

Fragility analysis of a historical reinforced concrete arch railway bridge

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Geliş Tarihi (Received Date): 08.08.2019

Kabul Tarihi (Accepted Date): 04.03.2020

Abstract

In this study, fragility analysis of a reinforced concrete arch railway bridge with a total length of 285 m having seven spans of 35 m, a height of 34 m and 15 % slope were performed. The bridge constructed in 1928 still continues to give service. Because the bridge is located in a seismically active region in the southern part of Turkey and on a road, which is critical and important for national railway transportation, it was aimed to perform a probabilistic seismic assessment of the bridge. For this purpose, firstly, 3D finite-element model of the bridge was generated with the software SAP2000 according to the original constructional drawings. Then, the initial FE model was verified using its natural frequencies and mode shapes obtained from in-situ field acceleration measurements. Nonlinear time-history analyses were performed to obtain the seismic demands for 60 different real earthquake records. Probabilistic seismic demand model (PSDM) was derived to determine relations between engineering demand parameter (EDP) and intensity measure (IM). Lateral displacements of the mid-spans were considered as a damage state for three different service velocities. Finally, fragility curves of the bridge were derived.

Keywords: Reinforced concrete arch bridge, railway bridge, fragility analysis, intensity measure, engineering demand parameter.

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Tarihi betonarme kemer bir demiryolu köprüsünün kırılma analizi

Öz

Bu çalışmada her biri 35m açıklık geçen 7 kemere sahip toplam 285m açıklık geçen, 34m yüksekliğinde ve 15 ‰ eğimli betonarme kemer köprüsünün kırılma analizi gerçekleştirilmiştir. İncelenen köprü 1928 yılında inşa edilmiş ve halen hizmet vermeye devam etmektedir. Köprü Türkiye'nin güneyinde aktif tektonik faaliyetlerin sıklıkla gözlemlendiği bir bölgede yer alması ve ülkenin tarihi demiryolu hattı için büyük öneme sahip olması nedeni ile olasılık bazlı sismik değerlendirilmesinin yapılması gereksinimi ortaya çıkmıştır. Bu amaçla, ilk olarak köprüsünün 3D sonlu elemanlar modeli SAP-2000 programı yardımı ile imalat paftaları kullanılarak oluşturulmuştur. Oluşturulan sonlu elemanlar modeli köprü testinden elde edilen ivme ölçümleri yardımı ile belirlenen mod şekilleri ve frekans değerleri kullanılarak iyileştirilmiştir. Köprüde oluşan sismik taleplerin belirlenmesi için 60 farklı gerçek deprem kaydı kullanılarak zaman tanım alanında analizler gerçekleştirilmiştir. Sismik talep ile sarsıntı şiddeti arasındaki ilişkinin tanımlanabilmesi için olasılık bazlı sismik talep modeli kullanılmıştır. Üç farklı kullanım hızı için köprü açıklığının orta noktasının yatay yer değiştirmeleri hasar parametresi olarak kullanılmıştır. Bunlara bağlı son olarak köprüsünün kırılma eğrileri elde edilmiştir.

Anahtar kelimeler: Betonarme kemer köprü, demiryolu köprüsü, kırılma analizi, sarsıntı şiddeti, sismik talep parametresi.

1. Introduction

Bridges, especially old bridges are one of the most fragile and critical components of transportation systems in terms of seismic vulnerability. To minimize potential human losses during, or after a strong earthquake, providing connectivity and operability of transportation network is vital, and so fragility analysis of bridges on a network is very important and inevitable from network vulnerability perspective. As there are many old historical bridges in Turkish railway lines, seismic assessment of these bridges needs to be done to reduce seismic losses. Fragility curve is one of the effective tools used to determine the seismic performance of a bridge. Fragility can be defined as the probability of seismic demand exceeding structural capacity under seismic events [1-3].

