15th ASCE Engineering Mechanics Conference
June 2-5, 2002, Columbia University, New York, NY

PROBABILISTIC SEISMIC DEMAND MODELS FOR MULTI-SPAN HIGHWAY OVERPASS BRIDGES

Kevin Mackie[1], AM ASCE
and Bozidar Stojadinovic[2], AM ASCE

ABSTRACT

Probabilistic Seismic Demand Models (PSDMs) are an important component of the PEER (Pacific Earthquake Engineering Research Center) probabilistic performance-based seismic design framework. A PSDM defines a relationship between a seismic hazard Intensity Measure (IM), such as spectral displacement at the fundamental period of the bridge, and a structure-specific Engineering Demand Parameter (EDP), such as maximum column curvature. It provides a probability of exceeding an EDP value, given a value of associated IM. PSDMs are computed using a probabilistic seismic demand analysis procedure.

This paper presents PSDMs developed for multi-span highway overpass bridges typical for California. These overpass bridges have two, three or four spans, circular columns with integral pile shafts, and continuous box girder superstructures. Finite element models of different configurations for each span type were developed in OpenSees. Parameters of these base models were then varied to cover a range of typical bridge designs. Each parametric bridge model was subjected to 80 ground motions recorded in California, and non-linear time-history response computed.

A statistical analysis of the computed responses was used to develop a suite of bridge PSDMs. In this paper PSDMs that demonstrate commonality with optimal PSDMs derived for single-bent bridges are presented. The influence of multiple bridge spans on their seismic behavior, such as the effect of higher mode response, is also investigated. PSDMs help to illustrate the effect of variation of span length and span number on bridge performance and give designers a tool to optimize the layout of the bridge for the expected level of seismic performance.

Keywords: performance-based design, seismic hazard intensity measures, structural demand measures, probabilistic seismic demand model, multi-span bridges.

INTRODUCTION

The advances in performance-based earthquake engineering using such probabilistic frameworks as that developed by the Pacific Earthquake Engineering Research Center (PEER) have motivated more careful scrutiny of each framework component. System performance objectives are de-aggregated into several interim models involving measures of capacity, demand,

[1] Graduate Student, Dept. of Civil & Env. Eng., University of California, Berkeley, CA

[2] Assistant Professor, Dept. of Civil & Env. Eng., University of California, 721 Davis Hall #1710, Berkeley, CA 94720-1710. E-mail: boza@ce.berkeley.edu

and hazard. Structural performance in the demand model can be evaluated in terms of Engineering Demand Parameters (EDPs), as affected by ground motion hazard evaluated in terms of Intensity Measures (IMs).

These intermediate variables should be chosen such that probability conditioning is not carried over from one model to the next. Additionally, the uncertainties over the full range of model variables needs to be systematically addressed and propagated, making the selection of each interim model critical to the process. Much research in the structural and geotechnical areas has gone into the selection of the optimal IM-EDP pairs for given classes of structures, specifically California highway bridges in this case.

Findings from earlier work on this relationship for single-bent highway bridges is extended to PSDMs developed for multi-span bridges in this paper. Of particular interest is whether optimal PSDMs remain optimal for all bridge bent configurations considered because as bridge designs become more complex, it is less likely they can be expected to respond in single degree of freedom fashion. PSDMs give designers a tool to enable optimization of bridge layout for the expected level of seismic performance specific to their site. Not only can they be used to assess design parameter variation on response, but also to demonstrate whether higher mode response contributes significantly as irregularity of the bridge is increased.

MULTIPLE-SPAN BRIDGE MODELS

In this study, typical new California highway overpass bridges are selected as the class of structures for demand analysis. Configurations in this study are limited to 1-, 2-, and 3-bent, single column per bent, reinforced concrete bridges, with spans that are not necessarily equal. This selection of spans is intended to cover a large range of frames used in stand-alone analyses. These are typical of overpass bridges in longitudinal arrangement separated by expansion joints, where individual frames are expected to perform independently.

