SEISMIC VULNERABILITY ASSESSMENT OF SKEW BRIDGES

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ABSTRACT :
The operability of the highway network after an earthquake is extremely important. The rescue operations and basic supply after a disaster mainly depend on the proper functioning of the highway network and its bridges. Past earthquakes have shown, that bridges are the most vulnerable components in a transport network and highly susceptible for severe damage during seismic events. Especially skewed bridges are at higher risk than straight bridges due to effects engaged by the activation of coupled transverse and longitudinal modes.
The fragility curves for the skew bridges with a skew angle of 0°, 12.33° and 45° have been developed. Fragility curves are the most reliable tool for seismic vulnerability assessment, when a rapid estimation is needed. For the present investigation fragility curves for four different damage states ranging from slight damage to complete destruction have been developed. An investigation of the effect of skew angle on the seismic vulnerability has been carried out by comparing the fragility curves for bridges with different skew angles in the different damage states. As the bridge piers and bearings are the most vulnerable components, different fragility curves for each of these components have been adopted for the analyses. A non-linear 3-D finite element bridge model was used for the investigation in order to be able to include all torsional effects arising from the bridge skew. Up to 60 synthetic acceleration time history analyses have been applied on the different bridge configurations. Peak responses from all time histories for each vulnerable component has been extracted and reviewed. Regression analyses of the peak responses have then been carried out to develop a probabilistic seismic demand model (PSDM). The developed fragility curves have been applied in the framework of a case study to a typical European highway bridge. The evaluation of the selected bridge shows that the bridge bend is extremely vulnerable to the seismic intensity range of 0.2 g to 0.4 g. But for bearings the range of seismic intensity which will cause severe damage is 0.45 g to 0.7 g. The effect of skew angle on the overall seismic vulnerability has been analyzed by comparing the fragility curves of the different bridge configurations

KEYWORDS:
Skew bridge, vulnerability, fragility curves, seismic, damage, FEM

1. Introduction

While performing the seismic risk analysis of a highway system, it is imperative to identify the seismic vulnerability of bridges associated with various states of damage, ranging from loss of serviceability to collapse. The failure of the civil infrastructure during seismic events can disrupt human life as well as economy. The seismic vulnerability of bridge structures depends on several aspects of structural form such as whether single or multispan; continuous or internal supports or simply supported; single column or multi column bend; the abutments are straight or skewed. Amount of confinement, age of the structure, soil condition etc. also influences the seismic vulnerability of bridge structure. The fragility curve can be used to assess the level of safety of transportation network immediately after a seismic event. Also, the rational decision on the existing bridge structure, whether the bridge should be retrofitted or replaced can be made from fragility curve. The first step in an analytical process is establishing an appropriate numerical model of the bridge. The second step is choosing a set of earthquake acceleration time histories, which covers various levels of ground shaking intensity. The accuracy of the vulnerability analysis depends on the ground motion data, which should be the representative of the area of interest. After quantifying the numerous uncertainties in the bridge to establish a set of earthquake-site-bridge samples, a nonlinear time history calculation of each of these samples is performed to simulate a set of bridge response data. The seismic vulnerability in the form of fragility curve, generated by considering the uncertainties in the seismicity, structural characteristics, soil-structure interaction are still under development. The prediction of the response of the existing structure is extremely challenging because of the randomness associated with the seismic loading and the resistance of the structure.

The most vulnerable bridge type in a transportation network is skew bridge. Because of its geometry, the response of the skew bridge during earthquake is more severe than for a straight bridge with identical mass and dimensions. The skew bridges have coupled response in global co-ordinate system and the seismic response increases with the increase in skew angle. The bridges with skew angle more than 45° are not recommended in highly seismic zones (Eurocode 8).

Skewed bridges are often encountered in highway design when the geometry cannot accommodate straight bridges. These bridges are found to be quite susceptible to severe damage during past earthquakes. When the skew angle gets large it can significantly alter the response of the bridge. Although the static and dynamic behavior of skew bridges has been investigated by various researchers, some phenomena are yet to be defined. The studies have revealed that if the deck is not rigidly connected with the abutment the dynamic response of the bridge is dominated by the in-plane rigid body motion rather than torsional and flexural modes. Due to symmetry the centers of mass and stiffness coincide, and the torsional vibration is independent of the translational vibrations. Hence the bridge will not experience torsional vibration unless the base motion is torsional.

The variables involved in the transverse vibration of the bridge are: stiffness of the pier, stiffness of the elastomeric bearings and the bridge mass. Mass is a function of geometry and the material of the bridge. The skewed bridge has directional coupling behavior. The period of vibration and inertial forces induced in the structure depends on the stiffness of supporting members in two non orthogonal directions. This stiffness affects the vibration modes. In case of straight bridge these two stiffnesses are mainly uncoupled.

For simplified dynamic analysis, the common idealization for bridge deck is the use of rigid beam elements. In case of a skew bridge the idealization provides inaccuracy for finding the natural vibration modes. The geometry of the deck slab also influences the modes of vibration. The study by Meng and Lui (2002) reveals that the first two natural frequencies are function of skew angle. It is observed that for the usual elastomeric bearing stiffness the first period decreases with the increase in skew angle, but the second period remains constant for all skew angles.

