Performance-Based Earthquake Engineering: Opportunities and Implications for Geotechnical Engineering Practice

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ABSTRACT: Great advances have been made over the 40-some years in which geotechnical engineers have actively been involved in the practice of earthquake engineering. Most recently, advances have come through the development of performance-based earthquake engineering, which seeks to predict the seismic performance of structures and facilities in ways that are useful to a wide variety of stakeholders. Performance-based earthquake engineering requires the integrated, collaborative efforts of several groups of earthquake professionals, including geotechnical engineers; as such, it will affect the practice of geotechnical engineering in seismically active areas. This paper reviews the evolution of performance-based earthquake engineering, discusses the notion of performance and its description, and describes a recently developed framework for performance evaluation. The nature and effects of the many uncertainties that apply to the prediction and description of ground motions, system response, physical damage, and loss are described. The paper gives examples of different manners in which performance-based earthquake engineering can be implemented into practice. Finally, a series of challenges and opportunities presented by performance-based earthquake engineering for geotechnical engineering practitioners are identified and discussed.

INTRODUCTION

Geotechnical engineers have always based evaluation and design on their perception of performance, so the concept of performance-based engineering is, in a broad sense, nothing new. The manner in which performance is characterized, however, has become more refined over the years. In early geotechnical practice, the notion of performance was essentially binary – poor, or unacceptable, performance constituted "failure" and the lack of failure was taken as evidence of satisfactory performance. The evaluation or prediction of performance was generally based on comparison of the shear stresses required to maintain static equilibrium with the available shear strength of the soil, and expressed in terms of a factor of safety.

Design-level factors of safety were selected with consideration of uncertainty and the consequences of failure, but were also based on experience, precedent, engineering judgment, and, to varying degrees, expedience. These design-level factors of safety were generally high enough that the average induced shear stresses were limited to a small enough fraction of the average shear strength that large strains, and consequently large deformations, were avoided. While geotechnical engineers recognized that serviceability was more directly related to deformations than to stresses, the unavoidable fact that stresses could be predicted much more accurately than deformations helped support the continuing use of stress-based prediction of geotechnical performance.

As the profession has developed, however, it has become possible to define, characterize, and predict performance in ways that were not previously possible. In more recent years, the ready availability of powerful computers and development of improved numerical tools has led to greatly improved capabilities for prediction of deformations under static conditions. Commercial software packages, such as FLAC and PLAXIS, with convenient graphical interfaces are being used with increasing frequency in professional practice. The design of many important geotechnical systems, such as braced excavations, dams, foundations, and tunnels, is increasingly performed with explicit consideration of limiting static (short- and long-term) deformations.

Consideration of deformations is particularly important for earthquake-related problems. Performance-based earthquake engineering (PBEE) is a relatively new paradigm that is gaining widespread acceptance in the broad field of earthquake hazard mitigation. PBEE implies that structures and facilities can be designed and evaluated in such a way that their performance under anticipated seismic loading can be predicted. The purpose of this paper is to review the basic concepts of seismic performance, to describe a framework for the implementation of PBEE, and to describe the opportunities and implications of PBEE for geotechnical engineering practice.

PERFORMANCE IN EARTHQUAKES

The development and implementation of PBEE requires that earthquake professionals be able to define performance in terms that are understandable and useful to the wide range of technical and non-technical professionals who make decisions on the basis of performance predictions. The term "performance" can mean different things to different people. To a seismologist, spectral acceleration may be a good descriptor of the potential performance of a building subjected to earthquake shaking. To an engineer, maximum interstory drift would likely be a better descriptor of performance. To an estimator preparing a bid for repairs, crack width and spacing could be more useful measures of performance. Finally, to an owner, the economic loss associated with earthquake damage could be the best measure of performance.

These different notions of performance lead to an intuitive way of viewing the earthquake process. As illustrated in Figure 1, an earthquake produces ground motion, which leads to dynamic response of a structure. That response can lead to physical damage, and that damage leads to losses. The prediction of losses, therefore, requires that we also be able to predict ground motion intensity, system response, and physical damage. If losses are the ultimate measure of performance (and the fact that they are usually of greatest importance to those who make the final decisions on seismic design, repair, and retrofitting efforts suggest that they should be), PBEE should focus on predicting losses as accurately, consistently, and reliably as possible. Working from the end to the beginning, the following sections describe losses and the progression of events that produce them.

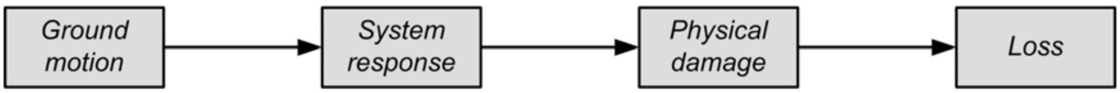

Fig. 1. Schematic illustration of the progression that leads to earthquake losses.

Losses

Loss can have many components, but they are usually divided into three categories often described informally as the "three D's" – deaths, dollars, and downtime.

Death and serious injury are certainly the worst kinds of losses, and their prevention has been the fundamental basis of seismic design since the concept of designing for earthquakes was first contemplated. While the goal of preserving life safety during earthquakes has largely been achieved in many urban areas of well-developed countries, millions of people remain at risk in the many areas around the world where, for economic, political, or other reasons, earthquake-resistant design and construction are not practiced.

The economic losses associated with earthquake damage are many, but are often divided into two categories – direct and indirect losses. Direct losses are those associated with the repair and/or replacement of structures and facilities damaged by earthquake shaking. Direct losses also include those associated with the contents of structures, which in some cases (e.g., museums, data centers, medical research laboratories, etc.) can be far more valuable than the structures that house them. Indirect losses include those associated with delayed or lost business, environmental damage, compromised infrastructure, etc.

Downtime, which refers to the period of time in which structures or facilities are unavailable for their intended use, is among the most important of indirect economic losses and can produce, for critical systems, losses that far exceed direct losses. The loss of a major bridge or water supply line in a non-redundant system, for example, can lead to inefficiencies in moving goods, services, and people or to health problems that have very real, and very high, economic consequences.

While desirable, efforts at expressing all three dimensions of loss in common terms have been understandably difficult. The most obvious metrics of loss are economic – quantities such as present value, expected annual loss, and probable maximum loss can and have been used to express performance in economic terms. Such quantities lend themselves very well to the description of direct losses. Putting indirect losses in economic terms can be more difficult, particularly given the wide spatial and temporal scales on which earthquake damage and recovery occur, but the problem at least seems tractable. The expression of deaths and serious injuries in economic terms has long been studied, and approaches considering factors such as age and future earnings potential have been developed to address the purely economic part of the problem; the emotional and social components of such losses, however, remain unquantifiable.

Physical Damage

The various forms of loss described in the previous section result from physical damage to structures and facilities. Consider a building, for example, subjected to earthquake shaking of various intensities. At low levels of shaking, some unrestrained objects within the structure may shift or fall – this could be as innocuous as books falling from a shelf or as serious as bottles of acid falling in a laboratory; in either case, losses can occur. At stronger levels of shaking, sheetrock and interior partitions can crack, plumbing pipes can break, cover concrete can spall, and windows or cladding can break and fall, all leading to further losses. At even stronger levels of shaking, beams can crack, joints can fail, foundations can rock and settle, diagonal braces can buckle – this is structural damage, which results in even greater losses. At very strong levels of shaking, ground movement can occur, foundations can fail, welded connections can fracture, columns can lose capacity and collapse can occur – such severe physical damage can lead to extremely high losses. In order to predict the losses associated with these and other forms of physical damage, it is necessary to identify the specific form(s) of physical damage that contribute most strongly to the losses of interest, and to be able to predict the physical damage associated with the response of the system of interest.

System Response

The types of physical damage described in the preceding section occur when the response of a structure and its components, which can be expressed in terms of force/stress or displacement/strain, exceed their capacities. Structures, whether comprised of steel, concrete, or soil, are compliant and therefore respond more strongly at some frequencies than others. They exhibit generally linear behavior at very low levels of loading but can become highly nonlinear and inelastic at higher levels of shaking. Their stiffnesses can change dramatically from the beginning of an earthquake to the end and even as in the cases of liquefiable soils and damage to reinforced concrete buildings, within a given cycle of loading. Response to earthquake loading will depend on the mass, geometry, stiffness, and damping characteristics of the structure and its foundation and on the amplitude, frequency content, and duration of the ground motion. In order to predict the damage associated with structural response, it is necessary to identify the measures of response that are most closely related to damage, and to be able to predict the response caused by earthquake ground motion.

Ground Motion

The response of a compliant system can vary dramatically from one earthquake to another, and from one location to another in the same earthquake, because of differences in the intensity of the ground motion. Earthquake engineers characterize the intensity of ground motions using parameters such as peak acceleration, spectral acceleration, duration, etc. The most useful ground motion parameters are those to which the response of the system of interest is most closely related. The optimum parameters for predicting response should be recognized as being different for different types of systems.

EVOLUTION OF PERFORMANCE-BASED EARTHQUAKE ENGINEERING

The concept of PBEE is a natural extension of the basic concepts that have underlain seismic design and evaluation for many years. Even the first edition of the Structural Engineers Association of California Blue Book (SEAOC, 1959) described the intention of its lateral force requirements as ensuring that a structure would be able to resist:

- a minor level of shaking without damage (non-structural or structural),
- a moderate level of shaking without structural damage (but possibly with some non-structural damage), and
- a strong level of shaking without collapse (but possibly with both non-structural and structural damage).

In this early document, basic performance goals are specified (albeit in terms of somewhat vaguely defined levels of "damage") and different performance goals are specified for different levels of ground motion.

