics and the structural response.

In this methodology, the hazard analysis of PBEE is bypassed; basement motion is taken from accelerometer data. To account for uncertainties in the structural characteristics and structural response, a stochastic structural model is created. Various model parameters are taken as uncertain. Here, five parameters are treated as uncertain: viscous damping (denoted by ζ), initial stiffness (denoted by K_0), post-yield stiffness (denoted by K_1), soil spring stiffness (denoted by K_S), and structural strength (denoted generically by F). We ignore uncertainty in mass because it appears to have modest impact on repair cost (Porter et al. 2002).

To create a stochastic structural model, a nominal (best-estimate) structural model is first created. Each parameter in the nominal model is then multiplied by an uncertainty term ε. For example, let $E[K_{0i}]$ denote the expected value of the initial stiffness of some spring number i in the structural model, and let ε_{K0} denote an uncertain variable with mean value of 1.0 and the same coefficient of variation (COV) as K_{0i}. The uncertain initial stiffness is then taken as the product of $E[K_{0i}]$ and the uncertainty term ε_{K0}. A vector of such uncertainty terms is given by

$$\varepsilon_X = \left[\varepsilon_\varsigma, \varepsilon_{K0}, \varepsilon_{K1}, \varepsilon_{KS}, \varepsilon_F \right]^T \tag{1}$$

One creates S realizations of the stochastic structural model. For each, nominal structural parameters are multiplied by the appropriate component in a realization of ε_X. Each term in ε_X is taken as independent and lognormally distributed with a mean value of 1.0 and a COV denoted by δ. We take the COV of stiffnesses δ_{K0} = δ_{K1} = 0.05. As in Beck et al. (2002), we estimate uncertainty in damping as $\delta_\zeta = 0.4$, and uncertainty in strength as δ_F = 0.11. For uncertainty in soil stiffness, we used Jones et al. (2002) to select δ_{KS} = 0.35. We used Latin Hypercube simulation to produce S samples of each ε as follows:

$$\varepsilon = \exp\left(\Phi^{-1}(u)\beta + \ln\hat{\varepsilon}\right) \tag{2}$$

where $u = \frac{u_1 + u_2}{S}$, $\beta = \sqrt{\ln\left(1+\delta^2\right)}$, $\hat{\varepsilon} = 1\big/\sqrt{1+\delta^2}$ (3)

In Eq. 3, u_1 is sampled from {0, 1, … *S-1*} with equal probability and without replacement and $u_2 \sim U(0,1)$, i.e., u_2 is uniformly distributed between 0 and 1. This approach ensures that the tails of the distribution of ε will be sampled as well as the body, and speeds convergence. In each realization of the structural model, each variable in Eq. 1 is sampled once. Thus, in each realization, all springs have their nominal initial stiffness multiplied by the same sample of ε_{K0}, and similarly for all other parameters.

The basement accelerogram is then used with each realization of the structural model in a nonlinear time-history structural analysis to produce a sample estimate of the structural response. The response is measured with a vector of engineering demand parameters (*EDP*), including member forces, member deformations, story drifts, etc. Before taking account of recorded upper-story motions, each sample *EDP* vector is equiprobable, because each sample of the structural model is equiprobable. However, some upper-story motions are known from upper-story sensors. Each sample *EDP* vector also includes an estimate of these same upper-study motions. With these two pieces of data, the prior distribution formed by the simulated *EDP* vectors and the sensor observations, one can use Bayesian updating to change the probability distribution of the structural model.

We developed two methods to do so. The first is a particle filter, capable of employing all of the data in the upper-story acceleration time histories. This filter was found to be impractical to implement with the structural analysis program we had available. The interested reader is referred to Porter et al. (2004) Appendix A for detail about the particle filter and a sample application. The second method uses only peak values of acceleration at the location of each upper-story accelerometer to update the probability distribution of the structural model.

