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Selection of Intensity Measure in Probabilistic Seismic Risk Assessment of a Turkish Railway Bridge

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Abstract**—**Fragility curve is an effective common used tool to determine the earthquake performance of structural and nonstructural components. Also, it is used to determine the nonlinear behavior of bridges. There are many historical bridges in the Turkish railway network; the earthquake performances of these bridges are needed to be investigated. To derive fragility curve Intensity measures (IMs) and Engineering demand parameters (EDP) are needed to be determined. And the relation between IMs and EDP are needed to be derived. In this study, a typical simply supported steel girder riveted railway bridge is studied. Fragility curves of this bridge are derived by two parameters lognormal distribution. Time history analyses are done for selected 60 real earthquake data to determine the relation between IMs and EDP. Moreover, efficiency, practicality, and sufficiency of three different IMs are discussed. PGA, Sa(0.2s) and Sa(1s), the most common used IMs parameters for fragility curve in the literature, are taken into consideration in terms of efficiency, practicality and sufficiency.

Keywords**—**Railway bridges, earthquake performance, fragility analyses, selection of intensity measures.

I. Introduction

THE railway system is an important transportation system for both passenger and load transportation. In Turkey, 82% of railway bridges were built before 1960 [1], and without any seismic code requirements. The seismic behavior of these bridges needs to be determined to control the safety and continuity of this railway system. Fragility analysis is an effective tool to determine the seismic behavior of bridges [2]. 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.

There are four types of fragility curve: Empirical, Expert opinion, Analytical, and Hybrid. Analytical fragility curve is most easy to apply because of no need for any information about past events and records and reports. All information is obtained by the numerical analysis of the structure. Although there are variety of nonlinear static and dynamic procedures, nonlinear time history analysis is the most reliable procedure [3]. Obtaining the result of the nonlinear time history analysis, probabilistic seismic demand model (PSDM) of structure can be derived [4]. Two coefficients of linear regression analysis are obtained by deriving power model for PSDM. There are three methods Cloud, Incremental Dynamic Analysis, and Stripe to derive PSDM [5]. Cloud method is used in this study that includes ground motion without any prior scaling.

An important step to derive PSDM is a selection of IM. There are several proposals to assign the optimum IM defined as practicality, effectiveness and sufficiency [5]-[8]; moreover, there are many different parameters that are used as IMs such as PGA and Sa. Fragility curves can be expressed as two-parameter (median and log standard deviation) lognormal distribution function [3]. The median and log standard deviation is the only needed variables to derive the fragility curve. The SAC-FEMA methods and Maximum Likelihood methods can be used to determine these parameters [4], [9].

This paper presents a discussion on optimum in IM for simply supported steel riveted railway bridge. PGA, Sa(0.2s) and Sa(1s) are chosen as the IMs and the practicality, efficiency and sufficiency of these IMs are discussed in detail, and the fragility curves of the bridge are derived.

II. Illustration of Steel Railway Bridges

In Turkey, the railway construction started with the contribution of European countries such as England, France, and Germany. The main aim of the railway network usage harbors for easy transfer of goods to Europe and beyond. The first railway line was constructed by an English company between Izmir and Aydın in 1856, and the total length is 130 km [1]. Railway lines are divided into seven regions in Turkey for ease of maintenance, repair and operation. The total length of the railway lines is 8,722 km, with 25,443 culverts and bridges in the inventory; 82% of which were built before 1960. Hence, the Turkish railway line includes many historical and monumental bridges.

This study focuses on a simply supported steel railway bridge that is located on the Manisa-Uşak-Dumlupınar-Afyon railway line. The bridge is a simply supported one span bridge with 22.4 m length and main girder of the beam's height is 1.83m (built up section composed of plates and angles connected to each other with rivets to form I section), the beam with steel plate and angle elements are connected with rivets. The stringers and transverse beams are IPN450, IPN300. The bridge is a riveted bridge, and used steel quality is st 37 (S235).

