Development of Analytical Seismic Fragility Curves for Ordinary Highway Bridges in Turkey

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SUMMARY:
Seismic fragility curves are conditional probability functions which give the probability of a bridge attaining or exceeding a particular damage level for an earthquake with a certain intensity level. In this study, analytical fragility curves are developed for the ordinary highway bridges in Turkey constructed after the 1990s. Bridges are first grouped into certain major bridge classes based on their structural attributes and sample bridges are generated to account for the structural variability. Nonlinear response history analyses are conducted for each bridge sample with their detailed 3-D analytical models under different earthquake ground motions having varying seismic intensities. Several engineering demand parameters are employed in the determination of seismic response of bridges. Fragility curves are obtained from the probability of exceeding each specified damage limit state for each major bridge class. Skew and single-column bent bridges are found to be the most vulnerable ones compared to the other bridge classes.

Keywords: Bridge, Vulnerability, Fragility Curve, Damage Limit State, Intensity Measure

1. INTRODUCTION

Damaging earthquakes have led to significant research in seismic risk analyses with the purpose of determining vulnerability of structures to find out the seismic risk resulting from the failure of the structures. Bridges that are one of the most important components of highways are among the critical structures that are considered in seismic risk analyses. Although, comprehensive research has been conducted in Turkey focusing on the development of fragility curves of buildings, no such studies have been carried out for bridges. Fragility curves of ordinary highway bridges in Turkey are necessary for the assessment of their seismic risk and vulnerability. Fragility curve, which is a fundamental component of seismic risk assessment methodology, is a probabilistic tool used to assess the potential seismic damage to highway bridges at a given seismic hazard level. As given in Eqn. (1.1), fragility function simply depicts the probability that the seismic demand imposed on the structure (D) is greater than or equal to the capacity of the structure (C_{LS}) for the investigated limit state (LS). This probability statement is conditioned on a selected seismic intensity measure (IM) representing the level of seismic action for a specific damage LS.

$$P(LS \mid IM) = P\left[(D \geq C_{LS}) \mid IM\right] \qquad (1.1)$$

Analytical fragility curves are employed for assessing the seismic performance of highway bridges when the actual bridge damage data or any expert opinion is not available. In this method, bridge analytical models are formed and ground motions with various intensity levels are considered for the seismic simulation of the bridge damage by executing numerous analyses. Fragility curves are highly sensitive to the choices made for the analysis method, structural idealization, seismic hazard, and damage state definitions (Kwon and Elnashai, 2006; Nielson and DesRoches, 2007).

Several analytical and empirical fragility curves have been developed for highway bridges especially in US and Japan (Karim and Yamazaki, 2003; Elnashai et al., 2004; Nielson and DesRoches, 2007). The available fragility curves are employed as vulnerability functions for the corresponding highway bridges, which do not represent the general properties of the highway bridges in Turkey. Moreover seismic source characteristics of the mentioned countries for which the fragility curves developed, and the seismic source properties of Turkey are not identical. Because of these reasons, performing seismic vulnerability assessment and loss estimation due to earthquake damage for ordinary highway bridges in Turkey considering existing fragility curves, which were developed for other regions especially in the US and Japan, can lead to unrealistic outcomes. Therefore in order to perform a reliable seismic vulnerability assessment of highway bridges, it is very important to have bridge fragility curves, representing the general attributes of the highway bridge structures as well as the seismic source characteristics of the bridge sites in Turkey. The main objective of the study is to generate analytical fragility curves of the ordinary highway bridges in Turkey constructed after the 1990s for the assessment of their seismic vulnerability. A comprehensive and original combination of modelling, analysis, damage state definition and the quantification of seismic vulnerability procedures have been used.

2. PROPERTIES AND CLASSIFICATION OF BRIDGES

In order to make a representative classification, a group of 52 bridges reflecting the general characteristics of the highway bridges constructed after the 1990s in different parts of Turkey were selected (Avşar et al., 2011). The group of bridges investigated can be defined as ordinary standard bridges according to Caltrans (2010). The investigated ordinary highway bridges in the inventory data are dominated by the multi-span simply supported bridges with cast-in-place continuous deck. C40 concrete class (the characteristic strength is 40 MPa) is used for the prestressed girders and C25 is used for the rest of the reinforced concrete bridge components. The quality of reinforcement steel is S420 (min yield strength = 420 MPa) for all RC members. Schematic drawings of a sample bridge and the components that constitute the general attributes of the bridges are shown in Figure 1.