Fragility curves are the conditional probability statements which give the probability of a bridge reaching or exceeding a particular damage level for an earthquake of a given intensity level. The fragility functions are derived based on the comparison of seismic demand due to expected earthquakes and the available capacity of the structure. The structural demand and capacity are random because of the randomness associated with the structural parameters and that of seismicity. The fragility curves can be developed based on expert opinion, empirically as well as analytically. A hybrid approach is also possible in which the analytically derived fragility parameters can be modified with the empirical data. Fragility curve methodologies using analytical approaches have become widely adopted because they are more readily applied to bridge type and geological regions where seismic damage records are insufficient. The recent development in the computational technology has lead to the inclusion of a number of response features such as shear-flexural-axial interaction, local buckling of reinforcement, interactive confinement of concrete members etc increases the accuracy of the analytical procedure.

In the past, numerous pioneer works have been carried out by various researchers for the fragility curve development. Some works on seismic vulnerability assessment and the dynamic behavior of skew bridges are briefly reviewed in the following:
Nielson (2005) developed analytical fragility curve by considering the fragility of all major vulnerable components of a bridge system. He has developed the component fragility curve and combined the component

fragility curve using join probability concept to get the parameters for system fragility curve. Yia et. al. (2004) presented a probability density function (PDF) interpolation technique for the evaluation of seismic fragility curves as a function of the return period. Seismic fragility curves for bridges have been developed as a function of the return period and compared with those based on PGA. Karim and Yamazaki (2003) adopted an analytical method to develop fragility curve for the bridge piers, using damage indices and ground motion parameters. There, the Park-Ang damage model has been used to obtain a damage index. Saxena et. al. (2000) have developed fragility curves for multi-span reinforced concrete bridges by considering the stochastic spatial variability of the seismic ground motion on the seismic response. Basöz and Kiremidjian (1997) have developed empirical fragility curves by logistic regression based on the bridge damage observations after the Northridge earthquake. After defining 11 bridge classes based on substructure material (e.g., concrete, steel, concrete/steel, timber, masonry, etc.) and on superstructure material and type (e.g., concrete girder, steel girder, concrete truss, suspension/cable stayed, arch, etc.), empirical fragility curves have been developed for bridges grouped by these structural characteristics. Shinozuka (1998) has developed empirical fragility curve on the basis of the damage resulting from the 1994 Northridge Earthquake under the assumption that the fragility curve can be expressed in the form of a two parameter lognormal distribution function. The parameters such as mean and standard deviation of the distribution have been estimated by maximizing the likelihood of observing the damage data. Uncertainties in the ground motion and the structure are considered using a sample of 10 “nominally identical, but statistically different” bridges and 80 ground motion time histories. Fragility curves have been estimated by fitting a lognormal distribution to the failure/no failure data obtained from numerical simulations. Basöz and Mander (1999) have developed the fragility curves with an approach similar to that of Singhal and Kiremidjian (1998). Each fragility curve is assumed to be a standard lognormal cumulative distribution function with unknown location parameter such as mean and known constant scale parameter such as standard deviation, meant to incorporate epistemic uncertainty and aleatory variability of both capacity and demand. Singhal and Kiremidjian (1998) have developed fragility estimates by Bayesian analysis of observed damage data for subclasses of structural systems. They have used the damage index derived by Park and Ang (1985) to quantify the damage to a structure as a function of structural capacity and demand. The fragility is then defined as the conditional probability that the damage index exceeds a certain threshold for a given ground motion. Singhal and Kiremidjian have assumed that the randomness in the damage index at a specified ground motion level can be represented by a lognormal distribution with unknown median and known constant standard. Hwang and Huo (1998) presented an analytical method for generating fragility curve based on numerical simulation of the dynamic behavior of specific structures. The uncertainties in the earthquake site- structure system are quantified by considering the parameters in the system as random. Krawinkler and Seneviranta (1998) summarized the basic concepts for which the pushover analysis can be used. They have identified the conditions under which the pushover will provide adequate information and also identified the cases in which the pushover prediction will be inadequate or even misleading. Yamazaki et al. (2000) have developed a set of empirical fragility curves for bridges based on the actual damage data from the 1995 Kobe earthquake, considering 216 bridge structures and assuming a lognormal distribution for fragility functions. Maleki (2000) has worked on the seismic modeling of skewed bridges with elastomeric bearing and side retainers. In all his works on skew bridges he considered the cross frame stiffness which is oriented in the direction of skew at the abutment. The effect of skew angle on the seismic demand is analyzed by him by considering the skew angle varying from 0° to 60°. In another work he has examined the effect of elastomeric bearing stiffness on the seismic response of the skew bridge. He performed the dynamic analysis of different skew angles with identical mass in longitudinal and transverse direction.

