Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 380

EVALUATION OF GROUND MOTION INTENSITY MEASURES FOR THE FRAGILITY CURVES OF ORDINARY HIGHWAY BRIDGES IN TURKEY

O. Avsar[1] and A. Yakut[2]

ABSTRACT

Four different ground motion (GM) intensity measures (IMs) have been considered to find out their correlation with the seismic damage of the ordinary highway bridges in Turkey. The investigated GM IMs are the peak ground acceleration (PGA), peak ground velocity (PGV), PGA/PGV and acceleration spectrum intensity (ASI). A number of analytical bridge models were generated to represent the typical features of the ordinary highway bridges in Turkey. After the development of 3D analytical models, nonlinear response history analyses of these bridges were performed under a suit of GMs considering biaxial seismic excitation. Superstructure displacement and column curvature in both principal axes were considered to be the engineering demand parameters for quantifying the bridge seismic damage. Engineering demand parameters were compared with the corresponding ground motion intensity parameters and correlation between them were investigated through coefficient of determination to realize the most appropriate IMs to be utilized in the development of analytical fragility curves. The results revealed that IMs of ASI and PGV have a better correlation with the seismic damage of highway bridges in comparison with PGA and PGA/PGV.

Introduction

One of the key components of seismic risk assessment for the highway bridges is fragility curves that represent the seismic damage imposed on the bridge in terms of the selected GM IM. These vulnerability curves are conditional probability functions which give the probability of a bridge attaining or exceeding a particular damage level for an earthquake of a given intensity level as shown in Fig. 1. Fragility curves are developed for a certain group of structures having similar structural attributes. The variability in the structural parameters of the bridges and damage state definitions as well as the uncertainty in the GM parameters make the development procedure of the bridge fragility curves a very challenging task. Among various parameters, uncertainties involved with the parameters of earthquake GM are the most influential ones affecting the reliability of the fragility curves considerably (Kwon and Elnashai 2006).

[1]Civil Engineer, Earthquake Research Dept., General Directorate of Disaster Affairs, Ankara, Turkey
[2]Assoc.Prof. Dr., Dept. of Civil Engineering, Middle East Technical University, Ankara, Turkey

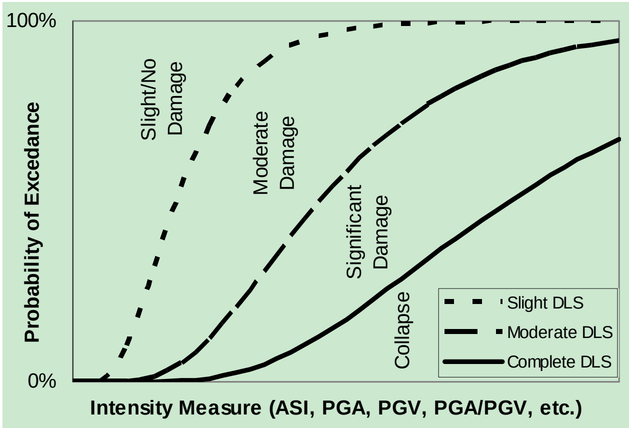

Figure 1. A sample fragility curve for different damage limit states.

In addition to the uncertainty involved in the selection of GM records, selection of GM IM, which is the abscissa of vulnerability functions (Fig. 1), has a direct influence on the development of fragility curves. Therefore, choice of GM IM has a considerable effect on the reliability of fragility curves. GM IMs represent the seismic hazard level of the selected earthquake GMs. Different IMs can be employed in the development of fragility curves. While some of the intensity parameters can be easily determined from GM records, some others are computed through equations considering structural properties. The essential point in selecting the appropriate IM is that it should have a certain level of correlation with the seismic damage of bridges. The main objective of this study is to investigate the correlation between seismic damage of the highway bridges in Turkey with different GM IMs such as PGA, PGV, PGA/PGV and ASI. Sample highway bridges were generated from bridge inventory data and their analytical models were developed to be used for the nonlinear response history analyses under a suit of GMs considering biaxial seismic excitation. GM IMs were compared with respect to several engineering demand parameters, which are utilized to quantify the seismic damage of highway bridges. GM IMs having better correlation with the bridge seismic damage were recommended for the development of fragility curves.

