Title: **Bridge Abutment Model Sensitivity for Probabilistic Seismic Demand Evaluation**

Authors: Bozidar Stojadinovic (Contact person)
Assistant Professor
Dept. of Civil & Env. Engineering
University of California
721 Davis Hall #1710
Berkeley, CA 94720-1710
Phone: (510)
Fax: (510)
E-Mail: boza@ce.berkeley.edu

Kevin Mackie (Presenter)
Graduate Student Researcher
Dept. of Civil & Env. Engineering
University of California
721 Davis Hall #1710
Berkeley, CA 94720-1710
E-Mail: mackie@ce.berkeley.edu

Bridge Abutment Model Sensitivity for Probabilistic Seismic Demand Evaluation

Kevin Mackie[1] and Bozidar Stojadinovic[2]

INTRODUCTION

Modeling the contribution of bridge abutments to overall bridge seismic response has been the focus of a significant research effort in the past decade. Many recent studies have shown how that abutment response significantly influences the response of short-and medium-length bridges. Some of these studies are based on sensitivity analyses using deterministic bridge models with varying abutment characteristics and capacities, and a relatively small number of earthquake ground motions.

In this paper, a comparison of Probabilistic Seismic Demand Models (PSDMs) obtained using Probabilistic Seismic Demand Analysis (PSDA) is used to conduct a sensitivity study on how bridge abutment models affect seismic demand for short- and medium-length bridges. The advantages of this method are in a large number of earthquake ground motions used in the sensitivity analysis, and in a probabilistic interpretation and comparison of bridge demand. Abutment capacity is not considered at this point because of a shortage of reliable abutment capacity data. PSDMs are a part of a performance-based seismic design framework developed by the Pacific Earthquake Engineering Research (PEER) Center. This design framework is based on the principle of de-aggregation of uncertainties that define probabilistic performance-based seismic design. Given a class of structures, such as typical highway overpass bridges, PSDMs define the relation between seismic hazard and structural demand. They are typically used to compute the probability of exceeding an Engineering Demand Parameter (EDP), such as drift, given the value of a seismic hazard Intensity Measure (IM), such as Arias intensity. Comparison of PSDMs for bridges with systematically varied abutment models gives a probabilistic characterization of the effect of abutments and their importance in the overall seismic demand for a bridge.

PROBABILISTIC SEISMIC DEMAND MODEL

The PSDM is the outcome from a Probabilistic Seismic Demand Analysis (PSDA) [1]. The procedure used to formulate the PSDMs of interest involves five steps. First, a set of ground motions is selected and categorized according to Intensity Measures (IMs) descriptive of their content and intensity. Secondly, the class of structures to be investigated is defined, along with a suite of Engineering Demand Parameters (EDPs) that can be measured during analysis to assess structural performance. Thirdly, a finite element analysis model is generated to model the class of structures selected, with specific allowances for varying the abutments through parameters. Fourthly, nonlinear dynamic analyses are performed until all motions and abutment model combinations have been exhausted. Finally, a demand model is formulated between resulting IMs and structural EDPs. The PSDA process, IM and EDP definitions, and application to various PSDMs are detailed elsewhere [2].

---

[1]Kevin Mackie, Graduate Student, Dept. of Civil & Env. Engineering, University of California, 721 Davis Hall #1710, Berkeley, CA 94720-1710
[2]Bozidar Stojadinovic, Assistant Professor, Dept. of Civil & Env. Engineering, University of California, 721 Davis Hall #1710, Berkeley, CA 94720-1710

In this study, the IMs were limited to Peak Ground Velocity (PGV), Cumulative Absolute Displacement (CAD), and Arias Intensity only. The bridge EDPs used are strictly displacement based, including global drift ratio (Δ) and column section displacement ductility (μ_{Δ}). Nonlinear models were generated for each bridge configuration and analyzed using the PEER OpenSees (www.opensees.org) platform. The EDPs are assumed to follow a log-normal distribution [3]. As a result, each of the following PSDMs is plotted in log-log space, with the EDP on the abscissa and the IM on the ordinate. Coefficients of a linear, or piecewise-linear regression then yield linear equations. The dispersion values for each regression analysis are listed in each corresponding figure. Each demand model is constructed in the longitudinal and the transverse direction independently.