2. DESCRIPTION OF THE INVESTIGATED SAMPLE BRIDGE

The starting point for the present seismic vulnerability assessment was a classical skewed highway bridge having a skew angle of 12.33 degree, located in Germany and is situated in the highest seismic zone. Based on this sample bridge, two bridges models with identical mass, having skew angles of 0° and 45° were considered in this study for analyzing the influence of skew angle on the seismic vulnerability. All the three bridges are pre-stressed continuous girder concrete bridge having two spans of 30.26 m each. The girder bridges have a total span length of 60.52 m and a width of 14.7 m. The thickness of the deck slab is 0.325 m. The T-beam deck having a cross section dimension of 1.15 × 1.2 m. The bridge bent consists of two columns having transverse spacing of 7.42 m and a length of 5.21 m. Each column has a 0.95 m diameter, reinforced with 20 mm diameter and with 40 longitudinal bars in two layers. The grade of concrete and steel are C25 and BSt500s respectively. The girders are supported by elastomeric bearing over the pier as well as over the abutment. The bearings over the piers are restrained in the transverse direction. Rectangular elastomeric bearings with 600 × 700 mm dimension are used to connect the abutment with the girder. Circular bearings of 800 mm diameter are used to connect pier with girder. The total thickness of elastomeric bearing is 110 mm with an elastomer thickness of 80 mm. The shear modulus of the bearing is 1 N/mm^2.

2.1 Finite Element Model

The finite element modeling is performed with the general purpose finite element software ANSYS. The 3-D model developed to perform the nonlinear dynamic response analysis of bridge is shown in figure 1.

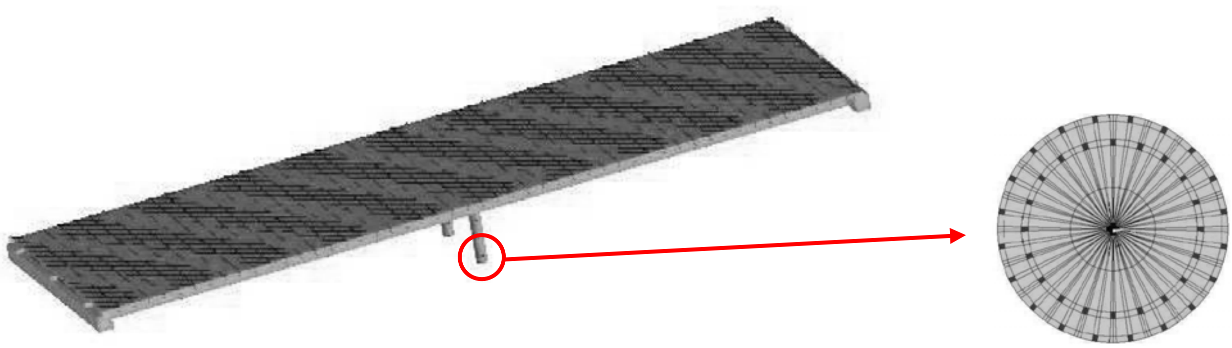
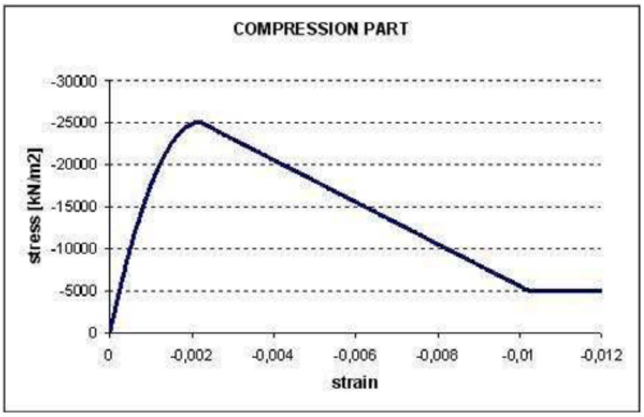


Figure 1 FEM Model of 12.3° skew bridge (l.) and modeled reinforced concrete column cross section (r.)

The bridge deck is assumed to behave linear at all levels of ground motion and thus, is modeled as linear. It is accomplished using four node linear elastic plane shell elements. The girders are modeled with linear beam elements, which are rigidly connected with the deck slab. The girders are restrained in vertical direction at each end which indicates the abutment is rigid in vertical direction. However, in the lateral directions, the end of the girder is attached to springs representing the lateral stiffness of the elastomeric bearings. The rotation about the longitudinal axis of the deck is restrained at the abutments. However, in plane translation and rotation about the vertical axis is allowed. This assumption is commonly used and it represents the actual support condition of the deck accurately enough. The nonlinear behavior of the elastomeric bearing is accomplished using nonlinear spring elements. The pier is modeled using the beam element, which has nonlinear capability and a detailed representation of the cross section. The bases of the column bent are modeled as fixed.

2.2 Analytical Concrete Column Model

Concrete is difficult to model due to the non-linear behavior and the different stress-strain relationships for compression and tension. A user defined material model has been used to represent the confined concrete. The non-linear material law for concrete has been programmed in FORTRAN and implemented in ANSYS. The concrete material model for the nonlinear column element in compression and tension are as shown in Figure 2. This model is able to properly represent the actual stress strain behavior of concrete including the cyclic behavior. The reinforcement is modeled as bilinear.



