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Research Article

Seismic assessment of a curved multi-span simply supported truss steel railway bridge

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ABSTRACT

Fragility curve is an effective method to determine the seismic performance of a structural and nonstructural member. Fragility curves are derived for Highway Bridges for many studies. In Turkish railway lines, there are lots of historic bridges, and it is obvious that in order to sustain the safety of the railway lines, earthquake performance of these bridges needs to be determined. In this study, a multi-span steel truss railway bridge with a span length of 25.7m is considered. Main steel truss girders are supported on the abutments and 6 masonry piers. Also, the bridge has a 300m curve radius. Sap 2000 finite element software is used to model the 3D nonlinear modeling of the bridge. Finite element model is updating according to field test recordings. 60 real earthquake data selected from three different soil conditions are considered to determine the seismic performance of the bridge. Nonlinear time history analysis is conducted, and maximum displacements are recorded. Probabilistic seismic demand model (PSDMs) is used to determine the relationship between the Engineering Demand Parameter (EDP) and Intensity Measure (IMs). Fragility curve of the bridge is derived by considering the serviceability limit state, and results are discussed in detail.

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1. Introduction

Turkey is under the influence of North Anatolia, Southeast Anatolia, and Western Anatolia earthquake zones. Therefore, 42% of the country is located in the very high seismic hazard zone. The railway lines in Turkey were started to build up in the 19th century. %82 of the bridges, and culverts were built before 1960 (Çağlıyan and Yıldız, 2013). To determine the seismic vulnerabilities of the bridges fragility curves must be derived. Fragility curve is a most used effective tool to determine the seismic performance of bridges. Fragility is the probability of a structural or nonstructural member which will exceed certain performance limit under an earthquake condition (Pan et al., 2010b). Fragility curves can be derived in three different way; Expert base, empirical and analytical (Shinozuka et al., 2000a; Nielson, 2005).

To derive expert base and empirical fragility curve past earthquake reports and expert opinion about the damage state of the bridge are a need. It is not possible to obtain this information for all bridges, so that the analytical method needed to derive the fragility curve becomes important. Linear and nonlinear analyses are being used to determine the relation between EDP and IM to derive analytical fragility curve. Nonlinear time history analysis is a frequently used and an effective tool to derive analytical fragility curve (Banerjee and Shinozuka, 2007; Bignell et al., 2004; Shinozuka et al., 2000b; Mackie and Stojadinovic, 2001; Kumar and Gardoni, 2014). Analytical fragility curves are also commonly used to determine the seismic performance of the bridges (Mackie and Stojadinovic, 2003; Pan et al., 2010a and 2010b; Shinozuka et al., 2007a and 2007b).

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In this study Bekdemir railway bridge is investigated. 60 different real earthquake data are selected, and nonlinear time history analysis is performed to determine PSDMs. Fragility curve of the bridge is derived by using two-parameter log-normal distribution function. Serviceability limit states for three different serviceability velocities are considered. Results are discussed in detail.

2. Finite Element Model of the Bridge

2.1. Description of the bridge

The bridge is located in Haydarpaşa-Eskişehir railway line at 235+470 km and built at 1980. There are 7 steel truss spans on the bridge. The multi-span truss girder bridge is supported by abutments at the edges and 6 piers at the middle spans. One edge of steel truss span is simply supported, and the other edge is designed as sliding support on rollers and length of a span is 25.7m while total length of the bridge is 187 m. The bridge has a 300 m curve radius. Steel girders on spans are composed of angle section, IPN, UPN hot-rolled sections, steel plates and built-up sections. The bridge is a deck-type truss bridge. The old and new conditions, of the bridge are shown in Fig. 1.

2.2. Finite element model of the bridge

Commercial finite element software to be used to model the 3D bridge. All the elements of the bridge were modeled by 2-node beam element according to the shop drawings and site visual inspections. Computer model was developed by including elements, supports, all irregularities and their connections to each other. Due to centerline differences of the connected beam members, eccentricity at the connection points was taken into account during modeling of the bridge. The weight of the sleepers, ballast, and rails were applied to the dead load at the appropriate nodes. Bridges have 40cm ballast under sleepers. Steel material of the bridge was used as St 37 given in the project. Elastic modulus of the masonry piers was taken as 28 GPa.

Fig. 1. General views of the Bekdemir railway bridge.