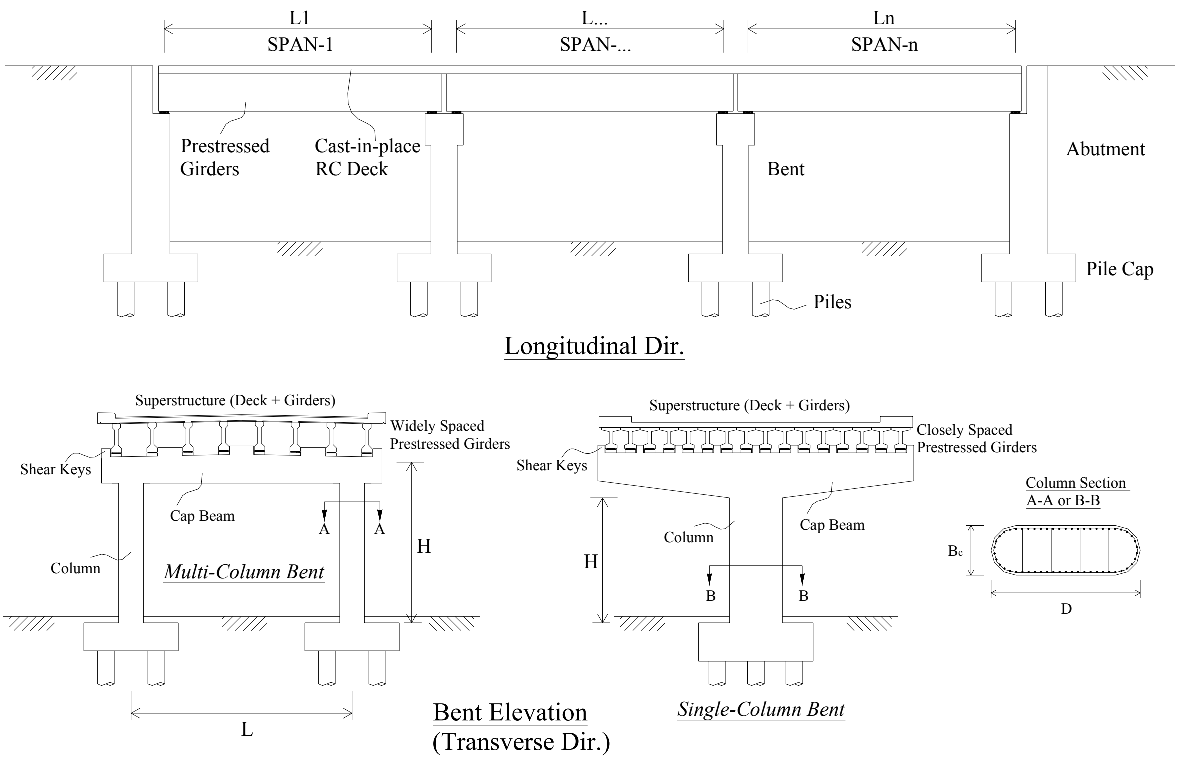

Figure 1. General properties of the ordinary highway bridges in Turkey

2.1. Major Bridge Classes and Sampling

Considering each bridge in the inventory individually and obtaining its fragility curve is neither feasible nor practical when the total number of bridges is concerned. Although each bridge has its own structural characteristics, they have some similarities at various aspects. Therefore, it is a rational way of classifying bridges into different groups considering their certain structural attributes. The classification is made such that the bridges representing a specific bridge class have some similarities in the basic structural attributes and their seismic response to the same earthquake ground motion is expected to be similar. Based on examination of data available from past earthquake reports and previous studies; span number, bent column number and skew angle were designated as the primary structural attributes for the associated bridge inventory data. The rest of the structural attributes are specified as the secondary structural attributes (Avşar et al., 2011). Single-span bridges are considered to be less vulnerable in comparison with MS bridges for seismic actions as per AASHTO LRFD (2007) and Nielson and DesRoches, 2007 and thus are not included in this study. Similar to previous studies (Basoz and Kiremidjian, 1997 and HAZUS (FEMA, 2003)), the classification according to column bent number is made as either single-column bent or multiple-column bent. The bridges are classified into two groups according to their skew angle: the bridges with negligible skew angle and the bridges with significant skew angle. In order to specify the two bridge types, a limiting skew angle value is required. In this study, the limit for skew angle was taken as 30°, which is also considered as per AASHTO (1996). Four major bridge classes presented in Table 1 were determined based on the primary structural attributes mentioned above.

Table 1. Major bridge classes

No.	Bridge Classes	Abbreviation
1	Multi Span_Multiple Column_Skew Less than 30°	MS_MC_SL30
2	Multi Span_Multiple Column_Skew Greater than 30°	MS_MC_SG30
3	Multi Span_Single Column_Skew Less than 30°	MS_SC_SL30
4	Multi Span_Single Column_Skew Greater than 30°	MS_SC_SG30

Bridge samples are generated by utilizing Latin Hypercube Sampling (LHS) method, which considers a constrained sampling approach instead of randomly selected samples (Ayyub and Lai, 1989). With the help of this method, instead of considering each structural attribute randomly, statistical distributions of the structural attributes are taken into account during selection. Statistical distributions of the primary and secondary structural attributes given by (Avşar et al., 2011) are employed for generating 10 bridge samples for each major bridge class.