Class of Structures and Analytical Model

Typical new California highway overpass bridges are selected as the class of structures. The bridges presented in this paper are designed according to Caltrans specifications [4] for reinforced concrete bridges. Configurations in this study are limited to two-equal-span overpasses with seat-type abutments on either end. Common to all bridges is a single column bent continuing below grade into a Type I integral pile foundation.

The base bridge configuration includes two 60 ft spans, a single-column bent 30 ft above grade, with a 5.25 ft diameter circular column, 2% longitudinal perimeter bar reinforcement, and 0.7% transverse spiral reinforcement. The base bridge is on a NEHRP C soil site. Bridge columns and pile shafts are modeled using three-dimensional flexibility-based beam-column elements. This element is limited to axial-flexural interaction, hence no shear failure is modeled. P-Δ effects were included for the column, but no other methods of softening were incorporated into the model. Soil-structure interaction was modeled using bilinear *p-y* springs placed at varying depths along the length of the pile shafts. The bridge deck was designed as a typical reinforced concrete box girder section for a three-lane roadway. The deck was assumed to remain elastic, therefore input into the model using elastic elements with cracked stiffness properties. Each abutment model comprises a five-element array of gap-spring elements in the transverse direction, two gap-spring elements in the longitudinal direction, and five gap-spring elements in the vertical direction. Different bridge instantiations were generated by varying parameters as described below.

Abutment Models

Bounds for abutment stiffness and mass variation are based on existing abutment methods. First group of methods is based on the Caltrans procedure for determining longitudinal and transverse stiffness' and strengths of an abutment [4,5]. Longitudinal direction values are formulated by combining passive backwall pressure, shear strength of the wall itself, and strength and stiffness of the pile groups supporting the abutment. These stiffness and strength values have been validated in large-scale abutment tests [6]. Transverse direction values are computed assuming a seat-type abutment with resistance from the wing walls, shear keys, and pile group. Obtained transverse stiffness and strength values are smaller than those obtained using Caltrans procedure. An idealized trilinear backbone force-displacement response curve was used to model the abutments, with varied stiffness and strength values shown in Table 1. These values were used as input parameters for the gap-spring elements in the OpenSees bridge model, described above.

A second group of methods involves deriving the response of the soil embankments in the transverse direction. Assuming a symmetric embankment with defined cross-section geometry, soil shear moduli and unit weight, Wilson and Tan determine the stiffness per unit length of the embankment [7]. The total stiffness is then derived from an estimated wing wall length. This length is purposely underestimated to make the transverse stiffness conservatively low. Wilson and Tan assume that abutment longitudinal stiffness is the same as transverse and calculate the vertical stiffness using the same cross-section approach. Using a similar procedure, Zhang and Makris generalize to any embankment geometry and calculate transverse and vertical stiffness and damping values for dynamic abutment response calculations [8]. Only the stiffness value is used herein. As in Wilson, longitudinal and transverse stiffness' are assumed the same.

TABLE 1. ABUTMENT STIFFNESS AND MASS PARTICIPATION VALUES

Proposed by	**Longitudinal Ka_l (k/in)**	**Transverse Ka_t (k/in)**	**Vertical Ka_v (k/in)**	**Participating mass (k s^2/in)**
Caltrans [4, 5]	2215	627	NA	7.4
Maroney [6]	1080, 168	487	NA	5.7 [9]
Wilson & Tan [7]	587	587	1643	7.4
Zhang & Makris [8]	1006	1006	2817	12.6

The other fundamental factor governing abutment response is the inertial force it generates during an earthquake. This force is included in the analysis using a concentrated mass at the abutments. Coupled with the stiffness quantities defined above, the abutment becomes a single degree of freedom oscillator attached to each end of the bridge. Determining the mass participating in abutment response is highly uncertain and is usually approximated by a critical length of the embankment. Different researchers have proposed participating lengths that best match recorded data [8, 9] as shown in Table 1. As suggested Wissawapaisal and Aschheim [10], this critical length may vary with earthquake intensity. Hysteretic damping is not included in any of the abutment models, but 2% Rayleigh system damping is.

