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Analysis of Covariance to Capture the Importance of Bridge Attributes on the Probabilistic Seismic Demand Model

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ABSTRACT: Determining the probability distribution of structural demand conditioned on the ground motion intensity measure, known as the Probabilistic Seismic Demand Model (PSDM), is one of the crucial steps in the generation of fragility curves and seismic vulnerability assessment. To capture the uncertainties associated with geometric parameters and configurations for a particular bridge class, it is common to generate a PSDM resulting from nonlinear time-history analyses for a large number of random bridge samples. This study selects a typical reinforced concrete box-girder bridge in California, and probabilistic numerical bridge models are developed accounting for uncertainties in material properties and structural system. Nonlinear time-history analysis is performed using a set of ground motions developed for the seismic hazard in California to generate the PSDMs. The effect of column configuration (e.g., column diameter, number of columns per bent), bearing types (rocker, elastomeric or friction) and abutment configurations (supported on piles or on spread footing) on the PSDM is assessed using the Analysis of Covariance (ANCOVA) technique. ANCOVA will be used to determine whether the PSDMs of different bridge configurations with distinct attributes vary in terms of slope and/or intercept. This study underscores the necessity to consider the importance of various attributes on the PSDM of box-girder bridges in California.

1 INTRODUCTION

Most fragility curves developed for California bridges (Mackie & Stojadinovic 2001; Mackie & Stojadinović, 2005; Zhang & Huo, 2009) are structure specific, and thus are beneficial for the risk assessment of a specific bridge structure. These fragility curves, however, cannot be used for the risk assessment of regional bridge inventories. Ramanathan et al. (2015) addressed this issue and developed fragility curves that are applicable to a portfolio of bridges. They considered geometric uncertainties such as span length, column height, number of spans, superstructure type and material uncertainties such as concrete compressive strength, reinforcement yield strength, and so on. However, their study is limited to circular column bridges, elastomeric bearings and specific abutment types. A detailed review of the bridge plans and use of in-house databases such as BIRIS obtained from the California Department of Transportation (Caltrans) shows that their study addresses only a specific class of bridges and is necessary to extend it to various bridge configurations. Bridges can be classified based on the number of columns per bent, abutment types, bearing types, cross-section types, etc. However, such a classification leads to a number of subclasses and it would be cumbersome to compute the seismic vulnerability of each class of bridges. Additionally, it is not warranted that such a detailed classification would lead to a better refinement of the seismic assessment or not. The objective of this study is to check whether different classes of bridges can be grouped together or not in the fragility analysis.

One of the main steps in the generation of fragility curves is the development of probabilistic seismic demand models (PSDMs). PSDMs are the probability distribution of structural demands (D) conditioned on the ground motion intensity measure (IM). Here, the demands are monitored through a set of non-linear time history analyses (NLTHAs). The PSDM is a regression model for pairs of D and IM in the logarithmic space. To capture uncertainties associated with geometric parameters and configurations for a particular bridge class, it is common to generate a PSDM by performing NLTHAs for a large number of random bridge models with various bridge attributes such as different column sections, bearings, abutment types etc. The intention of the current study is to examine whether the PSDMs differ significantly between the bridge groups with different attributes. If they differ significantly, it is necessary to generate PSDMs and fragility curves separately for bridges with those attributes and mixing the bridge attributes to a single PSDM leads to non-realistic estimation of the PSDMs and fragility curves.

This study is limited to older era (constructed before 1970s) two span box girder bridges in California and further studies would extend it to multi-span and newer era bridges (constructed after 1970s). The effect of skew, curvature and retrofitting is not considered in this paper.

2 VARIOUS BRIDGE ATTRIBUTES

Box girder bridges account for the bulk of the California bridge inventory and a detailed plan review is carried out in the current study using the in-house database of bridges assembled by Caltrans engineers. The bridges considered in this study are classified based on abutment types: (1) Diaphragm and (2) seat type abutment bridges. Diaphragm abutments are cast monolithic with the superstructure while seat type abutments provide a bearing support to the superstructure. The movement of the superstructure is restrained longitudinally by the abutment backwall and transversely by the shear key in case of bridges with seat abutments.