Common to all bridges are single-column bents anchored below grade by Type I integral pile foundations. Superstructures utilized are simple reinforced concrete box girders without pre- or post-tensioning. To allow for modeling of abutments or expansion joints at either end of the frames, the extreme span ends are given seat-type configurations. All designs follow specifications in the Bridge Design Specification and Seismic Design Criteria (Caltrans 1999).

For each configuration, bridge design can be varied through a series of design parameters. These include such quantities as skew, span length (L), span to column height ratio (L/H), steel and concrete nominal strength properties (f_y, f'_c), amount of longitudinal and transverse column reinforcement ($\rho_{s,long}$, $\rho_{s,trans}$), ratio of column diameter to superstructure depth (Dc/Ds), soil properties along the pile shafts (K_{soil}), additional bridge dead weight (Wt), and various abutment models. The single-bent base bridge configuration includes two 18.2 m (60 ft) spans, a single-column bent 7.6 m above grade (30 ft), with a 1.6 m (5.25 ft) diameter circular column, 2% longitudinal reinforcement, and 0.7% transverse reinforcement. The 2-bent base configuration includes 27.4-36.6-27.4 m (90-120-90 ft) spans, 7.6m (30 ft) above grade columns of diameter 1.7 m (5.6 ft), 2% longitudinal, and 0.8% transverse reinforcement. The 3-bent continues the same configuration as the 2-bent, adding another 36.6 m (120 ft) interior span. All base bridges are on a USGS class B (NEHRP C) soil site.

PROBABILISTIC SEISMIC DEMAND ANALYSIS

The Probabilistic Seismic Demand Model (PSDM) is the end product of Probabilistic Seismic Demand Analysis (PSDA), detailed elsewhere (Luco 2001). As utilized, PSDA involves five steps. First, a set of ground motions, representative of regional seismic hazard, is selected or synthesized. Intensity Measures (IM) descriptive of their content are computed. Second, the class of structures is defined as above. Associated with multiple span bridges are a suite of Engineering Demand Parameters (EDP) which can be used to assess structural performance under the considered motions. Third, a nonlinear finite element model is generated to model the bridges, with specific allowances for varying the design through design parameters. Fourth, dynamic analyses are performed until all motions and bridge model combinations have been exhausted. Finally, a demand model is formulated between resulting ground motion IMs and structural EDPs.

PSDA Ground Motions

This study explicitly involves the ground motion bin approach for formulating PSDMs. The bin approach (Shome 1999) is used to subdivide ground motions into arbitrary bins based on moment magnitude (M_w), closest distance (R), and local soil type. The bins in this study are delineated at a magnitude of 6.5 and a distance of 30 km. Four bins with 20 ground motions (Mackie 2001) each were obtained from the PEER Strong Motion Database (peer.berkeley.edu/smcat), and are characteristic of non-near-field motions (R >15 km) recorded in California. All ground motion records were scaled by a factor of two and sampling frequency reduced to 50 Hz.

The IMs associated with the above motions are limited to the spectral quantities, Arias Intensity, and peak ground velocity (PGV) only. First mode spectral displacement (Sd_{T1}), acceleration (Sa_{T1}), and velocity (Sv_{T1}) are used interchangeably, as the dispersions in the PSDMs are independent of the choice of spectral quantity.

PSDA Model

Bridge columns and pile shafts are modeled using three-dimensional flexibility-based beam-column elements with fiber cross-sections in the PEER OpenSees (www.opensees.org) platform. This element is limited to axial-flexural interaction, therefore columns develop plastic hinges in flexure rather than experiencing shear failure. P-Δ effects were included for the column, but no other methods of softening were incorporated into the model. The circular column cross-sections have perimeter longitudinal reinforcing bars and spiral confinement. Steel models include an elastic-plastic oscillator with Bauschinger effect, hysteretic damping, and 1.5% strain hardening for the steel. Confined and unconfined reinforced concrete use the Kent-Scott-Park constitutive relations.

