EPISTEMIC UNCERTAINTY OF SEISMIC RESPONSE ESTIMATES FOR REINFORCED CONCRETE BRIDGES

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ABSTRACT :

Considerable effort has recently been directed toward developing analytical probabilistic seismic response estimates at different earthquake intensities. However, probability distributions of response quantities are sensitive to not only the randomness in the ground motion input, but also the assumptions made in the nonlinear finite element models used in the analyses. There are epistemic uncertainties inherent in selection of model parameters as well as the capabilities of individual analysis platforms themselves. This paper develops quantitative estimates of the epistemic uncertainty, above and beyond the aleatory variability, of typical reinforced concrete overpass bridge structures in California. A series of nonlinear time history analyses of 6 typical existing reinforced concrete bridge structures in California using 60 ground motions is carried out using SAP2000 and OpenSees finite element programs. The response of the bridges in terms of peak displacements at the column top is related to the intensity measure defined as the spectral displacement at the first mode period. A procedure using natural logarithmic regression is applied to the data to estimate the seismic response of the structure and the uncertainty in such estimation. Two methods of obtaining response estimate bias factors between the two programs are illustrated for the 6 bridges under three typical seismic hazard levels (2%-, 5%-, and 10%-in-50 year probabilities of exceedance).

KEYWORDS: epistemic uncertainty, reinforced concrete bridges, seismic response

1. INTRODUCTION

Uncertainty associated with probabilistic estimates of seismic response of structures is either aleatory (due to randomness) or epistemic (due to lack of knowledge). Several sources of aleatory variability in seismic response assessment of buildings and bridge structures have been identified (Miranda and Aslani, 2003). Among them are: response variations corresponding to ground motion intensity measure at a site, seismic hazard characteristics, structural response parameters due to different seismic loadings applied to the structure, and damage experienced by structural and nonstructural components of the system. Another source of variability in seismic response estimates is the epistemic uncertainty of the structural model. Epistemic uncertainty is developed due to limited sample size of simulations or data, uncertainty in the modeling assumptions and parameters used in the analysis procedures, as well as the variability related to the interpretation of data and results.

The epistemic uncertainty related to analytical modeling and analyses conducted to estimate the seismic response of 6 typical reinforced concrete bridge structures is investigated in the present study. Modal, pushover and nonlinear time history analysis results obtained from two different structural analysis programs are compared. The remaining sources of variability in the structural response assessment are reduced by the use of identical load patterns, a large set of ground motions and similar numerical solution methods and parameters in both programs. The effects of the epistemic uncertainty related to modeling are investigated and computed as bias factors for the peak displacements obtained from dynamic non-linear analysis of these bridge structures.

2. METHODOLOGY

A total of 6 existing reinforced concrete bridge structures located in California, with different geometries and column cross sections, are modeled and analyzed using SAP2000 (CSI, 2005) and OpenSees (McKenna et. al, 2000) structural analysis programs. The selected bridge structures represent Ordinary bridges in California with box-girder superstructure, typical column bent details, and simple geometric regularity. The principal characteristics of the bridges are presented in Table 1.

Table 1 Charateristic of the bridge used in this study

Bridge	Type	No. Spans	Length (ft)	Width (ft)	No. Col's	Column Diameter (ft)	Column Height (ft)	Superstructure Depth (ft)	Cap Beam Dimensions (ft)
Route 14	Multi-Col.	2	286	53.7	2	5.42	37.9	5.74	7.55x5.74
La Veta	Multi-Col.	2	299	75.5	2	5.58	25.4	6.23	7.55x6.23
Adobe	Multi-Col.	2	203	41.0	2	4.00	26.6	4.10	7.00x4.10
LADWP	Multi-Col.	3	262	41.6	4	4.49	25.6	4.27	6.56x4.27
MGR	Single-Col.	3	366	42.3	2	6.00	39.1	6.23	-
W180-N168	Single-Col.	4	674	41.2	3	6.00	26.4	7.74	-

The bridge superstructures and cap beams are modeled (in both programs) as elastic beam-column elements divided into 5 discrete segments per span with translational and rotational tributary mass lumped at each node. The cap beam is assigned a rigid torsional stiffness due to the monolithic construction of the superstructure and cap beam into a single element. Expected material strength properties are used for all steel and concrete elements and fibers, rather than nominal properties. Elastic shear deformation is included for all beam and column elements. The P-Delta transformation is used for static and dynamic analysis in both programs.

