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Discussions

Seismic Assessment Of Multi-Span Steel Railway Bridge In Turkey Base On The Nonlinear Time History Analyses.

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Abstract. It has been seen that bridges are vulnerable to earthquakes by the research studies after important earthquakes like the San Fernando earthquake (1971 USA), the Northridge earthquake (1994 USA), Great Hanshin earthquake (1995 Japan), and Chi-Chi earthquake (1999 Taiwan). These studies show that to do the seismic risk assessments for bridges, fragility curves are useful tools. There are the most used two ways to generate the fragility curves; empirically or analytically. If the damage reports from past earthquakes are available then empirical fragility curves may be developed but otherwise seismic response analysis of structures may be used to develop analytical fragility curves. In Turkey, earthquake damage data are very limited so to develop the fragility curves for the Alasehir bridge, the analytical method is used in this study. The bridge that is studied on is lying on the Manisa-Afyon railway line that is very important for both transportation and freightage. As the most of the country land covers the seismically active zones it is a necessity to find out the vulnerability of the Alasehir bridge. The Alasehir bridge is consists of six 30m length truss system span with a total span length of 189.43m supported by 2 abutments and 5 truss piers with height of 12.5 m, 19m, 26m, 33m and 40 m. Sap2000 is used for computer model of the Alaşehir bridge and the model is refined by using field measurements. Then selected 60 different real earthquake data are used for the analysis by using the refined model.. Both material nonlinearity and Δ-δ are considered during the analysis. With this study, seismic behavior of Alasehir steel railway bridge is determined. Truss piers reaction and displacements are used to determine the seismic performance of the Alasehir bridge. Different IMs are compared in terms of efficiency, practicality, and sufficiency. Component and system fragility curve are derived for most proper IMs.

1 Introduction

Fragility is a conditional probability show that a structure or structural component will meet or exceed a certain damage level for a given ground motion intensity such as PGA or Sa. Fragility analysis is the important tool to determine the seismic vulnerability of bridges.(Pan et al., 2007). Fragility curves can be derived by three-way expert base, empirical and analytical. Empirical fragility curves can be derived in the form of the two-parameter lognormal distribution function.(Shinozuka et al., 2000b) Damage state of bridges and exposed ground motion intensity is determined to depend on an expert report to determine the expert base fragility curves, To derive the empirical fragility curves damage state of each bridge or structure with the exposed ground motion intensity need to be determined from damage reports. (Shinozuka et al., 2000b).when the damage state and ground motion intensity are unknown, analytical methods are used to determine the damage state and responding ground motion with these data analytical fragility curves are derived.(Nielson, 2005).

An important issue to derive fragility curve is determining the relation between IMs and EDP. There is three way to determine it nonlinear time history analysis, incremental dynamic analysis, and capacity spectrum analysis. Time history analysis is the most common used tool to derive fragility curve. (Banerjee and Shinozuka, 2007) (Bignell et al., 2004) (Shinozuka et al., 2000a) (Mackie and Stojadinović, 2001) (Kumar and Gardoni, 2014) Time history analysis gives more realistic results but it is time-consuming and includes high computation cost. Likewise, the incremental dynamic analysis used to determine the earthquake response of a structure and used to derive fragility curve (Lu et al., 2004) (Kurian et al., 2006) (Liolios et al., 2011). Because nonlinear time history analysis and incremental dynamic analysis are time-consuming and have more computation cost, capacity spectrum analysis is used to derive fragility curve as a faster way. (Banerjee and Shinozuka, 2007) (Shinozuka et al., 2000a) Determining fragility curve for retrofitted bridge systems, component fragility and railway fragility are other issues currently becoming famous. (Padgett et al., 2007a) (Chuang-Sheng et al., 2009) (Alam et al., 2012)(Tsubaki et al., 2016) what is more Energy base approach is recently used in fragility analyses (Wong, 2009)

Determining damage state of bridges that expose to an earthquake is another important issue. There are expert base and analytical ways to determine the real damage state of bridges. The expert based method depends on past earthquake and damage data recorded in seismic events. (Shinozuka et al., 2000c) Analytically determining damage state depends on the results of nonlinear analysis. Earthquake demands are obtained as displacement and rotation. To define damage state displacement limits and rotation limits need to be determined.(Choi and Jeon, 2003; Padgett et al., 2007b; Pan et al., 2007)

In this paper, we describe the nonlinear behavior of a selected railway bridge and derive the fragility curves of this bridge. The bridge is in a key role (has an important role on the line) bridge for railway transportation systems and unicity's of structural designation increase the importance of bridges. Cornel (2002) methods are used to derive the fragility curve of bridges(Cornell et al., 2002). Different intensity measures are taken into considered and most suitable intensity measure are determined. Both capacity limits and serviceability limits are used to derive fragility curve. Component fragility curve and system fragility curve are derived separately.