There are mainly two approaches to derive a fragility curve, i.e. analytical methods and empirical methods. To derive empirical fragility curve, past earthquake reports or experimental studies and surveys are required, however, it is usually not possible for many bridges. Therefore, analytical fragility curves become more important. To derive an analytical fragility curve, linear or nonlinear dynamic analyses are used. There are lots of studies to derive fragility curves for highway bridges but unfortunately limited such studies for railway and arch bridges. Mackie and Stojadinovic [4] presented a probabilistic seismic demand model for typical reinforced concrete highway overpass bridges in California. Choi et al. [5] developed fragility curves based on various damage states of bearings and columns for four typical highway bridge types in the Central and Southeastern United States and concluded that the most vulnerable bridge types are the

multi-span simply supported and multi-span continuous steel-girder bridges, and the least vulnerable bridge is the multi-span continuous pre-stressed concrete-girder bridge. Nielson [6] derived fragility curves for an ordinary highway bridge in California and determined damage state for supports, abutments, and piers, as well as updated analytical damage limits using past earthquake reports. Banerjee and Shinozuka [7] developed fragility curves using a nonlinear static method for a typical reinforced concrete highway bridge in California. Ozgur [8] obtained fragility curves based on the probability of exceeding specified damage limit states for reinforced concrete highway bridges constructed after the 1990s in Turkey, and it was found that skew and single-column bent bridges are the most vulnerable ones. Kumar and Gardoni [9] proposed probabilistic models to predict the effect of past earthquakes on the structural properties of RC highway bridge columns and their steel reinforcement, and then developed degradation models were used to assess the effects of seismic degradation on the seismic vulnerability of a RC highway bridge with one single-column bent. Yılmaz and Çağlayan [10] used both lateral displacement limit state and capacities of its structural members to derive fragility curves of a multi-span simply supported steel truss railway bridge. Pela et al. [11] performed nonlinear static (pushover) analysis and nonlinear time history analyses for seismic capacity assessment of an existing brick masonry triple-arched bridge and showed that the pushover results can slightly overestimate the time-history average predictions. De Santis and De Felice [12] proposed a fiber beam-based methodology to assess the seismic capacity of masonry arches and arch bridges using pushover analyses under different load distributions and nonlinear incremental dynamic analyses under earthquake ground motions. Pellegrino et al. [13] presented that use of in situ and laboratory tests for seismic vulnerability assessment of bridges may be a useful instrument to improve seismic assessment. Marefat et al. [14] carried out pushover analyses for seismic assessment of plain concrete arch railway bridges using two-dimensional finite element models calibrated by using results of in-situ field dynamic load tests.

In this paper, the probabilistic seismic assessment of an existing reinforced concrete arch railway bridge was performed using 3D finite-element modeling. The bridge having a span length 285 m long is an upper-deck arch bridge with reinforced concrete (RC) arch ribs, spandrel columns and a deck slab. A 3D finite-element model was generated using the general-purpose structural analysis and design software SAP2000 [15]. The FE model of the bridge was verified using the results of dynamic in-situ field tests. 60 real earthquake recordings were selected to represent seismicity of the site considering PGA, moment magnitude, central distance, and fault type. These records were used for nonlinear time history analysis of the FE model. Lateral displacements of the mid-spans were considered as slight damage for three different service velocities given by EN 1990 Annex2 [16]. Finally, the demand levels of the bridge were probabilistically compared with the bridge capacity using the obtained FE analysis results.

2. Description of the bridge

This existing reinforced concrete arch bridge (see Fig. 1) with a total length of 285 m and having seven same arch spans of 35 m was designed and built by “Nydqvist & Holm A.B.; J. Saabye & O. Lerche; Kampmann, Kierulff & Saxild” in 1928. Deck floor is supported by vertical members which is carried by the arch system. Thickness and

width of deck floor carrying about 35 cm deep ballast layer, concrete sleepers and rails is 35 cm and 4 m, respectively. The bridge having a single-track railway line is composed of the reinforced concrete deck floor, vertical members, arches having a span length of 30.5 m and a height of 17 m besides massive concrete piers varying in heights from 3 m to 18 m. While thickness of the reinforced concrete arches increase from 70 cm at the mid-spans to 110 cm at the cap of the piers, their widths vary from 4 m to 5 m, respectively.