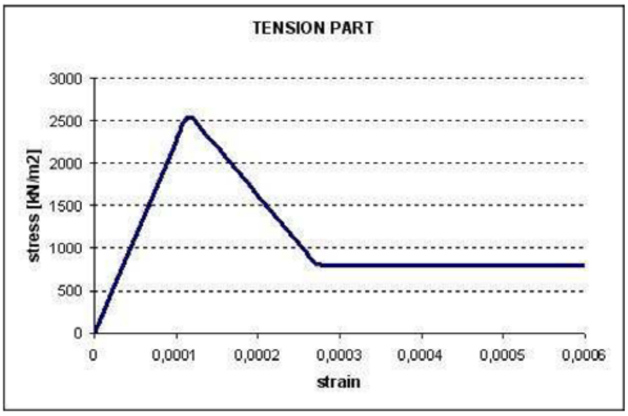


Figure 2 Concrete material model for compression and tension

2.3 Bearing Model

The bearings for the sample bridge are elastomeric pads. Rectangular bearings with a dimension of 700 × 600 mm are used between the abutment and the girder. The bearing over pier has a diameter 800 mm. The behavior of the elastomeric bearing is characterized by sliding. The bearing can accept the load until the coefficient of friction exceeded. The sliding behavior is characterized by the initial stiffness which accepts the load until the coefficient of friction is exceeded. Once it exceeded, the stiffness change to a value that is nearly zero (Schrage, 1981). At the abutment the bearing with steel dowels are used to ensure linear response during service load condition. The elastomeric pad transfers horizontal loads by developing a frictional force. For the model the fixed bearing the effect of the pad and dowels are developed separately and combined in parallel to get the appropriate composite action. The modeling of the elastomeric pad has been accomplished through the use of an elastic perfectly plastic material. A nonlinear spring element is used for the elastomeric bearing pad. This element has a capability to include a user defined force-displacement relationship. Initial stiffness of 5.25 10^3 KN/m and a yield force $Fy = \mu \times N$ are used in this study. The coefficient of friction is taken as 0.29. The shear modulus in bridge bearings range between 0.66 MPa and 2.2 MPa, depending on their hardness. A shear modulus of 1 MPa is assumed for this study.

2.4 Moment-Curvature Relationship

For verification purpose, a moment-curvature analysis of the user defined reinforced concrete cross section is carried out in ANSYS. The cross section properties can be influenced by the axial force on the section. The moment-curvature analysis has been done with an axial force from the dead load of 2850 KN. For verifying the accuracy of the user defined fiber cross section with user defined nonlinear concrete model, the moment curvature relationship from ANSYS has been compared with the result obtained for the cross section analysis software UCFyber (now called XTRACT). Figure 3 shows the comparison of the two results.

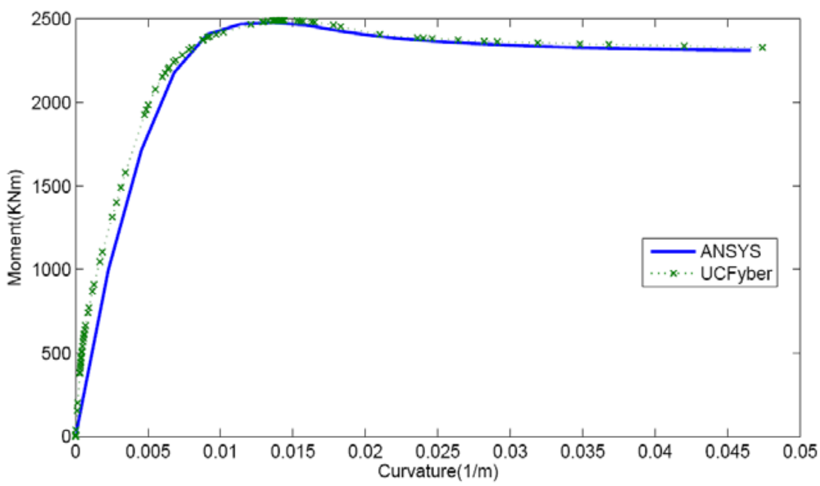


Figure 3 Moment-Curvature relationship comparison

2.5 Pushover Analysis

The push over analysis of the multi column bend is carried out in ANSYS with an axial load from the superstructure. Displacement at the column tips are monitored along with the shear force induced in the columns, like base shear in building. The ultimate load corresponding to the maximum displacement is 2760 KN and the maximum displacement of the bend is 0.101 m having an yield displacement of 0.0244 m.

3. SYNTHETIC GROUND MOTION

The acceleration time history varies, depending on the characteristics of the medium through which the wave is traveling. Since a sufficient amount of strong ground motion records do not exist for the investigated region, synthetic acceleration time histories are the alternative way for vulnerability analysis. Thus, synthetic ground acceleration time histories consistent with the Eurocode 8 spectrum have been used for the nonlinear time history analysis of the bridge model. For this study the software SeisPro (2008) is used, which generates spectrum compatible time histories based on random superposition of phase shifted harmonic signals.

For this seismic vulnerability assessment 60 synthetic acceleration time histories are generated consistent with Eurocode 8 elastic response spectrum, with ground type C. A general study on the influence of skew angle on the seismic vulnerability has been performed with the synthetic acceleration time histories by considering different soil type. The synthetic ground motion consistent with five soil class has been generated for this comparative study of non-skew and 45 degree skew bridges.

4. DYNAMIC CHARACTERISTICS OF BRIDGE MODEL AND SEISMIC RESPONSE

The fundamental periods in the longitudinal and transverse direction of the straight bridge and the skew bridges having a skew angle of 12.33° and 45° are represented in Table 1.