2 Analytical methodology and simulation

Fragility curve is an effective tool determines seismic capacity of structure or structural component. Fragility is defined as conditional probability of seismic demand (EDP) placed upon structure or structural component exceeding its capacity (C) for a given level of ground motion intensity (IM).(Padgett and DesRoches, 2008) as shown in Eq. (1):

$$Fragility = P[EDP \geq C \mid IM] \quad 1.$$

Seismic demand and capacity need to be determined to derive fragility curve. In this study, PSDMs is used to determine structural demand and capacity. The PSDMs require analytical modeling and analyzing of structure. Nonlinear time history

Nat. Hazards Earth Syst. Sci. Discuss., doi:10.5194/nhess-2017-168, 2017
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Discussion started: 17 May 2017

analysis is used to determine the PSDMs. Following sections include more detailed information about nonlinear analysis and probabilistic seismic demand model.

2.1 Probabilistic seismic demand model

When using analytical procedure PSDM describes the seismic demand of a structure or structural component in terms of approximate intensity measure. PSDMs can be written as Eq. (2):

$$P[EDP \geq d \mid IM] = 1 - \phi\left(\frac{\ln(d) - \ln(\widehat{EDP})}{\beta_{EDP|IM}}\right) \qquad 2.$$

Estimation of median EDP describes as a power model as given in Eq. (3): and linear representation is shown in Eq. (4): on the base of logarithm.[4]

$$\widehat{EDP} = aIM^{b} \qquad 3.$$

$$\ln(EDP) = \ln(a) + b\ln(IM) \qquad 4.$$

IM is the seismic intensity measure and a and b are the regression coefficient. ϕ is the standard normal cumulative distribution function, $\widehat{EDP}$ is the median value of engineering demand, d is the limit state to determine the damage level and $\beta_{EDP|IM}$ (dispersion) is conditional standard deviation of the regression as shown in Eq. (5):

$$\beta_{EDP|IM} \cong \sqrt{\frac{\sum(\ln(d_i) - \ln(aIM^{b}))^2}{N-2}} \qquad 5.$$

2.2 Component and system fragility

Component fragility shows the seismic behavior of different component under same level of damage. With using component fragility the weakest component of the bridge can be determined easily. Buckling capacities of all members are calculated. Fragility curves for Truss Piers, Truss, and Stringer members are derived using PSDM. Truss piers are most fragile component because of the slenderness of elements and total length of the truss piers.

However, here the point of interest for system fragility is to determine all possible damage probabilities in the system. To derive the fragility curve of bridge system, participation of all components needed to be considered. The damage probability of bridge for a chosen limit state is equal to the union of probabilities of each component of the bridge system for the same limit state. Eq. (6): shows the relation of system failure and component failure.

$$P[Fail_{system}] = \bigcup_{i=1}^{n} P[Fail_{component-i}] \qquad 6.$$

The correlation coefficients between the demands of components are estimated to characterize the conditional joint normal distribution. 10^6 samples of demand are generated by Monte Carlo simulation and using Eq. (6): system fragility curves are derived.

Nat. Hazards Earth Syst. Sci. Discuss., doi:10.5194/nhess-2017-168, 2017
Manuscript under review for journal Nat. Hazards Earth Syst. Sci.
Discussion started: 17 May 2017

3 Description of case study

In Turkey, the railway construction started with the contribution of European countries such as England, France, and German and the first aim of the railway is to transport agricultural good and valuable mineral to Europe from harbors. The first railroad line that was constructed by England company at 1856 placed on İzmir – Aydın corridor with a total length of 130km [1]. The railroad line in Turkey is divided into 7 regions. The total length of railroads is 8722 km. 81% of the total number of 25443 culverts and bridges were built before 1960. Hence, there are many historical bridges and some of them were certified as historical heritage.