Soil-pile interaction was modeled using bilinear *p-y* springs at varying depths over the length of the pile shafts. The *p-y* spring properties were determined using soil parameters corresponding to assumed soil properties. The bridge deck was designed as a typical Caltrans reinforced concrete box girder section for a three-lane roadway. During the nonlinear analyses, the deck was assumed to remain elastic, therefore input into the model using elastic elements and cracked stiffness'. Abutments are modeled as simple elastic-plastic spring-gap elements, with allowance for longitudinal, transverse, and vertical stiffness and mass. In order to maximize the column demand, the stiffness' are set to zero, creating a roller condition at the abutments.

The bridge EDPs were chosen from the PEER database of experimental results for concrete bridge components (Hose 2000). The database details specific discrete limit states for each of the EDPs considered. By mirroring the component database, it is possible to directly evaluate damage in a bridge, given the analysis demands. The EDPs include global (drift), intermediate (moment), and local quantities (stresses). EDPs in this study are limited to the column drift ratio (Δ), maximum column moment (M_{max}) and maximum displacement (u_{max}).

PSDA Analysis

For each bridge design parameter variation, nonlinear models were generated and analyzed in OpenSees. Each analysis involves a static pushover analysis to determined yield values, a modal analysis to determine natural frequencies, and also a dynamic time-history analysis to determine demand. While single bent bridges use standard static pushover techniques for both longitudinal and transverse directions, the introduction of multiple bents requires a modification of the procedure in the transverse direction. Similar to the modal pushover procedure for buildings (Chopra 2001), the distribution of lateral forces is determined from the shape of the fundamental transverse mode, weighted by tributary mass. Displacements at column tips are then monitored along with shear forces induced in the columns, like base shear in buildings.

Modal analysis of each bridge configuration yields natural frequency and mode shape information. For the assumed roller boundary condition at the abutments, the fundamental mode for all three bent types is in the transverse direction. This mode involves a simple transverse translation of the deck (Fig. 1). The second mode involves a longitudinal translation of the superstructure, coupled with small rotations of the columns and supports (Fig. 2). When abutment models are added, transverse stiffness becomes dominant due to the gap in the longitudinal direction before abutment impact. Hence the fundamental mode of the bridge becomes longitudinal, and the second mode transverse.

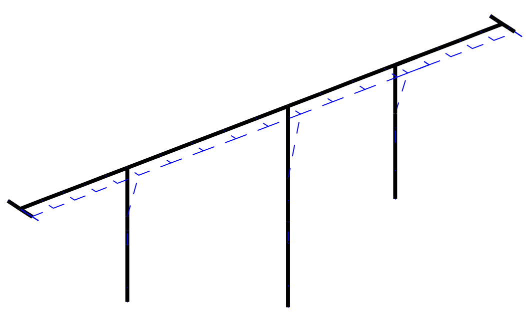

FIG. 1. Three bent bridge, 1st mode (transverse)

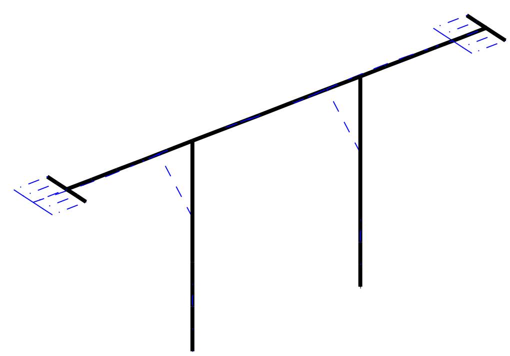

FIG. 2. Two bent bridge, 2nd mode (longitudinal)

With the availability of a fully three-dimensional model and transverse mode information, a new regularity-like index (RI^*) was developed to indicate the expected introduction of higher mode response. While the index proposed by Isakovic and Fischinger (Isakovic 2001) depends on the analysis method, RI^* is a function of the model only.

$$RI^* = \left(\frac{1}{L} \int_0^L \bar{\phi}(x) dx \right) * 100\% \approx \frac{\sum_{i=1}^{n} \frac{dx_i}{2} \left(\bar{\phi}_i + \bar{\phi}_{i-1} \right)}{\sum_{i=1}^{n} dx_i} * 100\% \tag{1}$$

Nonlinear time-history analyses were performed on all bridge configurations, using earthquakes in all bins. The resulting distribution of bridge demand values allows assessment of aleatory variability attributed to the randomness in the ground motion source parameters.