The plastic-hinge zone of the column bents is modeled in SAP2000 as a lumped-plasticity fiber hinge model, while the column outside the plastic-hinge zone is modeled as an elastic beam-column element with effective cross-sectional properties. The OpenSees model of the column bent consists of a distributed-plasticity fiber model with nonlinear force formulation and 5 integration points. The concrete constitutive model used in OpenSees is *Concrete02* that has Kent-Scott-Park behavior and includes tensile strength. The steel fibers utilize *Steel02* that has Menegotto-Pinto behavior with ultimate strains specified according to the Seismic Design Criteria (Caltrans 2004). The discretization of the cross section into fibers is carried out similarly in both programs according to Berry and Eberhard (2003).

The column foundations are modeled as fixed and pinned boundary conditions, for single and multi-column bent bridges, respectively. In the case of the longer MGR and W180-N168 bridges (exceeding 300 ft), the superstructure ends are assigned a roller support, since nonlinear abutment behavior does not control the response of the structures. A more elaborate abutment model defined as the simplified model (Aviram et. al, 2008) is used for the remaining 4 shorter bridges. The simplified model accounts for gap closure in the longitudinal direction, vertical stiffness of the elastomeric bearing pads, and soil embankment elastic-perfectly-plastic resistance in the longitudinal and transverse directions. The comparison between SAP2000 and OpenSees of the 6 bridge models is carried out using modal, nonlinear pushover and nonlinear time history analysis results.

3. DISCUSSION OF RESULTS

3.1. Modal Analysis

The elastic periods of the 6 bridges analyzed, obtained through eigenvalue analysis in OpenSees and SAP2000 assuming effective cross-sectional properties, match within 5% for the primary mode shapes of the structures. Therefore, the dynamic properties of the bridge models coincide for the elastic range of response. The nonlinear model in OpenSees with an initial uncracked column cross section yields another set of modal periods for the elastic range of response. Table 2 presents the results for La Veta bridge.

Table 2 Modal periods of La Veta bridge obtained from SAP2000 and OpenSees

Mode n	**$T_{n,SAP2000-Elastic}$ (sec)**	**$T_{n,OpenSees-Elastic}$ (sec)**	**$\Delta T_{OpenSees-SAP}$ (%)**	**$T_{n,OpenSees-Nonlinear, Pre-EQ}$ (sec)**
1- Longitudinal translation	1.183	1.176	0.6	1.091
2- Transverse translation	0.643	0.641	0.3	0.645
3- Vertical superstructure deformation (S-shape)	0.561	0.558	0.5	0.542
4- Global torsion	0.447	0.465	3.9	0.464
5- Vertical superstructure deformation (W-shape)	0.429	0.426	0.7	0.425

3.2. Pushover Analysis

Pushover analysis is carried out for all 6 bridges in SAP2000 and OpenSees. Figure 1 presents the pushover curves for La Veta bridge computed in the longitudinal (along the deck) and transverse directions of the structure. The initial stiffness of the bridges computed using the two programs matches within 5% for all bridges, while the computed ultimate base shear capacity differed by only 15% in most cases. As observed, degradation of strength is not captured in the ductile SAP2000 fiber hinge model, where the ultimate capacity of the fibers is extrapolated beyond the failure point. An estimate of ductility capacity of the bridge column bents is carried out separately according to empirical formulas of Caltrans SDC 2004 (Caltrans, 2004).