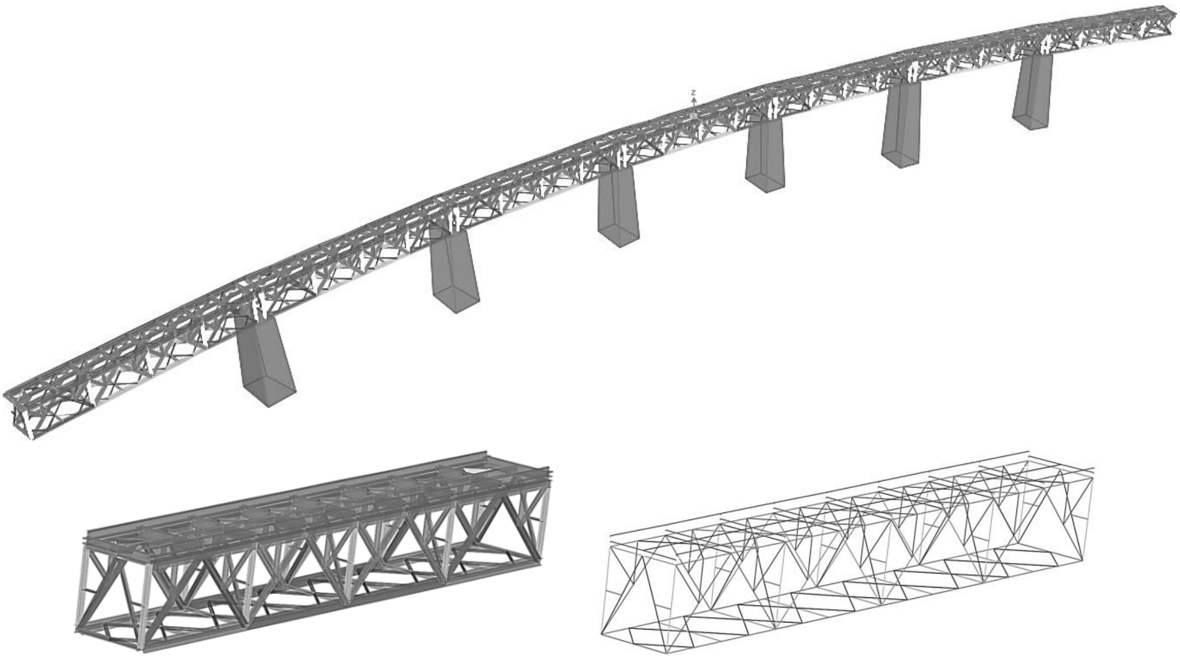

Fig. 2. Finite element model of the Bekdemir bridge.

3D model of the bridge includes 1471 points, 2054 frame elements, and 276 link elements. 16 uni-axial accelerometers were used to measure accelerations of the bridge caused by the passage of test vehicle. The accelerometers were attached to a heavy steel angle base having three short adjustable pointed legs, so that the combined unit, when placed without any attachments and easy to level, would pick up the bridge accelerations. Mode shapes and modal frequencies of the bridge model were compared with the mode shapes and frequencies that were calculated by processing the field test acceleration signals and 3D model of the bridge was updated. Frequencies of first longitudinal and transverse directions are calculated as 3.92s and 5.15s, respectively. Nonlinear time history analyses were conducted using selected earthquake data. Also, both material and geometric nonlinearity were considered in the analysis. Piers were defined as a linear elastic element. Concentrated plastic hinge approach was used to define the nonlinear behavior of the material. SAP2000 (2017) PMM plastic hinge acceptance which is given FEMA 356 Equation 5-4 was used to define the plastic joints of the bridge elements. Mode shape and modal frequency of bridge model are compared with the field test results and 3D model of bridge is updated.

3. Selection of the Earthquake Record

There are different methods that can be used to determine the demand of a structural system under a seismic effect. Linear static analysis, nonlinear pushover analysis, nonlinear time history analysis and incremental dynamic analysis are some of them. Nonlinear time history analysis needs expensive computational effort in terms of time and money, but gives the most realistic result. So nonlinear time history analysis is commonly used (Özgür, 2009; Choi and Jeon, 2003; Shinozuka et al., 2000a).

Earthquake data were selected by considering different soil types, moment magnitude (4.9-7.4), PGAs (0.001-0.82 g), and epicentral distances from earthquake epicenter (2.5-217.4 km). Fig. 3 shows the distribution of moment with central distance. Sixty real earthquake data were selected for soil types A, B, and C, and unscaled earthquake data were used for time history analysis.