PROBABILISTIC SEISMIC DEMAND MODELS

The final step in PSDA is to combine all the analyses into PSDMs, which relate ground motion specific IMs to bridge-specific EDPs. It was determined (Mackie 2002a) that the optimal PSDMs for single bent bridges were Sa-Δ and Sa-M_{max}. Optimal is defined as practical, sufficient, effective, and efficient. This analysis is extended to multiple-bent bridges below. Specifically, the PSDMs are evaluated in terms of efficiency and sufficiency.

It is assumed the EDPs follow a log-normal distribution (Shome 1999). Efficiency describes the scatter about the linear fit in log-log space. The measure used to evaluate efficiency is dispersion (σ), defined as the standard deviation of the logarithm of the demand model residuals. Sufficiency is required to determine whether the total probability theorem can be used to de-aggregate demand and hazard in coupled formulations such as that used by PEER (Cornell 2000). This cannot be done if there are any residual dependencies on M_w and R.

In each of the following models, the data is plotted on a log-log scale, with the EDP on the abscissa and the IM (dependent variable) on the ordinate. The dispersion values for each regression analysis are listed in each corresponding figure window. Each demand model is constructed in the longitudinal and the transverse direction independently.

Comparison to Single Bent

To assess the applicability of the optimal PSDMs from single-bent bridge analyses, the data for all bent types are presented on the same plot. Fig. 3 shows the PSDM formed between Sa_{T1} and Δ. Of interest is the decrease in dispersion between using Sa_{T1} over Sa_{T2}, even though the fundamental mode of the 2- and 3-bent bridges is in the transverse direction. Note, however, that this does not mean there is a relationship between transverse spectral quantities and longitudinal deformations. Instead, this suggests the spectral quantity producing optimal dispersions may not be a simple function of bridge modes, but a combination of them. This approach is investigated in part using the single-bent bridge elsewhere (Mackie 2002b). The PSDM between Sa_{T1} and maximum column moment (M_{max}) is shown in Fig. 4.