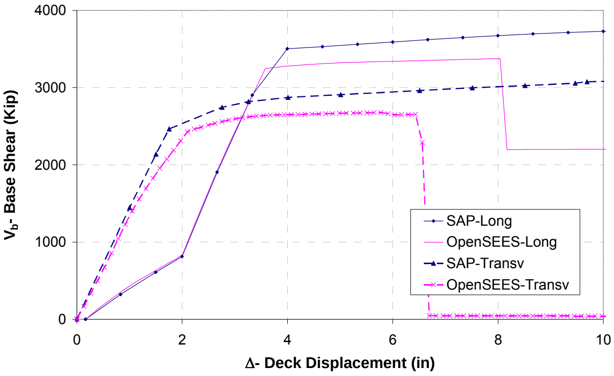
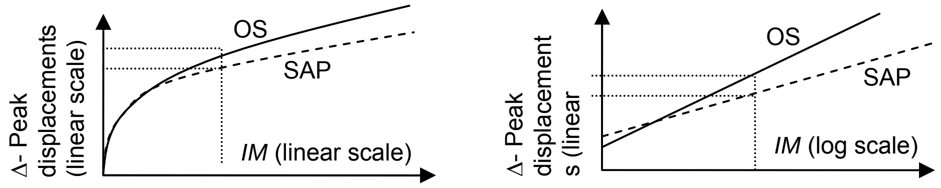


Figure 1 Pushover curves for La Veta bridge obtained using SAP2000 and OpenSees models

3.3. Time History Analysis Results

A set of 60 ground motions (GMs) with 3 components each from the I880n, I880p, and Van Nuys sets (Sommerville and Collins, 2002) is used to conduct nonlinear time history seismic response analyses. A uniform scale factor of 2.0 is used for all motions to induce significant nonlinear behavior in the bridge columns. The self-weight of the bridge is included during the time history analysis. The responses of the bridge in the longitudinal and transverse directions are used to estimate the diagonal response of the bridge at an angle with respect to its orthogonal axes by the SRSS combination rule at each time step. The transient analysis is performed using direct integration with Newmark's average acceleration time integration method and assuming 5% Rayleigh damping.

Analysis of the dynamic results is carried out for all 6 bridges relating peak displacements of a monitored point of the bridge to an intensity measure (IM) for each record, defined as the spectral displacement at the first mode period of the bridge ($S_{d,T1}$) (Mackie and Stojadinović, 2003). The first mode of each bridge analyzed corresponds to either the transverse or longitudinal translation. The monitored point in the bridge model is the intersection point between the superstructure and column top centerline. A natural log fit is used for the median of the data for all bridge models, as seen in Figure 2.



Displacements: $\ln(\Delta) = \beta_1 + \beta_2 \ln(IM)$ $\Delta = e^{\beta_1} \cdot IM^{\beta_2}$

Figure 2 Schematic comparison of SAP2000 and OpenSEES time history analysis results

Three levels of seismic hazard are defined to compute the bias factors between SAP2000 and OpenSees time history results. The low, moderate and high seismic hazard levels are defined as ground motions with 50%, 10% and 2%-in-50-year probabilities of exceedance (PE) for a high seismicity zone in California (e.g., Berkeley). The probabilistic uniform hazard curves provided by USGS (http://eqint.cr.usgs.gov) are used to obtain the elastic spectral displacements ($S_{d,elastic}$) corresponding to the first mode period of each bridge, for each hazard level.

The 60 points representing the ground motion intensity and the peak column top displacement computed using OpenSees, are shown in each of the graphs for each of the column directions in Figure 3. Also shown are the dispersion and the logarithmic regressions computed using OpenSees program for La Veta bridge are presented in Figure 3. Similar plots are generated for all 6 bridges from OpenSees and SAP2000 nonlinear time history analysis results, in both linear and logarithmic scales. The computation of the bias factors is carried out using 2 methods, described as follows.