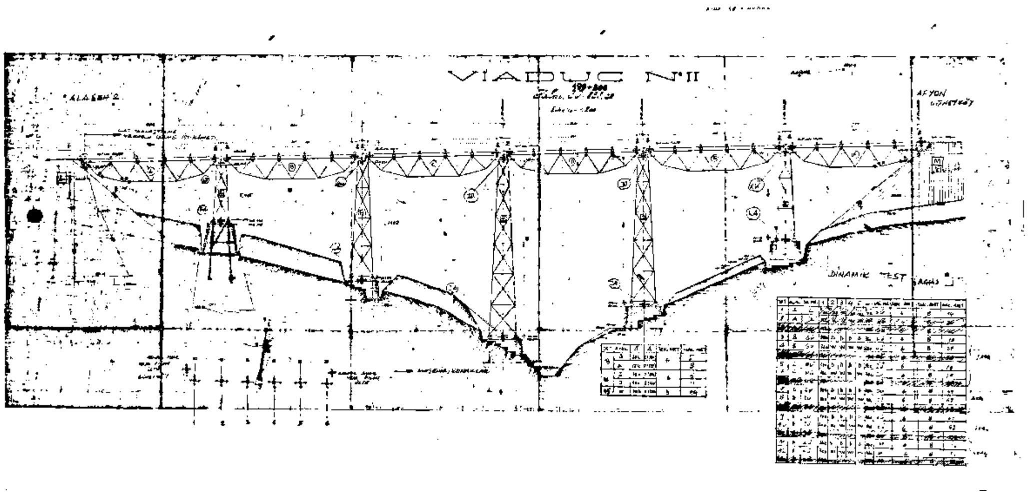

Figure 1 Project of Bridge

Figure 2: Photos of Bridge

Nat. Hazards Earth Syst. Sci. Discuss., doi:10.5194/nhess-2017-168, 2017
Manuscript under review for journal Nat. Hazards Earth Syst. Sci.
Discussion started: 17 May 2017

The Alasehir bridge takes place on the Manisa-Uşak-Dumlupınar-Afyon railroad line which is about 200 km long. The bridge was built by Ateliers De Construction De Jambes Namur Company at 1923. The bridge is composed of 6 over steel truss sub-bridges with 30m span length each and has 5 truss piers which are 12.5 m, 19m, 26m, 33m and 40 m in height. The total length of the bridges is 189.43m. The bridge has 300m horizontal curve radius. The curvature of the road is applied to the bridge via the location of the rails on the over truss bridge and this is why one side truss strength is higher than the other side truss. The slope of the railroad is nearly %2.7. Truss systems are simply supported between abutment-pier / piers-piers / pier-abutment. Bridge piers are connected to the foundation with long and thick steel anchorages. The bridge is constructed by using angel sections and build up sections. Span elements consist of L80x80x12, L100x100x12, L120x120x11, L120x120x15, PL20x420, PL20x200 and PL20x400 and pier element are consist of L80x80x10, L100x100x10, L100x100x12, L120x120x12, L120x120x14 and UPN300 element There are walkways at both sides of the sub-spans . The sleepers rest over the stringers which are mounted over the transverse girders.

4 Analytical Modelling and Simulation

4.1 Ground Motion Suites

The effect of ground motions on the structure is obtained by using linear or nonlinear mathematical model of a structure. A nonlinear dynamic time history analysis is used to minimize the uncertainty of structural responses. A relation between Ground motion intensity measure IMs and engineering demand parameter EDP are obtained depending on the time history analysis results. These relations can be obtained by using one of the third methods named cloud (direct) (Shome, 1999) method, Incremental dynamic analysis (IDA) (Vamvatsikos and Allin Cornell, 2002) and stripes method. (Sofiyanti et al., 2015) In these study cloud method is used to represent the relation of IM and EDP. Cloud method includes numerous selection of real ground motion record and uses the selected ground motion without any prior scaling (Mackie et al., 2008).

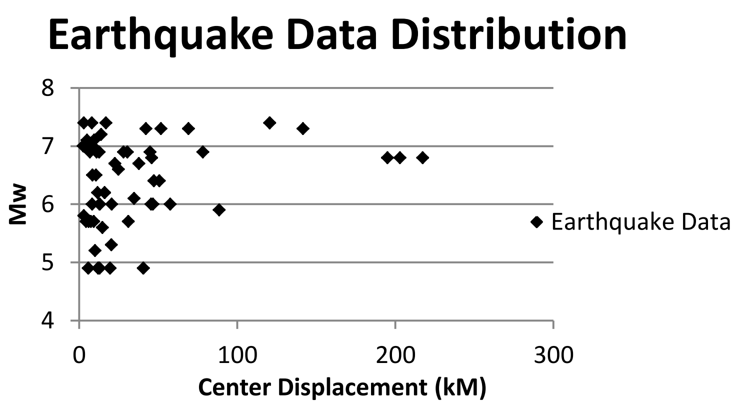

Figure 3: Earthquake Data Distribution.

