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BRIDGES OF EGNATIA MOTORWAY IN NORTHERN GREECE: SEISMIC RISK ASSESSMENT BY FRAGILITY CURVES

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Dedicated to the memory of Yordan MILEV, (15.2.1960 – 8.1.2017), Late Professor at the University of Architecture, Civil Engineering and Geodesy (UACEG), Sofia, Bulgaria

Keywords: *earthquake structural engineering, Egnatia Motorway bridges, vulnerability functions*

ABSTRACT

The present paper deals with the problem of computing analytically fragility curves of bridges. The methodology presented involves the use of the Finite Element Method (FEM) and of the non-linear static pushover analysis for the computation of the capacity curve of the bridges in combination with inelastic demand spectra for the estimation of the degree of damage for a given peak ground acceleration (PGA). Emphasis is given here for applying the developed methodology to some special bridges of the motorway Egnatia Odos, in Northern Greece. The vulnerability analysis of these bridges represents a critically important step in their seismic damage estimation process. The relevant fragility curves provide the probability that a specific damage level will be exceeded for a given intensity of a seismic event.

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1. Introduction

As is well known, see e.g. [1 – 5], a key element in formulating mitigation and disaster planning strategies in Earthquake Engineering is the estimation of the urban seismic risk. In this respect, development of vulnerability relationships for both the existing and under design Civil Engineering structures represents a critically important step in damage estimation process. Scope of the vulnerability analysis is the creation of the so-called fragility curves [1 – 4, 9 – 11], through which the probability that a specific damage level will be exceeded for a given intensity of a seismic event may be quickly estimated, supporting significantly the decision-making procedures. So, fragility curves for Civil Engineering Structures, such as buildings and especially bridges, are a useful tool for the assessment of the damage they may sustain for a certain level of earthquake shaking. In combination with seismic hazard analysis at the bridge sites, they can lead to a reliable assessment of the seismic risk of highways. Furthermore, they can even be used by the authorities in charge to prioritize the on site aftershock inspections, in order to check the structural integrity of the bridges subjected to a severe seismic event.

In recent literature, several methodologies dealing with the assessment of fragility curves for bridges can be found, based on either empirical or analytical procedures [2 – 3, 9 – 11]. Also, methodologies originally proposed for buildings can sometimes be extended for use in the case of bridges [1 – 4, 14].

The present paper deals with a simplified analytical methodology for the evaluation of vulnerability curves for bridges having deck on precast beams, seating through elastomeric bearings on the piers and with seismic stoppers. The methodology combines the nonlinear static pushover procedure and the capacity spectrum method [1 – 4, 9 – 11], and in connection to the details of [4] is applied for establishing fragility curves for an existing reinforced concrete bridge in the Kavala section of Egnatia Motorway, in the county of East Macedonia, Northern Greece.

Egnatia Odos is a new motorway that crosses Northern Greece in an E-W direction. It is currently the largest and technically the most demanding highway project in Greece, and one of the biggest ones under recent (2008 – 2009) construction in Europe. Moreover, for the design and construction of Egnatia Motorway, a lot of Applied Mechanics topics are involved, e.g. structural and seismic mechanics, geotechnical and transport engineering, hydraulic and environmental engineering, etc. So, Egnatia Motorway can be considered as an active field of Applied Mechanics and Structural Engineering. Its main axis has a length of 670 km and includes about 1900 special structures (bridges, tunnels and culverts). These structures are expected to withstand several minor or moderate earthquakes during their life, and may be damaged if they are subjected to a major (catastrophic) earthquake. So, the construction of their fragility curves is very significant. The Kavala bridge examined herein is structurally a representative one of many bridges in Egnatia Motorway, and in Greece more generally.