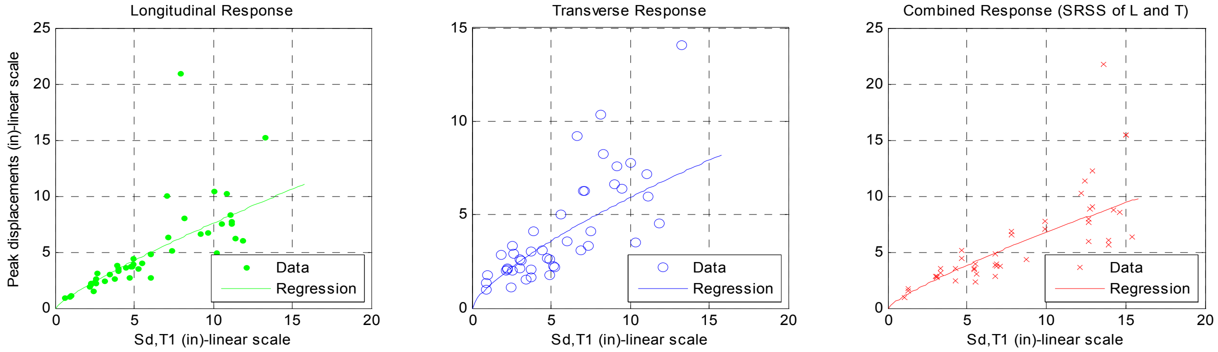
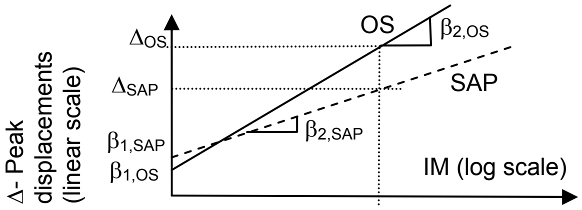


Figure 3 Nonlinear time history analysis results for OpenSees model of La Veta bridge

In method 1 (see Figure 4) the natural logarithmic regression is used for all peak displacements and the seismic intensities in the IM range of 0 to $1.2S_{d,elastic}$ for each bridge, representing a reasonable demand limit for the lifetime of the structures. The bias factors are then computed by comparing, at each hazard level, the peak displacements estimated through the regression coefficients of SAP2000 and OpenSees results.



Unparallel regression lines: $\beta_{1,SAP} \neq \beta_{1,OS}$; $\beta_{2,SAP} \neq \beta_{2,OS}$

Bias factor: $\dfrac{\Delta_{OS}}{\Delta_{SAP}} = e^{(\beta_{1,OS} - \beta_{1,SAP})} \cdot IM^{(\beta_{2,OS} - \beta_{2,SAP})}$

Figure 4 Method 1 for OpenSees- SAP2000 bias factor computation

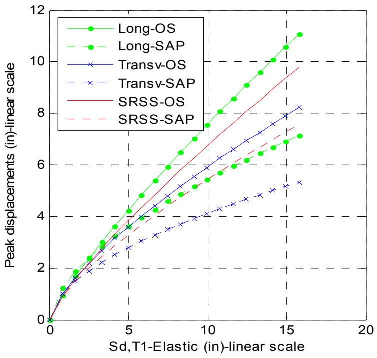


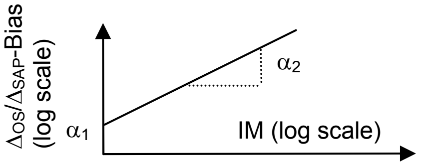
Figure 5 Comparison of SAP2000 and OpenSees regressions for La Veta bridge

Figure 5 presents the regressions for peak displacements obtained from SAP2000 and OpenSees for La Veta bridge model. The regressions computed for different directions are compared for the low, moderate and high hazard level demand corresponding to each bridge, and the bias factors, representing the ratio of OpenSees over SAP2000 peak displacement values, are recorded in Table 3. The seismic demand at each hazard level is computed for each bridge as the spectral displacement at the first mode period obtained from USGS hazard curves for a high seismicity zone in California.