Early efforts at seismic design were scenario-based, i.e., based on the identification of one or more "design earthquakes" (e.g., maximum credible and maximum probable earthquakes) typically specified by source (fault), magnitude, and location. The ground motions associated with the design earthquakes were estimated deterministically, initially by heuristic specification of peak ground acceleration, but later using median values from early attenuation relationships.

A major step forward in seismic design came in the late 1970s with the publication of the ATC-3-06 (1978) report. ATC-3-06 built on the probabilistic seismic hazard analysis concepts of Cornell (1968) and the mapping work of Algermissen and Perkins (1976) to express design seismic loading in a probabilistic manner. Recognizing the dramatic differences in seismic activity across the United States, ATC-3-06 presented contour maps of effective peak acceleration (*EPA*) and effective peak velocity (*EPV*) which, together with soil profile coefficients, could be used to develop design response spectra for structures. The use of these two measures of ground motion intensity accounted for differences in the high- and low-frequency characteristics of ground motions, and the soil profile coefficient controlled spectral shape. ATC-3-06 recommended that design be based on a single level of ground shaking – that with a 10 percent probability of exceedance in a 50-yr period, i.e., a 475-yr return period. On the structural side, ATC-3-06 provided guidance for the allowance of inelastic behavior of components through the provision of ductility, and based performance on the relationship between estimated and allowable interstory drifts. The allowable story drifts were presented for four seismic performance categories in three seismic use groups. Thus, the use of deformation-based response and capacity measures was introduced.

Subsequent U.S. building codes have maintained the basic approach of ATC-3-06 but have refined many of the details. Instead of basing design spectra on *EPA* and *EPV*, they are now based on short-period (0.2 sec) and long-period (1.0 sec) spectral accelerations. Soil effects are accounted for by more refined soil classification systems with site class coefficients that account for basic effects of nonlinear response. Design response spectra within UBC are based on 2/3 of the values of 2,475 yr (2% probability of exceedance in 50 yrs) spectral accelerations rather than 475-yr values, a change of somewhat convoluted logic that increases design requirements in areas subjected to relatively infrequent occurrence of large earthquakes (e.g., central and eastern United States) without substantially increasing them in more active areas (e.g., California).

The first document widely recognized as establishing procedures for performance-based design of new structures was the Vision 2000 report (SEAOC, 1995). Vision 2000 described procedures intended to produce structures "of predictable performance" with respect to a series of discrete hazard levels. Figure 2 shows how Vision 2000 coupled four discrete performance levels (fully operational, operational, life safe, and near collapse) with four ground motion hazard levels (frequent, occasional, rare, and very rare) for structures with performance objectives for three categories of structures (basic, essential/hazardous, and safety critical). The Vision 2000 report describes the general levels of damage to various building components and provides allowable inter-story drift limits associated with the four performance levels. These limits are expressed deterministically but are intended to be conservative; the degree of conservatism, however, is not known. Thus, Vision 2000 provides for design based on multiple levels of performance at multiple hazard levels, with performance related to deformation-related quantities (e.g., inter-story drift) that are closely related to damage.

Subsequent efforts, such as the investigations summarized in FEMA 273 (Applied Technology Council, 1996a), FEMA 274 (Applied Technology Council, 1996b), FEMA 356 (American Society of Civil Engineers, 2000), and ATC-40 (Applied Technology Council, 1996), used performance-based frameworks similar to that of Vision 2000, but differ in the manner in which performance and hazard levels are defined and in their recommended procedures for estimating force- and displacement-related demands.

In Europe, Eurocode 8 (EC 8, 2004) provides two design levels of ground motion: a damage limitation level (95-yr return period) and a no-collapse level (475-yr return period), along with importance factors for special structures (1.2 for high-occupancy structures; 1.4 for essential structures). Individual countries, however, are given a degree of flexibility in how they choose to implement certain aspects of the code. The Building Standard Law of Japan was revised in 1998 and enacted in 2000 (Kuramoto, 2006) with performance-based concepts. Two limit states are considered: a life safety limit state in which collapse of the entire structure or any floor of the structure does not occur, and a damage limitation limit state in which structural

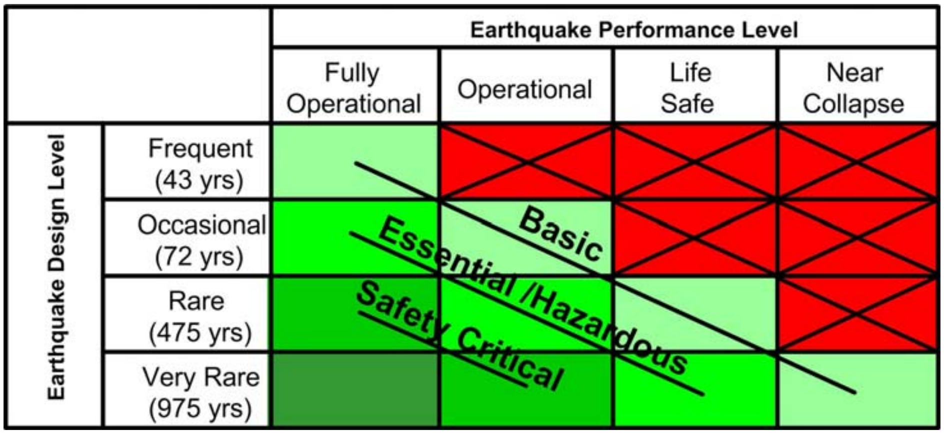

Fig. 2. Combinations of earthquake hazard and performance levels proposed by Vision 2000.

damage sufficient to cause future exceedance of the life safety limit state criterion does not occur. The life safety limit state must be satisfied for an "extremely rare" ground motion expected to occur once in approximately 500 yrs and the damage limitation limit state for a motion one-fifth as strong.

Thus, the current state of PBEE practice can be characterized by design for specific, discrete performance goals at up to about four ground motion hazard levels. The performance goals are generally expressed in terms of limiting values of response parameters (e.g., interstory drift) or, for structures, in terms of limit states (e.g., collapse). In geotechnical engineering, the ground motions associated with the design hazard levels are determined probabilistically but the response predictions and performance goals are generally deterministic. The implicit assumption is that limiting system response to certain levels will limit physical damage and losses to acceptable levels, but the actual amounts of expected damage and loss are not explicitly considered.

PERFORMANCE PREDICTION

Although the previously described PBEE frameworks are based on small integer numbers of discrete performance levels, there is no fundamental reason why performance cannot be considered on a continuous scale. Evaluations of the performance of existing structures or assessment of the performance of new structural designs require an ability to predict the anticipated performance of a structure subjected to seismic loading. As suggested by the discussion in the preceding section, performance depends on physical damage, physical damage depends on system response, and system response depends on ground motion intensity.

Terminology

In order to describe these quantities and the relationships between them, the notation developed by the Pacific Earthquake Engineering Research (PEER) Center will be adopted. The level of ground motion produced by earthquake shaking can be characterized by one or more *Intensity Measures*, *IM*s, which could be any of a number of ground motion parameters (e.g., a_{max}, S_a, Arias intensity, etc.). The response of the system of interest (e.g., excess pore pressure, interstory drift, etc.) to the ground motion can be described by *Engineering Demand Parameters*, or *EDP*s. The physical damage associated with the response (e.g., slab cracking, wall tilt, etc.) is expressed in terms of *Damage Measures*, or *DM*s. Finally, the losses associated with the physical damage (e.g., casualties, repair cost, downtime, etc.) are expressed in a form that is useful to decision-makers by means of *Decision Variables*, *DV*.

With this basic terminology in place, the performance prediction process can be viewed as moving from ground motion (*IM*) to response (*EDP*) to damage (*DM*) and, finally, to loss (*DV*). As illustrated in Figure 3, this progression takes place through the use of three predictive models: a response model, a damage model, and a loss model. A *response model* predicts the response of a structural system due to imposed ground motion, i.e., it predicts *EDP* from *IM*. Due to the various uncertainties in ground motions, response model parameters, and the response model itself, the response prediction process must be viewed as uncertain. A *damage model* predicts the physical damage associated with a given level of response (*DM* from *EDP*). The level of damage caused by a given level of response depends on the capacity of the system and capacity is, for a number of reasons, uncertain. As a result, uncertainty is associated with the damage model. Finally, a *loss model* predicts losses from physical damage, i.e., *DV* from *DM*. Due to uncertainties in quantities and unit costs, which can be affected by uncertain factors such as future material and labor costs, interest rates, repair times, etc., considerable uncertainty also exists in loss modeling.

Estimator Characteristics

Statistical inference states that good estimators should be unbiased, consistent, robust, efficient, and sufficient. The first three of these requirements are well-known and relatively intuitive, but the latter two are important enough to consider in some detail. A variable, *A*, is an *efficient* estimator of *B* if the uncertainty in $B|A$ (e.g., $\sigma_{B|A}$) is small. For example, peak acceleration (as an *IM*) is a relatively efficient estimator of slope displacement (as an *EDP*), but duration (by itself) is not. The variable, *A*, would be a *sufficient* estimator of *B* if the uncertainty in *B* is not reduced by additional information (i.e., $\sigma_{B|A,\boldsymbol{X}} = \sigma_{B|A}$ where $\boldsymbol{X}$ is a vector of additional variables). Peak acceleration, for example, is an insufficient predictor of liquefaction potential, as evidenced by the need for a duration proxy (i.e., magnitude, in the form of the magnitude scaling factor) to enable accurate predictions of liquefaction potential. A perfectly efficient and sufficient estimator would be one to which the estimated variable is uniquely related. Such estimators do not exist in the process of moving from *IM* to *EDP* to *DM* to *DV*, but some estimators are considerably more efficient and sufficient than others. As will be shown, the benefits of working with efficient and sufficient estimators in PBEE are great enough to make their identification and use worthwhile.