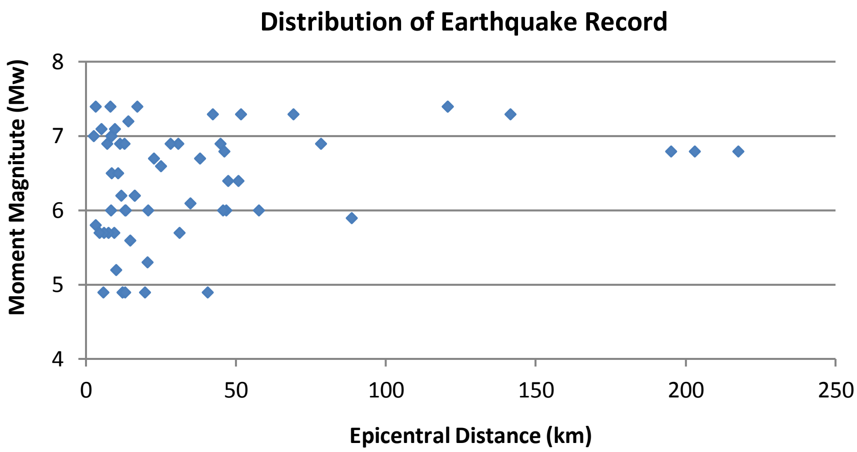

Fig. 3. Distribution of earthquake record in terms of epicentral distance and moment magnitude.

4. Fragility Curve

4.1. Probabilistic seismic demand model

Probabilistic seismic demand model (PSDMs) defines the structural demand in terms of intensity measure (IMs). Seismic performance of a structural and nonstructural member can be calculated by using PSDMs (Choi et al., 2004). Eq. (1) can be used to derive PSDMs.

$$P[EDP \geq d|IM] = 1 - \theta\left(\frac{ln(d) - ln(E\widehat{D}P)}{\beta_{EDP|IM}}\right), \quad (1)$$

where, θ is standard normal distribution function, $E\widehat{D}P$ is the median value of engineering demand, d is the limit state to determine damage level and $\beta_{EDP|IM}$ (dispersion) is the conditional standard deviation of the regression. $E\widehat{D}P$ can be estimated by Eq. (2). Eq. (3) is obtained in linear form if the two sides of Eq. (2) are taken as *ln*. $\beta_{EDP|IM}$ can be calculated using Eq. (4) while a and b are regression coefficients.

$$E\widehat{D}P = aIM^b, \quad (2)$$

$$ln(EDP) = ln(a) + bln(IM), \quad (3)$$

$$\beta_{EDP|IM} \cong \sqrt{\frac{\sum\left(ln(d_i) - ln(aIM^b)\right)^2}{N-2}}. \quad (4)$$

4.2. Determining serviceability limit state

During the usage period, different damages and displacements of the bridge elements have been observed. However, there are also some geometrical features that the bridges must have so that the railway line can be safely used. In EN 1990 Annex 2 (2001), lateral displacement limits of bridges are defined separately for three different train speeds, for single span and multi-span bridges as can be seen in Table 1.

Table 1. EN 1990 Annex 2 (2001) lateral displacement limits.

Speed Rating V (km/h)	Single Span	Multi-Span
	Radius (1/m)	Radius (1/m)
$V \leq 120$	1700	3500
$120 < V \leq 200$	6000	9500
$V > 200$	14000	17500

This study considered the lateral displacement limits state in EN 1990 Annex 2 (2001) and derived fragility curve to determine whether the displacement limits were exceeded under earthquake motion or not. Trainloads were not considered while nonlinear time history analyses were running and lateral displacements were recording.

4.3. Derivation of the fragility curve

Mid-span displacements gathered from 60 nonlinear time history analyses were used to derive a probabilistic seismic demand model (PSDMs). PGA was used as an intensity measure (IMs). By using PSDMs lateral displacement of bridge at mid-span points were determined in terms of IMs. For the nonlinear time history analysis runs, earthquake records were applied to all bridge piers and edge supports simultaneously and equally. Lateral displacement obtained at the mid-span of the bridge could also be affected by the displacements between the bridge piers, by the curvature of bridge geometry and by the differences of bridge piers foundation's soil condition. Therefore the assumptions made during the analysis should be studied carefully while the PSDMs is being developed, and fragility curve is being used.

Fig. 4 shows PSDMs of mid-span displacements of the bridge. PSDMs can be defined based on Eq. (2) by using a power function of IMs and Eq. (3) by regarding the linear function of IMs. Regression coefficients a and b are shown in Fig. 4. b value shows the correlation between the EDP and IMs. The highest value of b shows more correlated IMs and EDP (Padgett et al., 2008).