Nat. Hazards Earth Syst. Sci. Discuss., doi:10.5194/nhess-2017-168, 2017
Manuscript under review for journal Nat. Hazards Earth Syst. Sci.
Discussion started: 17 May 2017

In this study, Earthquake data are selected separately considering different soil types, moment magnitudes, PGAs and central distances of the earthquake record. The moment magnitudes are changing from 4.9 to 7.4 and PGAs are changing from 0.01g to 0.82g and the central distances of earthquake records are changing from 2.5kM to 217.4kM. The distribution of moment magnitude to central distances are shown in Figure (3). 60 different real earthquake data are chosen for A, B and C type of soil for this study. Unscaled earthquake data are used for time history analysis.

4.2 Analytical bridge models.

All the elements of the bridge are modeled by 2- node beam element. Supports and releases of the members are all applied according to as-built drawings and site visual inspections. The difference between the center points of the members and connections are all considered as rigid bars to take the moments into account which can occur due to eccentricity.
The weight of the sleepers and rail profiles are calculated and applied to the stringer beams as dead load (mass and load). Steel material of the bridge is assumed as ST37 which fits for the construction years of the bridge. Only the dead load calculation is done by finite element program according to the given members' properties (area, length, and density). Finite element models are composed of 1609 frame members, 832 nodes, and 120 link elements.

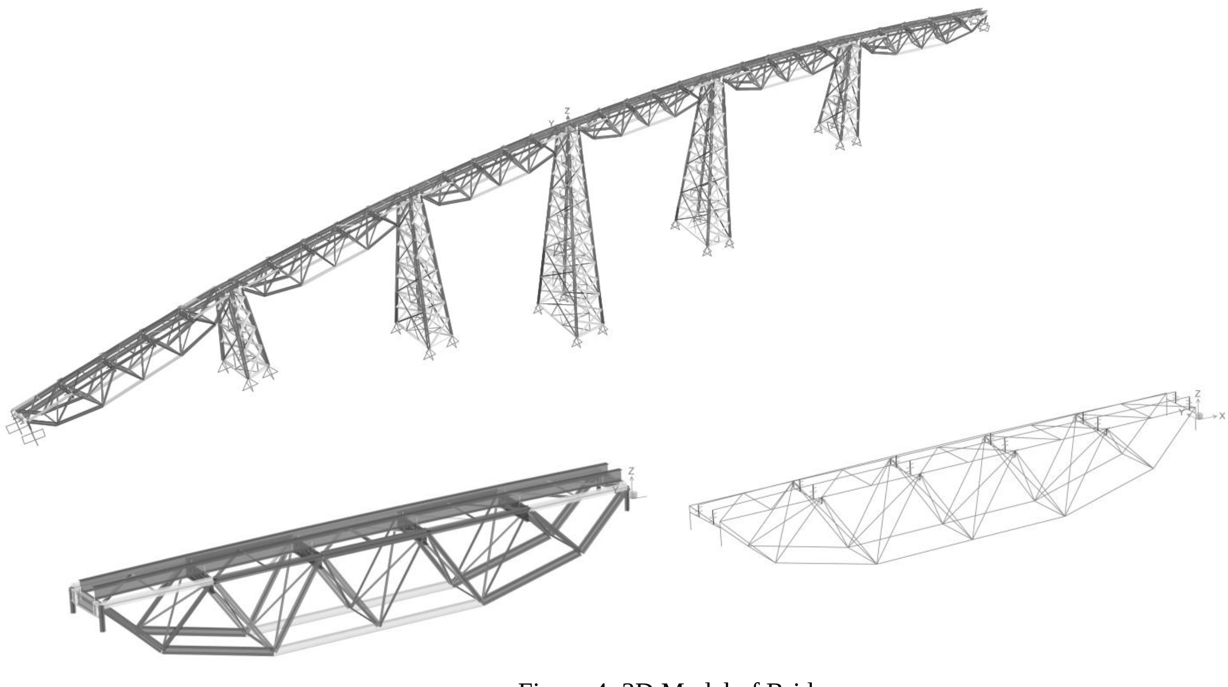

Figure 4: 3D Model of Bridge.

Time history analyses are applied to the model with considering both material nonlinearity and geometrical nonlinearity. Plastic hinges are defined with steel interacting PMM plastic hinges as defined in FEMA 356 Equation 5-4. In order to detect any hazard on bridges, plastic hinges are defined at the start, the mid, and the end points of all frame members. Geometric

Nat. Hazards Earth Syst. Sci. Discuss., doi:10.5194/nhess-2017-168, 2017
Manuscript under review for journal Nat. Hazards Earth Syst. Sci.
Discussion started: 17 May 2017

nonlinearity is defined as Δ-δ and large displacement and Newmark direct integration is used in the analysis. All three components of the earthquake (one longitudinal and two horizontal) are defined in the time history process.