Description and Sampling of Bridges

Considering each bridge in the inventory data of the specified bridges individually and obtaining its fragility curve is neither feasible nor practical when the total number of bridges is concerned. Therefore, bridges were classified 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 GM is expected to be similar. For this purpose, a group of 52 bridges representing the general characteristics of the ordinary highway bridges constructed after the 1990s in different parts of Turkey were selected. Schematic drawings of a sample bridge and its components that constitute the general attributes of the bridges are shown in Fig. 2. Number of spans, number of bent columns and skew angle of the bridges are considered as the primary structural attributes affecting the seismic behavior of the highway bridges considerably. Span length, total length, column height, column aspect ratio, cap beam to column inertia ratio, seat length, girder spacing, deck width, etc. are considered to be the secondary structural attributes of

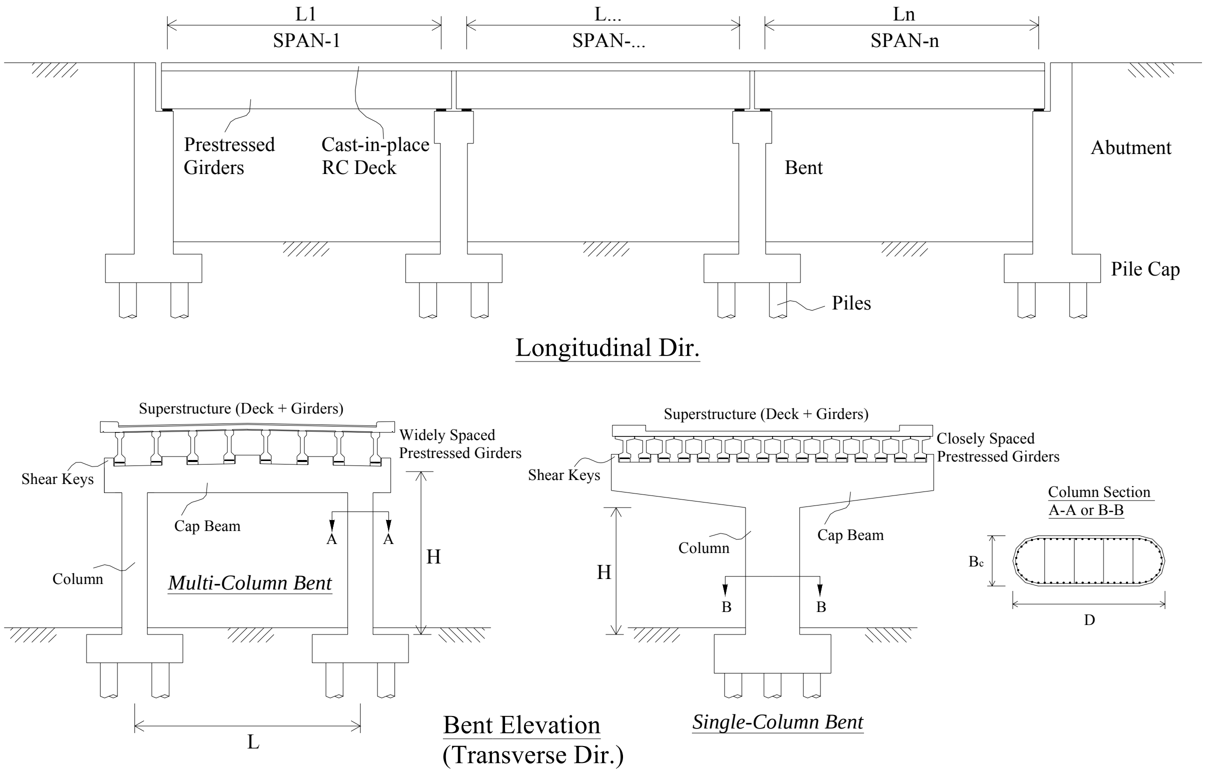

Figure 2. General properties of the ordinary highway bridges in Turkey.

highway bridges. Major bridge classes were developed based on the primary structural attributes of the bridges. The secondary structural attributes were considered for the generation of bridge samples, which are representative for each major bridge class. Bridge sampling was made by using Latin Hypercube Sampling Technique (Ayyub and Lai 1989) together with the distribution of each structural attribute in the inventory data.

Overall structural displacements, member forces, and local deformations of the bridge samples were analytically determined using mathematical models and the analysis tool to quantify the seismic response of the bridges. Detailed 3D analytical models of each bridge sample were developed in the OpenSees platform. Further information can be found in the study of Avsar (2009). Maximum seismic demand of each bridge sample was determined by performing nonlinear response history analysis under a suit of selected GM records.