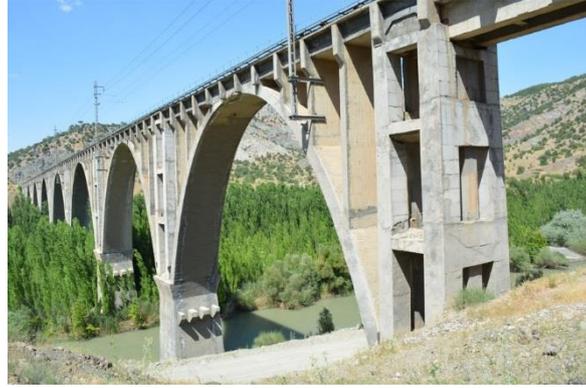


Figure 1. A general view of the multi-span reinforced concrete arch bridge.

This bridge, crossing Göksu River and having a height of 34 m and 15 ‰ slope, is located in about 105 km south-west of Malatya city and 8 km north-east of Gölbaşı county of Adıyaman city on Narlı-Malatya railway line (see Figs. 2 and 3). Besides, the bridge is placed in an active seismic prone region in the southern-east of Turkey. Specially, the bridge location is very close to the Gölbaşı-Türkoğlu and Çelikhan-Gölbaşı segments of the East Anatolian Fault [18] which compose a left lateral strike-slip fault zone resulting from intersection of the Anatolian plate and the Arabian plate (see Fig. 3). Considering historical destructive earthquakes larger than the magnitude $M_w=6.4$ produced by the East Anatolian Fault segments [18], it is predictable that each segment may generate an earthquake having magnitude $M_w \geq 7.0$ in the future.

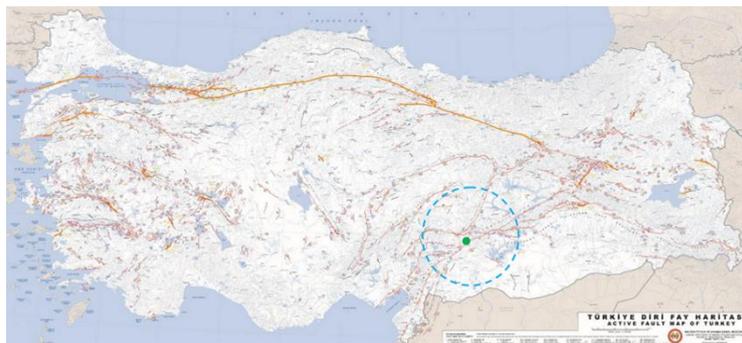


Figure 2. Bridge location (green circle) on the active fault map of Turkey produced by Emre et al. [17].

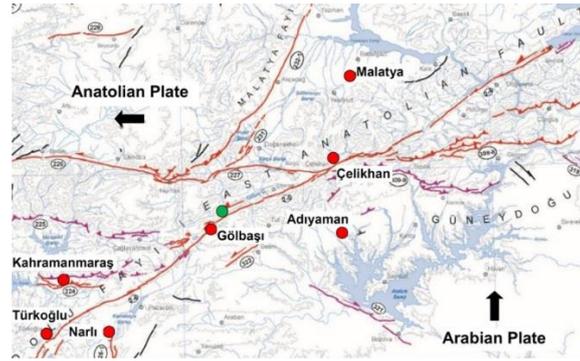


Figure 3. Close-up view of the bridge location on the active fault map of Turkey produced by Emre et al. [17].

3. In-situ field dynamic tests of the bridge

To obtain the acceleration response of the bridge, a test train (see Fig. 4) consisting of a DE24000 type diesel locomotive and passenger cars provided by Turkish State Railways Administration (TCDD) was used for twelve passages to and fro, so that data records in vertical, lateral and longitudinal directions for each passage were collected by using a total of sixteen accelerometers placed on the mid-span points at each side of the bridge (see Figs. 5 and 6) coupled with a dynamic data-acquisition system having 16 channels. Each passage was performed after the bridge damped out.



Figure 4. Dynamic loading by test train with DE24000 type locomotive.