Let w_s denote the weight assigned to each structural realization. Its value before updating (its prior probability) is denoted by w'_s. Before updating, each realization is equiprobable, i.e., $w'_s = 1/S$ for all s. The updated (posterior) value, denoted w''_s, is given by

$$w''_s = \frac{p\left[o \mid m_s\right] w'_s}{\sum_{s=0}^{S-1} p\left[o \mid m_s\right] w'_s}, \quad \text{for } s = 0, 1, \ldots S-1 \tag{4}$$

where $p[o|m_s]$ = likelihood of observing peak accelerations o, given model realization s

$= \Pi_j[\phi(z_{sj})]$

ϕ = Gaussian probability density function

$z_{sj} = \ln(o_j/\hat{o}_{sj})/\beta_j$

o_j = peak acceleration recorded by sensor j

$\hat{o}_{sj}$ = peak acceleration estimated for sensor j by structural analysis, using realization s

$\beta_j = 0.15$, an assumption of the model explained in Porter et al. (2002)

One now has S vectors of *EDP* calculated using nonlinear time-history structural analysis of the recorded basement motion and a stochastic structural model, each *EDP* vector having a posterior probability w''_s. The damage and loss analyses, described next, are similar to that described in Porter (2003), except that the samples are not equiprobable.

Damage analysis. In the next stage, one calculates damage to each damageable component for each sample *EDP* vector, using assembly fragility functions. Damage is simulated *T* times for each of *S* vectors of *EDP*. Each simulation proceeds as follows. After an assembly is subjected to a certain *EDP*, it is in an uncertain damage state *DM*, indexed by dm = 0, 1, 2, … N_{DM}, where $dm = 0$ indicates the undamaged state. We assume that the damage states can be sorted in increasing order, either because an assembly in damage state $dm = i + 1$ must have passed through damage state *i* already, or because the effort to restore an assembly from damage state $dm = i + 1$ necessarily restores it from damage state $dm = i$. The probability that an assembly will reach or exceed damage state *dm*, given that it experiences excitation $EDP = x$, is given by the fragility function, commonly taken as a lognormal cumulative distribution function:

$$\begin{aligned} F_{dm}(x) &= P\left[DM \geq dm \mid EDP = x\right] \\ &= \Phi\left(\frac{\ln(x/x_m)}{\beta}\right) \end{aligned} \tag{5}$$

where x_m denotes the median value of the distribution, and β denotes the logarithmic standard deviation of the distribution.

For a component with N_{DM} possible damage states (in addition to $dm = 0$), the conditional distribution of *DM*, given $EDP = x$, is given by:

$$\begin{aligned} F_{DM|EDP=x}(dm) &\equiv P\left[DM \leq dm \mid EDP = x\right] \\ &= 1 - F_{dm+1}(x) \qquad 0 \leq dm < N_{DM} \\ &= 1 \qquad dm = N_{DM} \end{aligned} \tag{6}$$

where $F_{DM|EDP=x}(dm)$ denotes the cumulative probability distribution of damage state *DM* evaluated at *dm*, given $EDP = x$. One can simulate the damage to each assembly by inverting Eq. 6 at a random probability level *u*, given by

$$dm = F^{-1}_{DM|EDP=x}(u) \tag{7}$$

where $u \sim U(0, 1)$ and can be simulated by Latin Hypercube. The simulated damage state of each assembly is then recorded in the vector *DM*, whose elements are the value of damage measure of each component. Using Eq. 7, one now has *T* simulations of structural damage for each of *S* realizations of the structural model. The *conditional* probability that a component is *in* damage state *dm* , given $EDP = x$, is given by:

$$\begin{aligned} P\left[DM = dm \mid EDP = x\right] &= 1 - F_1(x) \qquad dm = 0 \\ &= F_{dm}(x) - F_{dm+1}(x) \quad 0 < dm < N_{DM} \\ &= F_{dm}(x) \qquad dm = N_{DM} \end{aligned} \tag{8}$$

The *marginal* probability that a component is in damage state *dm* (conditioned only on the model and the observed basement acceleration and upper-story motion) is given by:

$$P\left[DM = dm\right] = \sum_{s=0}^{S-1} w''_s P\left[DM = dm \mid EDP_s = x\right] \tag{9}$$

where w''_s is calculated using Eq. 4 and $EDP_s = x$ indicates that the structural response applied to the component is taken from the structural analysis of realization *s* of the stochastic structural model. At this stage, we have the estimated probability that a given component is in damage state *dm*. Using these probabilities, one can identify the structural members most likely to be damaged by the earthquake, even if they are concealed behind building finishes. Damage and safety investigations can now be targeted to those components.