Ground Motion Intensity Measures

The most commonly utilized IM for bridge fragility curves is PGA and to a lesser degree PGV. One of the main reasons for PGA and PGV to be the most common IMs is that they can be simply obtained from GM records without any additional information about structural properties to be used in the calculation. Several earlier studies were conducted to compare the efficiency of different IMs for estimating the seismic damage with a certain level of confidence mostly for the building structures (Akkar et al. 2005; Yakut and Yılmaz 2008). The most important criterion in selecting an appropriate IM for fragility curves is that it should provide sufficient level of correlation between the degree of seismic damage sustained by the bridge and the hazard level of the GM.

In this study four different IMs were considered and their correlation with the seismic damage of the bridges was investigated. PGA and PGV are the two IMs that were considered in the calculations because of their common application in the earthquake engineering. Also, great majority of the available fragility curves were obtained using the two IMs. Additionally, PGA/PGV ratio was also regarded as a seismic IM. According to Kramer (1996), dominant frequency and energy content of the earthquake GMs can be represented by PGA/PGV ratio. Priestley et al. (1996) and Kwon and Elnashai (2006) mentioned that PGA/PGV ratio implicitly accounts for many seismo-tectonic features and site characteristics of earthquake GM records. Single period spectral acceleration was not considered because of higher mode effects and the period elongation due to inelastic response of the bridges. Instead of dealing with a single period value, considering a period range over response spectra of the GMs will be more reasonable. Moreover, fragility curves are developed for a group of bridges whose fundamental periods is not unique among the representative bridge samples. Acceleration spectrum intensity (ASI), which is the area under 5% damped elastic response spectrum within the boundary periods of T_i and T_f, was the fourth IM considered in this study. The definition of ASI is also presented schematically in Fig. 3. Von Thun et al. (1988) expressed the ASI as the area under the 5% damped elastic pseudo-acceleration spectrum between the periods of T_i=0.1s and T_f=0.5s. ASI was utilized as an IM for the seismic analysis of concrete dams, which generally have fundamental periods of less than 0.5s. For buildings, Yakut and Yılmaz (2008) mentioned that ASI correlate better with the response of building structures for the period range of T_i=0.1s and T_f=2.0s. It is obvious that the reliability of the ASI is highly dependent on the selection of period ranges T_i and T_f.

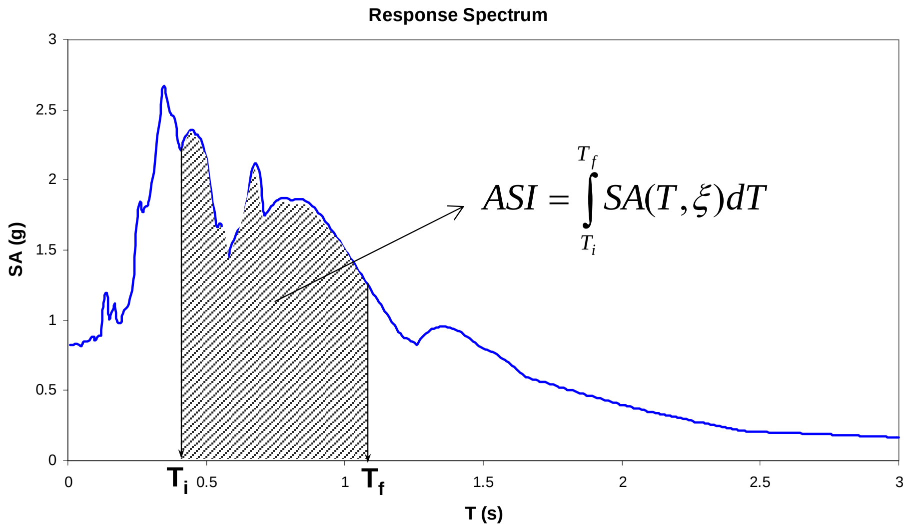

Figure 3. Definition of ASI.