Table 3 Bias factors computed for each bridge for different hazard levels, according to regression results: longitudinal, transverse, combined-SRSS, and mean responses

		Hazard level		
Classification	**Bridge**	**Low (50%-in-50yr PE)**	**Moderate (10%-in-50yr PE)**	**High (2%-in-50yr PE)**
Short (simplified abutment)	Route 14	1.18, 1.18, 1.19, 1.18	1.30, 1.18, 1.15, 1.21	1.37, 1.18, 1.13, 1.23
	Adobe	1.14, 1.12, 1.14, 1.13	1.28, 1.14, 1.11, 1.18	1.36, 1.16, 1.10, 1.21
	LADWP	1.03, 1.00, 1.10, 1.05	1.26, 1.16, 1.18, 1.20	1.40, 1.26, 1.22, 1.29
	La Veta	1.00, 1.17, 1.11, 1.09	1.30, 1.38, 1.21, 1.30	1.49, 1.50, 1.27, 1.42
Long (roller abutment)	MGR	1.09, 1.11, 1.14, 1.12	1.23, 1.17, 1.18, 1.19	1.32, 1.20, 1.19, 1.24
	W180-N168	1.25, 1.18, 1.05, 1.16	2.15, 2.02, 1.85, 2.01	2.87. 2.69, 2.48, 2.68

Method 2 for bias factor computation is carried out through a pair-wise comparison between OpenSees and SAP2000 nonlinear time history analysis results for each ground motion, in the longitudinal, transverse and diagonal directions of the bridge. A natural logarithmic regression is used relating the bias obtained for every ground motion to the intensity levels, as shown in Figure 6.



Bias factor: $\frac{\Delta_{OS}}{\Delta_{SAP}} = e^{\alpha_1} \cdot IM^{\alpha_2} = e^{(\beta_{1,OS}-\beta_{1,SAP})} \cdot IM^{(\beta_{2,OS}-\beta_{2,SAP})}$

Regression of bias factors:

$$\ln\left(\frac{\Delta_{OS}}{\Delta_{SAP}}\right) = \alpha_1 + \alpha_2 \cdot \ln(IM) = (\beta_{1,OS} - \beta_{1,SAP}) + (\beta_{2,OS} - \beta_{2,SAP}) \cdot \ln(IM)$$

Figure 6 Method 2 for OpenSees-SAP2000 bias factor computation

Figure 7 presents the dispersion of bias factors computed using Method 2 for each ground motion as the ratio of OpenSees and SAP2000 peak displacement results from nonlinear time history analysis of La Veta bridge. The bias factor corresponding to the low, moderate and hazard levels is computed using the new set of regression coefficients. The regressions for the bias factors in terms of the IM of $S_{d,T1}$ are obtained for all 6 bridges for the longitudinal, transverse and diagonal directions. The resulting bias factors for the low, moderate and high seismic hazard levels are presented in Table 4.

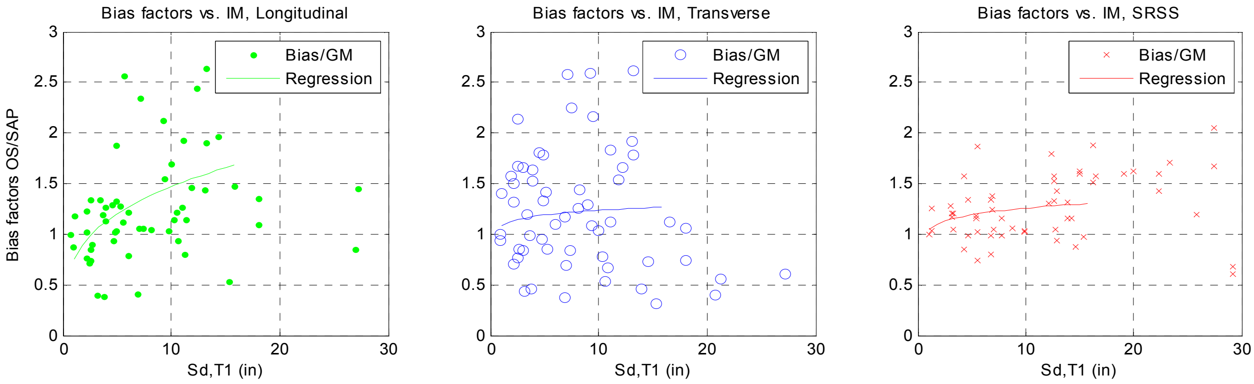


Figure 7 Bias factors computed for La Veta bridge in three directions

Table 4 Bias factors computed for the 6 bridges analyzed by regressions on bias factors obtained from OpenSees and SAP2000 pair-wise comparison: longitudinal, transverse, combined-SRSS, and mean responses