2. Methods for Assessing Structural Vulnerability

The vulnerability functions, required for the fragility curves, are expressed [2 – 4, 9 – 11] in terms of a Lognormal cumulative probability function in the form of the next eq. (1):

$$P_f\left(DP \geq DP_i \mid S\right) = \Phi\left[\frac{1}{\beta_{tot}} \cdot \ln\left(\frac{S}{S_{mi}}\right)\right]. \tag{1}$$

Here $P_f\,(\cdot)$ is the probability of the damage parameter *DP* being at, or exceeding, the value DP_i for the i-th damage state for a given seismic intensity level defined by the earthquake parameter *S* (here the Peak Ground Acceleration-PGA or Spectral Displacement-S_d), Φ is the standard cumulative probability function, S_{mi} is the median threshold value of the earthquake parameter *S* required to cause the i-th damage state, and β_{tot} is the total lognormal standard deviation. Thus, the description of the fragility curve involves the two parameters, S_{mi} and β_{tot}, which must be determined.

Now we consider briefly the problem of computing the vulnerability functions (1) for Civil Engineering Structures, such as buildings and especially bridges. For the latter ones, the case of reinforced concrete bridges with seismic stoppers is herein investigated. This case is a contact mechanics problem. So, such bridges can be considered as nonlinear elastic and inelastic systems with impacts which arise in mechanical and civil engineering applications. In Civil Engineering applications, such systems arise also, besides in the above analysis of bridges with seismic stoppers, in the analysis of pounding of adjacent buildings.

Next, the general problem of the seismic pounding of adjacent structures is briefly described. This problem belongs to the so-called Dynamic Inequality Problems of Mechanics, for which a strict mathematical treatment can be obtained by using the variational or hemivariational inequality concept. As is well known, the latter one has been introduced in Mechanics by P. D. Panagiotopoulos [5]. As far as their numerical treatment is concerned, many significant contributions are already available, see e.g. [5, 6]. So, for the case of two interacting structures (A) and (B), following e.g. the procedure of [7, 8], the problem is first formulated as an inequality one by using concepts of Non-Convex Analysis. Next, double discretization, in space by the Finite Element Method and in time by a direct-time integration scheme (e.g. the central difference method), and optimization methods are used. Thus, by piecewise linearization of the interface unilateral contact laws, at each time-step a nonconvex linear complementarity problem of the following matrix form with reduced number of unknowns is finally solved:

$$\mathbf{v} \geq \mathbf{0}\,, \quad \mathbf{A}\mathbf{v}+\mathbf{a} \leq \mathbf{0}\,, \quad \mathbf{v}^T.\left(\mathbf{A}\mathbf{v}+\mathbf{a}\right) = \mathbf{0}\,. \tag{2}$$

So, the nonlinear Response Time-History (RTH) for a given seismic ground excitation can be computed.

As was mentioned in the Introduction, the present study focuses on the simplified practical fragility analysis of bridges, that involve impacts due to the seismic stoppers designed to effectively withstand earthquake loads and reduce the size of the piers. For such a practical simplified analysis, these systems are represented by single and multi degree of freedom models with piecewise linear elastic stiffness elements that often involve strong inelastic behavior in parts of the system. So, the previous general approach for pounding of adjacent structures is simplified by considering the simple bridge with seismic stoppers shown in Fig. 1a. The bridge deck is connected to the piers by elastomeric bearings and seismic stoppers are added on the pier caps that have a small gap with the deck structure so that the elastomeric bearings are free to move under ambient or traffic loads, while they impact on the stoppers only under moderate or strong earthquake loads. Activation of the stoppers due to impact results in sudden increase of the stiffness of the structure. The gaps between the stoppers and the bearings are usually selected such that the impact with the stoppers occurs before the pier yielding.

Further, assuming a heavy undeformed deck of mass *M* and representing the stiffness of the piers and the elastomeric bearing by massless linear or inelastic springs, one can construct a single degree of freedom (SDOF) simplified model of the bridge as shown in Fig. 1a. For the case of stopper activation but no pier yielding, the springs are linear and the simplified system in Fig. 1a behaves as a SDOF piecewise linear elastic system. For the case of elastoplastic spring representing the inelastic behavior of the deck, the system in Fig. 1a behaves as a SDOF piecewise linear inelastic system with gap elements.