According to the modal analyses results of the sample bridges of major bridge classes, fundamental period values vary between 0.47s and 0.98s. These values were not used for the initial and final periods. In order to consider the higher mode effects a lower value of T_i was selected as 0.40s. After performing some sensitivity analyses, average period elongation of the sample bridges due to their inelastic response to seismic actions was computed as 1.10s on the average (Avsar 2009). Finally, it was assumed that ASI is determined considering the period range T_i=0.40s and T_f=1.10s for most of the ordinary highway bridges in Turkey.

Description of Ground Motion Database

A total of 25 individual unscaled GMs recorded at firm soil sites (Vs≥360m/s) and earthquakes having a strike-slip faulting mechanism were used in this study. The list of GM records and some of their important features and the investigated IMs are presented in Table 1. The main purpose in selecting the GM records is to compile a GM database representing wide range of seismic forces that impose various degrees of seismic damage on the highway bridges in order to represent the record-to-record variability in the fragility curves. If this can be accomplished, sufficient number of data points can be provided with a uniform distribution along the abscissa of the fragility curve. Otherwise, if the selected GMs impose similar seismic damage on the bridges, variation in the bridge seismic demands that are calculated from nonlinear response history analyses will be limited.

Both horizontal components of the selected GM records are used in the nonlinear response history analyses for biaxial excitation. Baker and Cornell (2006) mentioned that earth scientists typically use the geometric mean of the IM of the two horizontal components of GM for hazard analysis. In this study, IMs of each GM was obtained by calculating the geometric mean of IMs of the two horizontal components. Similarly, response spectrum of each GM was calculated by taking the geometric mean of the response spectrum of the two horizontal components. The response spectra of all the GMs and their mean are presented in Fig. 4.

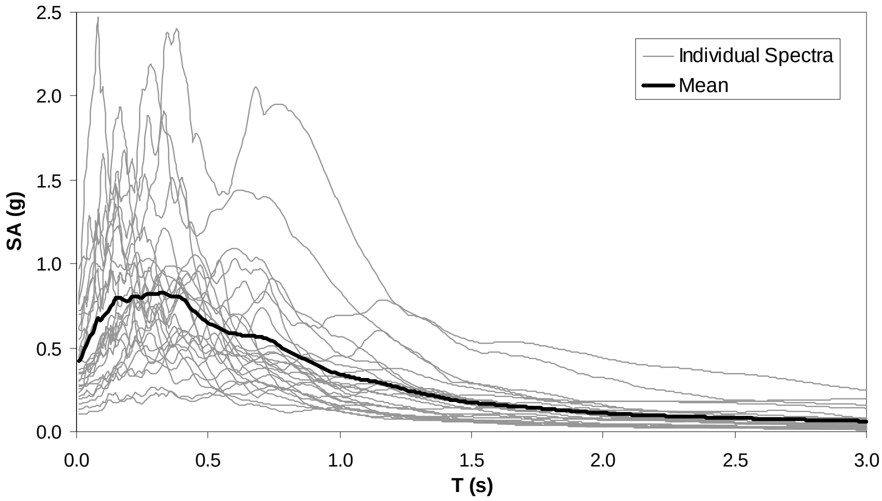

Figure 4. Response spectrum of the selected 25 GMs.

Table 1. Some important parameters of the selected 25 earthquake GMs.