		Hazard level		
Classification	**Bridge**	**Low (50%-in-50yr PE)**	**Moderate (10%-in-50yr PE)**	**High (2%-in-50yr PE)**
Short (simplified abutment)	Route 14	1.13, 1.12, 1.20, 1.15	1.20, 1.16, 1.12, 1.16	1.23, 1.18, 1.09, 1.17
	Adobe	1.10, 1.06, 1.13, 1.09	1.20, 1.12, 1.10, 1.14	1.26, 1.16, 1.09, 1.17
	LADWP	0.98, 0.97, 1.10, 1.01	1.25, 1.12, 1.19, 1.19	1.42, 1.21, 1.24, 1.29
	La Veta	1.00, 1.15, 1.13, 1.09	1.36, 1.22, 1.23, 1.27	1.59, 1.25, 1.28, 1.38
Long (roller abutment)	MGR	1.07, 1.10, 1.09, 1.09	1.35, 1.40, 1.34, 1.36	1.52, 1.58, 1.50, 1.54
	W180-N168	1.29, 1.17, 1.03, 1.16	2.31, 2.11, 2.02, 2.15	3.15, 2.88, 2.89, 2.97

Table 5 summarizes the results of the bias factor computation for peak displacements obtained from nonlinear time history analysis in OpenSees and SAP2000 programs. The average bias for the 6 bridges analyzed is presented, differentiating between the results for short and long bridges.

Table 5 Summary of bias factors results

		Hazard level		
Bridge Type	**Method**	**Low (50%-in-50yr PE)**	**Moderate (10%-in-50yr PE)**	**High (2%-in-50yr PE)**
Short	1-Regression	1.11	1.22	1.29
	2-Pair-wise	1.09	1.19	1.25
Long	1-Regression	1.14	1.60	1.96
	2-Pair-wise	1.13	1.75	2.25

Both methods of computing analysis software bias factors show a clear tendency for the bias to increase with increasing intensity of the ground motions. The average bias factor value greater than 1.0 for all cases indicates that the nonlinear time history analysis displacement response results obtained using OpenSees are consistently larger in magnitude than the corresponding response results obtained using SAP2000 for the 6 bridge models analyzed.

For the low hazard level, the bridges remain essentially elastic. Since the modal periods computed for SAP2000 and OpenSees are similar for all 6 bridges analyzed, the nonlinear time history analysis displacement results are comparable as well. The average bias obtained for short and long bridges in the low hazard level range is around 1.1 using the 2 methods of bias factor computation. For the moderate hazard level, the bias factors computed using both methods of computation show a clear tendency for the OpenSees displacement results to exceed the SAP2000 nonlinear time history analysis displacement results. An average bias of 1.2 and 1.6 is computed for the short and the long bridges, respectively. At the high hazard level, this tendency becomes more pronounced. Average bias of 1.25 and 2.0 are obtained for the short and long bridges, respectively.

The bias factors obtained for the corresponding hazard levels using both methods are similar. These methods are therefore considered adequate for computing bias factors. However, since very few bridges are used in the analysis, the bias factors presented do not represent a sufficiently large and representative data set. In order to obtain reliable values of OpenSees-SAP2000 bias factors, a larger number of bridges with different geometric configurations representing short and long bridges must be used. An estimate of the dispersion of these results for different hazard levels must be obtained as well.