Table 4.1 Modal properties of different bridge configurations

Skew	Mode	Period [s]	Modal mass participation [%]
0°	Transversal	0.3400	100
	Longitudinal	0.3156	98.23
12.33°	Transversal	0.3601	99.87
	Longitudinal	0.3656	99.5
45°	Transversal	0.3767	97.97
	Longitudinal	0.3127	95.95

If the deck is not rigidly connected to the abutment the dynamic response of the bridge is dominated by in-plane rigid body motion rather than by flexure and torsion. In this study the deck is connected with the abutment by using elastomeric bearings and the fundamental modes are in-plane rigid body motion as shown in Table 4.1. The fundamental modes are the translation along longitudinal and transverse direction. It can be noticed from Table 4.1 that the transverse period is increasing with the increase in skew angle. The period of vibration depends on the stiffness of supporting members in two non-orthogonal directions. In this study the elastomeric bearing at the support are oriented in the global direction and thus the period is changing with the skew angle because of the difference in alignment of column stiffness. Thus, the skew bridge having a bidirectional coupling behavior. The transverse period of the non-skew bridge is 0.34 s and is increased to 0.38 s with an increase in skew angle to 45°.

Non-linear time history analyses are performed to get the seismic response of the bridge. The acceleration time histories have been applied in two orthogonal directions to get the response. 60 synthetic ground motions with different peak acceleration have been generated. In this study the elastomeric bearing and column are considered as critical element in the bridge system. Hence, the fragility curves for these two components are developed.

The elastomeric bearing over pier is restrained in the transverse direction and thus the bearing response in transfer direction is more or less similar to that of the column. Thus, the column is vulnerable in transverse direction and bearings are the vulnerable component in longitudinal direction. Figure 4 shows the response comparison of column for the non-skew and the 45° skew bridge. It can be observed form the figure that the response for the column in transverse direction increases and the response of the elastomeric bearing in longitudinal direction decreases with the increase in skew angle.

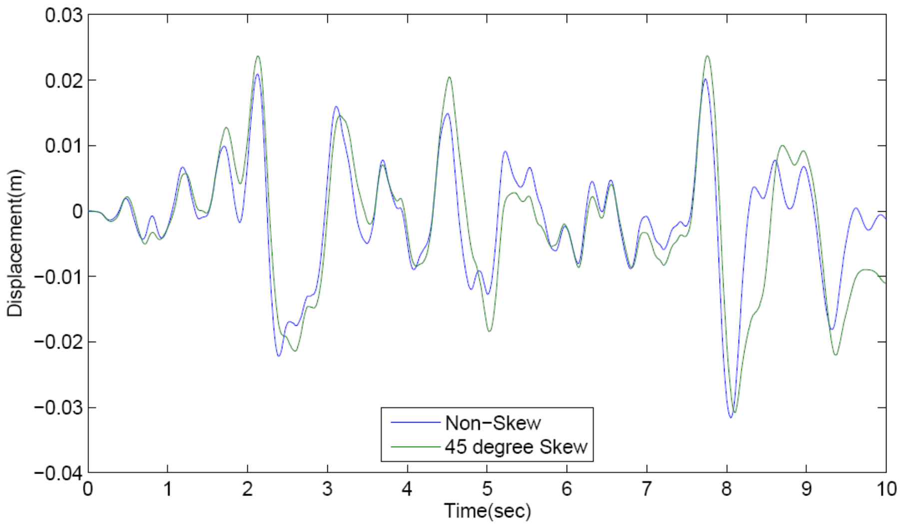


Figure 4 Colum response time history comparison for non-skew and 45° skew bridge

5. FRAGILITY ANALYSIS

Fragility curve methodology for assessing the seismic vulnerability has been used by various researchers in the past and is an emerging tool in seismic vulnerability assessment. Seismic fragility analysis was originally conducted to evaluate the seismic safety of nuclear power plants, which has recently been accepted as a reliable method for the evaluation of the seismic performance of civil infrastructures, such as bridges and buildings. For all these applications, fragility functions proved to be convenient, versatile and reasonably accurate tools of damage estimation. Fragility curve describe the probability of damage a bridge reaching a certain damage state given a specific ground motion parameter (commonly peak ground acceleration or spectral acceleration).

The procedure for the seismic fragility analysis of highway bridges is briefly described as follows:

- Establish an appropriate model of the bridge of interest in the study.
- Generate/chose a set of earthquake acceleration time histories, which covers the various levels of ground shaking intensity.
- Quantify the uncertainties in the seismic source, attenuation, local site conditions, and bridge modeling to establish a set of earthquake-site-bridge samples.
- Perform the nonlinear time history response analysis for each earthquake-bridge sample to simulate a set of bridge response data.
- Perform the regression analysis of simulated response data to establish the probabilistic characteristic of structural demand as function of ground shaking parameter, for example, spectral acceleration or peak ground acceleration.
- Define the bridge damage state and establish the probabilistic characteristic of structural capacity corresponding to each damage state.
- Compute the conditional probability that the structural demand exceeds structural capacity for various levels of ground shaking.
- Plot the fragility curves as a function of the selected ground shaking parameter.