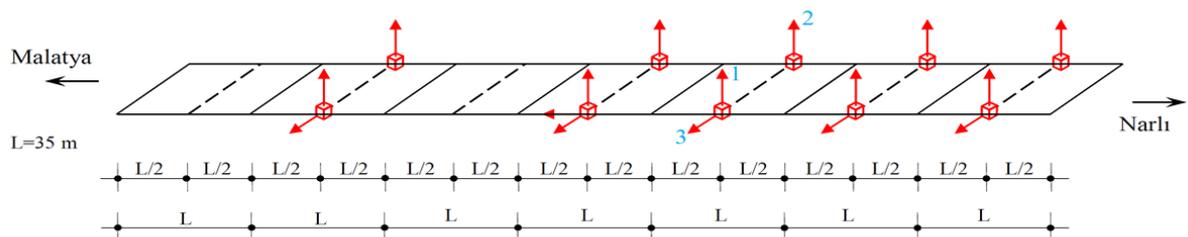


Figure 5. Layout of the accelerometers.



Figure 6. Views of accelerometers placed on the bridge deck; a) biaxial and b) single.

The free vibration data with a sampling rate of 100 Hz from each channel were captured after the test train had totally left the bridge. The signals in a raw form given in Fig. 7 represents a sample of acceleration records for a passage. After collected acceleration data in a raw form were preprocessed, the first three natural frequencies of the bridge were obtained by using Fast Fourier Transforms (FFT) technique (see Table 1).

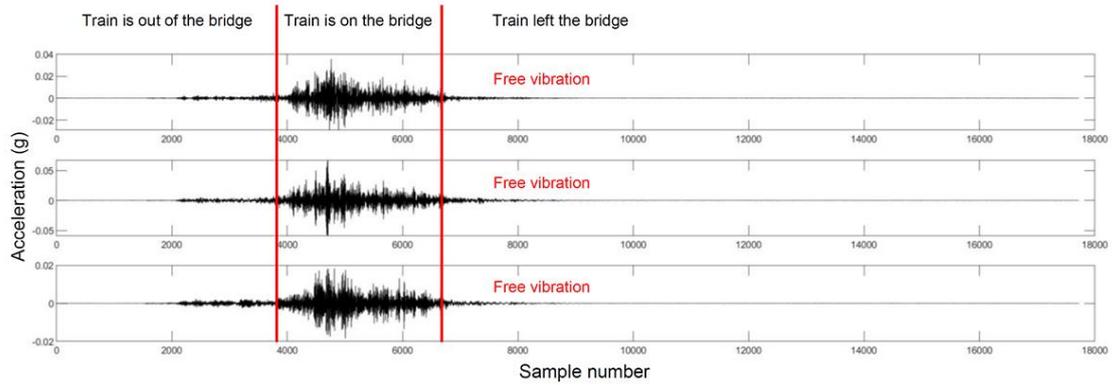


Figure 7. Sample acceleration records at points 1, 2 and 3 on the bridge, respectively.

4. Finite-element modeling of the bridge

3D finite-element model of the bridge was generated by using a general analysis and design software SAP2000 [15] based on its original constructional design drawings. The prepared FE model has a total of 8644 hexahedron solid (8-node brick) elements and 15947 nodes (see Fig. 8). Maximum size for the used solid elements was chosen as 1.2 m. All the bridge model rests on springs, and soil-structure connection was performed by using massless shell elements connected to these area springs.

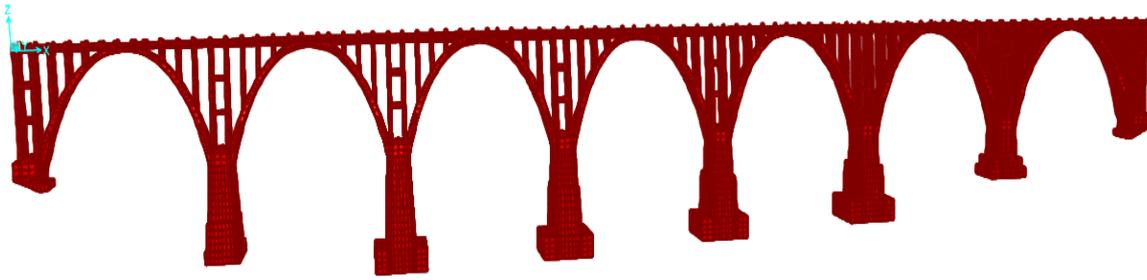


Figure 8. View of 3D finite-element model of the bridge.