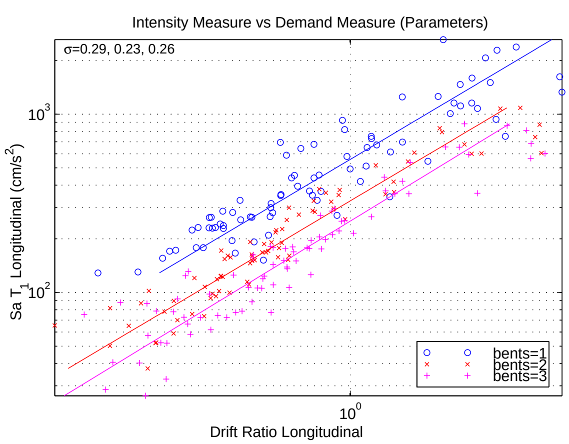

FIG. 3. Multiple bent PSDM, Sa_{T1}-Δ longitudinal

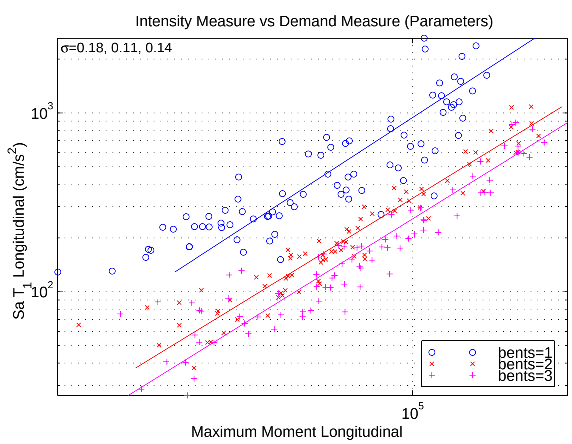

FIG. 4. Multiple bent PSDM, Sa_{T1}-M_{max} longitudinal

In order to better understand the trends indicated in a stiffness or mass varying PSDM, it is helpful to use a non-period dependent IM. The PSDM formed using Arias Intensity and Δ is shown in Fig. 5. While the dispersions increase, this PSDM allows the use of lines of constant intensity to observe an increase in demand as the number of bents is decreased.

Transverse Irregularity

All of the above PSDMs can be extended to the transverse direction as well. This is especially critical when considering bridges with irregular transverse response. Fig. 6 shows the Sa_{T1}-Δ PSDM for the case of 1-, 2-, and 3-bents all with a *RI** of 100% (regular). The trends and dispersions are similar to that in the longitudinal direction. Furthering the assessment of which period to use in the spectral computation, T_1 improves dispersion values over the use of T_2.

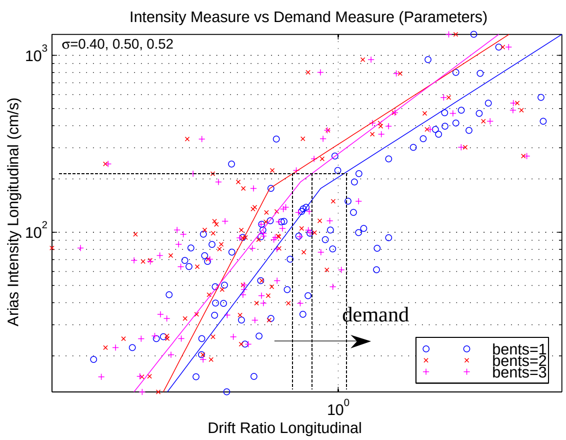

FIG. 5. Multiple bent PSDM, Arias-Δ longitudinal

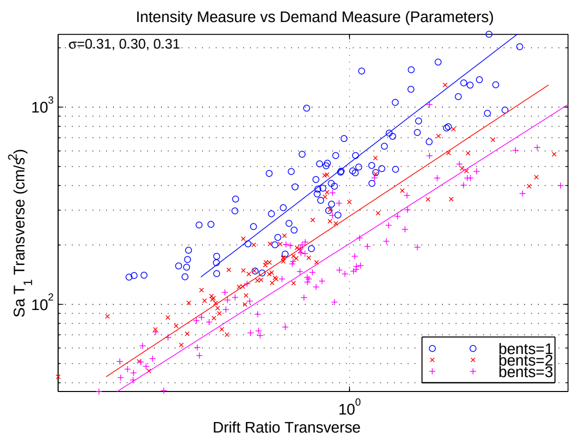

FIG. 6. Multiple bent PSDM, Sa_{T1}-Δ transverse

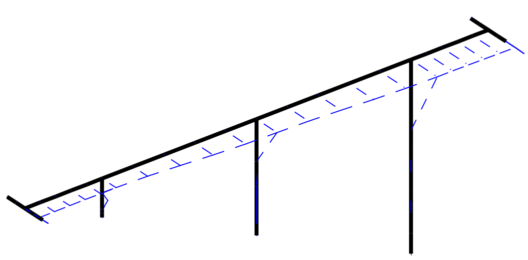

FIG. 7. Three bent irregular bridge (transverse)