Earthquake, Date	Station	Mw	D (km)	ASI (g*s)	PGA (g)	PGV (cm/s)	PGA/PGV (1/s)
Parkfield, 2004/09/28	Parkfield, CA - Gold Hill 3W; CSMIP station 36420	6.0	3.9	0.140	0.532	18.71	27.87
Landers, 1992/06/28	23559 Barstow	7.3	36.1	0.157	0.133	23.77	5.51
Parkfield, 1966/06/28	1438 Temblor pre-1969	6.1	9.9	0.161	0.312	18.0	17.02
Parkfield, 2004/09/28	Parkfield, CA - Cholame 2E; CSMIP station 36230	6.0	14.5	0.172	0.469	22.51	20.43
Landers, 1992/06/28	33083 Boron Fire Station	7.3	90.6	0.178	0.103	11.13	9.12
Coyote Lake, 1979/08/06	57217 Coyote Lake Dam (SW Abut)	5.7	3.2	0.187	0.209	14.81	13.87
Duzce, 1999/11/12	375 Lamont 375	7.1	8.2	0.249	0.706	27.15	25.51
Morgan Hill, 1984/04/24	57383 Gilroy Array #6	6.2	11.8	0.252	0.255	20.45	12.21
Parkfield, 2004/09/28	Parkfield, CA - Cholame 3E; CSMIP station 36450	6.0	14.8	0.260	0.620	25.24	24.08
Landers, 1992/06/28	5071 Morongo Valley	7.3	19.3	0.270	0.162	18.3	8.69
Parkfield, 2004/09/28	Parkfield, CA - Fault Zone 7; CSMIP station 36431	6.0	1.7	0.271	0.241	19.5	12.10
Westmorland, 1981/04/26	5051 Parachute Test Site	5.8	24.1	0.282	0.194	32.3	5.88
Denizli, 1976/08/19	Denizli Directorate of Meteorology	5.0	67.6	0.283	0.300	19.3	15.23
Bingol, 2003/05/01	Bingol Dir. of Public Works and Settlement	6.1	4.9	0.284	0.396	28.37	13.67
Landers, 1992/06/28	24 Lucerne	7.3	1.1	0.305	0.752	55.80	13.23
Coyote Lake, 1979/08/06	57383 Gilroy Array #6	5.7	3.1	0.346	0.370	34.72	10.46
Morgan Hill, 1984/04/24	1652 Anderson Dam (Downstream)	6.2	2.6	0.364	0.350	26.42	12.98
Victoria, 1980/06/09	6604 Cerro Prieto	6.4	34.8	0.383	0.604	25.1	23.62
Landers, 1992/06/28	23 Coolwater	7.3	2.1	0.416	0.344	32.9	10.24
Landers, 1992/06/28	22170 Joshua Tree	7.3	11.6	0.425	0.279	34.47	7.94
Superstitn Hills, 1987/11/24	286 Superstition Mtn.	6.7	4.3	0.528	0.781	37.03	20.68
Superstitn Hills, 1987/11/24	5051 Parachute Test Site	6.7	0.7	0.549	0.414	70.12	5.79
Parkfield, 2004/09/28	Coalinga, CA - Slack Canyon; Hidden Valley	6.0	32.1	0.552	0.271	36.42	7.29
Morgan Hill, 1984/04/24	57217 Coyote Lake Dam (SW Abut)	6.2	0.1	0.819	0.961	64.57	14.60
Kobe, 1995/01/16	0 KJMA	6.9	0.6	1.169	0.701	77.72	8.85

Seismic Response Calculation

Numerous nonlinear response history analyses of highway bridges were performed using their 3D analytical models under the selected suit of GMs through biaxial seismic excitation in the OpenSees platform. Seismic damage of highway bridges was determined by considering the engineering demand parameters of column curvature in both principal axis and superstructure longitudinal displacement. Maximum response of the bridge components were calculated by taking the absolute maximum of the response time history of the corresponding engineering demand parameters, which was obtained from the results of nonlinear response history analyses. A schematic representation for determining the maximum seismic response of bridge components in terms of different engineering demand parameters are presented in Fig. 5. Φ22 and Φ33 represent the column curvature in weak and strong axis, respectively.

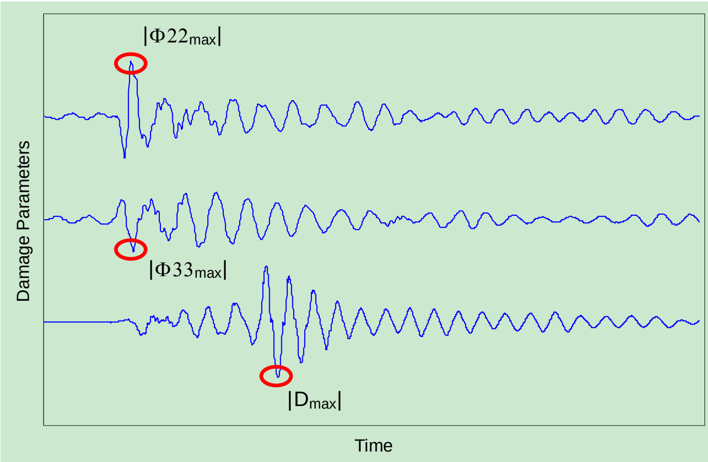

Figure 5. Schematic representation of max seismic response of engineering demand parameters.