5 Selection of Intensity Measure and Demand Models.

5.1 Selections Intensity Measures

PSDMs traditionally conditioned on a single intensity measure (IM), and the degree of uncertainty in the PSDMs is depended on the selected IMs. So determining optimum intensity measure is an important step to derive more realistic fragility curve. There are different intensity measure parameters such as PGA, S_{a_t} and. These study discussed intensity measure in terms of practicality, efficiency, and proficiency.

Table 1 Intensity measures

Intensity measure	Description	Units	Definition
PGA	Peak ground acceleration	g	$PGA = \max\left\|\ddot{u}_g(t)\right\|$
PGV	Peak ground velocity	cm/s	$PGV = \max\left\|\dot{u}_g(t)\right\|$
$S_{a-0.2s}$	Spectral acceleration at 0.2s	g	$S_a(T_i) = w_i^2 S_d(T_i)$
$S_{a-1.0s}$	spectral acceleration at 1s	g	$S_a(T_i) = w_i^2 S_d(T_i)$
I_A	Arias intensity	cm/s	$I_A = \frac{\pi}{2g}\int_0^{T_d}\left[\ddot{u}_g(t)\right]^2 dt$
I_V	Velocity intensity	cm	$I_V = \frac{1}{PGV}\int_0^{T_d}\left[\dot{u}_g\right]^2 dt$
CAV	Cumulative absolute velocity	cm/s	$CAV = \int_0^{T_d}\left\|\ddot{u}_g(t)\right\|dt$
CAD	Cumulative absolute displacement	cm	$CAD = \int_0^{T_d}\left\|\dot{u}_g(t)\right\|dt$
ASI	Acceleration spectrum intensity	cm/s	$ASI = \int_{T_i}^{T_f} SA(T,\xi)dT$

(PGA, PGV, $S_{a-1.0s}$) are characteristic parameter directly obtained from earthquake records. I_A, I_V , CAV, CAD, and ASI are other intensity measure offered to characterize seismic ground motion. (Hsieh and Lee, 2011)(Kayen and Mitchell, 1997)(Mackie and Stojadinovic, 2004)(ÖZGÜR, 2009)

Practicality is described as the correlation between an IM and demand screened on a structure or structural component. The more practical intensity measure means a higher correlation of demand and IMs. The practicality of an intensity measure can be evaluated with the regression parameter b of PSDM. Higher value of b shows more practical IMs.

Efficiency is described as alteration of demand for a given IMs and can be measured with dispersion. The less dispersion means the more efficient IMs. Proficiency is a proposed term that includes both practicality and Efficiency (Padgett et al., 2008) and Modified dispersion ζ is used to describe proficiency of IMs and the lower values mean more proficient IMs.

$$\zeta = \frac{\beta_{EDP|IM}}{b} \qquad 7.$$

There are many different intensity measure used to characterize seismic behavior. 9 different intensity measures parameter are selected for this study as shown in table (1).

Table 2 Demand models and IM comparisons

	Longitudinal			Transverse			Gravity		
	b	$\beta_{EDP\|IM}$	ζ	b	$\beta_{EDP\|IM}$	ζ	b	$\beta_{EDP\|IM}$	ζ
PGA	0.65851	0.50609	0.76853	0.55704	0.63446	1.13898	0.18694	0.82772	4.42775
PGV	0.63369	0.52072	0.82173	0.60588	0.52557	0.86744	0.28191	0.80852	2.86803
$S_{a-0.2s}$	0.61994	0.51074	0.82386	0.49221	0.65769	1.33621	0.25932	0.79767	3.07606
$S_{a-1.0s}$	0.52785	0.54505	1.0326	0.51245	0.5139	1.00283	0.3146	0.78627	2.49926
I_A	0.37559	0.50642	1.34832	0.34497	0.5539	1.60565	0.14998	0.80796	5.38716
I_V	0.27931	0.75571	2.70564	0.35374	0.66536	1.88095	0.19502	0.81545	4.1814
CAV	0.61123	0.61839	1.01173	0.62262	0.57814	0.92856	0.32622	0.80001	2.45235
CAD	0.32525	0.73245	2.25198	0.39189	0.6425	1.63949	0.21277	0.8118	3.81537
ASI	0.6741	0.45435	0.674	0.63134	0.50062	0.79294	0.37916	0.77549	2.04529

For longitudinal direction maximum b value is 0.67, for transverse-direction maximum b value is 0.63 and for gravity direction, maximum b value is 0.37. The result of these analysis shows that ASI is more practical than other parameters.
Considering the longitudinal direction minimum value of $\beta_{EDP|IM}$ dispersion is 0.45, with respect to transverse direction is 0.50 and gravity-direction is 0.77. ASI as an IMs gives smaller dispersion and more efficient results.
The minimum values of modified dispersions ζ are 0.67 for longitudinal-direction, 0.79 for transverse direction and 2.04 for gravity-direction The smaller modified dispersions obtained is ASI and ASI is more proficient than other parameter.