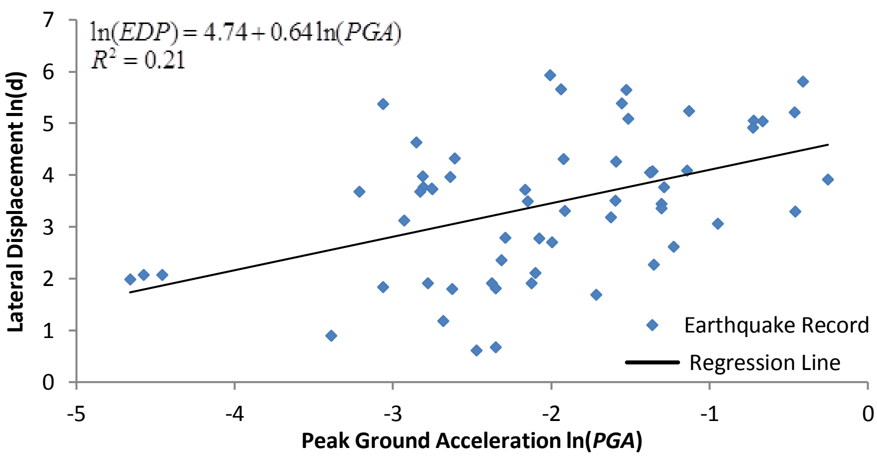

Fig. 4. Probabilistic seismic demand model.

Fragility curves of the bridge were derived considering both single span and multi-span condition. There was limited lateral displacements at the top of the bridge piers, because, the rigidity of the piers. Therefore lateral displacements recorded in the middle of the first span and fourth span were close to each other.

Fig. 5 shows the Fragility curve of the bridge in term of serviceability limit state. Fragility curves were derived for three different velocities that were V<120 km/h, 120 km/h<V<200 km/h and 200 km/h<V, respectively For the Bekdemir bridge %50 probability of exceeding the serviceability limits occured for V>200 km/h at PGA=0.1g for 120<V<200 km/h at PGA=0.175g and for V<120 at PGA=0.475g, respectively.

5. Conclusions

In this study, the earthquake performance of a multi-span steel railway bridge on the Istanbul-Ankara railway line, which is still in service, was determined with the help of fragility curves. Finite element model of the bridge was constructed, and nonlinear time history analyses were carried out for the bridge under the effect of selected 60 different real earthquake data. Relation between the lateral displacements, obtained for the mid-point of the girder bridge spans and IMs were determined by PSDMs. Multi-span lateral displacement limit states specified in EN 1990 Annex 2 were used to derive fragility curves. The PGA values were determined, which resulted in exceeding the boundary condition with %50 probability for multi-span fragility curve. It was seen that the bridge could exceed the limit considered for the serviceability conditions even in the case of small intensity measures. Moreover, the increase of the train speed enhanced the possibility of the bridge exceeding the damage limit state. In the direction of this study, it is suggested that train speeds needs to be limited to the related multi-span steel truss bridge. As a consequence, for the same type bridges, the speed is a limitation consideration that must be taken into account by the local authorities.

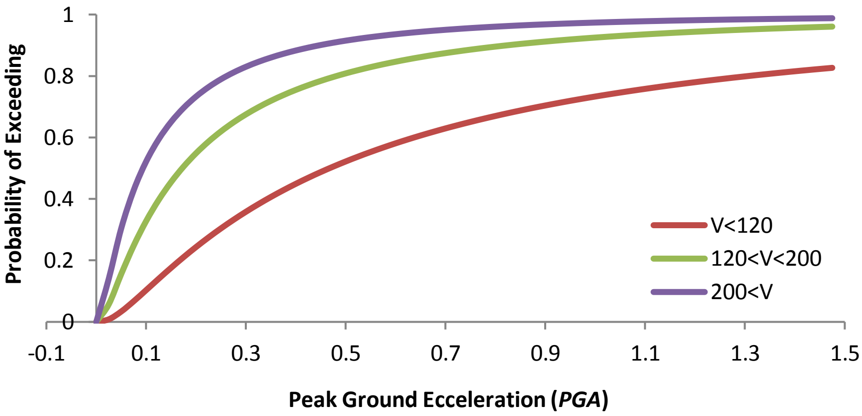

Fig. 5. Multi-span bridge serviceability fragility curve.

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