Bridge and ground motion attributes which are suspected to have a significant influence on the PSDMs are bearing type (elastomer vs rocker bearings), column cross sections (circular vs rectangular), abutment configuration (abutment on piles vs spread footing), abutment backfill (clay vs sand), column bent type (single column vs multi-column), and direction of ground motions (fault normal along the bridge axis vs fault parallel). Two types of bearings noted for bridges with seat abutments are rocker and elastomeric bearings. The response associated with elastomeric bearings is based on sliding, while it is characterized by rocking in case of rocker bearings. The column cross sections can be either rectangular or circular (Figure 1). Based on the footing type, abutments can be classified as abutments resting on piles and abutments resting on spread foundation. Depending upon the deck width, column bents can be of single or multiple column. Multi-column bents are assumed to be pinned at their base in the current study. The suite of ground motions developed by Baker et al. (2011), for the PEER Transportation Research Program, is adopted in this study. Each ground motion has two components: fault normal and fault parallel.

3 ANALYTICAL MODELING PROCEDURE AND BRIDGE CLASS CHARACTERISTICS

Figure 1 shows the typical configuration of a two-span continuous box girder bridge and associated numerical model of various bridge components. Numerical modeling is carried out with the help of the finite element package OpenSees (Mazzoni et al. 2006) incorporating both geometric and material nonlinearities. Longitudinal deck elements are modeled using elastic beam-column elements as they typically remain elastic during a seismic event. Properties of deck elements are calculated based on the composite section properties Columns are modeled using the fiber-type displacement-based beam-column elements, which have the distinct advantage of specification of material properties specific to different locations in a member cross-section.

The contact element developed by Muthukumar & DesRoches (2006) is used to model the pounding between the decks. This element model explicitly accounts for the loss of hysteretic energy. The maximum deformation, Δ_m is assumed to be 1.0 in. The yield deformation, Δ_y is assumed to be 0.1 Δ_m. The stiffnesses, K_1 and K_2 are recommended to be 1.0223×10^3 kips/in/ft and 351.755 kips/in/ft respectively. Abutment responses comprise of the earth pressure response and the structural response. Earth pressure on the abutment is due to the longitudinal response of the bridge deck and includes passive and active resistance. The passive resistance is developed when the abutment moves toward the backfill soil and the active resistance is activated when the abutment moves away from the backwall soil. Structural response of abutments consists of the pile response or the frictional response depending upon the footing configuration of the abutment. The passive response of the abutment backwall is simulated using the hyperbolic soil model proposed by Shamsabadi & Yan (2008). Piles provide longitudinal and transverse stiffness to the abutments in the case of abutments resting on piles. The trilinear force deformation response of the pile along with the modeling parameters is presented in Figure 1. The design recommendations of the Caltrans 2014 draft of bridge design aids on 'Permissible Horizontal Loads for Standard Plan and Steel HP Piles' are used to calculate the parameters at initial yielding (Δ_1, F_1). The plastic yielding parameters (Δ_2, F_2) are calculated based on results of modeling various pile systems at LPILE (2012). Frictional response applies to the abutments which are supported on spread footings. The maximum force (F_s) is calculated as the product of the co-efficient of friction and the dead load reaction at the abutment. The elastomeric bearing is assumed to be elasto–plastic and the yield force, F_y, is obtained by multiplying the normal force, N, with the co-efficient of friction (Ramanathan et al. 2015). Rocker bearings are modeled following the work of Mander et al. (1996) stemmed from an experimental study. This model includes a frictional component and the force-deformation behavior of the bearings in the longitudinal direction is shown in Figure 1. Shear keys help to constrain the relative transverse movement between the deck and abutments. The nonlinear model of the shear key is shown in Figure 1. P_{cap} denotes the capacity of the shear key and is computed as the product of dead-load reaction and the acceleration. Megally et al. (2002) conducted a series of experiments on shear keys and found that Δ_{Max}- Δ_{gap} equal to 3.5 in. is the deformation at which the capacity of the shear keys degrades to essentially zero.

