APPLICATION OF THE PEER PERFORMANCE-BASED METHODOLOGY FOR SEISMIC ASSESSMENT OF THE I-880 VIADUCT

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ABSTRACT
The PEER framework is the first PBEE approach that incorporates loss modeling and provides an economic basis for evaluating performance of structures in probabilistic terms. In this paper, the PEER methodology is applied to the evaluation of an existing highway bridge bent that forms part of the rebuilt section of the I-880 viaduct in California. The objective of the exercise is to demonstrate the value and applicability of the methodology in performance-based seismic engineering.

INTRODUCTION
FEMA-356, widely recognized as the first resource document that lays out a systematic approach to PBEE, is essentially a deterministic methodology. The range of uncertainties associated with predicting performance in a seismic environment has led to increased interest in probabilistic performance-based methods. A formal implementation utilizing a probabilistic treatment for seismic evaluation materialized with FEMA-350 (2000). The PEER performance-based framework may be regarded as an extension and an enhancement of the procedure developed for FEMA-350 (Cornell et al., 2002). The PEER evaluation methodology is summarized in Figure 1. The methodology comprises four distinct but related phases: *hazard analysis* that characterizes the seismicity at the site; *structural analysis* of a simulation model that yields the necessary force and deformation measures; *damage analysis* to enable transformation of response measures into physical damage states; and *loss analysis* that relates the damage to a measure of performance. Intensity Measure (IM) denotes a measure of the ground motion intensity (peak ground acceleration, spectral acceleration at some characteristic period of the structure, etc.) Engineering Demand Parameters (EDP) are the selected measures of seismic demand that are used to assess performance. Next, a set of damage measures (DMs), expressed as fragility functions of the form p(DM|EDP), are derived corresponding to each of the selected EDPs. Finally, the performance of a structure is defined as a discrete or continuous function with realistic decision-making potential. Such a loss-modeling measure is defined as a Decision Variable (DV) in the PEER framework. An example of a DV for bridges is the likelihood of closure of the facility. A conceptual description of the methodology is expressed as follows:

$$\nu(DV) = \iiint P(DV \mid DM)\, dP(DM \mid EDP)\, dP(EDP \mid IM)\, |d\lambda(IM)| \quad (1)$$

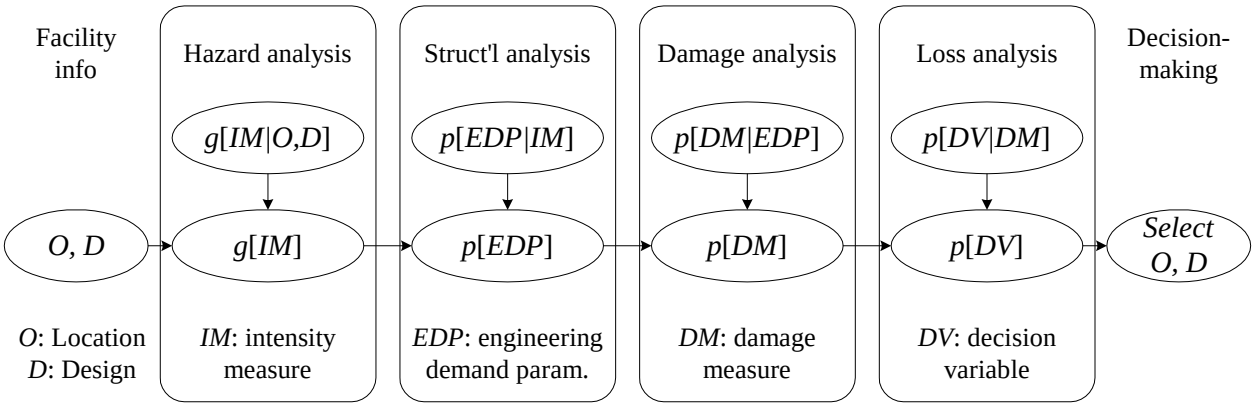

Figure 1: PEER performance-based evaluation framework

where $\nu(DV)$ is a probabilistic description of the decision variable (for example, the mean annual rate of exceeding a certain repair cost), *DM* represents the damage measure, *EDP* represents the engineering demand parameter (such as drift) and $\lambda(IM)$ is the mean annual frequency of the intensity measure. *P(DV|DM)* is the conditional probability that *DV* exceeds a specified limit for a given value of *DM* and the remaining terms of the form *dP(A|B)* is the derivative with respect to *A* of the conditional probability *P(A|B).*