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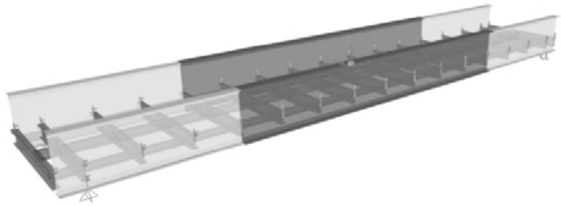

Fig. 1 3D extrude view of bridge model

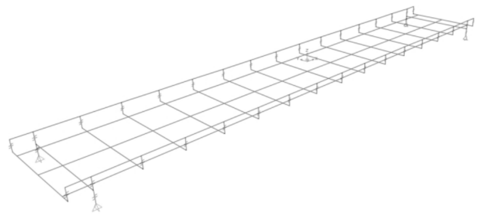

Fig. 2 3D frame view of bridge model

III. Selection of Earthquake Data

The effect of a ground motion on the structure is obtained by using linear or nonlinear mathematical models of the structure. A nonlinear dynamic time history analysis minimizes the uncertainty of structural responses. The relation between ground motion IMs and EDP can be obtained depending on time history analysis. These relations can be obtained by one of the third methods named cloud (direct) method [10], Incremental dynamic analysis (IDA) [11], and stripes method. In this study cloud method is used to represent the relation of IMs and EDP. Cloud method includes numerous selection of real ground motion record and uses the selected ground motion without any prior scaling [5], [12], [13].

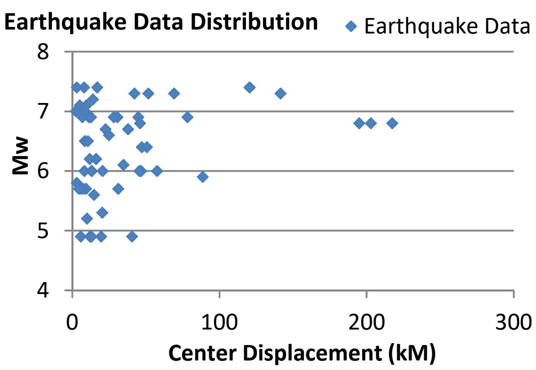

Fig. 3 Moment and Center Displacement Distribution of Earthquake Record

In this study, earthquake data are selected considering different soil types, moment magnitudes, PGAs and central distance of the earthquake records. The moment magnitudes are changing from 4.9 to 7.4 and PGAs are changing from 0.01 g -0.82 g, and the central distance of earthquake record is changing from 2.5 km - 217.4 km. The distribution of the moment magnitude to PGA and central distance are shown in Fig. 1. A total of 60 different real earthquake data are chosen for the study from A, B and C type of soil. Earthquake data are used for time history analysis without scaling.

IV. Finite Element Modeling and Analyzing

All the elements of the bridges are modeled by 2- node beam element. According to shop drawings and site visual inspections; the computer model is created more realistically with the elements, supports and their connections to each other. As an example, due to the height differences between the bridge members, eccentricity at the connection points is taken into account during modeling of the bridge.

The weight of the sleepers and rails are taken into account and applied to the dead load at the appropriate nodes. And the weight of perforated plates and gusset plates are ignored. Materials of the bridges are assumed as ST37, which fits for the construction years of the bridge. The only load that is considered during the modal analysis is dead load, but for the fragility analysis, the train load is also taken into account as mass. Sap2000 is used to model the bridge. Finite element model is composed of 114 frame, 38 link elements, and 124 nodes.

Time history analyses are applied to the model with considering both material and geometric nonlinearity. Plastic hinges are defined as steel interacting PMM plastic hinges from FEMA 356. In order to detect any hazard on bridges, plastic hinges are defined at the start point, end point and midpoint of all beams. Geometric nonlinearity is defined as Δ-δ with large displacement and Newmark direct integration is used in the analysis. Three components of the earthquake, one longitudinal and two horizontal directions are defined in the time history process.

V. Probabilistic Seismic Demand Analysis

When using analytical procedure PSMD describe the seismic demand of a structure or structural component in terms of approximate IM. PSDMs can be written as (1):

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

Estimation of median EDP are describe as a power model as given in (2) and (3) [4]:

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

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

IM is the seismic intensity measure, and a and b are the regression coefficient. ϕ is the standard normal cumulative distribution function, $\hat{EDP}$ is the median value of engineering demand, d is the limit state to determine the damage level and $\beta_{EDP|IM}$ (dispersion) is the conditional standard deviation of the regression in (4).