To calibrate the initial FE model, the experimental natural frequencies and corresponding mode shapes obtained from acceleration recordings of the dynamic tests were compared with the analytical values derived by using the initial FE model that was developed based on the original constructional design drawings. The FE model was updated by altering rigidities of spring elements at support points until a reasonable matching occurred (see Table 1). Adjustment of the analytical and experimental mode shapes was achieved by employing of the MAC (model assurance criteria) function (see Fig. 9). The MAC results were approximately obtained as 0.97, which can be assumed as a good match.

Table 1. Experimental and analytical frequencies of the bridge.

Mode	Experimental natural frequency (Hz)	Analytical natural frequency (Hz)	Relative error (%)
1st lateral bending	1.850	1.863	0.700
1st torsional	1.953	1.913	2.050
1st vertical bending	2.130	2.087	2.019

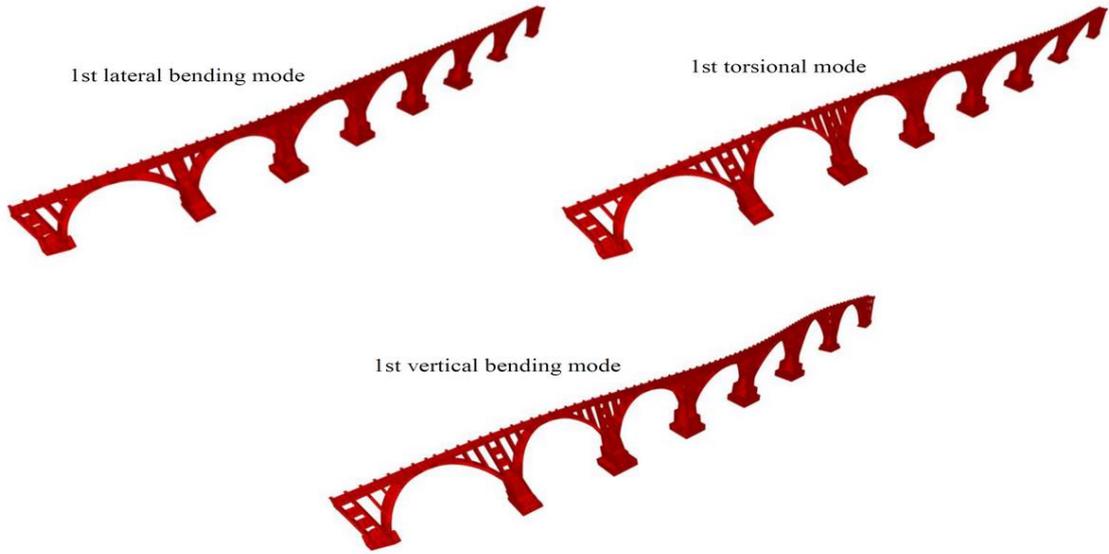


Figure 9. First three analytical mode shapes of the bridge.

5. Probabilistic seismic demand model

Probabilistic seismic demand model (PSDM) is derived to determine relations between engineering demand parameters (EDPs) and intensity measures (IMs) or used to describe seismic demand of a structure in terms of intensity measure, as follows,

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

The median EDP can be estimated by a power model described with Eq. (2), or a linear logarithm model can be given in Eq. (3).

$$\widehat{EDP} = aIM^b \quad (2)$$

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

Where IM is the seismic intensity measure, a and b are regression coefficients, ϕ Is the standard normal cumulative distribution function, \widehat{EDP} is the median value of engineering demand, d is the limit state used to assess the damage level, and $\beta_{EDP/IM}$ (dispersion) is the conditional standard deviation of the regression as given in Eq. (4).