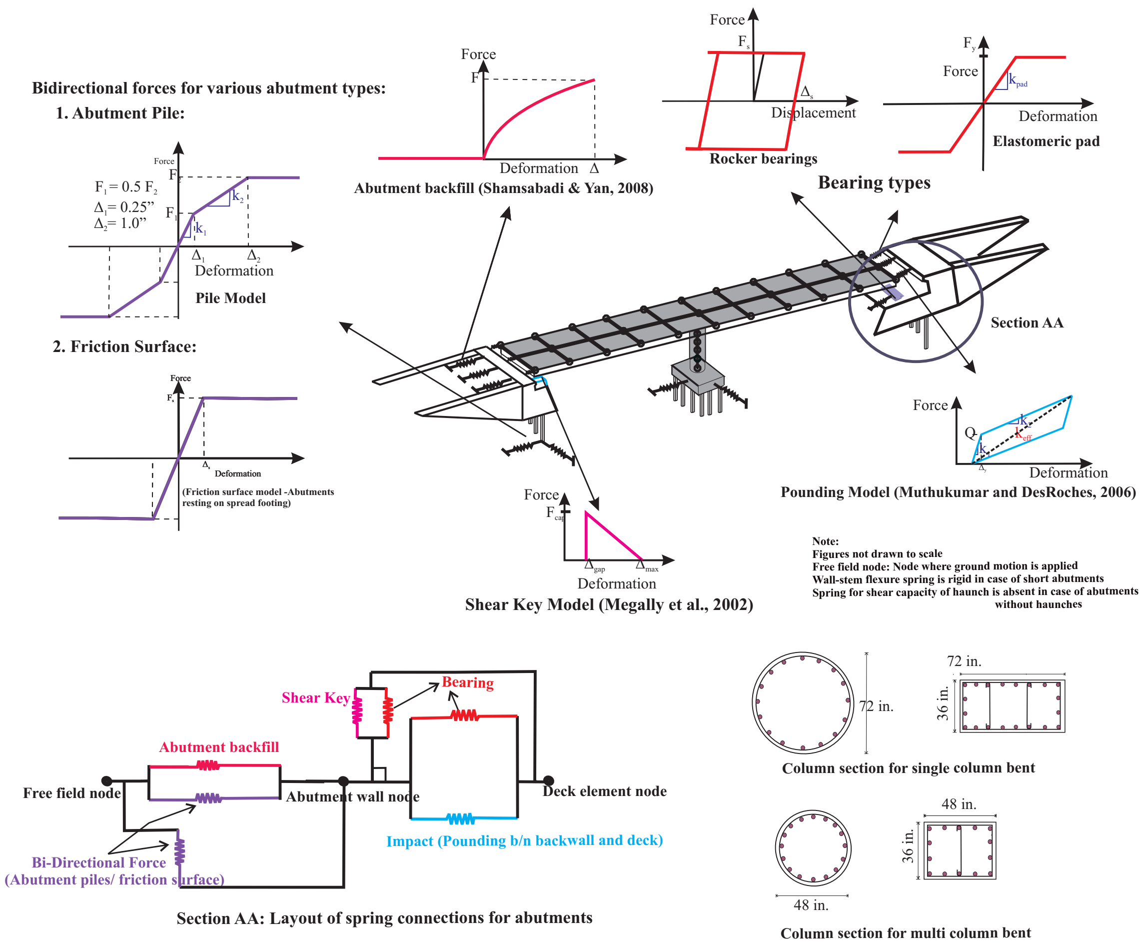

Figure 1: Modeling of various bridge components.

Translational and rotational springs are added to the base of the column to simulate the behavior of the footing. Zero length elements capturing the response of the abutment back fill soil and bi-directional force (abutment piles or frictional surface) are connected in parallel and are connected to the transverse deck elements in the case of diaphragm abutments. The abutment pile or friction surface model is selected, depending upon the type of footing (whether the abutment is resting on piles or on spread footings). Bearing pad elements and pounding elements are modeled with zero spring elements and are connected in parallel (Figure 1).