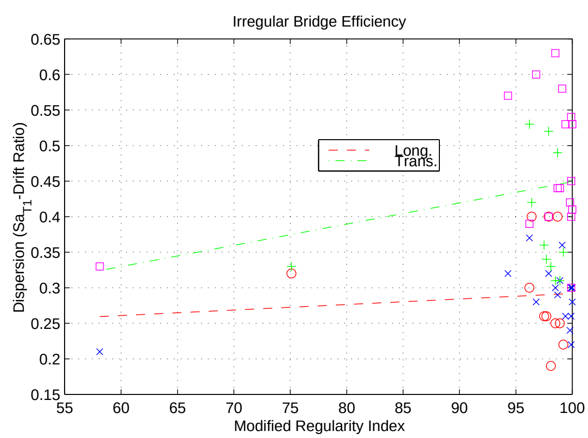

FIG. 8. Effect of irregularity on PSDM efficiency

Given a wide variation in the span length and column stiffness' in each bent, it was still not possible to generate a fundamental transverse mode with highly irregular behavior. The modified regularity index falls in the ranges of 55-100% for all configurations investigated. As it is possible to generate PSDMs with arbitrary IM-EDP pairs, it is constructive to investigate whether higher mode participation in highly irregular bridges considerably influences the efficiency of PSDMs using spectral quantities for the IM. A sample irregular bridge with transverse mode indicated is shown in Fig. 7. From a range of bridge configurations, the relationship between *RI** and dispersion for Sa_{T1}-Δ was developed. From Fig. 8 it is apparent that Sa_{T1} captures longitudinal and transverse response, regardless of *RI**.

Parameter Sensitivity

Designers can use PSDMs to assess the affect of changing certain design parameters on the response of the bridge. This is accomplished by varying bridge design parameters, in this case (Fig. 9) the 3-bent bridge is chosen and the middle span lengths ($L2$) are being varied. From the line of constant intensity shown, increasing $L2$ increases the bridge demand as expected. The EDP is shifted to maximum longitudinal displacement (u_{max}) rather than Δ as varying $L2$ also affects the column heights (L/H ratios remain constant). Note that maximum displacement and drift generate PSDMs with the same dispersion, therefore can be used interchangeably. Finally, this PSDM can be investigated to ensure there is no residual dependence on M_w and R, as assumed. R dependence is depicted in Fig. 10. As desired, the slope of the lines is approximately horizontal, indicating lack of R dependence. The same trend is true of M_w but not shown here.

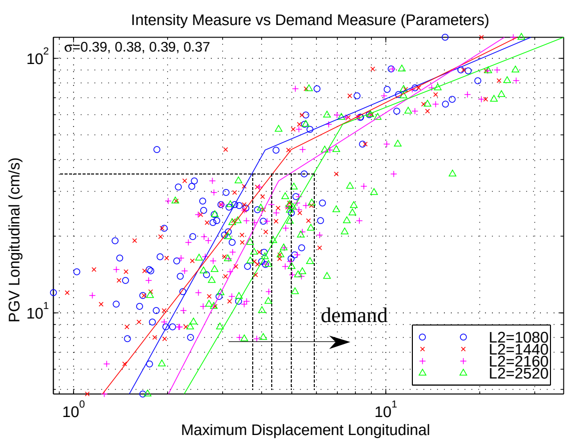

FIG. 9. L2 sensitivity, PGV-u_{max} longitudinal

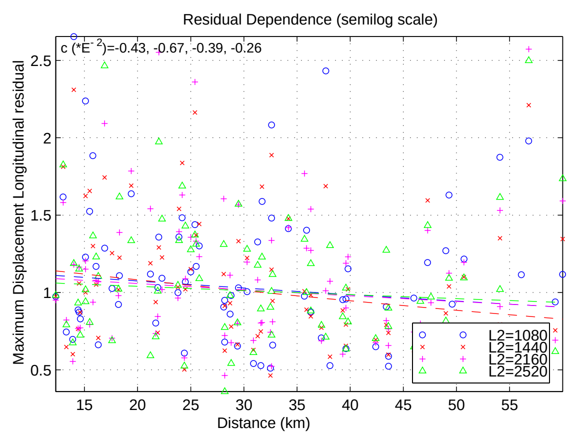

FIG. 10. R residual dependence, PGV-u_{max} longitudinal

CONCLUSION

PSDMs are a powerful tool, both as interim models in an overall probabilistic framework, and as individual tools for assessing effects of design changes on performance. When applied to multiple span highway bridges, the criteria required for de-aggregation are maintained in the resulting PSDMs, thus allowing seamless integration into the PEER framework. More importantly, the same IM-EDP pairs derived as optimal for single-bent bridges are also optimal for multiple-bent. The wide use of spectral acceleration in current practice makes it a useful property for bridges.