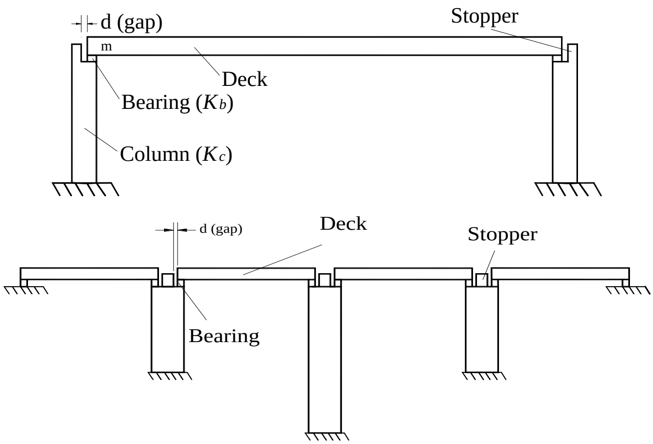


Figure 1. Schematic diagram of (a) single span bridge and (b) multi span bridge

From the previous analysis is obvious that the damage level depends on the input seismic excitation, i.e. the seismic ground acceleration. As is well known from the Structural Dynamics and Earthquake Engineering [1], since this input is not known for future earthquakes, the *spectral approach* is used according to various aseismic building codes, e.g. the Greek Aseismic Code EAK2000 [12]. So here, instead of a non-linear dynamic analysis, which is time consuming [1], the approach of [4, 14] is followed. According to equation (1), the description of the fragility curve involves only two parameters, S_{mi} and β_{tot}. The estimation of these parameters is obtained here as will be shown in the next section, where the proposed methodology is applied for an existing bridge of Egnatia Motorway. The first parameter S_{mi} is estimated on the basis of the capacity spectrum method [1], wherein the demand spectrum is plotted for a range of values of the earthquake parameter S (in spectral acceleration vs. spectral displacement format) and it is superimposed on the same plot with the capacity curve of the bridge. The earthquake parameter used in this study is the peak ground acceleration (PGA). The second parameter of Eq. (1) is the total lognormal standard deviation β_{tot} which takes into account the uncertainties in seismic input motion (demand), in the response and resistance of the bridge (capacity), and in the definition of damage states. This parameter (β_{tot}) can be estimated by a statistical combination of the individual uncertainties (in demand, capacity, and damage state definition) assuming these are statistically independent. On the basis of empirical fragility curves obtained from actual bridge damage data, the value of β_{tot} was set [4, 14 – 17] equal to 0,60 due to the lack of a more accurate estimation of uncertainties in capacity, demand and damage states.

Briefly, the proposed methodology comprises the following main steps:

(a) Due to elastomeric bearings, the system of the deck and prestressed reinforced concrete (r/c) beams is moving horizontally up to when the existing gaps of spans will close. Here, the shear stiffness of the system of elastomeric bearings is quite active.

(b) A Finite Element Model of the bridge is constructed using linear elements and lumped plastic hinges, for the end sections of the piers, the bents, the continuity slabs and the abutment’s ballast walls.

(c) The structural elements possess suitable effective flexural stiffness.

(d) The structural critical sections are analyzed in order to calculate the bilinear moment-curvature (M-C) diagram, as well as the moment-axial force diagram up to the yielding point by using a suitable material law for confined concrete.

(e) Transformations of bilinear diagrams M-C in bilinear diagrams M-R (moments-rotations) use a suitable length of each plastic hinge.

(f) The first translational mode-shape distribution of external static seismic lateral forces is considered in the nonlinear static pushover analysis, for both horizontal principal axes, which represent adequately the dynamic response of the bridge.

(g) The gravity loads of the system are in action.

(h) Static pushover procedure and capacity spectrum method are performed.

(i) The damage levels of the bridge are defined and finally the statistical lognormal function of probability distribution is used.

The presented methodology is here applied for establishing fragility curves for the 2nd Kavala ravine bridge on Egnatia Motorway that crosses northern Greece in an E-W direction.