ABUTMENT MODEL SENSITIVITY

Sensitivity studies were performed by varying, in turn, longitudinal stiffness, transverse stiffness, and participating mass of both abutments. Stiffness values range from 0 (no abutment case, only rollers) to 1000 k/in. Mass values range from 0 to 8 ks^2/in. To differentiate between the effect of increasing abutment stiffness and mass, longitudinal stiffness' are varied for the cases of no mass and a median mass. The resulting PSDMs shown below are not necessarily the optimal PSDMs for the given IM-EDP combinations. Optimal models utilize first mode spectral acceleration for the IM. Period independent IMs have been used in order to isolate the affect of abutment stiffness change on the response.

Longitudinal Stiffness

Longitudinal response in the presence of varying stiffness is difficult to evaluate in the case of bridges with seat-type abutments. Abutment stiffness is only activated once sufficient column deformations have caused the gap to close. Several studies were therefore performed. To evaluate stiffness only, response with the case of 0 abutment mass was performed first. For cases of large gaps (6"), response is identical in all except the high intensity region. In this study there is insufficient data in this range to assess sensitivity. Therefore, the gap was reduced to 2" to better assess stiffness sensitivity. Fig. 2 shows the response at varying stiffness levels. After gap closure, stiffer abutments reduce response. Even the lower stiffness bound provides improved response over the no abutment case. Finally, a median value of mass was added to the abutments and the 6" gap study repeated. The added inertia at the abutments is sufficient to cause significant gap closure. However, the mass appears to dominate the response as there is no appreciable difference between stiffness levels. At very high intensities, the 1000 k/in stiffness median response begins to decrease.

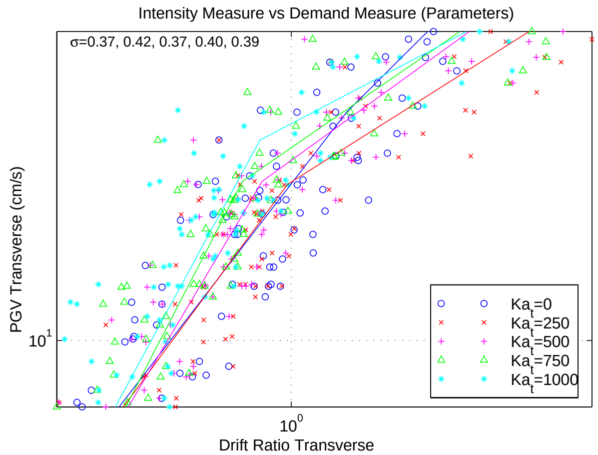

Figure 1. Ka_t sensitivity, PGV-Δ PSDM.

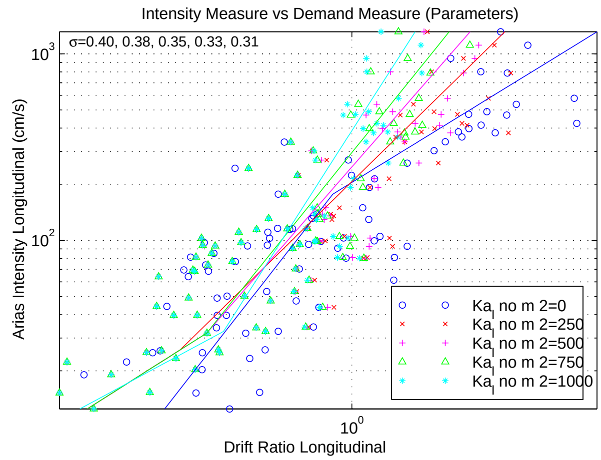

Figure 2. Ka_l (2") sensitivity, Arias-Δ PSDM.

Transverse Stiffness

The effects of increasing abutment stiffness are more readily investigated in the transverse direction as there is no gap before mobilizing the total abutment stiffness. The increase in transverse stiffness in the presence of embankment inertia has little effect on the response (Fig. 1). As expected, stiffer abutments reduce median response at all intensity levels. Excluding abutments from the model yields similar results as a low stiffness abutment in the presence of inertial forces, especially for smaller intensities. Making this assumption, however, is highly non-conservative in the high intensity region.