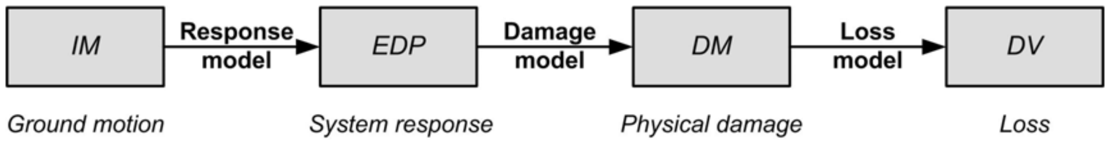

Fig. 3. Schematic illustration of performance prediction process.

THE PEER FRAMEWORK

The Pacific Earthquake Engineering Research Center (PEER) has proposed a framework for PBEE that is encapsulated in a "framing equation" formally presented in its most general form as

$$\lambda(\boldsymbol{DV}) = \iiint G(\boldsymbol{DV} \mid \boldsymbol{DM}) \left| dG(\boldsymbol{DM} \mid \boldsymbol{EDP}) \right| \left| dG(\boldsymbol{EDP} \mid \boldsymbol{IM}) \right| \left| d\lambda(\boldsymbol{IM}) \right| \tag{1}$$

In Equation (1), $G(a|b)$ denotes a complementary cumulative distribution function (CCDF) for a conditioned upon b (the absolute value of the derivative of which is the probability density function for a continuous random variable) and the bold type denotes vector quantities. From left to right, the three CCDFs result from the loss, damage, and response models; the final term, $d\lambda(\boldsymbol{IM})$ is from the seismic hazard curve. The framing equation implicitly assumes that the quantities used to describe ***IM***, ***EDP***, and ***DM*** are sufficient predictors of ***EDP***, ***DM***, and ***DV***, respectively. This triple integral can be solved directly only for an idealized set of conditions, so it is solved numerically for most practical problems; the numerical integration can be accomplished (assuming scalar parameters for simplicity) as

$$\lambda_{DV}(dv) = \sum_{k=1}^{N_{DM}} \sum_{j=1}^{N_{EDP}} \sum_{i=1}^{N_{IM}} P[DV > dv \mid DM = dm_k] P[DM > dm_k \mid EDP = edp_j] P[EDP > edp_j \mid IM = im_i] \Delta\lambda_{IM}(im_i) \tag{2}$$

where $P[a|b]$ describes the probability of a given b, and where N_{DM}, N_{EDP}, and N_{IM} are the number of increments of DM, EDP, and IM, respectively; accuracy increases with increasing number of increments.

The PEER framework has the important benefit of being modular. The discretized framing equation (Equation 2) can be broken down into a series of components, e.g.,

$$\lambda_{EDP}(edp) = \sum_{i=1}^{N_{IM}} P[EDP > edp \mid IM = im_i] \Delta\lambda_{IM}(im_i) \tag{3a}$$

$$\lambda_{DM}(dm) = \sum_{j=1}^{N_{EDP}} P[DM > dm \mid EDP = edp_j] \Delta\lambda_{EDP}(edp_j) \tag{3b}$$

$$\lambda_{DV}(dv) = \sum_{k=1}^{N_{DM}} P[DV > dv \mid DM = dm_k] \Delta\lambda_{DM}(dm_k) \tag{3c}$$

which means that hazard curves can be computed for EDP, DM, and DV and interpreted in the same manner as the more familiar seismic hazard curve (for IM) produced by a PSHA.

The conditional probability terms in Equations (3) can be expressed graphically in the form of fragility curves (Figure 4). Using Equation (3a) as an example, the fragility curves can express the variation of mean (or median) *EDP* with *IM* and the uncertainty in *EDP|IM* (represented by the response dispersion, β_R). The spacing of the fragility curves (for equal increments of *EDP*) reflects the linearity of the *EDP-IM* relationship and the steepness of the curves indicates the uncertainty in that relationship; a perfectly vertical fragility curve would correspond to no uncertainty in *EDP|IM*.

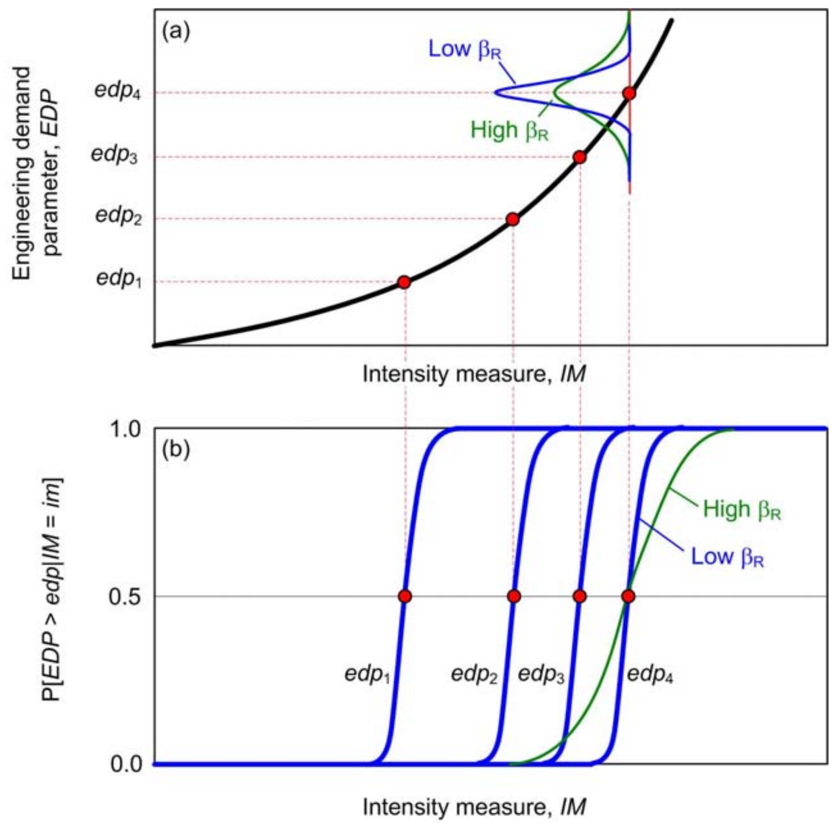

Fig. 4. Schematic illustration of relationship between (a) response relationship and (b) fragility curves. The effects of uncertainty on fragility curve shape is shown for $EDP = edp_4$.

The problem of performance evaluation can therefore be broken into four basic components – evaluation of ground motion hazard, evaluation of system response to the ground motions, evaluation of physical damage caused by the system response, and evaluation of losses associated with the physical damage. Using probabilistic response, damage, and loss models, the PEER framework allows this to be accomplished with proper consideration of uncertainty.

Randomness and Uncertainty

The PEER PBEE framework is inherently probabilistic, as evidenced by the chain of CCDFs that make up much of Equation (1). The need for probabilistic treatment is evident from previous discussion of the accuracy with which ground motions, response, damage, and losses can be predicted. The term "uncertainty" is usually used in discussions of statistical parameters and probabilistic models, but it can be helpful to distinguish between randomness and uncertainty when developing, implementing, and interpreting PBEE models.

Randomness, which is frequently described by the term "aleatory uncertainty," refers to the inherent or intrinsic variability of some quantity or phenomenon; as a result, it cannot be reduced by additional data or more thorough investigation. Randomness can manifest itself, for example, in the variability of response produced by different ground motions, even when scaled to the same *IM*. This record-to-record variability, which results from the apparently random, unpredictable nature of earthquakes, is a very significant component of the overall uncertainty in a typical PBEE investigation. Uncertainty due to lack of data or knowledge concerning the quantity or phenomenon is frequently referred to as "epistemic uncertainty." Epistemic uncertainty differs from aleatory uncertainty in that it can be reduced by the acquisition of new information, e.g., by additional data, more extensive investigation, or by new research.

The distinction between aleatory and epistemic uncertainties can be difficult, ambiguous, and confusing. In practice, the distinction often depends as much on pragmatic as theoretical concerns. While arguments can be made that all uncertainty is epistemic, practical considerations require that some be treated as aleatory; one could, for example, gain knowledge of the inherent variability of a natural soil deposit by drilling and sampling the entire site with boreholes on a six-inch spacing – an action so obviously impractical (and destructive) that it illustrates why such variability is treated as aleatory. The assignment of aleatory vs. epistemic uncertainty can also be situation-dependent. For example, uncertainty in the shear wave velocity of an existing earth dam would be characterized as epistemic if it is possible to measure it using various geophysical techniques; the shear wave velocity of a future earth dam, however, would be characterized as aleatory if the source of the fill material from which it is to be constructed is not known.

The nature of the models used to predict performance will also affect the aleatory-epistemic distinction. All predictive models should be recognized as mathematical idealizations of reality – they are not perfect. Model uncertainty, i.e., errors in model predictions, have two primary components: (a) the effect of missing predictive variables, and (b) the effects of inaccurate model form. Missing variables may be those not recognized as being influential or those that cannot be measured or otherwise characterized. Inaccurate model form may result from practical consideration of computational complexity/effort or lack of understanding of the basic physics of the problem. Both components of model uncertainty can potentially be reduced, by including additional predictive variables and/or the use of improved mathematical expressions, but there will usually be a limit to the number of variables that can be identified and/or measured or to the understanding of the physics of the problem of interest that will limit the degree to which uncertainty can be reduced. Therefore, model uncertainty will generally have both aleatory and epistemic components. The fact that different models are frequently of different form and use different predictive variables means that they will predict different output values. The variability of mean (or median) predictions from different plausible models, therefore, represents another component of epistemic uncertainty. This situation is familiar in the context of PSHA where different attenuation relationships, for example, are used with their weighted contributions accounted for through a logic tree. To properly account for epistemic uncertainty in response, damage, and loss predictions, multiple predictive models, where available, should also be used.