DESCRIPTION AND MODELING OF THE STRUCTURE

The 1989 Loma Prieta earthquake caused the collapse of the upper deck of a section of the double-decked Cypress Viaduct in Northern California. By mid-1997, a new non-decked viaduct was constructed approximately one-half mile from the old structure, just east of the San Francisco-Oakland Bay Bridge toll plaza. The new viaduct consists of two side-by-side single-bay spans and is known as the 5th and 6th Street Viaduct. A schematic of the eastbound bridge identifying the seven frames of the viaduct is shown in Figure 2. This segment of the I-880 consists of seven frames and twenty-five bents (Bent #2 – Bent #26). The frames are interconnected by expansion joints that comprise shear keys, restrainers and bearing surfaces. For the purpose of this study it was resolved that a multiple frame model incorporating at least a pair of expansion joints was essential. Consequently, the simulation model for the seismic demand analyses was derived to represent the section of the viaduct comprising frames 3 – 5 or bents 10 – 20. Expansion joints are located between bents 13 – 14 and bents 17 – 18.

Development of Simulation Model

Two models were developed for the simulation studies: (i) a model comprising three inter-connected frames (denoted in Figure 2 as Frame 3-4-5) which incorporates connection elements at the hinge region between two adjacent frames; and (ii) a simple model of a single bent that was identified as the region of maximum demand in the three-frame model. The three-frame, 11-bent model, shown in Figure 2, was originally prepared by Bauer (2003).

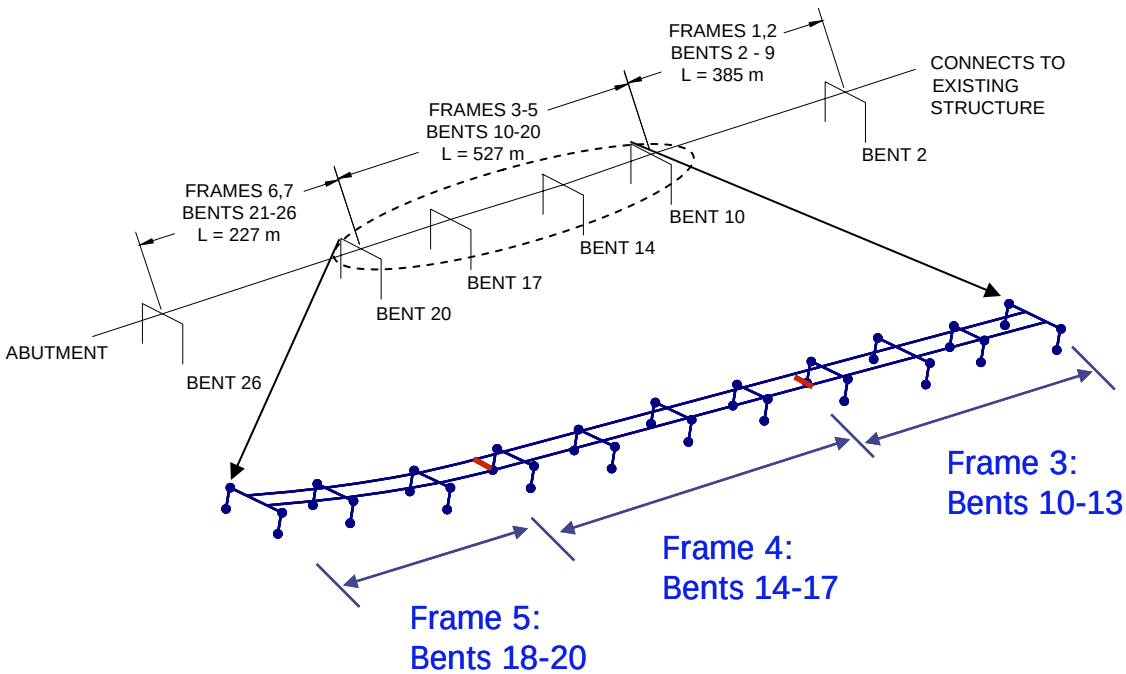

Figure 2: Schematic view of eastbound I-880 viaduct showing frame configuration and selected section used in analytical simulation of seismic demand