3. ANALYTICAL MODELING

Comprehensive 3-D analytical models for each of the bridge components were developed in the OpenSees (2009) platform as shown schematically in Figure 2. Nonlinear response-history analyses (NRHA) have been conducted using the analytical model developed for each bridge sample under earthquake ground motions having varying seismic intensities. Superstructure is modelled using standard prismatic elastic beam elements and assumed to remain in the elastic range per Caltrans (2010). Nonlinear modelling of bent components of column and cap beam is achieved by using fiber-based nonlinear elements to represent the distributed plasticity along the member length. Each fiber on the RC section is represented by uni-axial stress-strain relationship for reinforcement steel, unconfined concrete and confined concrete. Effect of abutment and its backfill soil on the bridge system is modeled using the approach presented in Caltrans (2010) provisions.

Elastomeric bearings are simply placed in between the superstructure and substructure components without any dowel or connecting device. Therefore, the only resisting force holding the elastomeric bearing at its place against lateral loads is the friction force between the rubber and concrete surfaces. The ultimate shear capacity due to friction depends on the level of axial load on the elastomeric bearings and the dynamic coefficient of friction between the concrete surface and bearings, which is specified as 0.40 by Caltrans (2010). The behaviour of the elastomeric bearings is characterized by an elastic perfectly plastic model. Superstructure and substructure components of the highway bridges are not continuous in longitudinal and transverse directions and there exists joints with a certain gap in-between. The opening and closing of expansion joints between bridge components introduce nonlinearities and discontinuities that affect the load path and hence the dynamic response of bridges. Upon the closure of joints, pounding takes place between the adjoining bridge components, which is modeled by pounding elements. Springs are employed for representing the force-deformation relation of the above mentioned bridge components and their numerical values are given in Avşar et al., 2011.