Discussion of Results

Comparisons of highway bridge seismic damage with the investigated IMs are made in the same graph for each engineering demand parameter and for each bridge sample. Since each bridge sample has its own structural attributes in order to represent the structural variability of the fragility curves, it is not feasible to compare the analyses results of each bridge sample together. To be consistent, seismic response of each engineering demand parameter of bridge samples were normalized according to their serviceability damage limit states defined by Avsar (2009). Section yield curvature was specified as the serviceability damage limit state for the curvature demand of reinforced concrete sections. Exceedance of the friction force between the concrete surfaces and the bearings was specified as the serviceability damage limit state for the superstructure longitudinal displacement. Since there is no connecting device between concrete components and bearings in the existing highway bridges, friction force is the only resisting force that holds the bearing in its place.

The normalized maximum seismic response of bridge samples with respect to investigated IMs of ASI, PGV, PGA, and PGA/PGV in terms of engineering demand parameters of superstructure longitudinal displacement and column curvature in both principal axis are presented in Fig. 6, Fig. 7, and Fig. 8, respectively. The IMs are calculated by taking the geometric mean of the two horizontal components of the GMs. Therefore, it is not possible to choose a GM data set having a uniform distribution among the investigated IMs at the same time. It is inevitable that some of the IMs accumulate at certain values, while there exist fewer data points at the other values of IMs. In spite of the uneven distribution of ASI and PGV, they have a better correlation with the maximum seismic response when compared with PGA and PGA/PGV through graphical examination of the data points. With the increasing values of PGA/PGV, there is not any explicit trend with bridge seismic response. Although PGA has some level of tendency with the bridge seismic damage, its correlation is worse in comparison with ASI or PGV.

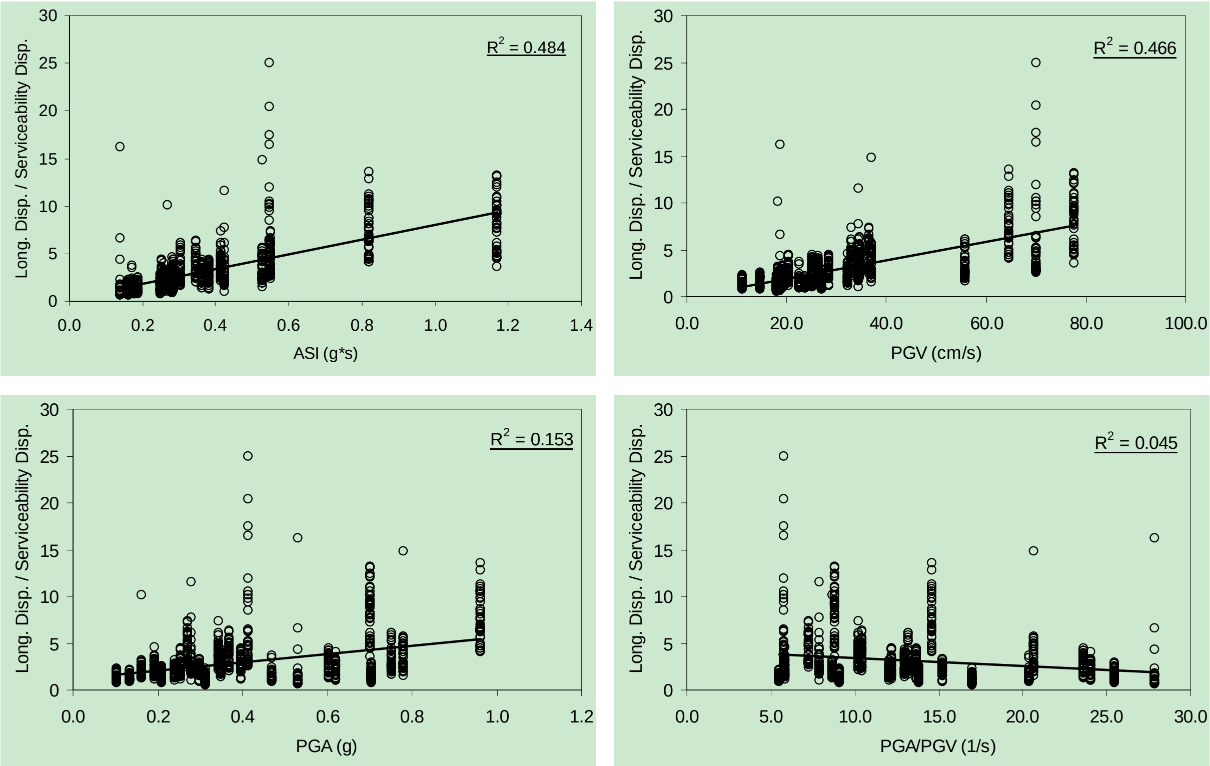

Figure 6. Normalized maximum superstructure longitudinal displacement.