3. The Investigated Case of an Egnatia Motorway Bridge with Seismic Stoppers: The 2nd Kavala Ravine Bridge

3.1. Bridge Description

On the part of Egnatia Motorway, northern Greece, that bypasses the city of Kavala, East Macedonia, a ravine bridge has been constructed, shown in Photos 1 and 2 (see also Fig. 1(b)). The bridge is a reinforced concrete one with seismic stoppers. It is a SLW 60/30 class bridge and crosses a 54,00 m deep ravine. Two similar separate bridges have been constructed, one for each traffic direction, named as “left branch” (the northern one) and “right branch”, (southern one, which is studied in this paper), respectively (Photo 2). The 180 m long southern branch of the bridge consists of four spans, each constructed using four 45 m long prestressed beams that rest on three piers and two abutments via elastomeric bearings. The decks of the bridge are straight in plan and have a longitudinal inclination of 3,04%. There are two traffic lanes and one emergency lane on each deck, with pavements on each side. Each deck has a total width of 13 m.

Each pier has a square hollow section of 4.00 × 4.00 m with 0,40 m thicknesses (Photo 2). The central pier (M2) has a height of 50,40 m while the other two (M1 & M3) of 28,00 m. It is a special case of pier design, usually met in Greek bridges constructed in the 1980 – 1995 period, and it is hence even more interesting to study its seismic vulnerability. The connection of the prestressed beams with piers M1 & M3 takes place via elastomeric bearings (GUMBA D650/195(155), $n = 10$, $t = 15$ mm ($E = 600000$ kN/m^2, $G = 1200$ kN/m^2). Also, the connection of the prestressed beams with the central pier M2 takes place via elastomeric bearings (GUMBA D600/300/52(37), $n = 4$, $t = 8$ mm ($E = 600000$ kN/m^2, $G = 1200$ kN/m^2). Two 900 × 900 mm bearings are used on each abutment (with an in-between distance of 10,70 m) and each has a total height of 120 mm (8 elastomeric layers, each 15 mm thick).

The soil at the site can be characterized as rock. The abutments rest on independent foundations, while the piers are founded on reinforced concrete circular wells, with diameter D = 6,00 m and effective height of 10,00 m, 9,80 m and 12,20 m for piers M1 M2 and M3 respectively. The concrete used is of class C35 for the prestressed beams of the deck and C25 for the piers, the two abutments and the foundations. Steel of class BSt 500/550 and 1700/1900 MPa prestressed cables (system PRECO) were used as reinforcement.

Photo 1. The Kavala Bridge on Egnatia Motorway

3.2. Application Steps of the Proposed Methodology

As mentioned, the above proposed methodology was used for the establishment of fragility curves for the Kavala Bridge. The computations were made for the right (southern) branch, and comprised the following steps:

1. A Finite Element Model of the bridge was developed using the SAP2000 Nonlinear program, using beam elements and plastic hinges at the locations of the superstructure where nonlinear seismic behaviour is expected to appear (in our case at the top and bottom of each column of the central pier). For the assessment of the plastic hinge properties, mean material values were assumed (i.e. $f_{cm} = f_{ck} + 8$ (in MPa) for concrete and $f_{ym} = 1{,}10\ f_{yk}$ for steel). A suitable confined-concrete material law proposed in [16] was used.

So, analysis of the critical cross-sections under biaxial moments and normal force was performed, to obtain Moment-Curvature (M-φ) and Moment-Normal force (M-N) diagrams. Using a suitably selected plastic hinge length for each location, the M-φ diagrams were transformed into Moment-Rotation (M-θ) ones.

2. A pushover analysis was performed for both the longitudinal and transverse directions using the 1st translational mode-shape of the lateral loads, for each principal direction. Since the most vulnerable direction of the bridge was not immediately evident, not even from the resulting pushover curves (Fig. 2 – 4), independent analyses were performed for

both directions. Note that the pushover curves are three-linear curves, where the first branch refers to displacement of elastomeric bearings until the 5 cm seismic gap that has been incorporated in the design closes. Afterwards, there is a structural natural interlocking between the prestressed beams and the piers through the existing seismic shear connectors (stoppers).