5.2 Probabilistic seismic demand models.

PSDMs are constructed from the peak transverse displacement on the top of the middle pier of bridge. 60 different nonlinear time history analysis results are used. ASI is selected as IMs as a result of analysis conducted previous section.

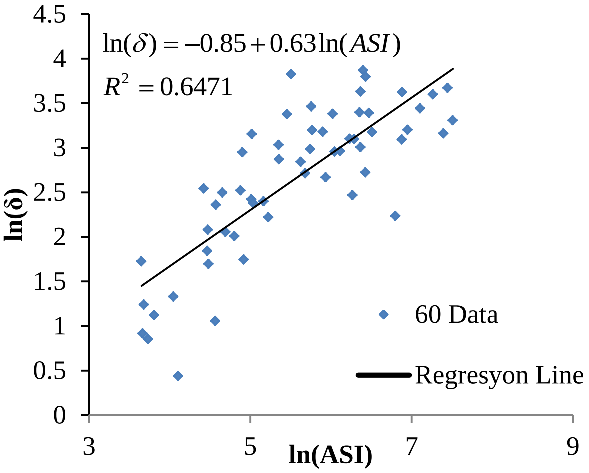

Figure 5: PSDMs for selected IMs

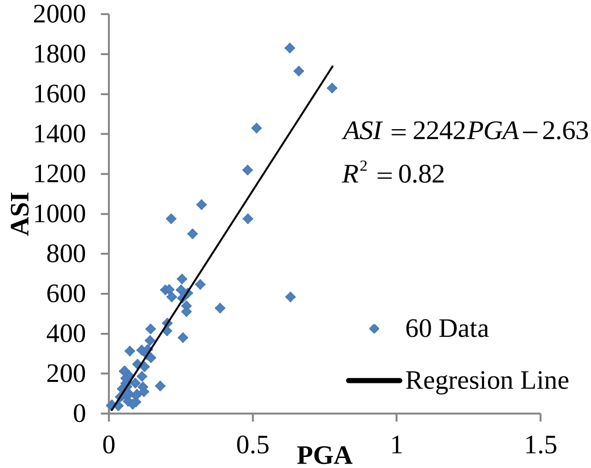

Figure 6: Correlation between ASI and PGA

Figure 5 shows that there is a good correlation between ASI and transverse displacement of the middle pier of bridges. Figure 6 shows that ASI and PGA have good correlations.

5.3 Limit state estimation.

There is limited information about damaged steel truss bridge in the literature. The limited information shows that main causes of damage of bridges are buckling of the upper and lower braced element and shear failure of the transverse element (corrosion quickens the phenomena). (Kawashima, 2012) (Bruneau et al., 1996) Reliability assessment of a steel truss bridge are generated by Shah PM (Shah et al., 2009) This study considers tension and compression capacity of steel element as a limit state. To calculate tension capacity effective net area of steel members are considered. During the calculations, buckling capacity of the members found smaller than the tension capacity of the members so buckling limit state is overcome for the compression members during the calculations. For material properties, as there were no specimen tested for the Alasehir bridge material are choose from literature. Capacities of structural elements are deterministically calculated.

Table 3 Maximum angular variation and minimum radius of curvature (m)

Speed range V (km/h)	Rotation (rad)	Curvature (1/m)
V≤120	0.0035	1700
120<V≤200	0.0020	6000
V>200	0.0016	14000

Serviceability is also taking into consideration in this study. Seismic action may cause overturning of train and derail of train. there is sample of overturning and derailment of train on the condition of 1999 Kocaeli Turkey earthquake.(Byers, 2004) Lateral displacement of bridge is considered as serviceability limit states. EN1990-Annex A2 include lateral displacement limitation for railway bridges.(EN1990-prANNEX A2 : Application for bridges, 2001) EN1990-Annex A2 describe different maximum angular variation and minimum radius of curvature to limited lateral displacement for different velocity level as shown in table (3).