When using analytical procedures, particularly nonlinear time history analysis, the seismic demand is described through probabilistic seismic demand models (PSDMs) which are given in terms of an appropriate intensity measure. In addition to the lognormal assumption, it has been suggested by Cornell et al. (2002) that the estimate of the median demand, S_d can be represented by a power model:

$$S_d = aIM^b \tag{5.1}$$

where *IM* is the seismic intensity measure of choice and both *a* and *b* are regression coefficients. After estimating the variability $\beta_{d|IM}$, which is conditional upon the intensity measure, the PSDM can be written as in equation (5.2), where *d* is the seismic demand (ductility, displacement, etc.) of the structure.

$$P_f(s_{d|IM}) = 1 - \phi\left(\frac{\ln(d) + \ln(aIM^b)}{\beta_{d|IM}}\right) \tag{5.2}$$

The actual regression used to estimate the parameters *a* and *b* from Equation (5.1) is more easily facilitated in a transformed natural logarithmic space.

$$\ln(S_d) = \ln(a) + b\ln(IM) \tag{5.3}$$

The fragility function, which is a conditional probability which gives the likelihood that a structure will meet or exceed a specific level of damage for a give ground motion parameter. The conditional probability is given by

$$Fragility = P[D > C \mid IM] \tag{5.4}$$

where *D* is the response measure of the bridge or bridge component. *C* is the limit state or damage level of the bridge or component and *IM* is the ground motion intensity measure.

Since damage states are related to the structural capacity *C* and the ground motion intensity parameter is related to the structural demand *D*, the fragility or probability of failure P_f can then be described as:

$$P_f = P[D/C > 1] \tag{5.5}$$

It is a widely adopted assumption among researchers that the structural capacity and demand are lognormal distributed functions. It can be verified from the central limit theorem that the composite outcome also can be lognormal. Thus the fragility curve can be represented by a lognormal cumulative distribution function and is given by (Melchers, 2001):

$$P_f = \phi\left(\frac{\ln(S_d / S_c)}{\sqrt{\beta_d^{\,2} + \beta_c^{\,2}}}\right) \tag{5.6}$$

Where S_c is the median value of structural capacity defined for the damage state, β_c is the lognormal standard deviation of the structural capacity, S_d is the seismic demand in terms of chosen ground motion intensity parameter and β_d is the lognormal standard deviation for the demand.

The equation can be rewritten in the following form, by substituting Equation (5.3) and where $\beta_{comp} = \sqrt{(\beta_d^{\,2} + \beta_c^{\,2}}$.

$$P_f = \phi\left(\frac{\ln(IM) - \ln(IM_m)}{\beta_{comp}}\right) \tag{5.7}$$

Hwang et al. (2000) proposed limit states for columns in terms of displacement ductilities, of 1.0, 1.2, 1.76 and 4.76 which correspond to yield, cracking, spalling and reinforcement buckling, respectively. It is pointed out in the Seismic Retrofitting Manual for Highway Bridges (FHWA, 1995) that, for poorly confined columns, longitudinal steel will buckle at a displacement ductility of 3.0 and is thus the value chosen for this study. The limit state for the bearings has been taken from Nielson (2005). The proposed values are 28.8, 90.9, 142.2, 195 mm for slight, moderate, extensive and complete damage.

In this study fragility curves have been generated using an analytical approach. Numerous nonlinear time history analyses have been performed to get a wide range of damage data. Regression analysis of the damage data has been carried out to get the parameters. Table 5.1 shows the median (Med.) and variability (Var.) of the fragility curves for non-skew, 12.33° and 45° skew bridges in all levels of damage suffered by the structure due to the given synthetic ground motions.

Table 5.1 Parameters for fragility curve for different skew angles (Med. in [g])

	No skew				12.33° skew				45° skew			
	Column		Bearing		Column		Bearing		Column		Bearing	
Damage state	Med.	Var.	Med.	Var.	Med.	Var.	Med.	Var.	Med.	Var.	Med.	Var.
Slight	0.302	0.6	0.307	0.870	0.177	0.6	0.216	1.00	0.239	0.6	0.278	0.970
Moderate	0.351	0.6	0.804	0.850	0.210	0.6	0.66	0.98	0.283	0.6	0.736	0.811
Extensive	0.468	0.6	1.033	0.836	0.317	0.6	0.981	1.01	0.406	0.6	1.143	0.875
Complete	0.998	0.6	1.312	0.797	0.956	0.6	1.384	0.96	0.931	0.6	1.472	0.840

The fragility curve for all damage levels are calculated using the parameters obtained from the statistical analysis of damage data. Figure 5 shows exemplarily the fragility curve of the column for the 12.33° skew bridge and that of the bearing in Figure 6. The response of bearings and the columns are identical in transverse direction, since the bearings over piers are restrained transversely. The bearing fragility curves are plotted by considering the response in transverse direction. It can be observed from the fragility curves that the fragility of columns are higher than that of the bearing in all levels of damage.