Photo 2. The two branches of the Kavala Bridge

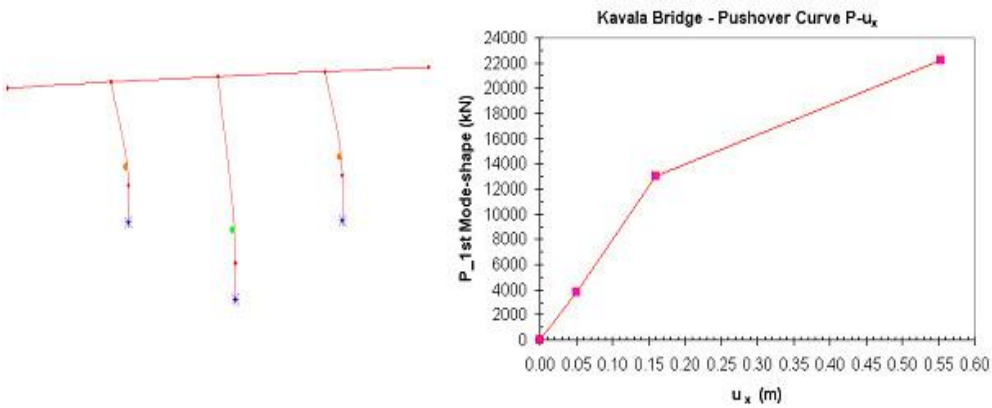


Figure 2. Kavala Bridge, longitudinal direction: (a) FEM model (b) pushover curve

3. The resulting pushover curves were expressed in terms of displacement of the centre of gravity of the deck vs. the normalized base shear (i.e. sum of the base shears at the supports divided by the total vertical service load). The pushover curves were transformed into capacity curves, which should then be in general combined with appropriate inelastic demand spectra. The particular bridge under examination has an equivalent single-degree-of-freedom period of about 1,46 sec in the x and in the y direction. In this high-period range, the displacements of

the inelastic system tend to coincide with those of the respective elastic one. This fact allows us to use the elastic branch of the pushover curve in combination with elastic demand spectra, for the whole range of PGA values under investigation. Two such spectra were used in the present work: (a) a mean elastic acceleration spectrum derived from a representative set of Greek earthquakes [12] and (b) one based on the acceleration design spectrum proposed by the current Greek Seismic Code (EAK2003 for Soil category A) (similar to the Type-1 spectrum of Eurocode 8).

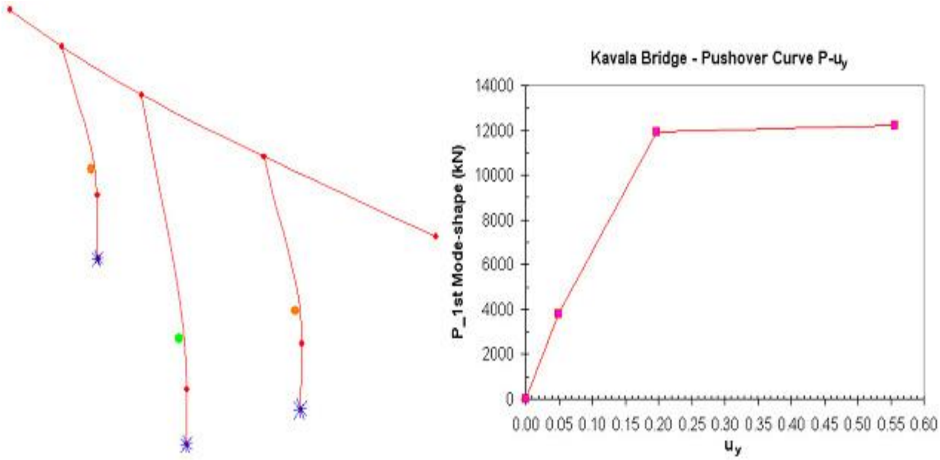


Figure 3. Kavala Bridge, transverse direction: (a) FEM model (b) pushover curve

4. Five damage states are also defined in terms of the displacement δ at the top of the central pier, normalized by the respective yield displacement δ_y. The description of the damage states and the corresponding threshold values of the damage ratio $D = \delta/\delta_y$ is presented in Tab. 1. The proposed values for damage ratio D are based on previous experience of the research team and are deemed appropriate for bridges that exhibit nonlinear behaviour at the piers.