Each bent, comprised of two columns joined by a single cap beam, is connected to the adjacent bents by a deck system. The deck is modeled in the present study by elastic beams connecting the column bents appropriately. Zero length inelastic springs are used to model the connection elements representing the expansion joints. Each hinge connection is composed of four springs representing the shear key, the longitudinal restrainer, the vertical restrainer, and the bearing plate. The properties of the longitudinal restrainer also model frame-to-frame impact in the compression direction of the spring following gap closure. The column elements were considered to be the primary nonlinear elements in the model. Hence, force-based nonlinear beam-column elements that consider the spread of plasticity along the element in conjunction with a fiber-section model accounts for inelastic axial flexure interaction in bidirectional bending were used to model the columns in the viaduct. Since the shear strengths of the column sections are large enough to preclude shear failure, only the elastic contribution to shear deformations are considered. The piers are connected to a foundation system, consisting of 5x5 pile groups, was modeled by three translational and three rotational springs. The spring properties were derived from separate three-dimensional finite element analyses of the soil-foundation system (Jeremic et al. 2004). The modeling choices were motivated by the capabilities of OpenSees (2005), the open-source computational platform used in the seismic analyses of the I-880 simulation model.

PERFORMANCE-BASED EVALUATION PROCEDURE

The objective of the evaluation is to establish the mean annual frequency of closure of the viaduct. As is evident from Equation (1), the evaluation process consists of several independent modeling tasks beginning with the selection of earthquake records to characterize the site hazard and ending with the probability of closing the bridge for the specified hazard.

Seismic Hazard Curve

Uniform hazard spectra for S_D (soil) site conditions were developed by Somerville and Collins (2002) for the bridge site corresponding to three hazard levels: events with a 50%, 10% and 2% probability of being exceeded in 50 years (Figure 3). The spectra were generated for both strike-parallel (SP) and strike-normal (SN) directions. Several earthquake records with the required magnitude-distance combinations from strike-slip earthquakes were then selected. The components in the strike normal (SN) directions of each of these records were scaled so that the spectral acceleration at the natural period matches the corresponding value at the same period on the hazard spectra. The scale factor obtained for the SN direction is also used for the SP direction since it preserves the relative scaling between all components of the recording. The intensity measure (IM) that was selected for the study is the 5% damped elastic spectral acceleration at the characteristic period of the structure. Selected ground motions are then scaled to this IM. The seismic hazard curve (also shown in Figure 3) was derived by plotting the return periods against the magnitude of the spectral accelerations at the characteristic structural period. The seismic hazard curve is approximated by a linear function on a log-log scale:

$$\lambda(IM) = k_0 (S_a)^{-k} \tag{2}$$

The parameters k and k_0 obtained by regression analysis along with the hazard curve is displayed in Figure 2. To utilize this in the PEER evaluation methodology, it is necessary to find the slope of the seismic hazard curve.

$$d\lambda(IM) = -kk_0 (IM)^{(-k-1)} dIM \tag{3}$$

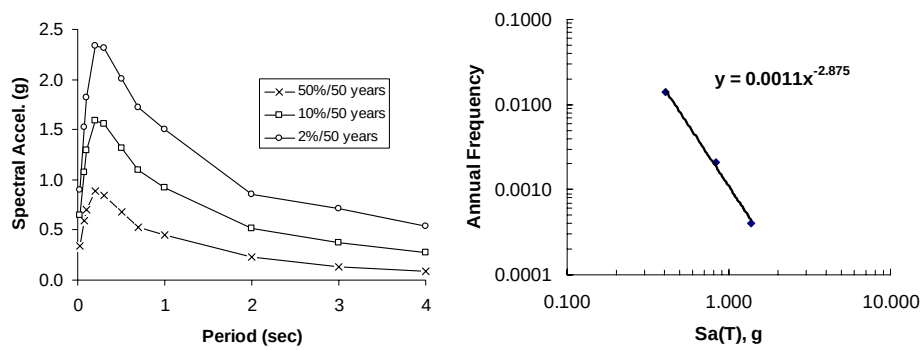

Figure 3: Uniform hazard spectra in fault-normal direction and resulting hazard curve