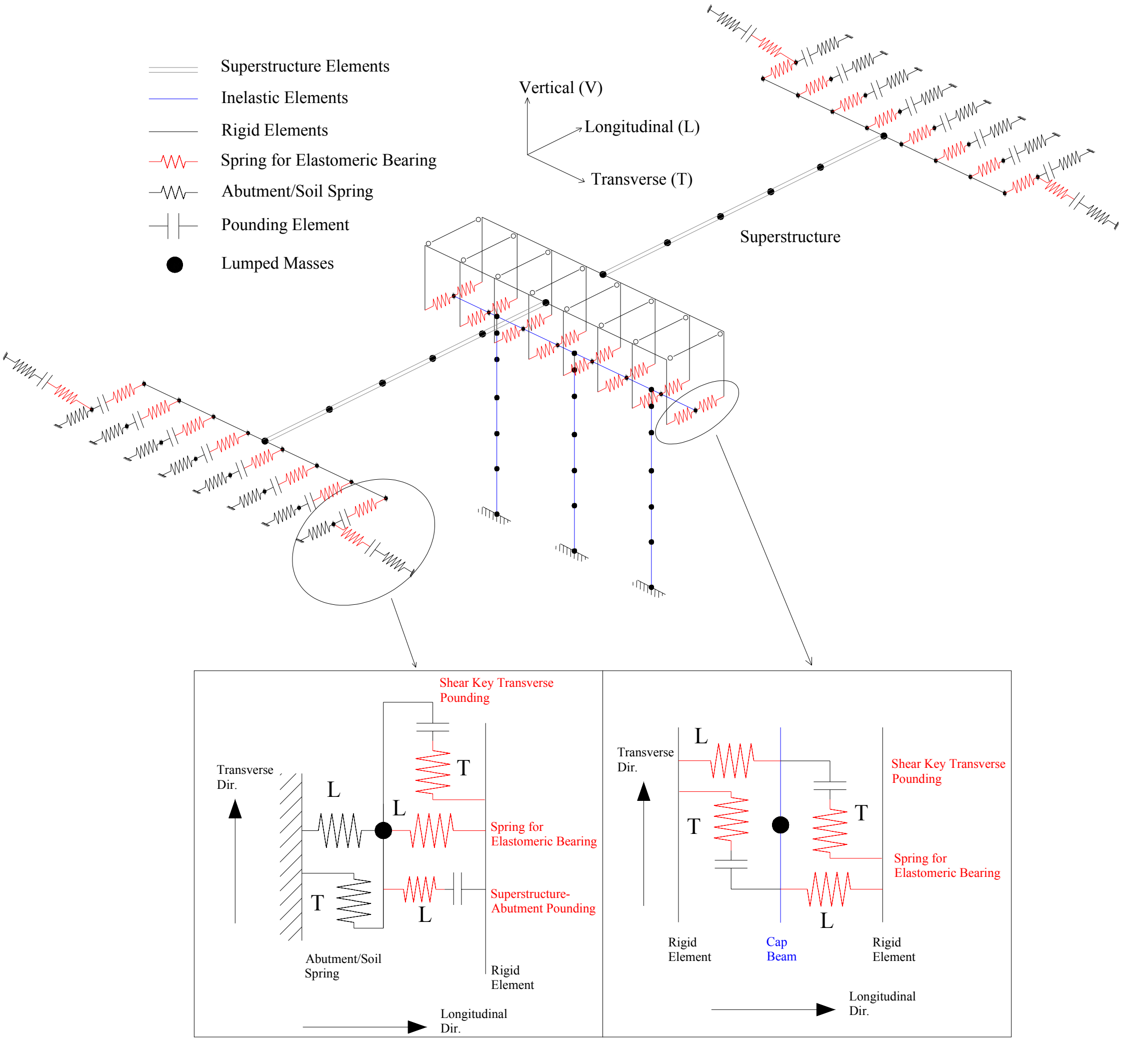

Figure 2. Detailed 3-D analytical model of the bridge and its components

4. EARTHQUAKE GROUND MOTION DATA SELECTION

The most important point in selecting the ground motions for fragility analysis is to compile a ground motion database representing wide range of seismic forces that impose various degrees of seismic damage on the bridges. A ground motion set that contains a total of twenty-five ground motions recorded in Turkey and in other regions having similar faulting mechanisms and seismic potential to Turkey is compiled without applying any scaling to represent the record-to-record variability. Moreover, the ground motion records having two horizontal orthogonal components are selected. Some of the important features of the earthquakes and several IM parameters of the ground motions are given in Avşar et al., 2011. The response spectra of all the selected ground motions and their mean are presented in Figure 3. The seismic hazard level of the earthquake ground motions can be represented by different ground motion intensity measures (IMs). The essential point in selecting the appropriate IM is that it should have a certain level of correlation with the seismic damage of highway bridges. In literature, the most commonly utilized IM for bridge fragility curves is PGA and to a lesser degree PGV. Spectral accelerations at certain periods are also employed in previous studies (HAZUS (FEMA), 2003; Nielson and DesRoches, 2007). Considering a single spectral acceleration can lead to unrealistic acceleration values the bridge is expected to experience due to higher mode effects and the period elongation due to inelastic response. Moreover, fragility curves are developed for a group of bridges whose fundamental periods is not unique among the representative bridge samples. Therefore, instead of dealing with a single period value, considering a period range over response spectra of the ground motions will be more reasonable. For this reason as a third IM, acceleration spectrum intensity (ASI) calculated from Eqn. 4.1 is employed (Avşar et al., 2011). T_i and T_f are defined as the initial and final periods and SA represents the 5 percent damped response spectrum. According to the modal analyses results of the sample bridges of major bridge classes employed herein the values of T_i=0.40s and T_f=1.10s are used for the ordinary highway bridges in Turkey (Avşar et al., 2011). T_i=0.40s is obtained from modal analysis results to account for the pre-yield fundamental period of the bridge samples as well as their higher mode periods. T_f=1.10s is selected to account for the post-yield period of the bridge samples. T_f=1.10s is calculated by averaging the elongated periods of the bridge samples after performing NRHA.

$$ASI = \int_{T_i}^{T_f} SA(T,\xi)dT \qquad (4.1)$$

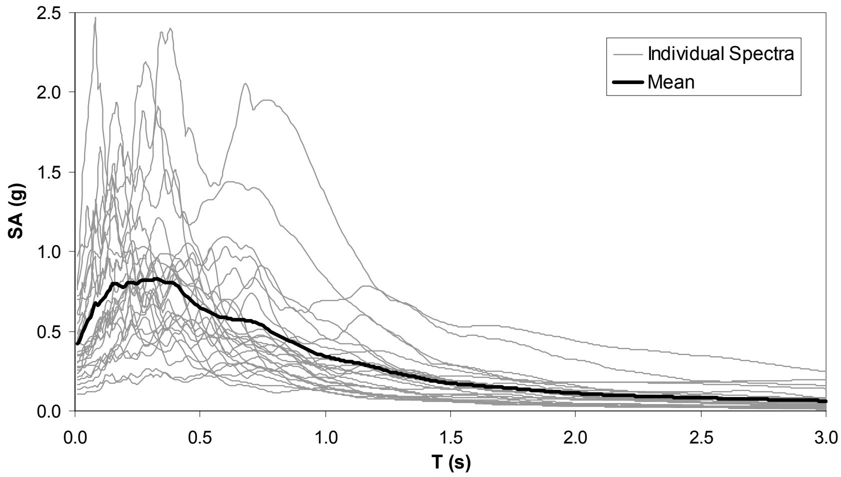

Figure 3. Response Spectrum of the selected 25 ground motions

5. METHODOLOGY USED FOR FRAGILITY CURVES

A specific methodology that relies on component based evaluation of the bridge models using NRHA is implemented. The following steps outline the methodology;