Loss analysis. The damage analysis produced *T* simulations of damage for each of *S* simulations of *EDP*. In the loss analysis for cost, each simulation of repair cost proceeds as follows. Each assembly type and damage state is associated with one or more possible repair measures, each with an uncertain repair cost. Let $C_{j,dm}$ denote the uncertain cost to restore an assembly of type *j* from damage state *dm*; it can be calculated by standard cost-estimation principles. Let $F_{C,j,dm}(c)$ denote its cumulative distribution function evaluated at *c*. We take these as lognormally distributed with median values x_m and logarithmic standard deviations β specified for each assembly type and damage state. To simulate $C_{j,dm}$, evaluate

$$C_{j,dm} = \exp\left(\Phi^{-1}\left(u\right)\beta + \ln x_m\right) \tag{10}$$

where *u* is sampled from U(0,1). The total repair cost, denoted here by *RC*, is given by

$$RC = \left(1 + C_{OP}\right)\sum_{j=1}^{N_j}\sum_{dm=1}^{N_{DM}} N_{j,dm} C_{j,dm} \tag{11}$$

where $N_{j,dm}$ is the number of assemblies of type *j* in damage state *dm*, determined in the damage analysis, and the contractor's overhead and profit, denoted here by C_{OP}, is taken as uniformly distributed between 15% and 20%. This completes one simulation of loss.

One now has *S* samples of *EDP*, each with posterior probability w''_s and each with *T* equiprobable samples of damage and loss. The cumulative distribution function for the total repair cost *RC* is calculated from the *S* x *T* simulations of loss by:

$$\begin{aligned} F_{RC}\left(rc\right) &= P\left[RC \le rc\right] \\ &= \frac{1}{T}\sum_{s=0}^{S-1} w''_s \sum_{t=0}^{T-1} H(rc - RC_{s,t}) \end{aligned} \tag{12}$$

where $H(x) = 1$ if $x \ge 0$; $H(x) = 0$ otherwise. One now has the probability distribution of repair cost, useful in arranging repair financing, filing insurance claims, etc.

To measure the qualitative performance of a building after an earthquake, we use an enhanced version of the building performance levels in FEMA 356 (ASCE 2000). The basic methodology for mapping from *DM* to building performance level is described in Porter and Kiremidjian (2001). We enhanced this methodology using fuzzy-set theory, which allows one to define fuzzy, quantitative boundaries between the qualitative component damage states and related system performance levels described in Chapter 1 of ASCE (2000)[5]. The methodology is not described here, because of space constraints. See Porter et al. (2004) for detail.

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[5] In Chapter 1 of FEMA 356, its authors state that the component damage descriptions associated with building performance levels are "approximate limiting levels … of damage … rather than precise predictions." That is, they are not exactly criteria for determining whether a building is actually in a given performance level. On the other hand, as approximations they are convenient and reasonable proxies for criteria, and are used as such here.

Sample Application

Case-study building*[6]*. The case-study building is a 7-story, 66,000-sf hotel in Van Nuys, CA, as it existed just before the 1994 Northridge earthquake. It was built in 1966 according to the 1964 Los Angeles City Building Code. The lateral force-resisting system is a perimeter nonductile reinforced-concrete moment frame on drilled piers. It was lightly damaged by the M6.6 1971 San Fernando event, 20 km to the northeast, and severely damaged by the M6.7 1994 Northridge Earthquake, whose epicenter was 4.5 km to the southwest. The building has been studied extensively, e.g., by Jennings (1971), Islam (1996), and Li and Jirsa (1998). Trifunac et al. (1999) and Browning et al. (2000) describe its damage in the 1994 Northridge earthquake.

The building is 63 ft by 150 ft in plan, 65 ft tall. Its first story contains a lobby, meeting rooms, and support services. Upper stories contain hotel suites arranged along a central longitudinal corridor. The replacement cost is approximately $7.0M. The column plan is shown Fig. 1; the south frame elevation, in Fig. 2. The building is clad on the north and south facades with aluminum window walls and cement asbestos board panels with an ornamental sight-obscuring mesh. The east and west endwalls are finished on the outside with stucco. Interior partitions are constructed of gypsum wallboard on metal studs. Structural and architectural designs were available from drawings on file in the city building department. Conditions of the mechanical, electrical, and plumbing components were determined by a facility walkdown.