The need for distinguishing between aleatory and epistemic uncertainty depends on the manner in which the results of the PBEE analysis will be used. The final result of the PEER PBEE framework is a mean annual rate of exceedance (or corresponding return period) of some loss level; the mean value is invariant with respect to the characterization of uncertainty as aleatory or epistemic. As such, that characterization doesn't matter in the end – the numerical value of the loss hazard will be the same regardless of whether some component of uncertainty is treated as aleatory or epistemic – what matters is properly capturing the total uncertainty. It should be noted that the aleatory/epistemic distinction can become important if design or evaluation is to be based on some percentile (rather than mean) loss value. In such cases, the value of interest can be sensitive to the manner in which uncertainty is divided into aleatory and epistemic components, and care must be taken to ensure that this division can be justified as fair and objective (i.e., not influenced by economic or competitive factors).

Even when the mean hazard is used, however, it is still useful to consider which components of uncertainty can and cannot be reduced and the costs and benefits of doing so. As will be illustrated shortly, increasing uncertainty tends to drive the ground motions, response, damage, and losses for a given return period higher in a performance-based evaluation. The ability to show the benefits of increased investment, for example, in additional subsurface investigation or more sophisticated response modeling, represents a tremendous opportunity for geotechnical earthquake engineering practitioners.

Idealized Performance Model

With the aid of some simplifying, but not unrealistic, assumptions, the PEER PBEE framing equation (Equation 1) can be solved in closed form (Jalayer, 2003) providing expressions that are very useful for understanding the effects of the relationships between ground motion, response, damage, and loss, and particularly of the uncertainties in those quantities on performance.

Many seismic hazard curves are nearly linear on a log-log plot, at least over significant ranges of ground motion intensity (Department of Energy, 1994; Luco and Cornell, 1998), which implies a power law relationship between mean annual rate of exceedance and *IM*, i.e.,

$$\lambda_{IM}(im) = k_0 (IM)^{-k} \tag{4}$$

In this expression, k_0 is the value of $\lambda_{IM}(im = 1)$ and k is the slope of the seismic hazard curve (in log-log space, in which Equation (4) plots as a straight line); it should be noted that the slope of the hazard curve describes the relative frequencies of low and high *IM* values, which vary from one location to another. If the response model is also assumed to be of power law form,

$$EDP = a(IM)^b \tag{5}$$

with lognormal dispersion ($\sigma_{\ln EDP|IM} = \beta_R$) that has statistically independent aleatory and epistemic components, $\beta_{R,A}$ and $\beta_{R,E}$, such that $\beta_R^2 = \beta_{R,A}^2 + \beta_{R,E}^2$, the resulting *EDP* hazard curve can be expressed as

$$\lambda_{EDP}(edp) = k_0 \left[\left(\frac{edp}{a} \right)^{1/b} \right]^{-k} \exp\left[\frac{k^2}{2b^2} \left(\beta_{R,A}^2 + \beta_{R,E}^2 \right) \right] \tag{6}$$

This equation describes the mean annual rate of exceeding some level of response, *EDP* = *edp*, given the seismic hazard curve, which is the result of a probabilistic seismic hazard analysis, and a probabilistic response model. It therefore considers *all* possible values of *IM* rather than only those corresponding to the small integer number of return periods considered by codes and even PBEE approaches such as that described in Vision 2000. Equation (6) is composed of two parts, the first of which is a function of *edp* and the constants (k_0, k, a, b) that describe the mean hazard curve and the median *EDP-IM* relationship (i.e., the response model). The second part depends on the slopes of the hazard curve and median response model relationship and, most significantly, on the uncertainty in the response model. The second term can be viewed as an "uncertainty multiplier" since its value is 1.0 when there is no uncertainty and becomes progressively greater than 1.0 as the response model uncertainty increases. This result shows that the mean annual rate of exceedance of a particular *EDP* value increases with increasing uncertainty. Put another way, the *EDP* value corresponding to a given mean annual rate of exceedance (or return period) increases with increasing response model uncertainty.

As an example, Figure 5(a) shows *PGA* hazard curves for downtown San Francisco at the six return periods available through the USGS National Seismic Hazard Map website (2002 maps) and an approximation of the form of Equation (4) (k_0 = 0.00015, k = -3.374) that is exact at return periods of 225 and 975 years. Figure 5(b) shows the permanent displacement of a rigid slope with yield coefficient, k_y = 0.05, in a M_w = 7 earthquake predicted by the empirical model of Bray and Travasarou (2007), and an approximation to that response in the form of Equation (5) (a = 277.1, b = 2.162). Using the closed form solution of Equation (6), slope displacement hazard curves can be computed for various levels of uncertainty in displacement given *PGA* as shown in Figure 5(c)). The displacements at each return period exceed the zero-uncertainty (β_R = 0) displacements by factors of 1.08, 1.36, 1.98, and 3.38 for β_R values of 0.25, 0.50, 0.75, and 1.00, respectively. Bray and

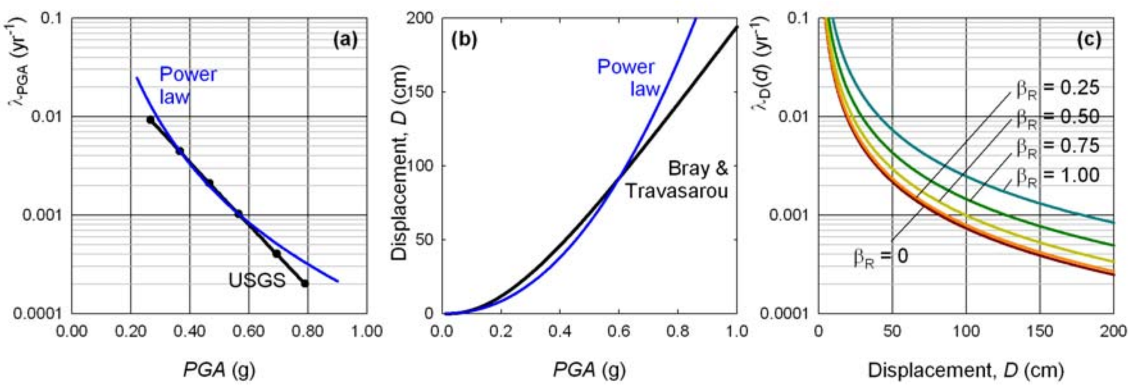

Fig. 5. Use of idealized power law models for (a) *IM* hazard curve and (b) *EDP-IM* response model to predict (c) *EDP* hazard curves for different response model uncertainties.

Travasarou (2007) indicate that β_R = 0.66 for the data on which their model was calibrated. Although a more accurate displacement hazard curve could have been obtained by numerically integrating Equation 3(a) over the USGS hazard curve using the Bray and Travasarou (2007) model, the simple idealized model shows clearly that the effect of uncertainty on response hazard is significant. While the Bray and Travasarou (2007) model is likely the most advanced empirical model available at this time, it is not the only plausible empirical model available; analysis of this problem with other empirical models would produce similar, but nevertheless different, mean (or median) response curves (Figure 5(b)) that would add additional epistemic uncertainty.

Assuming damage and loss models to also be of power law form, i.e., $DM = c(EDP)^d$ and $DV = e(DM)^f$ with lognormally distributed residuals, $\sigma_{\ln DM|EDP} = \beta_D$ and $\sigma_{\ln DV|DM} = \beta_L$, respectively, the DV hazard curve can be expressed in closed form as

$$\lambda_{DV}(dv) = k_0 \left[\frac{1}{a} \left\{ \frac{1}{c} \left(\frac{dv}{e} \right)^{1/f} \right\}^{1/d} \right]^{-k/b} \exp\left[\frac{k^2}{2b^2 d^2 f^2} \left(d^2 f^2 \beta_R^2 + f^2 \beta_D^2 + \beta_L^2 \right) \right] \quad (7)$$

Again, it is apparent that the loss curve consists of a term that depends on the mean *IM* hazard curve and the median response, damage, and loss model relationships, and a second term that depends on the various uncertainties in the prediction of response, damage, and loss (and the slopes of the *IM* hazard curve and median response, damage, and loss model relationships). Increased uncertainty in any of these models will increase the mean annual rate of *DV* exceedance.

Mackie and Stojadinovich (2006) produced a Matlab program called Fourway that performs the PBEE calculations described in the previous section. The program allows users to change hazard, response, damage, and loss model characteristics (median relationships and/or uncertainties) and see the corresponding effects on response damage, and loss hazard curves in a convenient graphical format.

Numerical Solution

While the assumptions underlying the development of the closed-form solution described in the preceding section are not grossly unreasonable, they do not capture all of the details of ground motion hazard, response, damage, and loss for specific structures located at specific sites. In such cases, the PEER framing equation must be solved numerically. Given the modular nature of the framing equation, the numerical integration process can be applied three times – first, to go from *IM* to *EDP*, then from *EDP* to *DM*, and finally from *DM* to *DV*. Considering the first step, the integration can be performed as

$$\lambda_{EDP}(edp) = \sum_{i=1}^{N_{IM}} P[EDP > edp \mid IM = im_i]\Delta\lambda_{IM}(im_i) \qquad (8)$$

The integration process involves two terms. The $\Delta\lambda_{IM}(im_i)$ term is a function of the seismic hazard curve, which comes from a PSHA. The PSHA can be site-specific or from the results of regional PSHAs such as those available from the USGS National Hazard Mapping Program (http://earthquake.usgs.gov/research/hazmaps/). The conditional probability term is a function of the response of the system which comes from a probabilistic response model and can be expressed in terms of fragility curves.