- Obtain 3D analytical model of each sample bridges and determine the response quantities for each component under each ground motion record.
- Define the damage limit states and corresponding demand parameters for all components.
- Determine the performance level of each component by comparing component demands from NRHA results with component damage limits expressed in terms of engineering demand parameters (EDPs). The EDPs employed in this study are: column and cap beam curvature, shear in both principal axes and deck displacement.
- Evaluate the global performance level of each bridge model for the given ground motion record.
- Determine the exceedance probabilities of each specified damage state for each ground motion.
- Plot the selected IM of the ground motion against the probability of exceedance for each

damage state and major bridge class to obtain the fragility points.
- Determine the fragility curves for each damage state and major bridge class by curve fitting the jaggedly varying fragility points through log-normal distribution functions characterized by median and dispersion.

A thorough discussion of the underlying concepts these steps rely on is given next.

5.1. Damage limit state definitions

A limit state can be defined as the ultimate point beyond which the bridge structure can no longer satisfy the specified performance level. Three damage limit states are termed as "serviceability" (LS-1), "damage control" (LS-2) and "collapse prevention" (LS-3) that are similar to the ones presented in TEC (2007). These damage states are specified in terms of deformations and seismic force demands for the local and global response parameters that are known as engineering demand parameters (EDPs). Therefore, capacity limits for each component EDPs are determined for each of the three damage limit states. The EDPs employed in this study are the column and cap beam curvature, shear in both principal axes and the superstructure displacement. Shear failure is a brittle type of failure without exhibiting any sign of damage before failure; member failure takes place suddenly when the shear capacity of the RC sections is exceeded by the seismic shear demand. Therefore, only collapse prevention limit state (LS-3) is defined for the shear capacity of columns and cap beams according to Caltrans (2010). For the remaining EDPs, limiting values are specified for each damage limit states. From the results of moment curvature analysis by Avşar et al., 2011, three limit curvature values have been specified for each damage limit state of the column and cap beams, which are expected to behave in the inelastic range. Section yield point determined from bilinear moment-curvature curve corresponds to the serviceability damage limit state at which column and cap beams have some minor repairable cracks. The damage control limit state is defined as section curvature at which the concrete cover spalling occurs (Priestley et al. 1996) at which these members can be repaired without closures of traffic. The ultimate curvature capacity of the column and cap beam sections is considered to be the collapse prevention damage limit state at which significant repair is required with traffic closure to service use with some exceptions for emergency use.

The displacement capacity of the bearings, beyond which the friction force is exceeded by the seismic forces, is accepted as the ultimate bearing displacement for defining the serviceability limit state (LS-1). Damage control limit state (LS-2) is specified for the displacement when the superstructure falls over pedestal on the cap beam. Finally, when the superstructure displacement exceeds the available seat length provided by the cap beam, it will fall over the bent and total collapse occurs, which is specified as the collapse prevention limit state (LS-3). Damage limits for each EDPs and major bridge classes are determined as discussed by Avşar et al., 2011.

5.2. Determination of Bridge Fragility

Maximum response of the bridge components are calculated by taking the absolute maximum of the response history of each defined EDPs. Seismic damage state of the bridge components under each ground motion is determined by comparing the corresponding threshold values of the damage limit states and the maximum seismic response of the bridge components. Since there does not exist any specific method that relates the overall bridge damage to the damage state of its components, a simple assumption is made for identifying the bridge damage state as a whole. If any of the bridge components attains or exceeds a damage limit state, bridge system as a whole is assumed to be in the same damage state regardless of the damage states of the rest of the bridge components. In this method, a series system for the bridge is assumed. This method is a conservative approach in the determination of overall bridge damage. Because correlation among the bridge component damage states and its influence on the overall bridge damage are not taken into account.

An example for the damage state assessment of a sample bridge is presented in Table 2. In the table, the parameters are given according to their section local axis in terms of 33 (strong axis) and 22 (weak axis). "K" and "V" represent the curvature and shear for the RC members, respectively. If the most critical bridge component has reached or exceeded a certain damage limit state, then the score of the bridge component for that limit state is assumed to be 1, otherwise 0. According to the assumption made in identifying the bridge damage state, if any of the bridge component has the score of 1, then the whole bridge is assumed to be in that damage state with the score of 1. That is the damage state of the whole bridge is dictated by the damage state of the most severely damaged component.