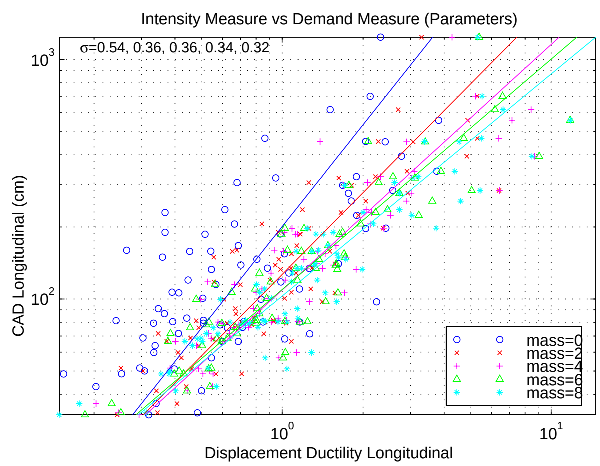

Figure 3. Mass sensitivity, CAD-μ_Δ PSDM.

Participating Mass

As indicated by the stiffness sensitivities in the presence of mass, the participating mass is more critical to bridge response. Fig. 3 shows the increasing contribution of participating mass to the total response at higher intensities. This verifies the observations in Aschheim [10], as reduced mass in required to maintain a response level as intensity is increased. Similarly, at constant intensities, the response increases with more participating mass.

CONCLUSION

As demonstrated, PSDMs can be used as a tool to assess the sensitivity of highway bridge overpass response to abutment parameters. Specifically, the participating length of the embankment, and hence the mass associated with the abutment, is the most critical parameter. Neglecting to include mass in the analysis under-predicts the response, even if large longitudinal or transverse stiffness' are applied. In the absence of inertia forces also, global response is insensitive to the selection of longitudinal stiffness. Therefore, any of the methods discussed are sufficient for approximating the longitudinal stiffness. Transverse stiffness values have a larger affect in the absence of inertia, however, given the predominance of mass, transverse stiffness sensitivity is also reduced to the point where calculated values are sufficient. Further studies are needed to investigate the dependence of the participating embankment length on bridge length, intensity and other factors.

As a simplified model, it is possible to conservatively analyze a given bridge with only rollers at the abutments. This assumption is valid only at lower intensities due to the trade-off introducing stiffness and mass to the abutment incurs. However, introduction of more complex abutment models do not necessarily improve the accuracy of the solution when improperly calibrated.

ACKNOWLEDGEMENT

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

[1] Luco, N., C.A. Cornell, and G.L. Yeo. 2001. "Annual limit-state frequencies for partially-inspected earthquake-damaged buildings," presented at 8th International Conference on Structural Safety and Reliability, Denmark.

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

[3] 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 Univ.

[4] Caltrans. July 1999. *Seismic Design Criteria 1.1*. California Department of Transportation.

[5] Goel, R.K., and A. Chopra. 1997. "Evaluation of bridge abutment capacity and stiffness during earthquakes." *Earthquake Spectra*, 13(1): 1-23.

[6] Maroney, B., B. Kutter, K. Romstad, Y.H. Chai, and E. Vanderbilt. 1994. "Interpretation of Large Scale Bridge Abutment Test Results," presented at 3rd Annu. Seismic Research Workshop, Sacramento, CA.

[7] Wilson, J.C., and B.S. Tan. August 1990. "Bridge Abutments: Formulation of Simple Model for Earthquake Response Analysis," *J. Engrg. Mechanics*, 116(8): 1828-1837.

[8] Zhang, J. and N. Makris. March 2001. "Seismic Response Analysis of Highay Overcrossings Including Soil-Structure Interaction," PEER Report 2001/02, College of Engrg., University of California Berkeley.

[9] Werner, S.D. 1994. "Study of Caltrans' seismic evaluation procedure for short bridges," presented at 3rd Annu. Seismic Research Workshop, Sacramento, CA.

[10] Wissawapaisal, C., and M. Aschheim. 2000. "Modeling the transverse response of short bridges subjected to earthquakes." Mid-America Earthquake Center CD release 00-05, Univ. of Illinois, Urbana-Champaign.