Table 1: Definition of damage states

i	Damage state	Necessary repair interventions	Duration of interventions	Damage ratio $D_i=\delta_i/\delta_y$
0	No damage	None	---	<0.7
1	Minor damage	Small-scale repairs	<3 days	>0.7
2	Moderate damage	Repair of structural elements	<3 weeks	>1.5
3	Extensive damage	Reconstruction of structural parts	<3 months	>3
4	Collapse	Reconstruction of bridge	>3 months	>μ_u

5. Using the above pushover curves with (a) the mean elastic demand spectrum of the Greek earthquakes, (b) the damage states and (c) the performance levels of the bridge we arrive

at the mean value S_{mi} of the maximum ground acceleration (PGA) that corresponds to the i-th damage state threshold (Fig. 4).

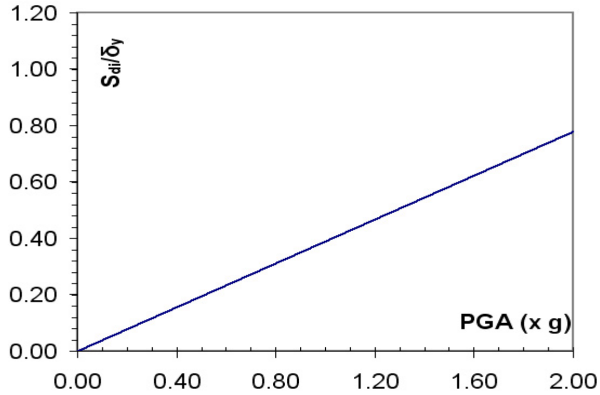


Figure 4. Damage index $D_i = S_{di}/u_y$ of Kavala Bridge as a function of PGA (y-direction) due to the Greek earthquakes

6. The mean expected peak ground acceleration value PGA_{mi} that corresponds to the D_i-th damage state threshold was then assessed as the point of intersection of the capacity curve with the demand spectrum.

7. Finally, fragility curves were computed, assuming a lognormal cumulative probability distribution for the damage ratio as a function of peak ground acceleration PGA and using the equation (1).

3.3 Results for the Kavala Bridge

The results of the numerical implementation of the methodology for the case of the Kavala Bridge are briefly presented as follows:

Since the most vulnerable direction of the bridge is not immediately evident, a pushover analysis was carried out for both directions, yielding pushover curves that were suitably transformed to the bilinear forms shown in Fig. 2 and 3. In the same figures, the activation of the plastic hinges at the piers of the finite element model during a step of the pushover analysis can be seen. The first translational mode-shape in each direction was used to define the shape of the corresponding lateral load distribution.

Even from the pushover curves, there can be no definite conclusion about the most vulnerable direction. So it was decided to compute the fragility curves of the bridge for both directions.

For comparison purposes, two alternative elastic demand spectra were used in the evaluation of the fragility curves. The first is a mean elastic spectrum derived from a representative sample of Greek earthquakes for which records exist. The sample consists of 67 records of 24 strong earthquakes that occurred within the Greek territory in the last 20 years, and which were recorded by 20 stations of the permanent accelerograph network of ITSAK (Institute of Technical Seismology and Aseismic Structures, Thessaloniki, Greece). The sample comprises earthquakes ranging in terms of magnitude between 4,4 and 6,9, and in terms of PGA between 50 and 400 cm/sec^2 [14]. The second alternative is the elastic demand spectrum proposed in the Greek Seismic Code EAK2003 (which is similar to the Type-1 spectrum proposed in EC8), for soil type A (which corresponds to soil conditions at the bridge site). Overall, four analyses were carried out, using both spectra for each horizontal direction.