Table 2. Determination of the damage state of the bridges

		Serviceability Limit State (LS-1)				
EQ#	Intensity Measure (ASI, PGV, PGA)	Col. K33	Col. K22	Cap K33	Deck Disp.	**OverAll**
EQ-1	IM-i	1	1	0	1	**1**
EQ-2	IM-i	0	1	1	1	**1**
-	-	-	-	-	-	**-**
-	-	-	-	-	-	**-**
EQ-N	IM-i	1	0	1	1	**1**

		Damage Control Limit State (LS-2)				
EQ#	Intensity Measure (ASI, PGV, PGA)	Col. K33	Col. K22	Cap K33	Deck Disp.	**OverAll**
EQ-1	IM-i	1	0	0	0	**1**
EQ-2	IM-i	0	0	0	0	**0**
-	-	-	-	-	-	**-**
-	-	-	-	-	-	**-**
EQ-N	IM-i	0	0	1	0	**1**

		Collapse Prevention Limit State (LS-3)							
EQ#	Intensity Measure (ASI, PGV, PGA)	Col. K33	Col. K22	Cap K33	Col. V2	Col. V3	Cap V2	Deck Disp.	**OverAll**
EQ-1	IM-i	1	0	0	1	0	0	0	**1**
EQ-2	IM-i	0	0	0	0	0	0	0	**0**
-	-	-	-	-	-	-	-	-	**-**
-	-	-	-	-	-	-	-	-	**-**
EQ-N	IM-i	0	0	0	0	0	0	0	**0**

0 = NOT Attained the Specified Damage Limit State
1 = Attained the Specified Damage Limit State

Damage states of each bridge sample in the four major bridge classes are identified under the selected ground motions. For a selected ground motion record with a certain IM value, the number of bridge samples that reached or exceeded a specified damage limit state is obtained. The IMs are calculated by taking the geometric mean of the two horizontal components of the ground motions. The ratio of the number of sample bridges, which reached or exceeded the specified damage limit state, to the total number of sample bridges gives the probability of exceeding the corresponding limit state of the bridge class for the investigated earthquake. After performing the same assessment for each earthquake ground motion in the set and for the three specified damage limit states, probability of exceeding the damage limit states is obtained for each earthquake consequently for each IM. Since fragility curves are developed for bridge classes, evaluation of the results of bridge samples is made for each bridge class separately.

When earthquake ground motions are represented with an appropriate seismic IM, distribution of exceeding probabilities with respect to the selected IM is obtained as schematically shown in Figure 4. In this graph, x-axis is the seismic IM of the ground motion and y-axis is the probability of exceedance of a certain damage limit state. In the seismic loss estimation studies, continuous functions of fragility curves will be more convenient in the calculations instead of jaggedly varying fragility points. Therefore, a mathematical expression is utilized to characterize the jaggedly varying exceedance probability points to achieve smooth fragility curves for a specific damage limit state and bridge class. A representative sketch is shown Figure 4 illustrating a function that is the best fit for the exceedance probability points. Fragility curves for all bridge classes are modelled as lognormally-distributed functions that give the probability of reaching or exceeding different damage states for a given level of ground motion. Each fragility curve is characterized by a median value and an associated dispersion factor (lognormal standard deviation) of ground motion, which is represented by seismic IMs.

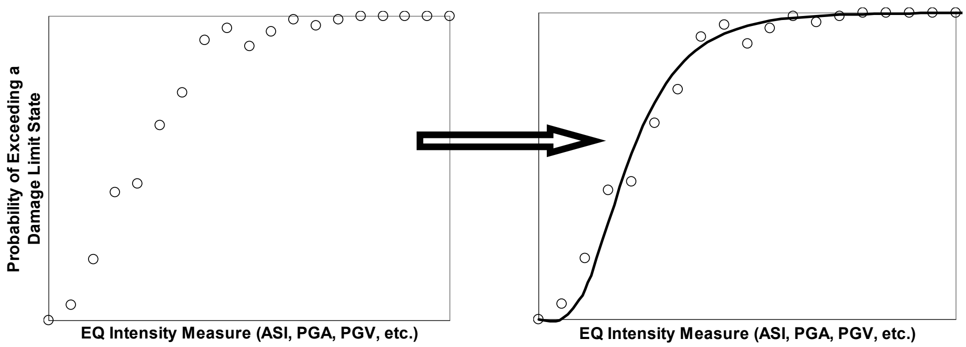

Figure 4. Schematic representation of a fragility curve

The median and the dispersion values of the cumulative lognormal probability distribution function are determined by employing the least squares technique to the exceedance probability points. Besides, to investigate the correlation between the exceedance probability points and the developed fragility curves, the coefficient of determination (R^2) is computed for each individual fragility curve and presented in Table 3. When the coefficient of determination values calculated for each IM is investigated, it is found out that ASI has the highest and PGA has the lowest R^2 values. This implies that fragility curves developed using ASI have better correlation with the corresponding exceedance probability points in comparison with the other IMs.