As tools for designers, the PSDMs for multiple span bridges are especially useful as they are not subject to deterioration of effectiveness or sufficiency due to bridge irregularity. Optimality of IM-EDP pairs is maintained across a broad range of irregular configurations. This standardization allows designers to evaluate bridge response without an initial requirement that a bridge design qualify as an "ordinary standard bridge" (Caltrans 1999).

ACKNOWLEDGMENTS

This work is supported by the Earthquake Engineering Research Center's Program of the National Science Foundation under Award EEC-9701568 as PEER Project #312-2000 "Seismic Demands for Performance-Based Seismic Design of Bridges." The opinions presented in this paper are solely those of the authors and do not necessarily represent the views of the sponsors.

REFERENCES

Caltrans (July 1999). Seismic Design Criteria 1.1. California Department of Transportation.

Chopra, A. K. and R. K. Goel (2001). "A Modal Pushover Analysis Procedure to Estimate Seismic Demands for Buildings," Pacific Earthquake Engineering Research Center Report No. 2001/03, University of California, Berkeley.

Cornell, C.A. and H. Krawinkler (2000). "Progress and Challenges in Seismic Performance Assessment," PEER Center News Spring 2000, **3**(2).

Hose, Y., P. Silva, and F. Seible (2000). "Development of a performance evaluation database for concrete bridge components and systems under simulated seismic loads," Earthquake Spectra, **16**(2), 413-442.

Isakovic, T. and M. Fischinger (2001). "Analysis and design of RC viaducts in moderate seismicity regions of Europe," Proceedings of the 4th International Symposium on Earthquake Engineering for the Moderate Seismicity Regions, Seoul, Korea, 2001.

Luco, N., Cornell, C.A., and G.L. Yeo (2001). "Annual limit-state frequencies for partially-inspected earthquake-damaged buildings," Proceedings of the 8th International Conference on Structural Safety and Reliability. International Association for Structural Safety and Reliability (IASSAR), Newport Beach, California, June 2001, in press.

Mackie, K. and B. Stojadinovic (Nov/Dec 2001). "Probabilistic Seismic Demand Model for California Highway Bridges," Journal of Bridge Engrg., **6**(6), 468-481.

Mackie, K. and B. Stojadinovic (2002a). "Optimal Probabilistic Seismic Demand Model for Typical Highway Overpass Bridges," Proceedings of the 12th European Conference on Earthquake Engineering, London, England, September 2002, in press.

Mackie, K. and B. Stojadinovic (2002b). "Design parameter sensitivity in Probabilistic Seismic Demand Models," Proceedings of the 7th US National Conference on Earthquake Engineering, Boston, Massachusetts, July 2002, in press.

Shome, N. and C.A. Cornell (1999). "Probabilistic seismic demand analysis of nonlinear structures," Reliability of Marine Structures Report No. RMS-35, Dept. of Civ. and Envir. Engrg., Stanford University.

APPENDIX I. NOTATION

The following symbols were used in this paper.

L = span length
M_{max} = maximum column moment
M_w = moment magnitude
R = closest distance
RI^* = modified regularity index
Sa_{T1} = 1st mode spectral acceleration
T_1 = 1st mode bridge period
T_2 = 2nd mode bridge period
Δ = drift ratio
$\bar{\phi}(x)$ = mode shape normalized to a maximum value of 1.0
$\bar{\phi}_i$ = value of $\bar{\phi}(x_i)$