Table 1: Uncertainty distribution considered in the fragility analysis

Parameter	Units	Distribution Type	Distribution Parameters[1] α	β	Source
Concrete compressive strength	ksi	Normal	4.21	0.52	Caltrans (2015)
Reinforcing steel yield strength	ksi	Lognormal	4.21	0.08	Ramanathan (2012)
Span Length	ft.	Empirical	114.8	40.5	Ramanathan (2012)
Deck width	ft.	Empirical	67.2	42.2	Ramanathan (2012)
Column height	ft.	Empirical	18.0	3.7	Ramanathan (2012)
Abutment backwall height					
Diaphragm abutments					
On piles	ft.	Lognormal	2.35	0.15	
On spread footings	ft.	Lognormal	2.20	0.35	Caltrans (2015)
Seat abutments					
On piles	ft.	Lognormal	1.95	0.20	
On spread footings	ft.	Lognormal	1.95	0.20	
Abutments on piles - Lateral capacity/ft of deck width					
Diaphragm abutments	kip/ft.	Lognormal	1.79	0.35	Caltrans (2015)
Seat abutments	kip/ft.	Lognormal	2.08	0.35	
Abutments on spread footing		Normal	0.40	0.075	
Coefficient of friction	in.	Uniform	0.5	1.0	Caltrans (2015)
Yield displacement					
Elastomeric bearing pad	kip/in/ft				
Stiffness per ft of deck width		Lognormal	0.40	0.35	Caltrans (2015)
Coefficient of friction for bearing pad		Normal	0.30	0.10	
Rocker bearings					
Coefficient of friction (long. direction)		Normal	0.04	0.01	Caltrans (2015)
Coefficient of friction (tran. direction)		Normal	0.10	0.02	
Gap					
Longitudinal (btw. deck & abutment	in.	Lognormal	-0.20	0.50	Caltrans (2015)
wall)	in.	Lognormal	-0.70	0.20	
Transverse (btw. deck and shear key)					
Mass factor		Uniform	1.1	1.4	Ramanathan (2012)
Damping		Normal	0.045	0.0125	Ramanathan (2012)
Acceleration for shear key capacity	g	Lognormal	0.00	0.20	Caltrans (2015)
Pile group – pile cap and piles					
Translational stiffness					
6 ft dia column – 1% long. Steel	kip/in	Normal	1700	800	
6 ft dia column – 3% long. Steel	kip/in	Normal	1400	600	
3 ft dia column – 1.5% long. Steel	kip/in	Normal	800	600	Ramanathan (2012)
Rotational stiffness					
6 ft dia column – 1% long. Steel	kip-in/rd	Normal	4×10^7	1×10^7	
6 ft dia column – 3% long. Steel	kip-in/rd	Normal	6×10^7	1×10^7	
3 ft dia column – 1.5% long. Steel	kip-in/rd	Normal	0*	0	

[1]α and β are the parameters of the distribution. These denote mean and standard deviation for a normal distribution and empirical distribution, lower and upper bound in case of uniform distribution and mean and standard deviation of the associated normal distribution (in log space) in the case of a lognormal distribution.
* Multi column bents are assumed to be pinned at their base

The suite of ground motions assembled by Baker et al. (2011) is adopted in this study. The entire suite of ground motions are scaled by a factor of 2 (Ramanathan et al., 2015) to have sufficient time histories of IM's higher than the Palmdale spectrum (highest probabilistic design hazard level in

California). Thus, the expanded suite of 320 ground motions (N) is used in this study. In addition to the uncertainty from the ground motions, geometric and material uncertainties are also considered in this study and are elaborated in Table 1. Statistically significant yet nominally identical 3-D bridge models are generated by sampling across the range of parameters (Table 1) using Latin Hypercube Sampling (LHS) and then paired randomly with the selected suite of ground motions.

Previous studies (Ramanthan et al., 2015; Mangalathu et al. 2015) on fragility analysis of box girder bridges have shown that columns are the most vulnerable component in the event of an earthquake and hence column curvature ductility (μ_ϕ) is considered as the demand parameter in the current study. Although several researchers have explored the issue of selection of ground motion IM, spectral acceleration at 1.0s ($S_{a\text{-}1.0s}$) is considered as the IM in the current study. The following section gives a brief outline of ANCOVA and the application of ANCOVA to PSDMs.