Seismic Demand Simulation

Numerous EDPs at the structural level (displacements, rotations), foundation system (deformations in the soil-foundation springs) and hinges (forces and deformations in equivalent springs at expansion joints) can be recorded for each simulation. However, only those EDPs that are relavent in damage analysis and loss modeling can be utilized in the next step of the PEER performance-based framework. The primary EDP selected for this illustrative testbed exercise is the peak tangential drift of the individual columns which is a measure of the larger relative lateral deformation from the inflection point to either the base or top of the column. The simulation model of the viaduct was subjected to a set of ten scaled and transformed earthquake recordings discussed in the previous section for <u>each</u> of the three hazard levels. The peak tangential drift values are displayed in Figure 4. These drift values are in the transverse direction

of the bent. Since three hazard levels were considered, the results of the simulations show ten realizations for each *IM*. The variation of *EDP* with changes in *IM* was assumed to have the following form:

$$EDP = a(S_a)^b \tag{4}$$

where *a* and *b* are constants determined through curve-fitting. This represents a straight line fit (Figure 4) in log-log scale, through the mean of the natural logarithm of the simulations.

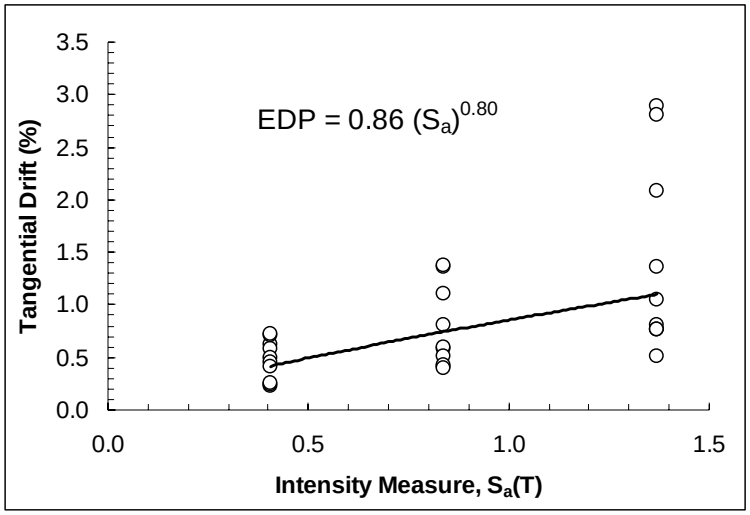

Figure 4: Seismic demand as a function of IM

Damage Modeling

In the context of the present study, the only components expected to experience damage in a seismic event are the bent piers. Hence, the damage measures selected for the evaluation of the viaduct are limited to visible damage of the column elements. Based on work carried out by Berry and Eberhard (2005), two primary damage measures were considered in this study: the onset of concrete spalling and reinforcing bar buckling. Spalling of the concrete cover is an important damage measure because it represents the first flexural damage states wherein the repair costs may be significant. The onset of longitudinal reinforcing bar buckling is another critical damage state because it significantly reduces the structure's functionality and may have implications for structural safety. Based on extensive statistical analysis of experimental data, Berry and Eberhard proposed two fragility functions that describe the probability of reaching or exceeding these damage states given a seismic demand in terms of the tangential drift in the column. The application of the Berry-Eberhard model to a typical column bent that experienced the largest demands results in the fragility curves shown in Figure 5.

Decision Variable (DV): Closure Probability

The final task in applying the PEER methodology involves developing a probabilistic description of a decision variable in terms of the observed damage. In the present illustration, a sample decision variable is selected in this study and its application to the methodology is demonstrated.