4. EPISTEMIC UNCERTAINTY

The ultimate goal in the determination of the measure of uncertainty in bridge response estimates is to obtain fragility curves for different levels of seismic intensity measures (IMs) that include both epistemic (modeling) and aleatory (ground motion) variability. Since the latter can be reduced significantly by using a large GM database, the schematic procedure to calculate the epistemic uncertainty is presented in Figure 8 and an example is provided in Figure 9. A more comprehensive approximation of the probabilities exceeding specified engineering demand parameter (EDPs) thresholds required for the estimation of the epistemic uncertainty can be performed using first or second order approximations of reliability analysis methods (FORM, SORM, etc.) or simulation methods.

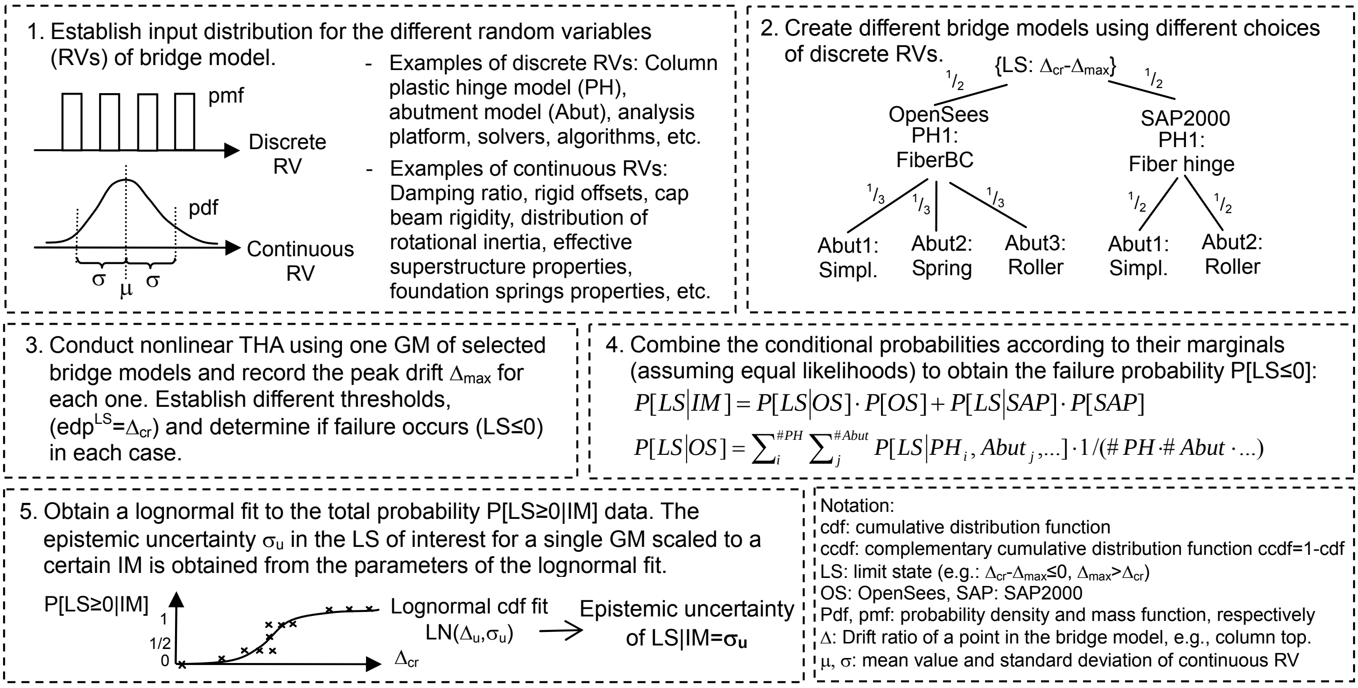


Figure 8 Procedure to evaluate epistemic uncertainty in bridge response estimates