The procedure involves dividing the hazard curve into N_{IM} hazard intervals, determining the *IM* values at the center of each interval, and then using the response model to compute the probability of exceeding the *EDP* value of interest for each *IM* value. Summing the products of those exceedance probabilities and the hazard rate increments produces a single point on the *EDP* hazard curve at *EDP* = *edp*. The accuracy of the numerical integration process depends on the number of hazard rate increments – if 100 hazard rate increments were used to define 50 points on an *EDP* hazard curve, a total of 5,000 response model calculations would be required. Depending on the complexity of the response model, which could range from the algebraic equation of an empirical response model to a dynamic nonlinear finite element model, these calculations could be quite time-consuming.

Rather than run response models repeatedly for each hazard rate increment, it is more common to run a series of response analyses to define the relationship between *IM* and *EDP* and to then characterize that relationship with relatively simple functions. Two such functions are required – one to establish the relationship between median *EDP* and *IM*, and one to describe the uncertainty in *EDP*|*IM*. Two basic procedures can be followed:

1. A suite of ground motions can be scaled to a common *IM* value and applied to the response model to predict *EDP*s. The fact that the computed *EDP* values are not all the same is an indication of the record-to-record variability that is inherent in earthquake ground motions. By repeating this process for a number of *IM* values (with due consideration of dominant source characteristics in selection of the motions corresponding to each *IM* value), a plot with a series of “stripes” of response data (Figure 6(a)) can be generated.

Median values of *EDP* for each stripe can be used to establish a median *EDP-IM* relationship, and the distributions of residuals at each *IM* level can be used to characterize uncertainty in *EDP*|*IM*; it is common for *EDP*|*IM* values to be characterized as lognormally distributed.

2. A series of ground motions spanning a wide range of *IM* values (again selected with consideration of source characteristics) can be identified and used as input to a series of response analyses. The resulting *EDP* values form a "cloud" of data points on a plot of *EDP* vs. *IM* (Figure 6(b)). Regression techniques can be used to establish a median *EDP-IM* relationship and the residuals of the regression can be analyzed to characterize the distribution of *EDP*|*IM*.

Both approaches allow estimation of parameters describing the conditional distribution of *EDP* given *IM*, i.e., the development of fragility curves. The stripes approach is generally more efficient when considering a single *IM* level, and the cloud approach when multiple *IM* levels are considered (Mackie and Stojadinovich, 2006); the use of multiple stripes, however, allows improved characterization of *IM*-dependent dispersion, which can be significant over the wide range of *IM*s considered in a PBEE analysis (Baker, 2007).

Implementation of PBEE

The basic concepts of PBEE described in this paper can be implemented into engineering practice in a number of different ways. The modular nature of the PEER framing equation lends itself to different levels of implementation. In the simplest approach, PBEE could be implemented at the response level, i.e., by specifying performance in terms of *EDP* values at different response hazard levels (or response return periods); this approach would involve the application of Equation 3(a). An intermediate approach would be to specify performance in terms of damage limit states, which involves the comparison of demand (response) and capacity; this approach would involve prediction of *DM*s with the use of Equations 3(a) and 3(b). The most complete level of implementation would be to involve the definition of performance in terms of losses (*DV*s), which would involve Equations 3(a) – 3(c), i.e., the entire PEER framing equation (Equation 2). These levels of implementation are described in the following sections.

Response-Level Implementation

A response-level implementation of PBEE would allow evaluation of the mean annual rate of exceedance (or return period) of various levels of response. By combining the results of a PSHA with a probabilistic response model, this approach would provide a more consistent and objective evaluation of seismic response hazards than current procedures (which consider a single level of ground motion at a time).

Performance-based procedures for liquefaction hazard evaluation have been developed (Marrone et al., 2003; Kramer and Mayfield, 2005; 2007) and permanent slope displacements (Travasarou et al., 2004). Kramer and Mayfield (2007)

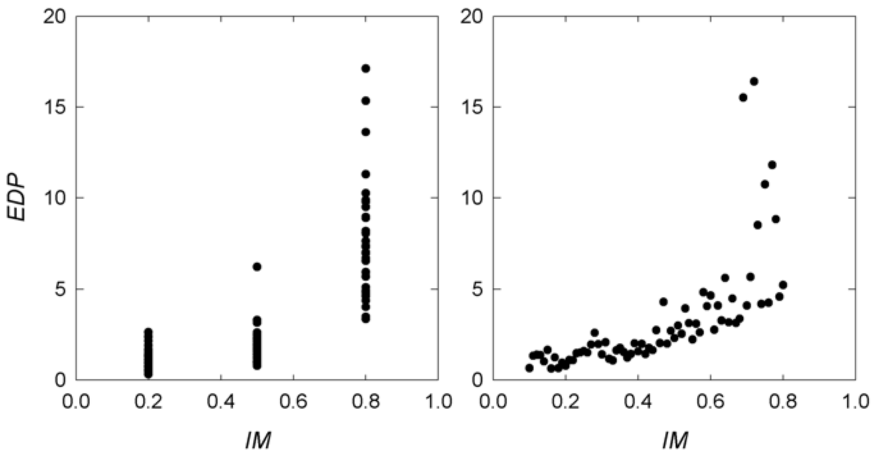

Fig. 6. Schematic illustration of (a) "stripes" approach, and (b) "cloud" approach to *EDP|IM* characterization.

described a performance-based procedure for evaluating liquefaction potential that resulted in factor of safety hazard curves (Figure 7) that account for all *PGA* levels and all magnitudes that contribute to those *PGA* levels, thereby rendering the question of which magnitude (mean or mode) to base magnitude scaling factor calculations on moot. Comparison of these hazard curves with factors of safety computed using conventional procedures (i.e., using one *PGA* level and one corresponding magnitude value) showed that typical designs (in this case, assuming FS_L = 1.2 for 475-yr *PGA* and mean magnitude) using conventional procedures produced highly variable actual likelihoods of liquefaction (i.e., return periods of liquefaction itself ranging from about 350 to 600 yrs) in different seismic environments. Basing design on a particular return period for liquefaction (Kramer et al., 2006) would provide more uniform performance than the current process of basing it on a deterministic factor of safety computed for a single ground motion hazard level.

Damage-Level Implementation

Once the response model has been used to compute the *EDP* resulting from a given *IM*, i.e $EDP = \mathcal{R}(IM)$, a probabilistic damage model can be used to estimate the *DM* resulting from a given *EDP*, i.e., $DM = \mathcal{D}(EDP)$. The damage model must be probabilistic so that it can predict the distribution of damage for a given level of response, i.e., the distribution of *DM|EDP*. The maximum allowable damage has frequently been referred to as a damage limit state in the structural engineering literature, where attempts at their explicit prediction are more advanced at this time than in geotechnical engineering.

Fragility Curve Approach

A probabilistic damage model can be used to develop damage fragility curves in much the same manner as probabilistic response models are used to develop response fragility curves. Characterization of damage using continuous *DM* scales has proven

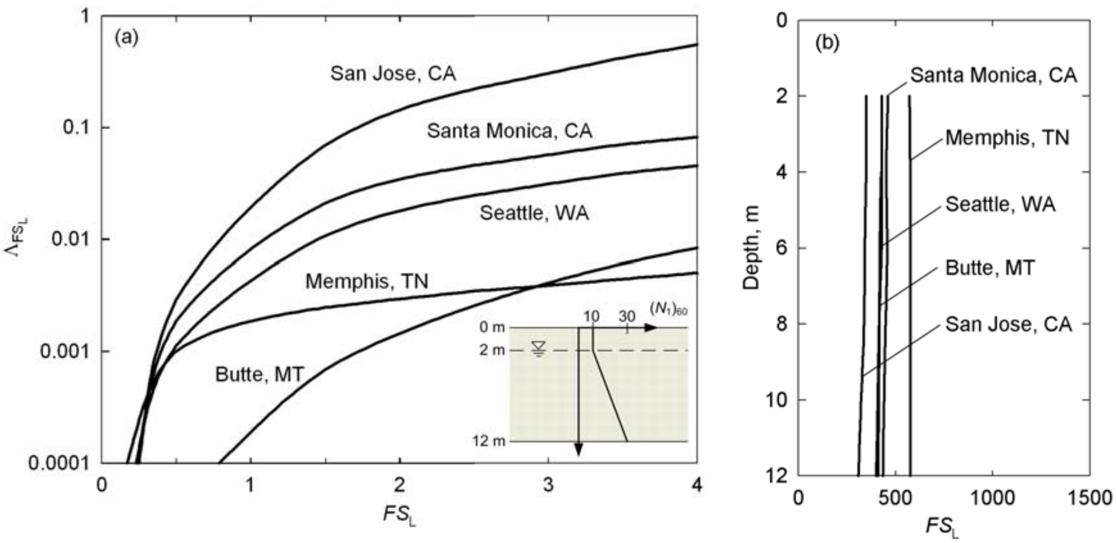

Fig. 7. (a) Mean annual rate of non-exceedance of factor of safety (factor of safety hazard curves) for an element of soil at 6 m depth in the standard soil profile (inset) defined by Kramer and Mayfield (2007), and (b) profiles of return periods of liquefaction (after Kramer and Mayfield, 2007).

to be somewhat problematic. In some cases, pertinent damage limit states (e.g., collapse of a structure or initiation of a flow slide) are essentially binary – they either occur and produce catastrophic damage or they don't. In other cases, loss estimators indicate that they do not use continuous scales, rather they consider small integer numbers of damage levels when estimating, for example, repair costs.