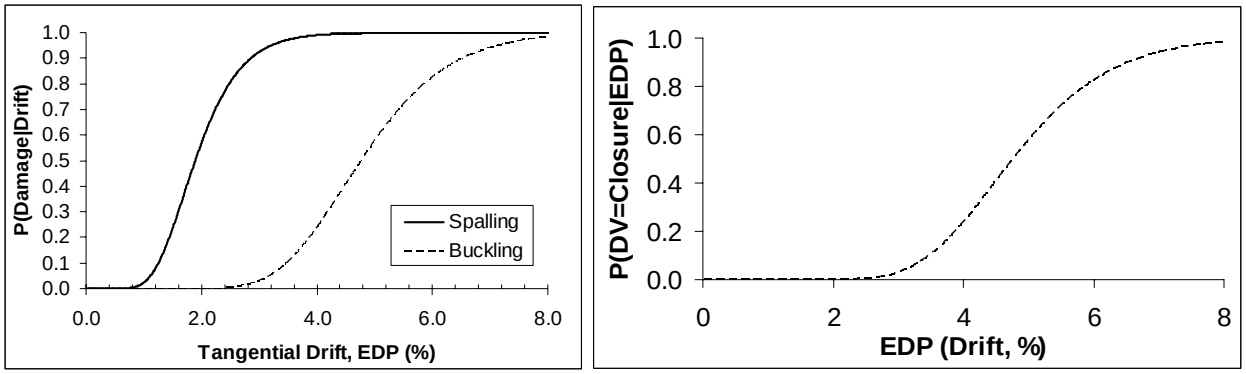

Figure 5: Damage probability conditioned on drift and closure probability conditioned on expected seismic demand

The decision variable utilized is the closure probability of the bridge. Of the two damage measures described in the previous section, it would be reasonable to assume that spalling of the concrete cover is a damage state that is not severe enough to result in the closure of the bridge. However, any observed buckling of reinforcing bars indicate large plastic deformations and the likelihood that the column capacity is significantly reduced. This damage state may prompt partial closure of the bridge until further inspection is completed. This results in the following discrete probabilities for bridge closure:

$$P\ (DV = Closure \mid Only\ Concrete\ Spalling = True) = 0.0 \tag{5}$$

$$P\ (DV = Closure \mid Bar\ Buckling = True) = 1.00 \tag{6}$$

The fragility functions which represent the damage probabilities for concrete spalling and reinforcing bar buckling (Figure 4) can now be combined with the discrete closure probabilities (Equations 5 – 6) to determine the probability of closing the bridge given an *EDP*, as follows:

$$P(DV \mid EDP) = \sum_{i=1}^{2} P(DV_i \mid DM) P(DM_i \mid EDP) \tag{7}$$

which reduces to:

$$P(DV \mid EDP) = P(\,Closure \mid Buckling\,) \cdot P(\,Buckling \mid EDP\,) \tag{8}$$

The resulting probability distribution is also shown in Figure 5.

Using the total probability theory, the probability of closure given an intensity measure (*IM*) is:

$$P(\,DV > dv \mid IM\,) = \int_0^{\infty} P(\,DV > dv \mid EDP\,) dP(\,EDP \mid IM\,) \tag{8}$$

In order to evaluate the second term that appears on the right hand side of the above expression, it is first necessary to determine *P(EDP|IM)*, for each hazard level. Assuming a lognormal distribution, and using the variation of median *EDP* with changes in *IM*, the resulting closure probability conditioned on the intensity measure for the bridge bent is displayed in Figure 6. Finally, the mean annual frequency of closure is given by:

$$\nu(DV) = \int_0^{\infty} P(DV > dv \mid IM) \left| \frac{d\lambda(IM)}{dIM} \right| dIM \tag{10}$$