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

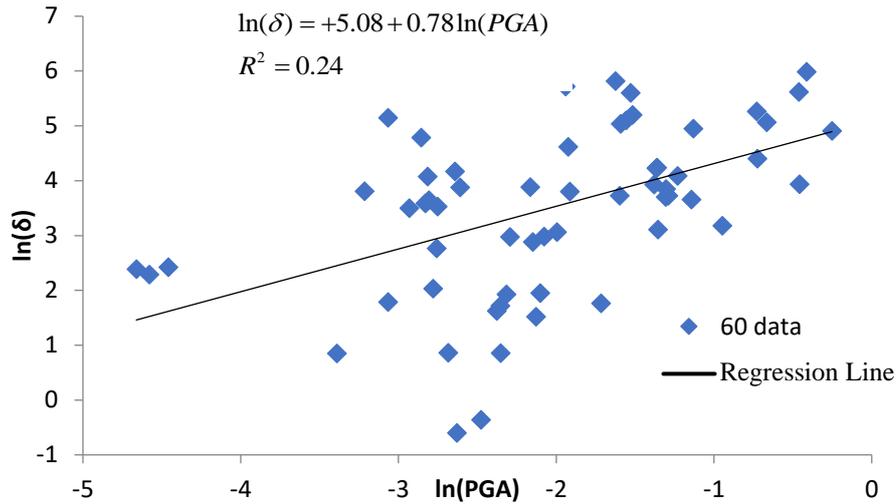


Figure 10. Probabilistic seismic demand model (PSDM) of the bridge

Fig. 10 shows the relation between EDP and IMs which can be defined by a linear equation. Eq. (3) is used to determine PSDM of the bridge based on results of the nonlinear time history analysis.

To derive PSDMs, the linear or nonlinear analysis needs to be performed. The nonlinear time history analysis gives more realistic results. There are three methods to derive PSDM depend on nonlinear time history analysis; these are cloud, incremental dynamic analysis and stripe method [19]. In this study, the cloud method was used. The cloud method includes results of nonlinear time history analyses achieved by using a group of earthquake records without scaling. As the results of the nonlinear analysis are depending on selected earthquake records, the earthquake record domain has an important effect on the PSDMs.

6. Selection of the earthquake records

Selection of an earthquake record is one of the important steps to derive an analytical fragility curve. Characteristic properties of a selected earthquake record constitute an important uncertainty in seismic demand [20]. It is aimed to represent different earthquake hazards through the selection of the earthquake records. One of the most important parameters affecting the characteristics of the earthquake is the soil type. In this study, totally 60 different earthquake records were selected considering different soil types, moment magnitudes, PGAs and central distances. The moment magnitudes are varying between 4.9 and 7.4, and PGAs are changing from 0.01g to 0.82g where the central distances of earthquake records are ranging from 2.5 km to 217.4 km. Distribution of moment magnitudes to central distances is shown in Fig. 11. As one of the most important parameters affecting the characteristics of the earthquake is the soil type, where the maximum accelerations were classified between 0.1s and 0.3s for soil type A, 0.15s and 0.5s for soil type B and 0.1s and 0.9s for soil type C. The selected earthquake records were used for time history analysis without scaling.

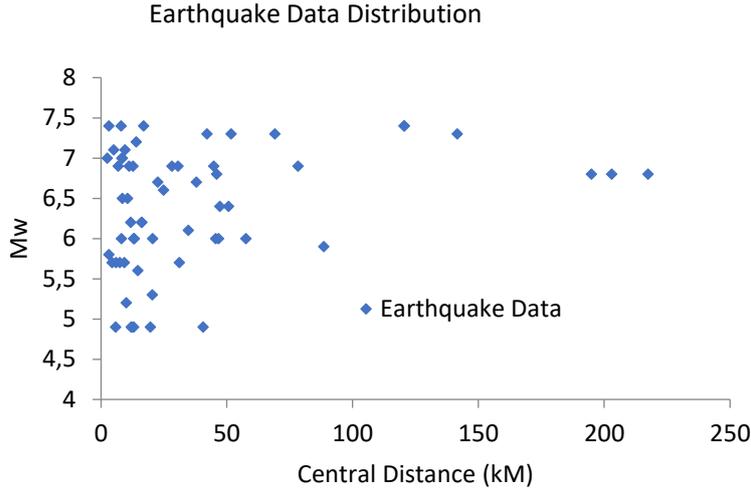


Figure 11. Moment magnitude and central distance distribution of the earthquake records selected.