Table 3. Fragility curve parameters of the bridge classes

MS_MC_SL30									
Intensity	LS-1: Serviceability			LS-2: Damage Control			LS-3: Collapse Prevention		
Measure	Median	Disp.	R^2	Median	Disp.	R^2	Median	Disp.	R^2
ASI (g*s)	0.121	0.401	0.758	0.592	0.290	0.748	0.693	0.308	0.902
PGV (cm/s)	11.238	0.454	0.299	59.678	0.573	0.569	72.287	0.628	0.619
PGA (g)	0.117	0.400	0.121	0.693	0.280	0.296	0.869	0.316	0.361

MS_MC_SG30									
Intensity	LS-1: Serviceability			LS-2: Damage Control			LS-3: Collapse Prevention		
Measure	Median	Disp.	R^2	Median	Disp.	R^2	Median	Disp.	R^2
ASI (g*s)	0.137	0.366	0.843	0.497	0.272	0.777	0.623	0.309	0.721
PGV (cm/s)	10.914	0.423	0.235	49.109	0.532	0.501	62.887	0.570	0.469
PGA (g)	0.094	0.500	0.128	0.583	0.350	0.176	0.756	0.380	0.205

MS_SC_SL30									
Intensity	LS-1: Serviceability			LS-2: Damage Control			LS-3: Collapse Prevention		
Measure	Median	Disp.	R^2	Median	Disp.	R^2	Median	Disp.	R^2
ASI (g*s)	0.133	0.381	0.779	0.438	0.389	0.846	0.593	0.368	0.937
PGV (cm/s)	11.083	0.354	0.307	44.434	0.486	0.602	57.340	0.529	0.643
PGA (g)	0.110	0.450	0.131	0.577	0.400	0.144	0.741	0.480	0.207

MS_SC_SG30									
Intensity	LS-1: Serviceability			LS-2: Damage Control			LS-3: Collapse Prevention		
Measure	Median	Disp.	R^2	Median	Disp.	R^2	Median	Disp.	R^2
ASI (g*s)	0.123	0.346	0.804	0.347	0.400	0.826	0.508	0.385	0.900
PGV (cm/s)	10.090	0.386	0.323	33.049	0.444	0.655	47.656	0.535	0.740
PGA (g)	0.100	0.420	0.124	0.482	0.360	0.223	0.613	0.400	0.218

Developed analytical fragility curves are grouped separately in Figure 5 for the three damage limit states (LS-1, LS-2, and LS-3) and three IMs (ASI, PGV, and PGA) to compare the effect of different bridge classes on the fragility curves. Bridge classes with larger skew are more vulnerable to seismic effects than the bridges with small skew angles. Bridges that fall into the bridge classes of skew greater than 30° have the fragility curve resulting higher probability of exceeding values in comparison with the fragility curves of bridge classes for skew angle less than 30°. This outcome is consistent with the response of the bridges observed in the Loma Prieta and Northridge Earthquakes (Buckle, 1994; Basoz and Kiremidjian, 1997). Bent column number also has a considerable effect on the fragility curves. Single-column bents are found to be more vulnerable compared to the multiple column bents. This finding is in accordance with the performance of bridges during the Loma Prieta and Northridge Earthquakes. Basoz and Kiremidjian (1997) mentioned that, bridges with single-column bent performed poorly during these earthquakes.