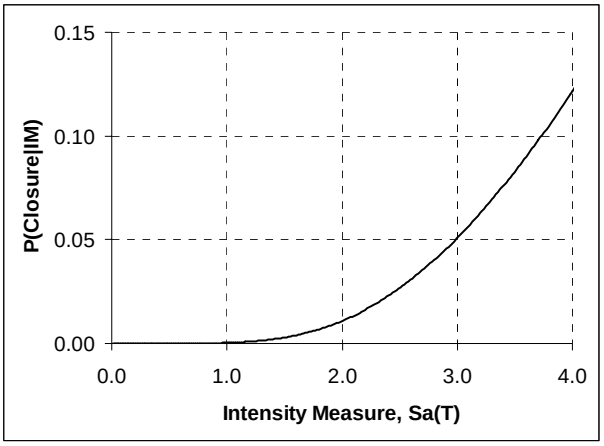

Figure 6: Closure probability as a function of IM

The integral is evaluated numerically to obtain the mean annual frequency of closure. If the mean annual frequency is small (< 0.01), then it will numerically be equal to the annual probability of closure. Hence, the probability of closure in *n* years is approximately given by:

$$P(Closure) \approx 1 - \left[1 - \nu(DV)\right]^n \tag{11}$$

Equations (10) and (11) assume that earthquake occurrence at the site follow a Poisson process and that cumulative damage effects are ignored. For the case study under investigation, following the application of Equations (10 – 11), the closure probability of the bridge in 50 years is estimated to be approximately 0.06%.

RELIABILITY ANALYSES AND TREATMENT OF UNCERTAINTIES

Structural reliability methods are employed in performance-based engineering to obtain probability estimates for various performance events. Structural performance is specified in terms of EDP's in the PEER terminology. EDP's, in turn, are functions of the ground motion intensity, or Intensity Measures (IM's). On the other hand, from an owner's or decision-maker's perspective, performance events must be defined in terms of Decision Variables (DV's), which characterize the cost associated with different structural performance outcomes, e.g., the costs of repair and loss of function of a bridge as a result of an earthquake. DV's in general depend on the state of the structure as characterized by a set of Damage Measures (DM's). For example, different demand measures may be used as indicators of different levels of damage to a bridge. DM's in general are functions of EDP's. Thus, using underlines to denote vector valued quantities, one can write $\underline{DV}(\underline{DM}(\underline{EDP}(\underline{IM})))$. Each of the relationships $\underline{DV}(\underline{DM})$, $\underline{DM}(\underline{EDP})$ and $\underline{EDP}(\underline{IM})$ is a mathematical model. Some of these models are well developed, while others are subjects of current research within and outside PEER.

In addition to the above, in reliability analysis we must consider the uncertain quantities affecting each of the measures $\underline{IM}$, $\underline{EDP}$, $\underline{DM}$ and $\underline{DV}$. These include material and geometric properties of the structure, as well as the applied loads. Additional uncertainties are present in the models used to describe the relations between these measures. Two types of uncertainty may be considered: (a) time-invariant uncertain quantities, such as material properties and structure geometry, and (b) time-variant uncertain quantities, such as the components of ground motion, which are characterized by a vector of stochastic processes. In the presence of these

uncertainties, obviously IM, EDP, DM and DV are also uncertain. Two objectives are then sought from reliability analysis: (a) estimation of the uncertainty in these measures arising from the two uncertainties listed above and (b) estimation of the probability of various events defined in terms of EDP's, DM's or DV's. An equally important objective is to identify variables or stochastic processes that are important sources of uncertainty. Several tools currently exist in OpenSees (2005) for such analysis. Further discussion on these issues or demonstrating the application of reliability and uncertainty analyses are beyond the scope of this paper. Interested readers will find additional information in Haukaas and Der Kiureghian (2004).

CONCLUDING REMARKS

The estimated closure probability for the I-880 is insignificant which suggests that the rebuilt section of the I-880 viaduct will perform satisfactorily for the hazard scenarios investigated. More importantly, the procedure outlined in this paper can be used to study the sensitivity of material and modeling parameters and then eventually to examine the impact of modeling considerations on the performance of the bridge. In this study, only a single demand variable was used. Additional measures such as plastic rotation or strain amplitude in the longitudinal reinforcing bar could have been selected. Likewise, different damage measures and decision variables may have been considered. Gaining an insight into the response of structures through probabilistic evaluations that incorporates a large parameter space offers advantages over other deterministic methods in performance-based seismic evaluation.

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