When discrete damage states are used, the process for estimating *DM* values remains straightforward. The continuous range of *EDP* levels must be discretized into an integer number of *EDP* intervals. For each damage state (e.g., negligible, slight, moderate, severe, and catastrophic), discrete distributions of *DM*|*EDP* can be defined by means of a matrix, perhaps $\boldsymbol{X}$, for which $X_{ij} = P[DM = dm_j \mid EDP = edp_i]$ and the sum of each row and column is unity. The matrix can be illustrated in tabular form as shown in Table 1.

Table 1. Damage state matrix for definition of *DM*|*EDP* relationship.

Damage State, *DM*	Description	*EDP* interval				
		edp_1	edp_2	edp_3	edp_4	edp_5
dm_1	Negligible	X_{11}	X_{12}	X_{13}	X_{14}	X_{15}
dm_2	Slight	X_{21}	X_{22}	X_{23}	X_{24}	X_{25}
dm_3	Moderate	X_{31}	X_{32}	X_{33}	X_{34}	X_{35}
dm_4	Severe	X_{41}	X_{42}	X_{43}	X_{44}	X_{45}
dm_5	Catastrophic	X_{51}	X_{52}	X_{53}	X_{54}	X_{55}

The total probability theorem can then be used to compute the probability of being in a given damage state using the conditional distribution of *DM*|*EDP* and the distribution of *EDP* ranges as

$$P[DM = dm_j] = \sum_{i=1}^{N_{EDP}} P[DM = dm_j \mid EDP = edp_i] P[EDP = edp_i] \quad (9)$$

Demand and Capacity Factor Approach

A damage-level implementation can also be stated in an LRFD-like format of demand and capacity factors (Jalayer, 2003). Such a format, which underlies recent steel design codes (Federal Emergence Management Agency, 2000a-c; Cornell et al., 2002), can also be applied to geotechnical damage. The format can be conveniently described using the closed form approximations described previously.

Loss-Level Implementation

The most complete performance evaluation can be accomplished by specifying performance in terms of losses. Such evaluations represent the ultimate expression of PBEE, and are likely to be justified primarily for particularly large and/or important projects in the near future. Nevertheless, they provide a useful and instructive look into the future of earthquake engineering practice.

Conte and Zhang (2007) describe a complete, detailed evaluation of the performance of the Humboldt Bay Middle Channel (HBMC) bridge (Figure 8) near Eureka in northern California. The 330-m-long, nine-span bridge, which was designed in 1968 and constructed in 1971, is supported on groups of precast, prestressed piles that extend through Tertiary and Quaternary alluvial soils. A 1.5 to 3-m-thick layer of soft to very soft organic silt blankets the entire site and is underlain by medium dense to dense silty sand (SP/SM) under the left abutment area, dense silty sand and sand (SP) in the central area, and soft sandy silt to loose silty sand (OL/SM) under the right abutment. These soils are underlain by dense and stiff soils. The river channel slopes toward the center of the channel at an average inclination of about 7 percent. The central piers (Piers 3-7, counting from the left) are supported on groups of 1.37-m-diameter, 1800 kN piles and the abutments and outer piers (Piers 1, 2, and 8) on groups of 356-mm-square, 400 and 625 kN piles. Expansion joints at the abutments and the tops of Piers 3 and 6 effectively divide the bridge structure into three frames. The fundamental period of the bridge-soil system was determined to be 0.71 sec.

The first-mode spectral acceleration, i.e., $S_a(T=0.71)$, was taken as the *IM* for this study. The results of USGS seismic hazard analyses were used to approximate the *IM* hazard curve for the site. A total of 51 ground motions were assembled (and scaled to appropriate $S_a(T=0.71)$ values) to represent the *IM* hazard at return periods of 72, 475, and 2,475 yrs (50-yr exceedance probabilities of 50%, 10%, and 2%, respectively).

A two-dimensional finite element model (Figure 9) using OpenSees (Mazzoni et al., 2006) was developed to estimate the response of the soil-foundation-structure system. The OpenSees model represented the nonlinear, inelastic behavior of cohesive and granular soils using pressure-independent and pressure-dependent multi-yield models (Yang et al., 2003), respectively. Piles in out-of-plane rows were

Fig. 8. Aerial view of Humboldt Bay Middle Crossing (Conte and Zhang, 2007).

lumped into composite piles and modeled as nonlinear beam-columns using a fiber model (although local interaction through *p-y* and *t-z* springs was not included). The superstructure was modeled using high-level nonlinear structural models that included expansion joints, bearings, and shear keys.

Examples of the computed response are presented in Figure 10, which shows the displacements of the tops and bases of all eight bridge piers in response to a single motion scaled to match the 2,475-yr *IM*. The response of the bridge is relatively coherent until the shear key at the Pier 6 expansion joint breaks (at t = 16.52 sec) after which the right frame responds differently (Figure 10(a)) than the other two frames which remain connected. The response at the bases of all piers (Figure 10(b)) is relatively coherent until about 28 sec at which time liquefaction occurs and lateral spreading causes the bases of the piers to move laterally toward the center of the river channel.

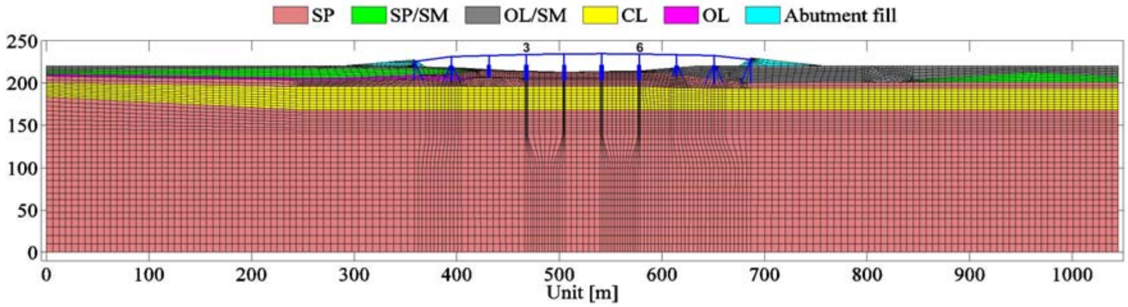

Fig. 9. Two-dimensional finite element model of Humboldt Bay Middle Crossing bridge site (Piers 3 and 6 are labeled); dimensions are in meters (after Conte and Zhang, 2007).

The results of even this single analysis illustrate some of the benefits of detailed soil-foundation-structure interaction analyses. While model development and calibration can be time-consuming, such analyses can explicitly model the response (and failure) of critical components like the Pier 6 shear key and account for whatever effects the soil and foundations might have on their response. They can also explicitly account for the generation of excess pore pressure in liquefiable soils, and for its effects on cyclic and permanent deformations of the soil and structure.

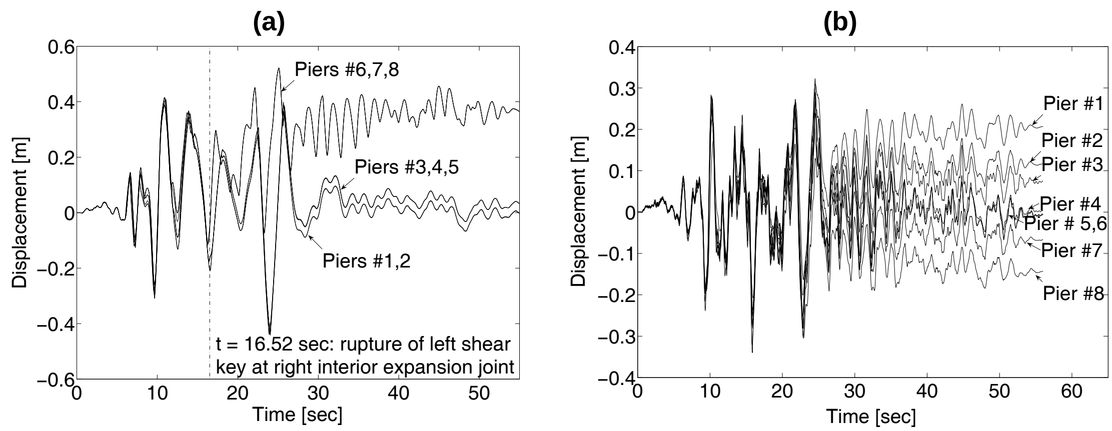

Fig. 10. Computed time histories of displacement: (a) tops of piers, and (b) bases of piers (Conte and Zhang, 2007).

Preliminary analyses had revealed that flexural failure in the regions of lap-splices at the bases of the piers, failure of unconfined shear keys, and unseating at the abutments and interior expansion joints were the most critical failure mechanisms for the HBMC bridge. *EDP*s considered to efficiently represent the response leading to those damage mechanisms were peak lateral bridge pier drift, peak shear key deformation, and peak unseating displacement at the expansion joints. The maxima of those *EDP*s over all components (i.e., all piers, shear keys, and expansions joints) were used to predict damage and loss.

The damage analyses were based on discrete damage states described by fragility curves developed from analytical and experimental investigations of pier flexure and shear key failure. Median displacements for unseating failure were taken as the width of the abutment seat or half the pier width at the interior expansion joints but, due to lack of experimental or field data on unseating, uncertainties in that damage mechanism had to be estimated. The *DM* fragility curves were combined with the *EDP* hazard curves to compute mean annual rates of exceedance for the various damage states. The results of these analyses, summarized in Table 2, showed that shear key failure was quite likely to occur (Damage State IV reached, for example, at a return period of 14.5 yrs), flexural failure of piers was also likely (Damage States III-IV-V reached at 47.6-yr return period), and collapse due to unseating was much less likely (return periods over 800 yrs).