4 ANALYSIS OF COVARIANCE (ANCOVA)

ANCOVA examines the influence of an independent variable (μ_ϕ here) on a dependent variable ($S_{a\text{-}1.0s}$ here) with various treatments (various bridge attributes, Section 2). ANCOVA is a general linear model which blends ANOVA (Analysis of Variance) and regression analysis (Rutherford 2001; Vidakovic, 2011). Two important assumptions in ANCOVA are (1) independence of the treatment effect, and (2) homogeneity of regression slopes. These two assumptions are valid in the current study as we are assuming each bridge attribute as independent and the damage measure increases with the intensity of ground motions. The purpose of ANCOVA is to compare two or more linear regression lines. The first step in performing an ANCOVA is to compute each regression line and check whether the slopes are significantly different or not. If the slopes are not significantly different, a regression line is drawn through each group of points, all with same slope and check whether the y-intercepts of regression lines with common slope are same or not. If not, it can be deduced that the regression lines are different at any point. It is hence possible in the current study through ANCOVA to check whether the PSDMs significantly differ between the bridge groups with different attributes or not.

In ANCOVA, the model is assumed as (Vidakovic, 2011):

$$D_{ij} = \mu + \alpha_{si} + \beta_s (x_{ij} - \bar{x}) + \varepsilon_{ij}\,,\ i = 1,...,a\,;\ j = 1,...,n \qquad (1)$$

where a is the number of treatments (different bridge attributes here), n is the common sample size, $\bar{x} = \frac{1}{an}\sum_{i,j} x_{ij}$ is the overall mean of xs ($S_{a\text{-}1.0s}$ here), β_s is the regression slope, α_{si} is the treatment effect, D is the demand parameter. The errors ε_{ij} are assumed independent normal with mean 0 and variance σ^2. The results of the current study are reported in terms of p - value which check the null hypothesis, H_0 : $\alpha_{si} = 0$ (i.e., the effect of treatment is zero). p - value is the probability that the variation of the damage measure due to different structural attributes are occurred by chance. Smaller p –values indicate stronger evidence for rejecting the null hypothesis (H_0) and a cut-off of 0.05 is adopted in the current study**.** For example, at cutoff value of 0.05, if the p-value of α_{si} from ANCOVA is less than the cut off it can be concluded that with 95% confidence the variation in the damage measure is not due to random chance but due to the influence of the different bridge attributes.

5 RESULTS AND DISCUSSION

In order to study the effect of various bridge attributes on PSDMs, the total simulation is split equally amongst the bridge attributes. For example, to estimate the effect of distinct bearings on PSDMs, 50 % of the simulations (160) are carried out on rocker bearings while the remaining 50% are carried out with elastomeric bearings. The models are chosen randomly to ensure similar propagation of uncertainties. The failed analyses and extreme outliers are removed from the PSDMs.

ANCOVA on various bridge attributes for bridges with seat type and diaphragm abutments are shown in Figures 2 and 3 respectively. The p–value for the cases are reported in Table 2. The column cross section, bearings and bent type significantly influence the PSDMs for bridges with seat abutments. However, the column cross sections are not significant for bridges with diaphragm abutments. It can be explained that the diaphragm abutments are stiffer than the adjacent bents and hence attract a large portion of the imposed seismic force and hence less seismic demand on bridge columns.