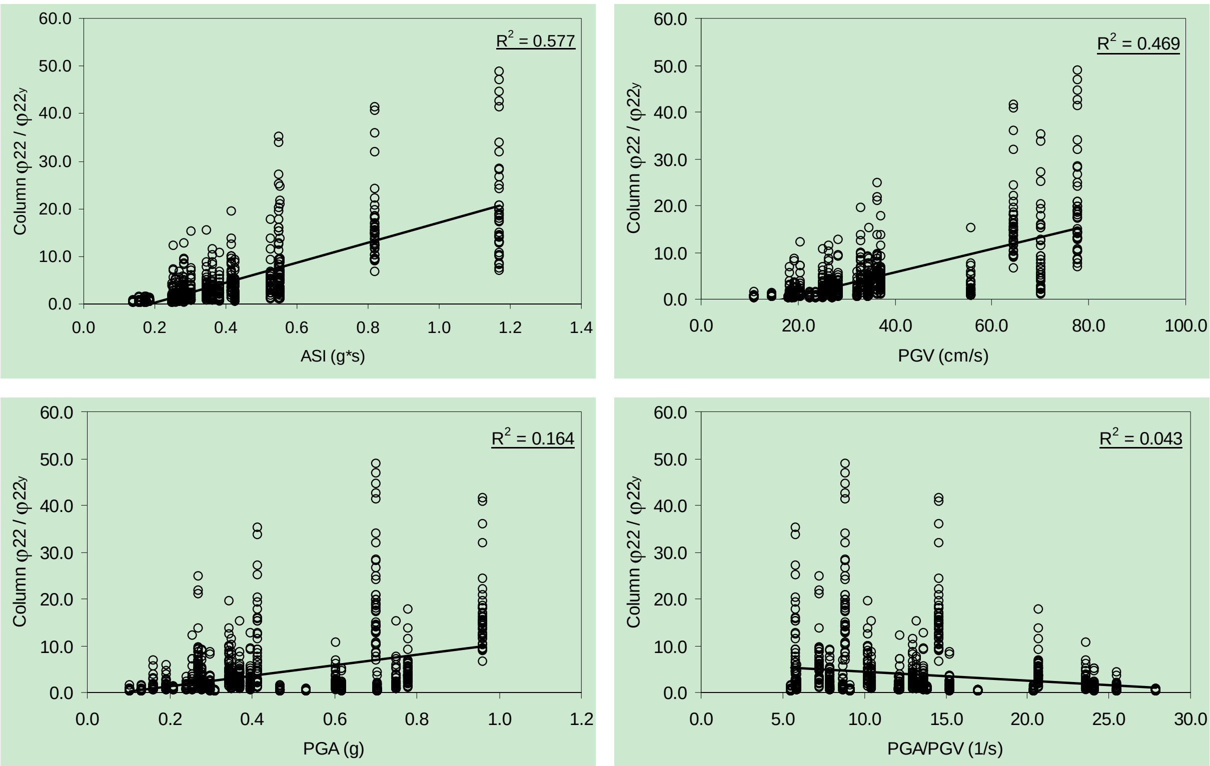

Figure 7. Normalized maximum column curvature-22.