TABLE I
DEMAND MODEL AND IM COMPARISON

	x			y			z		
	a	b	$\beta_{EDP\|IM}$	a	b	$\beta_{EDP\|IM}$	a	b	$\beta_{EDP\|IM}$
PGA	35.02	0.56	0.45	63.38	0.86	0.84	10.14	0.27	0.61
Sa0.2	18.77	0.52	0.44	16.09	0.30	1.13	14.17	0.21	0.63
Sa(1s)	29.13	0.49	0.40	33.52	0.55	0.97	11.04	0.24	0.60

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

VI. DISCUSSION ON IM

Practicality is described as the correlation between an IM and demand screened on a structure or structural component. The more practical IM means a higher correlation of demand and IMs. The practicality of an IM can be evaluated with the regression parameter b of PSDM. The higher value of b shows more practical IMs [8].

Efficiency is described as alteration of demand for a given IMs and can be measured with dispersion. The smaller dispersion means the more efficient IMs [8].

For x direction, the maximum b value is 0.56 and for y-direction, the maximum b value is 0.86, and for z direction, the maximum b value is 0.27. The result of this analysis shows that PGA is more practical than other IMs parameter.

The minimum values of $\beta_{EDP|IM}$ dispersions are for x-direction 0.40, for y-direction 0.84 and for z-direction 0.60. The smaller dispersions obtained are Sa(1.s) for x and z-direction PGA for y-direction.

Sufficiency describes the statistical independencies of IMs and ground motion characteristics such as M magnitude and R distance [8]. Sufficient IM needs to describe the structural demand independently probable earthquake scenario. Sufficiency of IMs can be determined by the statistical analysis of ground motion characteristics and IMs [6], and to determine the sufficiency of IMs residual regression analysis are conducted for IMs and ground motion magnitude, and p-values are determined. The smaller p-value shows the sufficient IM.

TABLE II
P VALUES OF RESIDUAL REGRESSION ANALYSIS FOR MAGNITUDE (M) AND DISTANCE (R)

		PGA	Sa (0.2s)	Sa (1s)
Magnitude (M)	x		0.1808	0.2216
	y	0.0558	0.3317	0.0519
	z		0.2293	0.0755
Distance R	x		0.0001	0.0157
	y	0.0004	0.0003	0.0237
	z		0.0030	0.0547

Magnitude (M) and Distance (R) are the chosen ground motion characteristics for determining the sufficiency of IMs. Table II shows the p values of residual regression analysis for magnitude (M) and Distance (R) Minimum p-value for magnitude is 0.051 and minimum p-value for Distance (R) is 0.0001. Sa(1.s) is more sufficient IMs for Magnitude and Sa(0.2s) is more sufficient IMs for Distance. Sa(1.0s) can be used for IMs as more practically, efficiently and sufficiently than other IMs but Sa(0.2s) is sufficient for distance. The conditional independence of Sa(1.0s) from magnitude is shown in Fig. 4 with respect to the conditional independence of Sa(0.2s) from distance, shown in Fig. 5.

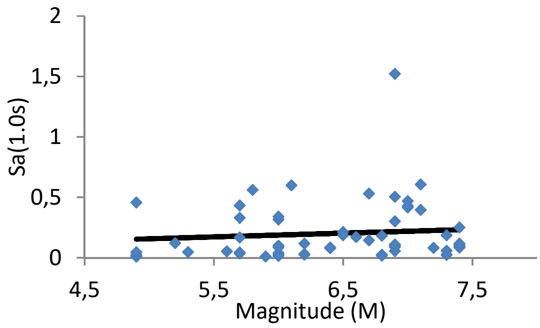

Fig. 4 Sufficiency of Sa(1.0s) for magnitude M

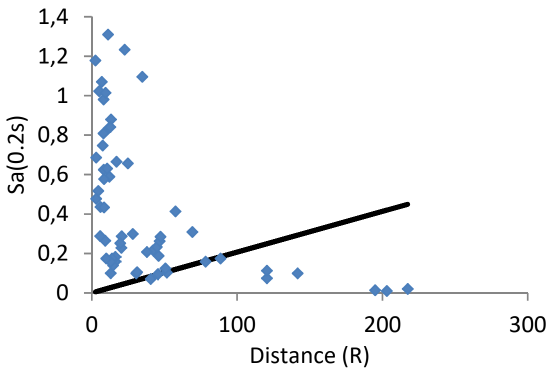

Fig. 5 Sufficiency of Sa(0.2s) for Distance R

VII. FRAGILITY ANALYSIS

Some 60 different nonlinear time history analysis of the bridge are performed and joint displacements are saved. The maximum span displacements and maximum PGA are plotted on a log-log scale, as shown in Fig. 6. A regression analysis is performed to show the relation between the EDP and Earthquake IMs (PGA). The relation between EDP and IMs are defined with (5) [4]:

$$\ln(\widehat{EDP}) = \ln(a) + b\ln(IM) \tag{5}$$

$$\ln(EDP) = 4.14 + 0.85x\ln(PGA)$$

$$R^2 = 0.49$$

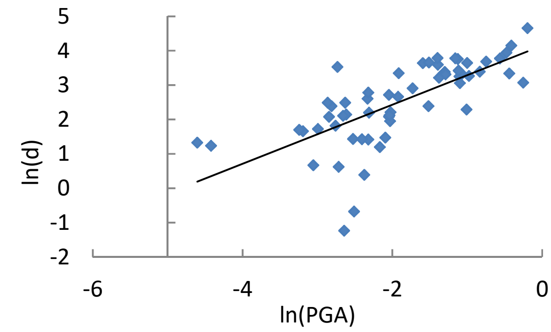

Fig. 6 Log-Log Distribution of EDP and PGA

IM is selected as PGA because of conducted practicality, efficiency and sufficiency analysis and both a and b are coefficients determined by least square linear regression analysis and (Dispersion) measured about standard deviation. PSDMs can be written as (6):

$$P[EDP > d \,|IM|] = 1 - \phi\left(\frac{\ln(d) - \ln(aIM^b)}{\beta_{EDP/IM}}\right) \quad (6)$$

Lateral deformation limit states for railway bridges are defined by EN 1990 Annex A2. As shown in Table III.

TABLE III
LATERAL DISPLACEMENT LIMIT STATE FOR RAILWAY BRIDGE

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

According to the limit state defined by annex2, lateral displacement limits for selected bridges are 12.35 mm for V≤120 km/h 3.56mm for 120<V≤200 km/h and 1.52 mm for V>200 km/h.

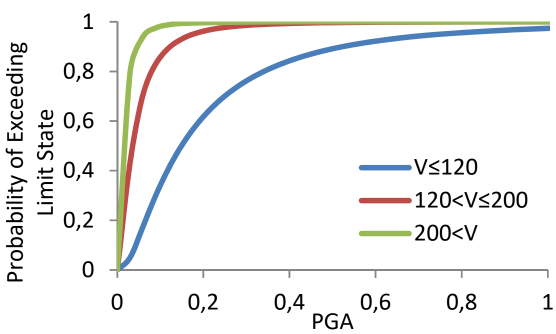

Fig. 7 Probability of Exceeding Lateral Displacement Limit State

While conducting nonlinear time history analyses, it is assumed that there is UIC train on the railway bridges and mass of the train is taken into account while conducting nonlinear time history analysis. Because the steel railway bridge is light weight compared with the weight of the train, the position of the train on the bridge is affecting the nonlinear behavior of bridge dramatically. Fig. 7 shows the probability of exceeding the lateral displacement limit state of the simply supported railway bridges under earthquake condition. Because less plastic deformation occurs on the main girder, there is limited residual lateral displacement, and thus, the railway bridge is mostly safe after earthquakes. But from the lateral displacement limit point of view under seismic action, even a small magnitude of seismic action acting on the bridge may cause the lateral displacement to exceed the limit state for service speed V>200 km/h when the train is on the bridge. This phenomenon is affecting the probability of exceeding of the limit state dramatically.

VIII. Conclusion

These studies include a selection of IM for a simply supported riveted bridge. PGA, Sa(0.2s) and Sa(1.0s) are selected as IMs and they are compared in terms of practicality, efficiency, and sufficiency. After conducting many statistical analyses PGA is found to be more practical, more efficient. Sa(1.0s) is more sufficient for Magnitude(M) and Sa(0.2s) is more sufficient for Distance (R). Moreover, fragility curve for a simply supported steel railway bridge is derived. Used lateral displacement limit state for railway bridges are given in EN 1990 Annex A2. The fragility curves show that during seismic action, railway bridges are more susceptible to exceed the lateral displacement limits. It is proposed that PGA can be used as an IM for a simply supported steel railway bridge and that the railway system needs to be temporally stopped during times of seismic activity.

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