6 Fragility Curve for Railway serviceability and bridge component

6.1 Fragility curve for railway serviceability

Under seismic condition accidental derailment and overturning can be occured. In the Kocaeli Turkey earthquake, there are 3 such events are visualized.(Byers, 2004) Serviceability limits to minimize such event are determined in EN1990 Annex A2 are used as limit state in this study. There is tree limit state for three different services speed. V<120, 120<V<200, 200<V. Railway becomes more and more important for both stuff transportation and human transportation and service speed is affect both capacity and quality of railway transportation system.

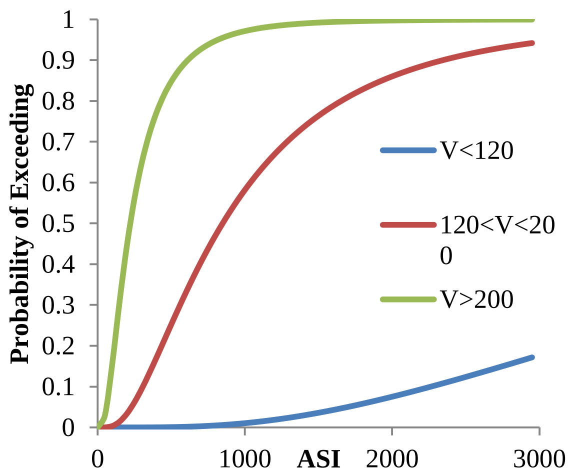

Figure 7: Probability of Exceeding /ASI

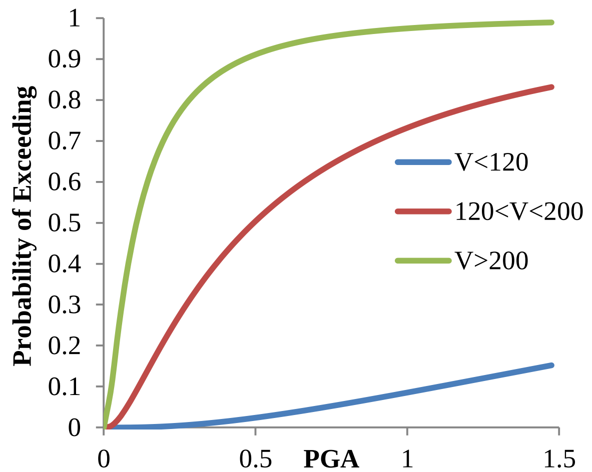

Figure 8: Probability of Exceeding / PGA

Probabilities of exceeding of serviceability limit states are shown in figure 7-8. Fragility curve in figure 7 gives more certain information about structure. For V> 200km/h %50 probability of exceeding is occur in 250cm/s for ASI and 0.11g for PGA. For 120<V<200 km/h %50 probability of exceeding is occur in 850cm/s for ASI and 0.5g for PGA. Finally for V<120 km/h %50 probability of exceeding is occur in 6250cm/s for ASI and 4.8g for PGA. There is a good correlation between ASI and PGA shown in Section 4.2. considering figure (7) and figure (8) fragility curve derived using ASI gives more safe and absolute results.

6.2 Fragility Curve for Bridge Component.

After conducting 60 different nonlinear time history analysis forces acting on structural elements are recorded. Buckling capacities of the elements are calculated using AISC 360-2010 specification requirements with considering both axial forces

Nat. Hazards Earth Syst. Sci. Discuss., doi:10.5194/nhess-2017-168, 2017
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Discussion started: 17 May 2017

and moment acting on bridge components. The results of buckling capacity are used to specify whether any damage occurs on the component or not. Fragility curves of components are derived with two parameter log-normal distribution.

Component fragility curve of bridges are shown in Figure 9. Truss Piers elements seem to most vulnerable element of bridges. There are 5 piers in bridges and lengths of them are changing from 12m to 40m and constituted by steel truss element. So the truss piers elements are slender and buckling capacities are less. On the other hand superstructures of the bridge are most safe component. The less probability of buckling is occurring in superstructure.

Seismic risk analysis of the side of bridge is needed to be determined in order to get more information about the reliability of bridge system.