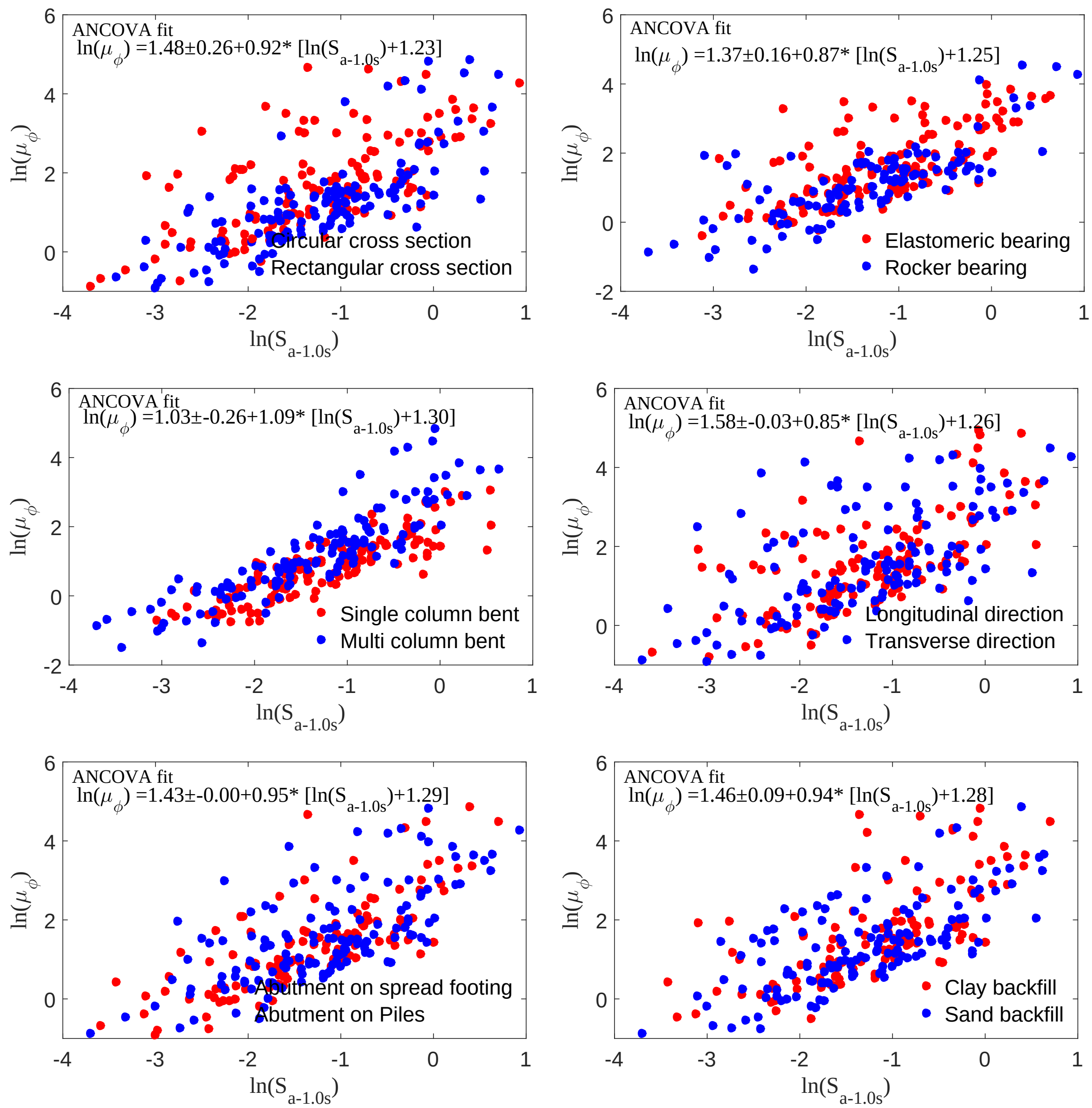

Figure 2: ANCOVA with various bridge attributes on bridges with seat abutments

Table 2: Results of ANCOVA on PSDMs

Bridge Attribute	**Seat type abutments**		**Diaphragm abutments**	
	p- value	**Statistically significant or not**	***p- value***	**Statistically significant or not**
Column cross section	0.000	Significant	0.194	Not significant
Bearing	0.001	Significant	-	-
Bent type	0.000	Significant	0.000	Significant
GM direction	0.569	Not significant	0.239	Not significant
Abutment	0.968	Not significant	0.564	Not significant
Backfill	0.092	Not significant	0.000	Significant

As the diaphragm abutments attract a major portion of the seismic force, the type of backfill is important, which explains why the backfill is significant for diaphragm abutment bridges and not significant for seat abutment bridges (Table 2). Multi-column bents are pinned at the base while single columns have a significant amount of rotational restraint, which induces high moment for single column bents (Priestley et al., 1996). It is noted that the type of abutment footing is not significant in either bridge cases. Consistent with the previous study (Mackie et al. 2011), the column damage measure is less affected by the ground motion direction.