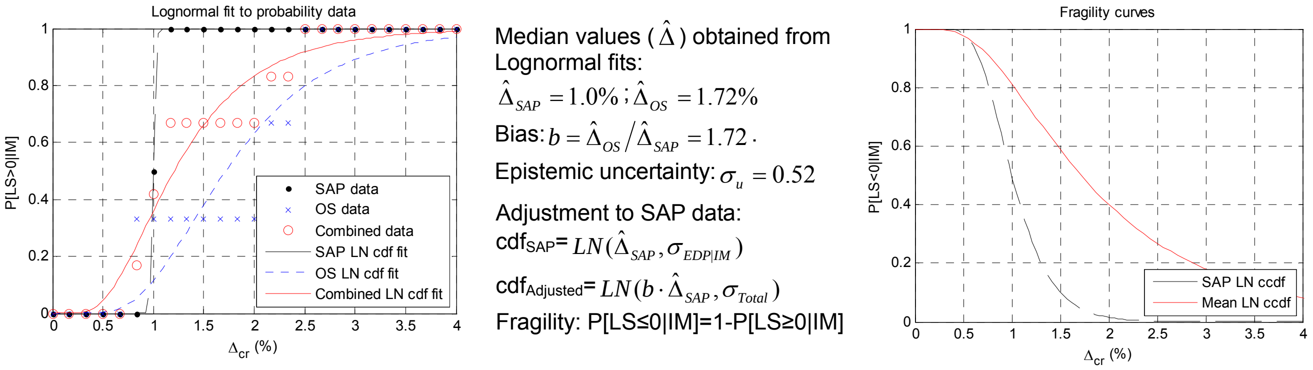


Figure 9 Example fragilities for longitudinal response of La Veta bridge for the scaled *andd* GM of the I880n set

The adjustment to SAP2000 seismic response estimates presented in Figure 9 is carried out for the IM level of the scaled *andd* record ($S_{d,T1}$=3.13 in) accounting for the aleatory variability of the GMs ($\sigma_{EDP|IM}$=0.32). The aleatory variability was obtained from a single bridge model in SAP2000 and 60 GMs scaled to a common IM. The mean fragility curve and total uncertainty were obtained according to Eqn. 4.1:

$$P[LS \le 0|IM] = 1-\Phi\left[\frac{\ln(\Delta_{cr})-\ln(\hat\Delta_{SAP})}{\sigma_{Total}}\right], \text{ where: } \sigma_{Total} = \sqrt{\sigma_{EDP|IM}^{2}+\sigma_{u}^{2}} = \sqrt{\sigma_{Aleatory}^{2}+\sigma_{Epistemic}^{2}} \qquad (4.1)$$

5. CONCLUSIONS

A total of 6 reinforced concrete bridge models are developed in OpenSees and SAP2000 structural analysis programs. Similar modeling assumptions are used for the superstructure, cap beam and abutment system. The column bent model in OpenSees consists of a distributed-plasticity fiber model with nonlinear force formulation and 5 integration points. Conversely, the column-bent plastic hinge model in SAP2000 consists of a lumped plasticity fiber model with an elastic beam-column element with effective cross section properties outside the hinge. Eigenvalue analysis and pushover analysis are similarly carried out in both programs, where the elastic periods, initial stiffness and ultimate base shear match in both programs within about 10% for all 6 bridges.

The bias factor, computed as the ratio of OpenSees over SAP2000 response parameter, was also computed by conducting nonlinear time history analyses. The set of 60 records used in these analyses includes near-field and far-field ground motions. Several near-field motions exhibit fault-normal and fault-parallel characteristics. Therefore, explicit considerations of directivity effects are not represented by the computed bias factors. Despite using a finite number of ground motions in the analysis, the use of an extended set (60 motions) helps to reduce the variability inherent in using a limited sample size for earthquake ground motion sampling and characteristics.

Despite similar modal and pushover analysis results for all bridges, the epistemic uncertainty related to column modeling and the different solution algorithms in each program introduce significant variations in the dynamic analysis results. The peak displacements computed using OpenSees uniformly exceed those computed using SAP2000 for all 6 bridge models. Bias factors are computed for a low, moderate and high seismic hazard level in California, differentiating between short and long bridge types. Furthermore, it can be observed that the bias and data dispersion between OpenSees and SAP2000 dynamic results grow with increasing intensity measure of the ground motion. Additional analysis is required to evaluate the epistemic uncertainty inherent in bridge response estimates according to the methodology presented using an extended set of bridges with different geometric configurations and properties.

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