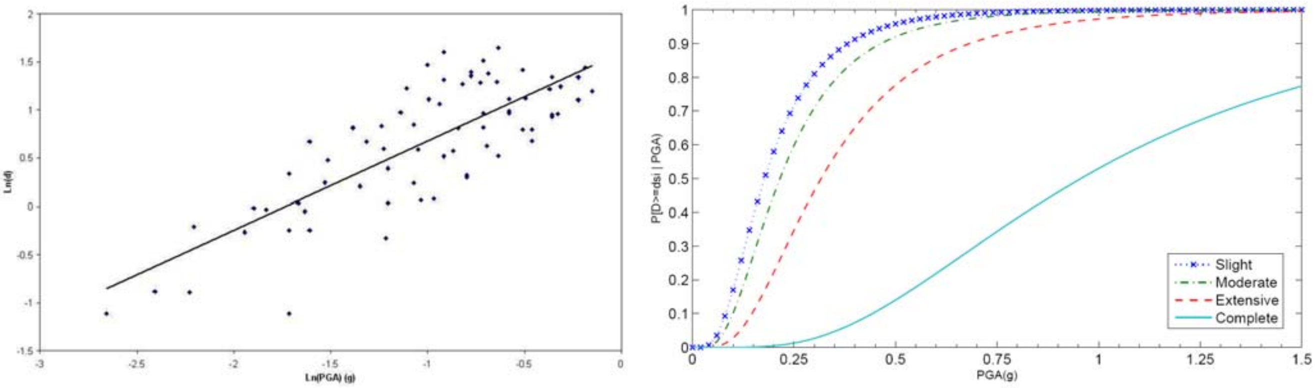


Figure 5 Linear regression of column for 12.33° skew bridge (l.), Fragility curve of column for 12.33° skew (r.)

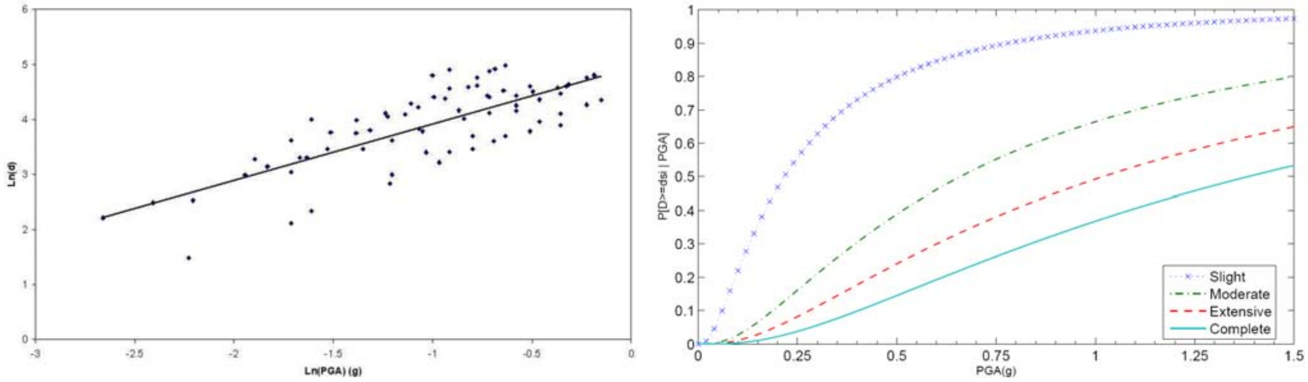


Figure 6 Linear regression of bearing for 12.33° skew bridge (l.), Fragility curve of bearing for 12.33° skew (r.)

The fragility curves and regression curves of the columns and bearings for the non-skew and 45° skew are shown in Figure 7 and 8.

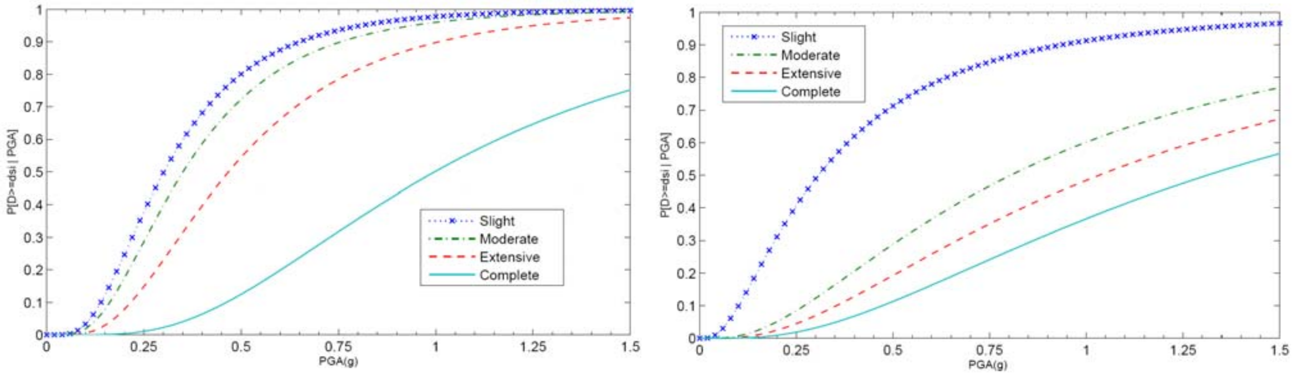


Figure 7 Fragility curve of column for no-skew (l.), Fragility curve of bearing for no-skew (r.)