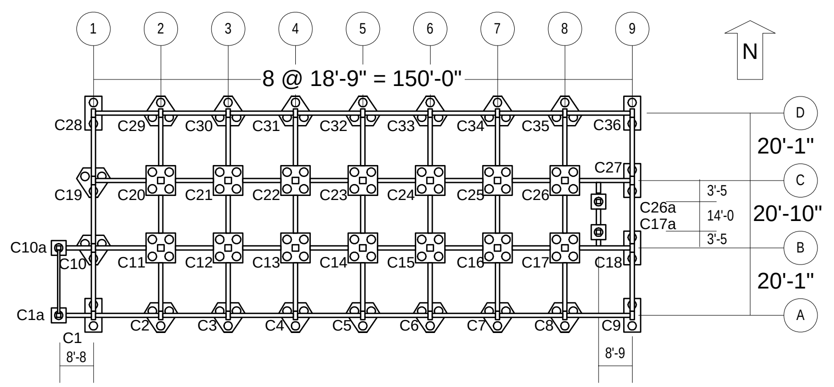

Figure 1. Foundation and column plan, showing designer's notation for column numbers.

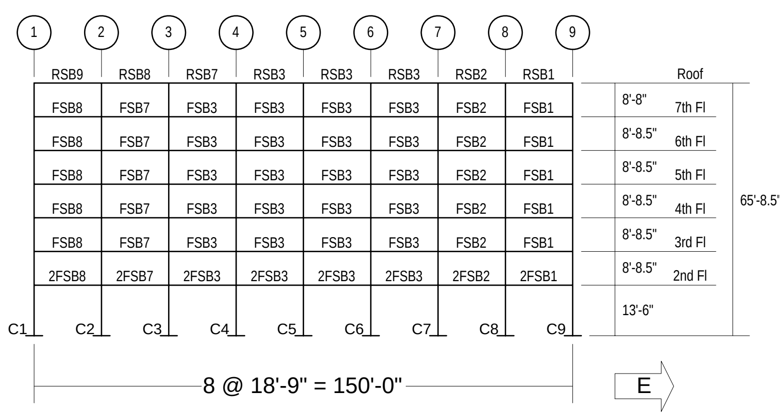

Figure 2. South frame elevation, omitting stair tower at west end

[6] A Japanese case study building was also examined, but is not described here.

Instrumentation, historic shaking, and damage. The building was strongly shaken by the M6.6 1971 San Fernando event, 20 km to the northeast, which caused primarily nonstructural damage, and in 11 subsequent events. According to available literature, only the 1971 event and the 1994 Northridge earthquake series caused significant damage. Shaking, structural response, and damage in the 1994 Northridge earthquake were more severe than in the San Fernando earthquake. Peak acceleration at the ground floor was 440 cm/sec^2 in the longitudinal direction, 390 cm/sec^2 transversely.

Damage surveys performed in early 1994 indicate extensive structural damage in the form of shear failure of columns and beam-column joints in the perimeter moment frame. The failures include spalling of the cover concrete over longitudinal bars, buckling of the longitudinal bars and through-cracks up to several inches wide. Damage to the south frame occurred at six locations on the 5th floor and one at the 3rd floor. Damage to the north frame occurred in the full-height infill masonry walls at the 1st story and at the base of three short columns at the 1st story. Damage to the north frame also occurred at or within the beam-column joint at 12 other locations at the 2nd through 5th floors.

Structural repairs after the Northridge earthquake involved the addition of shearwalls at three columns of the south frame and four columns of the north frame, and at several interior column lines. Base fixity is provided to the new shearwalls by the addition of grade beams spanning between pier groups.