7. Analytical fragility curves of the bridge

In past earthquakes, ground-motion-induced various damages observed for bridges were reported in literature as shear failure of piers, unseating of bridge spans due to excessive relative movement of spans, loss of supports due to liquefaction and excessive lateral movements, embankment failures, track damages such as broken rails and joints, buckled tracks, parapet and spandrel wall failures due to outward movement of piers and abutments parapets and spandrel walls, bearing and anchor bolt damages, derailments and overturning collapse of locomotives and cars due to settlement of tracks and ground shaking [21, 22]. In this study, as lateral displacement of bridges is a destructive case in terms of the most important vulnerability, lateral displacements of the bridge spans were considered as a serviceability damage state. The lateral displacement limits given in EN1990-Annex A2 [16] assuming different service velocities for railway bridges were used (see Table 2). The horizontal deflections of the bridge deck are limited by EN 1990-Annex A2 [16] to sustain traffic safety for the railway line. To ensure traffic safety, fragility curve of the bridge can be derived depending on horizontal displacement limits.

Table 2. Maximum angular variation and minimum radius of curvature [16].

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

Fragility curves of the bridge were derived considering maximum damage probability for all IM levels and using probabilistic seismic demand model [23]. Probabilities of exceeding of serviceability limit states are shown in Fig. 12. Serviceability limit states were considered as slight damage [3]. It was found that %50 probability of exceeding limit states for $V < 120$ is $0.35g$, $120 < V < 200$ is 0.1 and finally $200 < V$ is $0.05g$. The results show that the decrease in speed range increases the safety of the bridge.

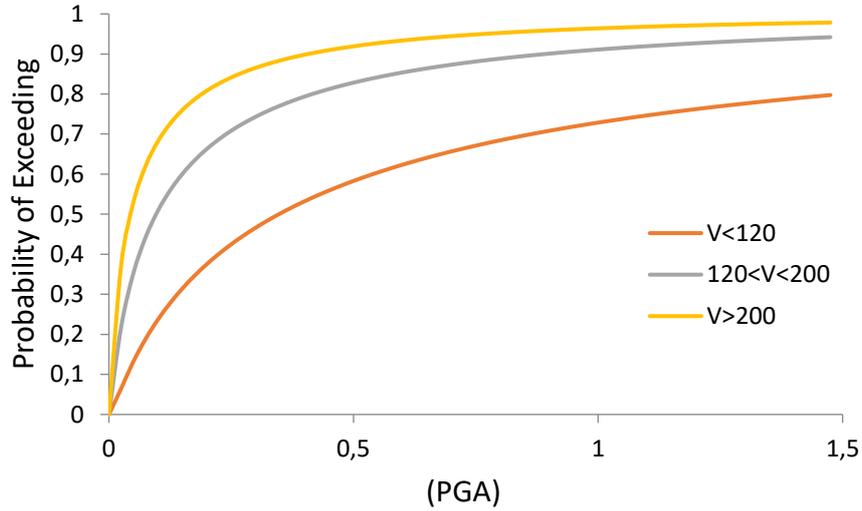


Figure 12. Fragility curves of the bridge.

8. Conclusions

This study presents the probabilistic seismic assessment of an old reinforced concrete arch railway bridge still in service on Turkish railway line between Narlı and Malatya. 3D finite element model of the bridge was generated, and the nonlinear time history analyses were performed for 60 different real ground motion records selected considering different characteristic properties. Finally, fragility curves for the bridge were derived based on probabilistic seismic demand model. Regression analysis was conducted to determine mean values and dispersion, and two-parameter log-normal distribution function was used to derive the fragility curve. Damage state for this railway bridge was determined considering lateral displacement of the spans. The fragility curves of the bridge were derived considering these damage limits. Serviceability damage limits were considered as slight damage state. It was found that to increase the safety of the bridge, the speed range should be decreased.

Acknowledgments

This research presented in this paper was supported by TUBITAK 114M332 project and Turkish Railways Administration (TCDD). Any opinions expressed in this paper are those of authors and do not reflect the opinions of the supporting agencies.

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