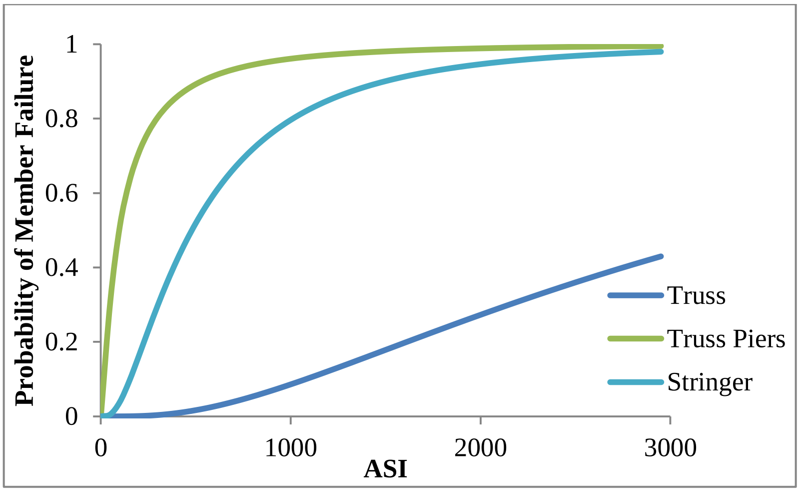

Figure 9: Component Fragility Curve

System fragility curve of entire bridge is derived using joint probabilistic seismic demand model (JPSDM) and limit state models in crude Monte Carlo simulation. 10^6 Sample Demand of all components are regenerated by using these simulation depends on median and standard deviation of PSDM and System fragility curve are derived using Eq. (6): Median value of bridge fragility is 100m/s. One other option for deriving system fragility curve is calculating upper and lower bounds on the system. For a serial system, the bounds are derived using Eq. (8):

$$\max_{i=1}^{n}[P(F_i)] \leq P(F_{system}) \leq 1-\prod_{i=1}^{m}[1-P(F_i)] \qquad 8.$$

Where $P(F_i)$ is the failure probability of component i and $P(F_{system})$ is failure probability of system. The maximum of component failure probability gives the lower bound. It means there is a correlation between demands of components and gives un-conservative result. Conversely, upper bound assumes there is no correlation between components demands and gives conservative results. It expected that system fragility curve must be between upper and lower bound fragility curve. When only one component significantly affects the system fragility curve the bounds become narrow. Contrariwise, if many components affect the system, the bounds become wider.(Nielson and DesRoches, 2007) Figure (10) shows bridge fragility curves and upper and lower bound fragility curves.

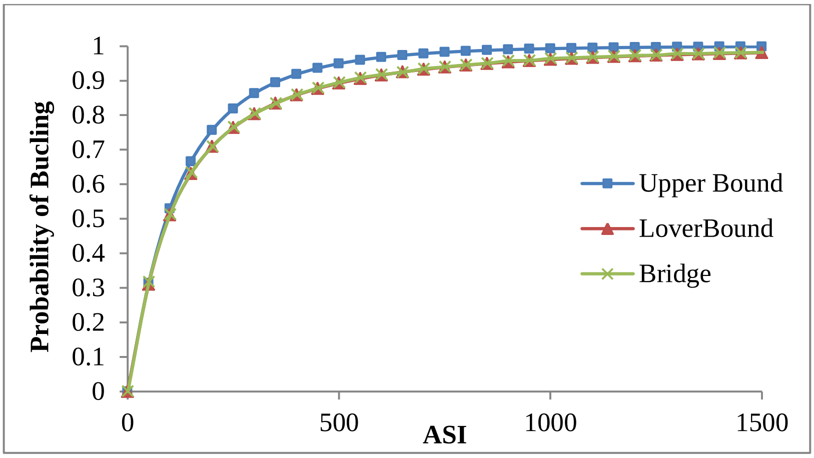

Figure 10: Bridge and system fragility bounds.

7 Conclusion.

This work presents a specific study on seismic assessment of multi-span steel railway bridge in Turkish railway systems. The main idea of the study is determining seismic behavior and safety of bridge under seismic condition. PSDMs of bridges are obtained by deriving 60 different nonlinear time history analysis and demand of bridge components are used to derive fragility curve of component and bridge. 9 different IMs characterize the seismic event are used. 4 of the IMs can be calculated directly from time history record. Practicality, efficiency, and proficiency of the IMs are discussed. For this bridge, it is shown that ASI is the most efficient, practical and proficient parameter and has good correlation with demand good characterizing strong ground motion.

Fragility curves of the Alasehir Bridge are derived for both serviceability limits and component capacity limits. Serviceability limits are determined by EN 1990 Annex 2 The results show that serves velocity limits has important effects on fragility curve of bridges. While velocity limit smaller than 120km/h the bridge is safer but while velocity limit is bigger that 200 km/h the bridge is more vulnerable. According to the result of this analysis, it is proposed that serves velocity limits of train line need to decrease on the bridge. Components and system fragility curves of bridge are derived considering buckling and fracture capacity of bridge element. Component fragility curves show that truss piers elements are most vulnerable elements in the system. But superstructure elements are most safe element in the system. Because Truss piers significantly affect bridge system fragility curve, upper and lower bound of system fragility curve give very narrow results. System fragility curve is close to lower bound, shows that there is a good correlation between demands of the component.

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