Estimated damage and loss. Our structural model is an enhancement of the one used in Beck et al. (2002), the addition being soil springs in place of the fixed-based columns. Hutchinson (2003) provides a model of soil-spring stiffness for this building. Base excitation from the Northridge earthquake is taken from a basement accelerometer. Observed upper-story accelerations are taken from instruments at the roof, 6th, 3rd, and 2nd floors. Fragility and repair-cost parameters are tabulated in Porter et al. (2004). We used S = 100 simulations of the structural model and *EDP*. The structural analyses took approximately 4 hr on an ordinary desktop computer. Because the subsequent damage and loss analyses can be performed much more quickly, we used each *EDP* vector T = 10 times, thus performing a total of 1,000 simulations of damage and loss.

We estimated extensive damage in 3rd-story columns and roof beams, whereas the actual damage was concentrated at the top of the 4th-story columns and in their joints with 5th-floor beams. None of our estimated most-likely damage locations actually experienced damage in the Northridge earthquake, implying either (a) actual structural conditions in the building differed from the construction documents, e.g., column reinforcement actually continues higher than shown in the drawings, (b) the fragility functions employed here are inappropriate for the structural components in the case-study building, or (c) both.

In every simulation, damage to beam and column elements exceeded that allowed under the life-safety performance level. This implies near-certainty that the building would not be in the life-safety performance level when exposed to the 1994 Northridge earthquake. Since that is in fact what did happen in Northridge, our model merely implies that, had such a real-time loss estimation system been in place at the time of the 1994 Northridge earthquake, it would have accurately and with near certainty predicted the system performance level.

The prior and posterior probability distributions of repair cost are shown in Fig. 3. Both distribution are almost perfectly lognormal, with median values of \$2.5 and \$2.6 million, respectively, and logarithmic standard deviations of 0.06 and 0.08, respectively. Actual repair costs after the 1994 Northridge earthquake are unknown, and in any event the building was not restored to its pre-earthquake condition but rather converted to a shearwall building. The damage was costly enough to cause a several-year closure of the building.

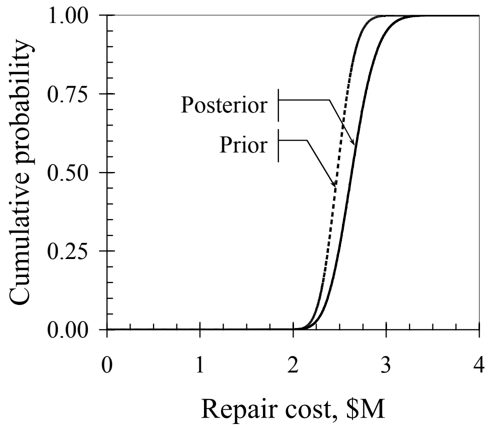

Figure 3. Hindcast repair cost distributions for the 1994 Northridge earthquake.

We observe that the Bayesian updating of the structural model does not substantially reduce uncertainty regarding repair cost, or even greatly modify its mean value, implying that uncertainty in repair cost may be largely produced in the damage and loss analyses, and that uncertainty in the structural model may not matter much. Perhaps no stochastic structural model needs to be created, and that one need only consider uncertainty in the damage and loss analysis. How true or how general this implication may be is not yet known, but if true, a real-time loss analysis can be performed very quickly, with just a single deterministic structural analysis, *and no need to update the model considering observed upper-story motions*.

Conclusions

Macroscopic versus component-level accuracy. Hindcasting of the overall system performance was good, but estimation of detailed damage locations was poor, perhaps because as-built conditions differ substantially from those shown on the drawings. We lack the data to validate the estimated repair costs, although based on closure duration and the general agreement of the extent of damage, the estimates appear to be correct on an order-of-magnitude basis.

Questionable need for stochastic structural model and upper-story accelerograms. We found that the above-base sensor data adds little information to the performance estimate, probably because structural uncertainties are relatively unimportant compared with the uncertainties in the damage and loss analyses. Real-time loss estimation may not need a stochastic structural model, and motions recorded by upper-story accelerometers may not substantially add to the accuracy of the performance estimate.

Opportunities for implementation. Because much of the cost of applying this real-time loss-estimation algorithm results from the cost of instrumentation and the effort of setting up a structural model, the readiest application would be to instrumented buildings whose structural models are already available. Furthermore, the methodology might produce the most value when applied to important facilities such as those required for emergency response, and buildings with high cost of testing for concealed damage, such as steel-frame structures.

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