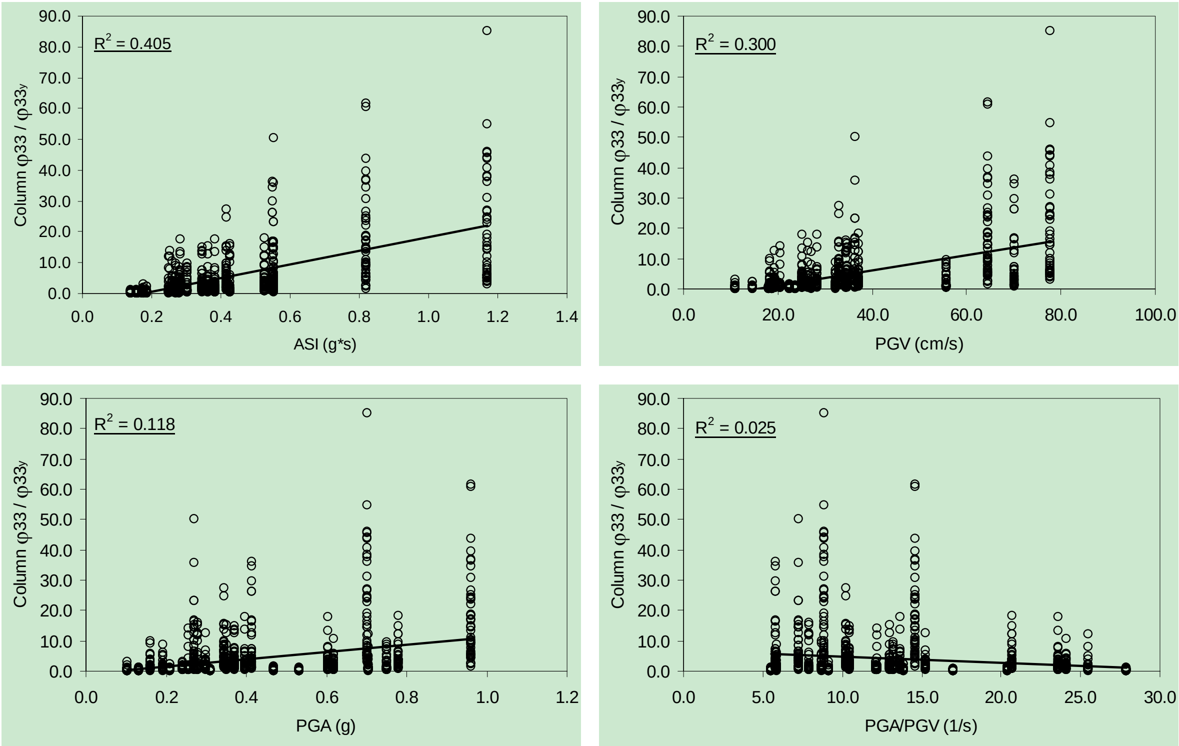

Figure 8. Normalized maximum column curvature-33.

In addition to the graphical examination of the data points, assessment of the IMs have been quantified by computing the coefficient of determination (R^2) for the linear relationship between the IM values and seismic response of engineering demand parameters to evaluate the relative adequacy of GM IMs. R^2 is an indicator varying between 0 and 1 that reveals how closely the estimated values by linear relationships correspond to the actual data of the seismic response distribution. The closer the R^2 value to 1, the higher the correlation between distribution data points of the bridge seismic response with the corresponding IM. The R^2 of ASI and PGV is much higher when compared with the R^2 of PGA and PGA/PGV.

Conclusions

Comprehensive research has been undertaken to investigate the destructiveness of GM IMs of ASI, PGV, PGA, and PGA/PGV through their correlation with the certain engineering demand parameters of highway bridges. Among the investigated GM IMs, ASI and PGV appear to be the ones that have better correlation with the highway bridge seismic damage, which is quantified with various engineering demand parameters. Therefore, the generated fragility curves based on ASI or PGV are expected to have higher reliability in the estimation of damage state of highway bridges. Although most of available fragility curves were developed using PGA, it was found out that PGA has a poor correlation with the seismic damage of highway bridges in comparison with ASI or PGV. PGA/PGV does not exhibit any trend with the seismic damage of highway bridges. By simply considering the ratio of PGA to PGV without any other IM of the corresponding GM may lead to incorrect interpretations. Two different earthquake GMs having different seismic intensity levels may have very similar PGA/PGV ratio. For this reason IM of

PGA/PGV should be considered together with an additional intensity parameter of the corresponding GM. Therefore, PGA/PGV is not an appropriate IM to be utilized in the development of fragility curves.

Boundary periods of T_i and T_f, which are used for the definition of ASI, have a considerable influence on the effectiveness of ASI. Defining ASI as the area under the 5% damped elastic response spectrum within the boundary periods of T_i=0.4s and T_f=1.10s lead to a reliable IM having a good correlation with the seismic damage of highway bridges. Therefore, it can be concluded that; boundary periods of T_i=0.4s and T_f=1.10s cover both elastic period range and the elongated period range of the ordinary highway bridges in Turkey.

Acknowledgments

A substantial support has been received through the collaborative EU FP7 project SYNER-G (244061). This grant is gratefully acknowledged

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