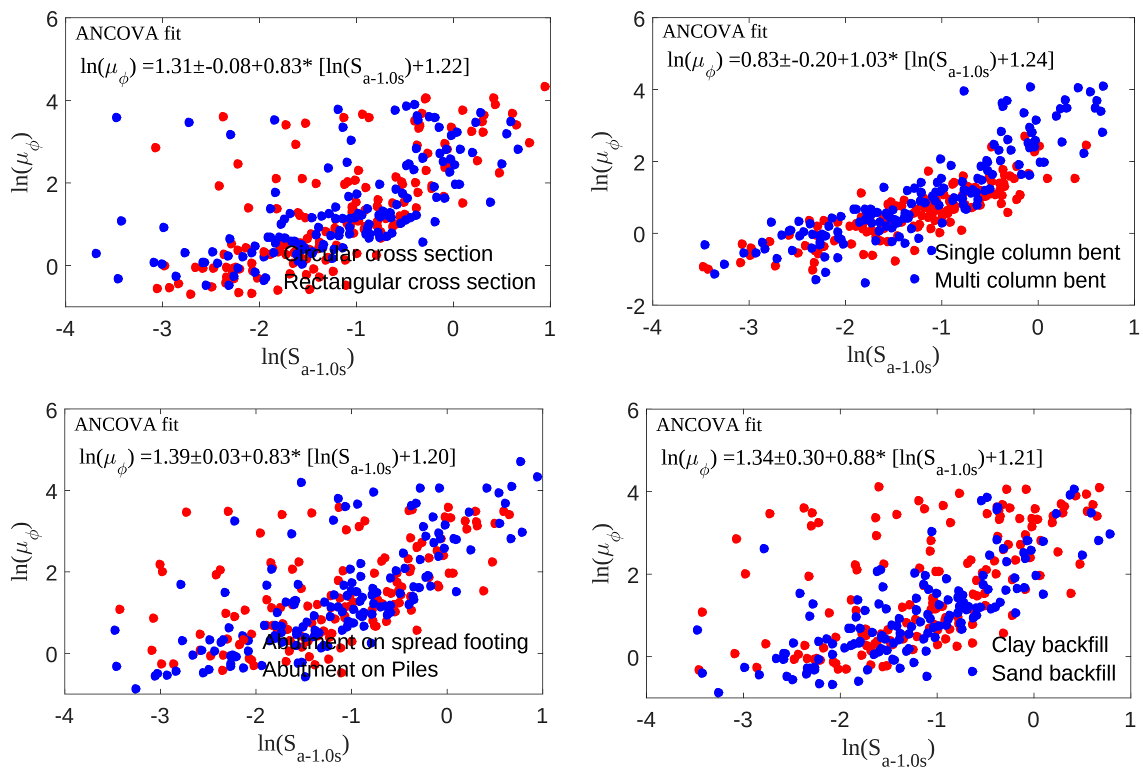

Figure 3: ANCOVA with various bridge attributes on bridges with diaphragm abutments

6 CONCLUSION

One of the critical aspects of the seismic vulnerability assessment is to understand and characterize the highway bridge inventory. In order to facilitate the regional seismic risk assessment of bridge inventories, it is imperative to have fragility curves applicable to a class of bridge structures. To ensure this objective, various bridge attributes should be considered. The current study suggests the bridge attributes which can be grouped without causing the significant variation of PSDMs.

Nonlinear time history analysis is carried out on the statistically significant yet nominally identical 3-D bridge models. The bridge models are generated by sampling across the range of parameters accounting for the geometric and material uncertainties. In order to estimate the significance of various bridge attributes, ANCOVA is carried out on PSDMs of bridge models with distinct design attributes. It is noticed that the bent type (single column bent or multi column bent) significantly affect the PSDMs and therefore PSDMs of bridge types with different bent types should be developed separately. The column cross section significantly affects the PSDMs of bridges with seat abutments while it is not significant for diaphragm abutment bridges. The type of backfill also significantly influences the PSDMs of bridges with diaphragm abutments.

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