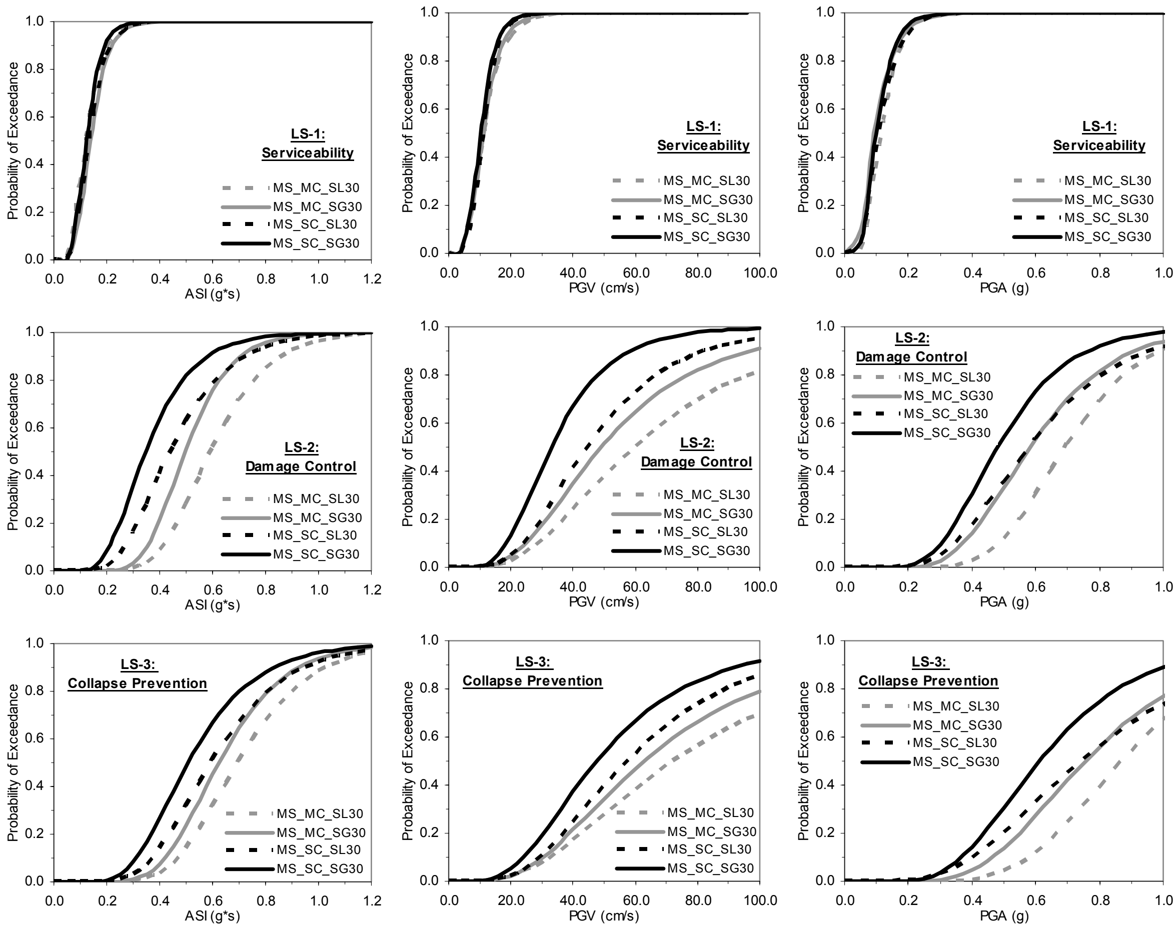

Figure 5. Fragility curves for different damage limit states and IMs (ASI, PGV, and PGA)

The effect of skew and bent type on the fragility curves depends on the GM IM. For instance when Figure 5 is investigated, the difference between the fragility curves at lower and higher IM values is negligible whereas in the intermediate IM values the difference is more pronounced. The relative relation among the three IMs considering the damage state exceeding probability depends on the major bridge types employed in the study. The difference between the fragility curves of all the bridge classes for the serviceability damage limit state is negligible regardless of the IM considered. Reaching or exceeding the serviceability damage limit state mostly occurs when the superstructure displacement exceeds the specified displacement limit, at which the friction force between the bearings and concrete surfaces can no longer hold the elastomeric bearing at its place. A single fragility curve can be utilized for all bridge classes for the serviceability limit state. This finding is in good agreement with the HAZUS (FEMA 2003) fragility curves. In HAZUS, a modification factor is employed for the skew of the bridges in the determination of fragility curves for the moderate, extensive, and complete damage limit states. Whereas, for the slight damage limit state no modification factor is considered. Namely, same fragility curve is considered for the fragility curve of slight damage limit state of the bridges having different skew angles.

6. CONCLUSIONS

In this study, analytical fragility curves were developed for the ordinary highway bridges in Turkey constructed after the 1990s. The following conclusions have been drawn:

- The most significant contribution of this study is the development of fragility curves for certain bridge classes common in the highway transportation system in Turkey. They can be used to determine the seismic risk associated with existing ordinary highway bridges in Turkey.

- Fragility curves of the highway bridges are developed for three damage limit states. Fragility curve for the Serviceability damage limit state is mostly governed by the superstructure relative displacement. Whereas, curvature demands of the column and cap beam dominate the fragility curves for the Damage Control and Collapse Prevention damage limit states. Developed fragility curves are original for the Turkish highway bridges in terms of the applied methodology as well as the acceptance criteria employed in the calculation procedure.
- When the fragility curves of the major bridge classes are investigated, it is found that the skew and single-column bent bridges exhibit higher vulnerability compared to the non-skew and multiple-column bent bridges. This result is in accordance with the past earthquake experiences.
- Among the investigated ground motion IMs (ASI, PGV, PGA), ASI and PGV appear to be the ones that have better correlation with the seismic damage of the bridge components due to the observed high reliability in presentation of the fragility curves. Therefore, the generated fragility curves based on ASI or PGV are found to be more realistic in the estimation of bridge damage states.
- The difference between the fragility curves of all the bridge classes for the serviceability damage limit state (LS-1) is almost invariant regardless of the IM considered. Effect of bridge skew angle or bent column number on the fragility curve for serviceability limit state is found to be insignificant. Therefore, a single fragility curve can be utilized for all bridge classes for the serviceability damage limit state.
- When the horizontal component of the seismic force is greater than the friction force, which is the case even at lower ground motion intensities, “walk-out” phenomenon takes place and superstructure starts to move. This can cause permanent displacement of the superstructure affecting the functionality of the bridge. Therefore, proposed fragility curves for the serviceability damage limit state result in higher probability of exceedance values.

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