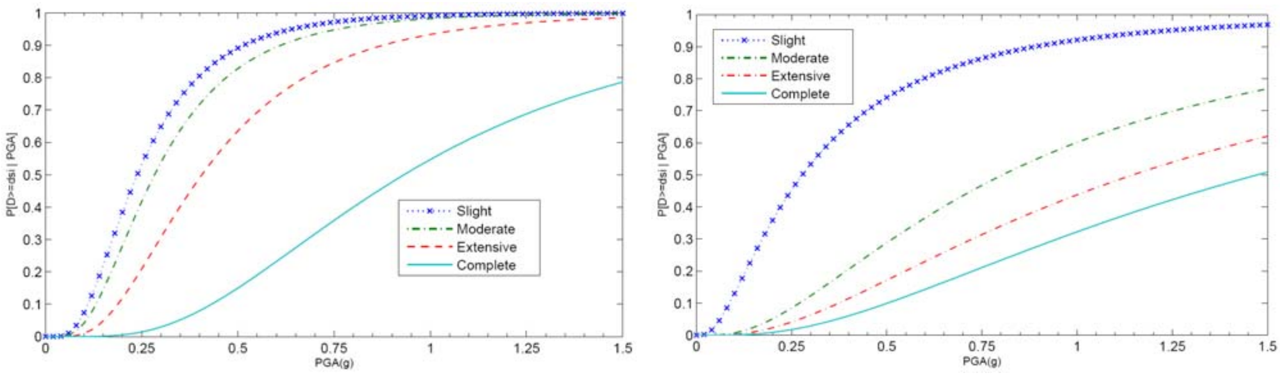


Figure 8 Fragility curve of column for 45° skew (l.), Fragility curve of bearing for 45°skew (r.)

As is well-known, in can be observed, that the seismic response of skewed bridges are higher than that of the non-skewed bridges in transverse direction. The orientation of the column stiffness affects the response in both longitudinal and transverse direction.

A comparison of the fragility curves in different damage states has been performed. The probability of getting damage increases with the increasing skew angle in transverse direction. Figure 9 shows the comparison of fragility curves for the column in different damage state. From this comparison, one can conclude that the fragility of the column is increase with the increasing skew angle in transverse direction. On the other hand for the longitudinal direction the influence of skew angle is reverse. The interested reader can look up the full set of results in the thesis of the first author (Pottatheere, 2007).

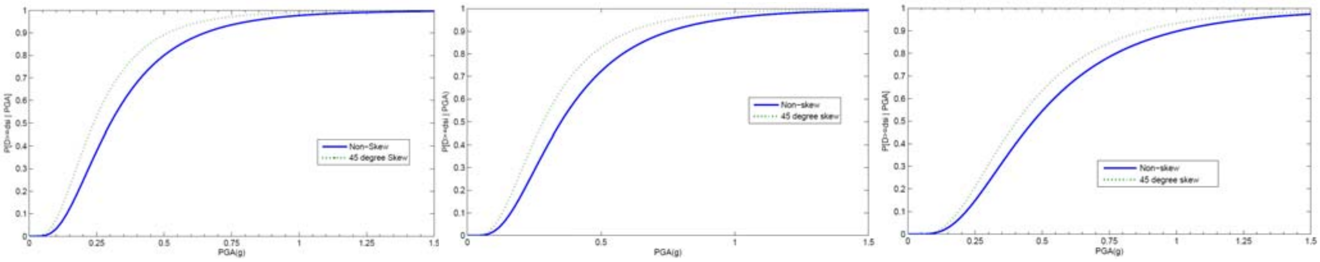


Figure 9 Comparison of column fragility curve for non-skew and 45° skew: Slight damage (l.), moderate damage (m.), extensive damage (r.)

6. CONCLUSIONS

Bridges are one of the most critical components in the transportation network. The seismic performance evaluation and the quantification of risk associated with the bridge structures is very important because its functionality may magnify the impact of natural disaster like seismic events. Skew bridges are commonly used as overpasses in highway intersection or interchanges. This bridge type is found to be more vulnerable during the seismic events than straight bridges. This case study represents a contribution to the better understanding of the seismic fragility curves for predicting the damage to skewed bridge structure.

This study investigated the seismic vulnerability assessment of skew bridges with different skew angles based on numerical simulation. The vulnerability study has been conducted by considering a sample highway skew bridge made of reinforced concrete. An analytical method for the generation of fragility curve have adopted in which the damage to the structure is derived from the response obtained from the push over analysis and time history analysis. The verification of the model has been done by comparing the moment curvature relationship of the column cross section with two different software packages. The structural damage has been derived from the static pushover and nonlinear time history analysis of the model. In this study the elastomeric bearing and bridge columns were found to be most vulnerable components. The fragility curve for both vulnerable components was generated using an analytical approach. From the comparison of the different skew angles, it can be conclude that the skew angle has a significant influence on the seismic vulnerability. The damage probability is increasing with the increase in skew angle from 0° to 45° with identical intensity measure (PGA) of earthquakes in transverse direction. The evaluation of the selected bridge shows that the bridge bend is extremely vulnerable to the seismic intensity range of 0.2 g to 0.4 g. But for bearings the range of seismic intensity which will cause severe damage is 0.45 g to 0.7 g.

In this study the pier foundation is assumed to be fixed at the base, and also the effect of the abutment on the overall response of the bridge has been neglected. The consideration of soil-structure interaction at abutments and piers, the movement joint characterization etc. could further improve the accuracy of the generated fragility curves. Furthermore, the bearing alignment over the abutment has a significant influence on the seismic response of skew bridges. This effect of alignment of the bearing stiffness on the fragility curve should further be studied.

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