Conte and Zhang (2007) used total repair cost as their decision variable. This cost was taken to be the construction cost of a new bridge (estimated at $24M) for global failure (i.e., collapse) damage states and the sum of the repair costs of all damaged components for non-collapse cases; indirect losses associated with downtime were not considered. Estimation of repair costs required identification of repair schemes for each damage state, and estimation of required repair quantities and unit costs for each repair scheme. These estimates were based on experience with retrofits on bridges in California, compiled unit cost data (California Department of Transportation, 2003), and consultation with experienced practitioners. Conte and

Table 2. Computed mean annual rates of exceedance and return period for various damage limit states.

Damage Mechanism	Limit State	λ_{DM} (yr^{-1})	Return Period (yrs)
Flexural failure of piers	II: Yielding of reinforcement	0.034	29.4
	III-IV-V: Initiation of failure mechanism; full formation of failure mechanism; strength loss	0.021	47.6
Shear key failure	I-II-III: Onset of cracking; reinforcement yielding; large open cracks and onset of spalling	0.080[a] 0.046[b] 0.064[c]	12.5[a] 21.7[b] 15.6[c]
	IV: Cracks and spalling over full region of key	0.069[a] 0.039[b] 0.058[c]	14.5[a] 25.6[b] 17.2[c]
	V: Loss of load-carrying capacity; fracture of reinforcement	0.010[a] 0.00008[b] 0.0058[c]	100[a] 12,500[b] 172[c]
Unseating	V: Collapse	0.0011[a] 0.0012[c]	909[a] 833[c]

[a]abutments; [b]continuous joints; [c]interior expansion joints

Zhang used a multi-layered Monte Carlo procedure to perform the integrations required to compute losses, producing the loss curve shown in Figure 11. The loss curve indicates that repair costs of $2M or more are relatively likely (return period on the order of 67 yrs), but costs exceeding $3.6M are quite unlikely (return period of about 1,000 yrs) since the loss curve drops quickly as repair costs exceed about $3M. A performance-based analysis also allows deaggregation of *EDP*, *DM*, and *DV* values just as *IM*s are deaggregated in PSHAs. Figure 12 shows the relative contributions of pier failure, shear key failure, and collapse as a function of *IM*. At *IM* levels less than about 0.2 g, losses are dominated by repair of shear key failures. At intermediate *IM* levels (0.2 g to 1.5 g), losses are associated with shear key and pier flexural failures. At *IM*s greater than about 2 g, collapse becomes more likely and increasingly dominates the expected losses. The unseating mechanism makes essentially no contribution to loss due to its relatively long return period (Table 2) and low repair cost. It should be noted that the repair costs in Figures 11 and 12 become asymptotic (at long return period/high *IM*) to the replacement cost of the bridge.

IMPLICATIONS AND OPPORTUNITIES FOR PRACTICE

The preceding sections have described a framework for PBEE and the basic mechanics of its use. The framework is clearly formulated to allow consideration of the many uncertain aspects of ground motion, response, damage, and loss estimation, and it has been shown that each of those quantities increase, for a given return period, with increasing levels of uncertainty. Performance-based concepts are making their

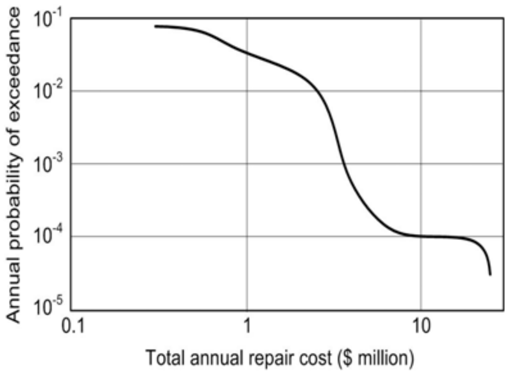

Fig. 11. Loss curve for Humboldt Bay Middle Crossing bridge (after Conte and Zhang, 2007).

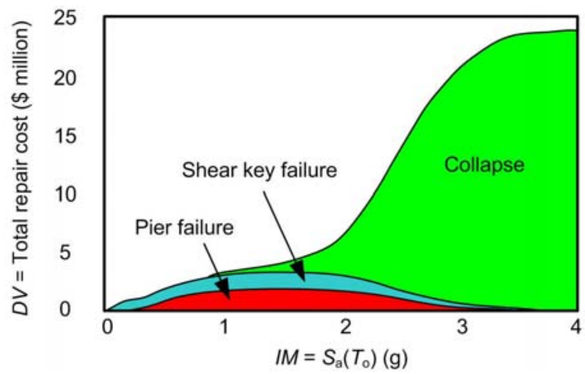

Fig. 12. Deaggregation of repair cost as function of ground motion intensity (after Conte and Zhang, 2007).

way into practice and into the codes and standards that strongly influence practice. The successful implementation of performance-based concepts into practice provides both challenges and opportunities for geotechnical engineering practice.

Challenges

In order to participate equally with other earthquake professionals in the future development, implementation, and practice of PBEE, geotechnical engineers will need to develop new awareness, knowledge, and tools; these can be thought of as challenges for the geotechnical earthquake engineering profession that will require its practitioners to:

1. *Understand the "big picture" and geotechnical engineering's place in it.* A variety of professionals are involved in the full process of estimating earthquake losses. Seismologists predict ground motion hazards for a reference site condition. Geotechnical engineers evaluate the effects of actual site condition on ground motion hazards and predict ground failure potential. Structural engineers evaluate structural response to the ground motions and predict the physical damage that occurs when response exceeds capacity. Loss analysts use physical damage estimates to predict direct and indirect economic and other losses. Finally, a decision-maker, typically the owner, will decide how to address earthquake risk – to accept (or ignore) it or to take steps to reduce it through planning, retrofitting, or insurance. Each of these professions play a key role in the big picture and each depends on the others for accurate and unbiased information. Each also deals with uncertainty, from the uncertainty in fault slip distribution that affects ground motions to the uncertainty in future interest rates that affects repair costs, and all of these uncertainties combine to affect estimated performance.

2. *Think probabilistically*. While it is not necessary for geotechnical practitioners to understand all of the daunting calculus of probability, it is necessary to develop a conceptual understanding of its basic tenets, and of the critical effects of uncertainty in performance estimation. Geotechnical engineers must recognize, and work to characterize, all sources of aleatory and epistemic uncertainty in their portion of the performance evaluation process. It is important that such characterizations be done accurately, i.e., without bias – the place for conservatism is not in the estimation of mean (or median) response. Uncertainties should be characterized realistically with proper recognition of the uncertainty associated with what is not known in addition to the uncertainty in what is known; the natural tendency for people to optimistically underestimate uncertainties in their estimates of things they are familiar with has been well documented by social scientists, and must be guarded against in engineering practice.

3. *Identify improved parameters*. Geotechnical engineers must strive to identify and validate the ground motion parameters (*IM*s) that correlate best to response, the response parameters (*EDP*s) that correlate best to physical damage, and the physical damage metrics (*DM*s) that correlate best to the losses of interest. These parameters should be efficient and sufficient, and the *IM*s must also be predictable (i.e., the uncertainty in their prediction by an attenuation relationship should be low – an *IM* that efficiently predicts *EDP* is not effective if its own value cannot be predicted accurately). For complex structures and facilities, more than one *IM* may be required to produce the least dispersed estimates of response. The use of vector *IM*s will require the development of ground motion "hazard surfaces" (the multi-dimensional equivalent of hazard curves) using vector-based PSHA, the basics of which have been laid out by Bazzuro and Cornell (2002) and Baker (2007). In such cases, the geotechnical engineer will have the added responsibility of estimating statistical correlation between the various parameters.

   For geotechnical-related response, optimal *IM*s will likely correlate well to deformations, which are related to strains. Basic wave propagation concepts show that strain amplitudes in a linear material are proportional to particle velocity, which suggests that optimal geotechnical *IM*s would likely lie in a frequency range closer to those associated with peak velocities than peak accelerations. Bray and Travasarou (2007) suggest the use of spectral acceleration at 1.5 times the fundamental period of a potentially unstable slope as an optimal *IM* for slope deformation predictions. Kayen and Mitchell (1997) suggested the use of Arias intensity as an *IM* for liquefaction analyses; Kramer and Mitchell (2003) identified another parameter, *CAV5*, as an optimal *IM* for liquefaction problems. Each of these parameters are associated with frequencies closer to those associated with peak ground velocity than to peak ground acceleration, and the use of each offers the potential to reduce the level of record-to-record variability that contributes strongly to uncertainty in *PGA*-based response models.

   Because geotechnical aspects of damage are generally related to deformations, optimal geotechnical *EDP*s are likely to consist of deformations, whether the permanent deformation (horizontal and/or vertical) of a sloping bridge approach or the peak curvature of a pile foundation. While traditional force-based (e.g., factor of safety) measures of response can be used in the PBEE framework described in this paper, the much higher uncertainty in damage given factor of safety (relative to damage given deformation) results in a substantial "penalty" in performance when they are used.

For a given type of structure or facility, identification of optimal *DM*s will likely require the input of loss analysts or construction estimators who would be asked questions such as "what type(s) of physical damage would most strongly control your estimated repair costs and repair times?" Continuous, measurable quantities such as crack width or wall tilt could be identified by respondents, but early investigations of such questions indicate that many professionals involved in estimating losses tend to use a small integer number (perhaps 3-4) of perceived damage states that are loosely defined functions of multiple damage observations. The use of discrete, rather than continuous, *DM*s presents no particular difficulty in the PEER framework, but the heuristic means by which damage is described can come with a substantial level of uncertainty.

4. *Develop improved response models*. For both site response and ground failure, improved response models that use optimal *IM*s as input and produce optimal *EDP*s as output are required. These models must be capable of predicting response over a wide range of ground motions – the PEER framework, for example, integrates all ground motion levels (ranging from the very weak motions associated with short return periods to the potentially very strong motions associated with long return periods) to estimate response. They must also be probabilistic, i.e., capable of producing the distribution of *EDP|IM*.

   Analytical models for estimating the response of soil and soil-structure systems have developed dramatically over the past 10-20 years. Early geotechnical models represented both ground motions and the physical systems of interest in crude, grossly simplified ways. Pseudo-static analyses, for example, replaced transient earthquake ground motions with constant, uni-directional accelerations, modeled physical systems as rigid and infinitely strong), and expressed response in terms of factors of safety. Subsequent geotechnical models allowed the use of actual ground motion time histories and accounted for at least some of the most basic characteristics of the physical system; Newmark-type sliding block analyses, for example, use an entire ground motion time history as input and model a slope using a rigid block bounded by a pre-determined failure surface with rigid-perfectly plastic (with shearing resistance equal to the average shear strength) force-displacement behavior. This type of analysis is more time-consuming than a pseudo-static analysis in that it requires identification of suitable input ground motions and a critical failure surface, and performance of the sliding block analysis itself, but it produces output in the form of an estimated slope displacement, a much more efficient *EDP* for damage prediction than pseudo-static factor of safety. In 2008, we have at our disposal advanced stress-deformation (finite element and finite difference) analyses that allow ground motions and the physical systems of interest to be represented with great rigor. Soil-foundation-structure interaction (SFSI) analyses can be performed without prior constraint of deformation mechanisms and with as complete a characterization of nonlinear, inelastic soil behavior as our constitutive models allow. The performance of such analyses is far more time-consuming than sliding block analyses, but can provide direct estimates of a wide range of geotechnical and structural *EDP*s without the constraints of *a priori* assumptions required by the simpler models.

Response models should be capable of predicting threshold levels of shaking below which response may not occur (for example, weak shaking may produce strains below the threshold shear strain and thereby produce no excess pore pressure in potentially liquefiable soils). They should also address upper limits to response (for example, when reasonable levels of uncertainty are applied to existing post-liquefaction settlement models, many of which include "estimated" volumetric strain contours that extend well beyond the levels observed in available laboratory tests, unrealistically high volumetric strains of 30-40% are produced with sufficiently high probability to significantly affect estimated settlements). Improvements are needed in both empirical and numerical response models. Empirical models need more case history data and better characterization of that data. Numerical models need user-friendly interfaces and sufficient computational efficiency to allow sensitivity and simulation (e.g., Monte Carlo) analyses to be performed and interpreted.

Characterization of response model uncertainty, from relatively simple empirical models to complex numerical models, is urgently needed. This will require application of the various models to case histories and, where appropriate, physical models. In such studies, uncertainties in the inputs must be characterized and accounted for to separate parametric uncertainty from model uncertainty. For years, the general public has asked why ground shaking hazards have continuously increased despite the acquisition of more and more data from recent earthquakes and expanded seismographic networks. A primary reason (Bommer and Abrahamson, 2006) is that uncertainties in previous hazard analyses were ignored or underestimated. Today, clients are frequently reluctant to allow engineers to perform the types of detailed response analyses they should perform because they fear a more conservative (hence, expensive) result. This situation may well exist because the uncertainties in simplified and/or empirical response analyses are being ignored or underestimated. A number of response models are based on limited and/or highly scattered data and should be recognized as such with high model uncertainties.

5. *Develop improved damage models*. A great deal more emphasis has been placed, particularly in research, on response prediction than on prediction of the physical damage (both structural and non-structural) resulting from that response. Geotechnical engineers must work with structural engineers and

loss analysts to identify the damage measures that most closely correlate to losses and to characterize the levels of response that produce physical damage for different structures. This procedure essentially requires the characterization of capacity, specifically the probabilistic characterization of capacity. Capacities have historically been thought of in force-related terms such as bearing capacity, shearing force, and bending moment, but can also be expressed in deformation-related terms. Since geotechnical aspects of damage are most closely related to deformations, particularly permanent deformations, capacities will need to be expressed in terms of permanent deformations for PBEE. Geotechnical engineers must be deeply involved in determining, for example, the distributions of slope displacements that cause various levels of damage to a bridge abutment or the amounts of settlement that cause various levels of damage to different building types. The classical studies of "tolerable" movements completed some 30-50 yrs ago (e.g. Skempton and MacDonald, 1956; Burland and Wroth, 1974) need to be updated with additional case history data, supplemented by modern soil-structure interaction analyses, and adapted to seismic conditions. Until such research becomes available, capacity characterization may take the form of elicitation of multiple expert opinions on issues such as the amount of displacement required for 50% probability of severe damage and 90% probability of severe damage under various scenarios. Opinion-based estimates of this type will necessarily be accompanied by relatively high uncertainties.

6. *Develop tools*. The performance-based procedures described in this paper require multiple calculations in the definition of median response, damage, and loss relationships and in the integration across distributions of *IM*, *EDP*, and *DM*. The calculations are not necessarily more complicated than those performed in current practice, but they need to be repeated many times. Furthermore, to properly account for epistemic uncertainty, multiple models should be used with user-determined weights and the variance in their results added to the variances from other sources of uncertainty.

7. *Consider the role and application of engineering judgment*. Geotechnical engineering has a long history of the beneficial application of experience in the form of engineering judgment. Because gravity never rests, the profession has developed a substantial history of performance under static loading conditions. The field of geotechnical earthquake engineering, however, is considerably younger than geotechnical engineering itself, and the relative infrequency of strong earthquakes provides fewer data on which to base engineering judgment. Furthermore, the loading induced by earthquakes is much more complicated than gravitational loading, and the relatively simple concepts on which conservatism can be induced in static evaluations (e.g., assuming a reduced strength) do not work the same way for dynamic problems. Engineering judgment can play an important role in geotechnical earthquake engineering as long as there is a strong basis for that judgment. The judgment that comes from carefully examined, evaluated experience can be used to reduce uncertainties in estimated response, damage, and loss. Peer

review may prove to play an important role in the validation of such judgment.

8. *Engage in professional practice issues*. The manner in which performance-based concepts are implemented will have a direct effect on the professional practice of geotechnical engineering in seismically active areas. Performance-based concepts have been making their way into codes in recent years and that trend is likely to continue in the future. Many of the seismic problems that geotechnical engineers deal with are significantly different than those faced by structural engineers, so it is important that geotechnical engineers be involved in the development of future codes and standards.

Opportunities

The development and implementation of PBEE also offers important and exciting opportunities for geotechnical practice. The fact that losses are so strongly affected by uncertainty and that geotechnical engineers frequently deal with high levels of uncertainty means that geotechnical engineering has great potential for reducing design and construction costs for new structures and for reducing earthquake losses in existing structures. The opportunities are for geotechnical engineers to:

1. *Produce improved products*. The application of PBEE principles, whether interpreted at the response, damage, or loss levels, will result in more consistent and uniform designs and evaluations. The levels of risk/safety can be made much more consistent from one geographic region (i.e., seismic environment) to another.

2. *Balance risk*. The application of PBEE principles, along with the use of improved analytical tools, can lead to more balanced designs by explicitly considering the effects of both geotechnical (e.g., foundation) and structural (e.g., superstructure) aspects of a particular facility in a consistent manner.

3. *Extend professional development*. The basic concepts of PBEE, and the skills required to implement it, will improve the technical skills of geotechnical engineers and will require them to work closely and communicate with earth scientists, structural engineers, and other earthquake professionals. These concepts, skills, and interactions are also transferable to other natural hazards and other aspects of geotechnical practice. The development and use of these skills can provide engineers with new and useful insights into the sources and effects of various uncertainties in many areas of geotechnical practice.

4. *Demonstrate value*. Current geotechnical earthquake engineering practice centers on the prediction of response, which is typically performed deterministically and presented with an informally determined level of conservatism. That conservatism frequently fails to adequately reflect the potential benefits (i.e., reduction of uncertainty) of factors such as additional

subsurface investigation, additional insitu testing, additional laboratory testing, and more detailed/sophisticated analyses. Each of these factors has the potential to reduce uncertainty, which has been shown in this paper to reduce losses at a particular return period. The PBEE framework provides an opportunity to demonstrate the value (or, to be fair, lack of value) that may be associated with different levels of geotechnical services.

5. *Engage in professional practice issues*. As stated previously, performance-based concepts will likely be implemented into future codes for buildings, bridges, etc., which presents geotechnical engineers with the opportunity to influence how that is done. Over the years, some codes have evolved from safety nets that ensured a minimum level of safety (and protected the public from the lowest levels of design/construction practice) to *de facto* design standards, as evidenced by the difficulty in gaining approval of a design, no matter how well-documented, that involves lower levels of loading or greater levels of resistance than specified by the code. Implementation of performance-based concepts such as those described in this paper into codes would allow the geotechnical engineer's ability to reduce uncertainty through comprehensive site investigation, detailed analysis, and application of relevant experience to benefit his/her client.

CONCLUDING REMARKS

This paper has reviewed the development of PBEE and shown how a modular PBEE framework can provide more objective, consistent, and accurate estimates of seismic performance. Different levels of implementation have been introduced, and a number of important challenges and opportunities that PBEE brings to the geotechnical engineering profession have been identified.

PBEE provides a framework in which the improved field, laboratory, and analytical tools now available to geotechnical engineers can be used to the advantage of the engineer and client. Its further development and implementation raises a number of important challenges that must be addressed in coming years, but it also provides important opportunities for geotechnical engineers to improve their products and demonstrate the value of the services they provide.

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