The fragility curves can then be evaluated for different PGA values using equation (1). The required lognormal standard deviation β_{tot} incorporates the uncertainties in the seismic demand, the response and the capacity of the bridge, but also in the definition of the damage index and damage states. If no explicit calibration is performed, a value of β_{tot} = 0,60 is proposed [4, 14 – 18].

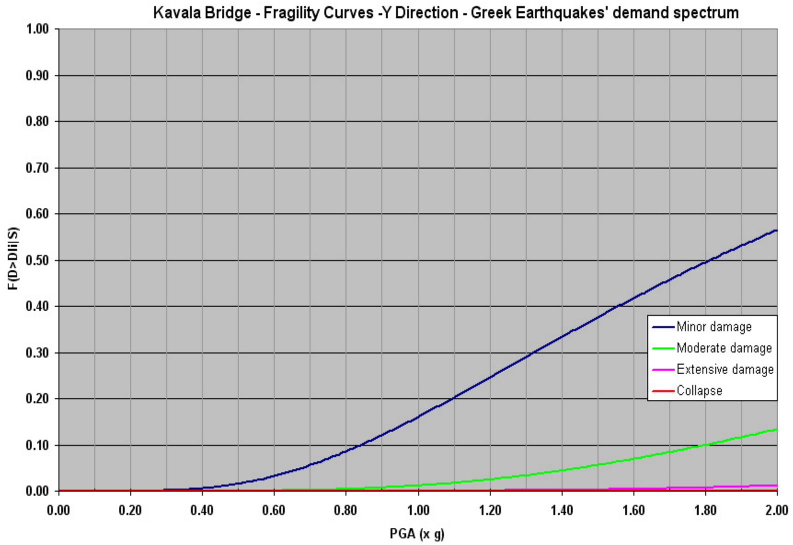


Figure 5. Fragility curves for Kavala Bridge (y -transverse- direction, mean demand spectrum from sample of Greek earthquakes)

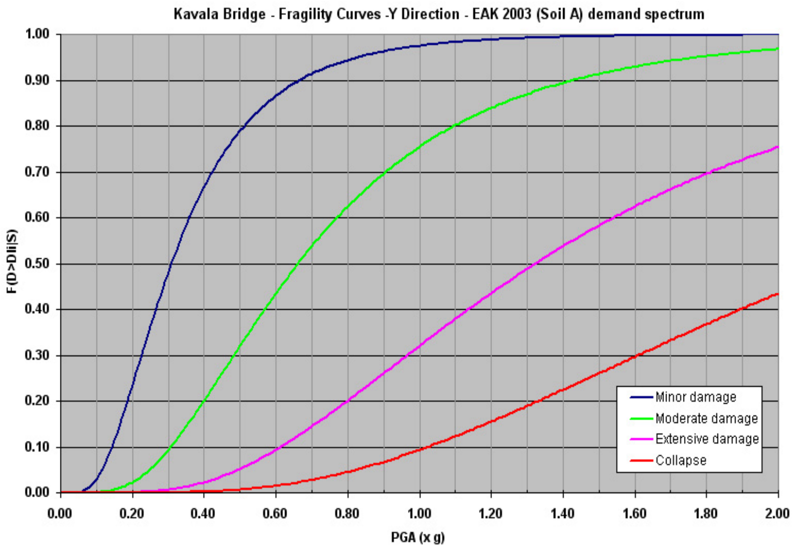


Figure 6. Fragility curves for Kavala Bridge (y-transverse-direction, EAK2003 elastic demand spectrum)

Based on the above, the fragility curves for the Kavala Bridge in the transverse (y) direction were derived and are presented in the next Fig. 5 and 6.

It is immediately apparent that the choice of the demand spectrum greatly influences the results. In our case, the EAK2003-compatible demand spectrum seems to lead to much heavier damage (for the same PGA value) than one derived from the representative sample of Greek earthquakes.

Further, two similar analyses were also carried out in the longitudinal (x) direction of the bridge. The respective fragility curves are presented in the following Fig. 7 and 8.

Conclusions, similar to those for the y-transverse direction, are also drawn for the x-longitudinal direction about the effect of the demand spectrum on the computed fragility curves. From numerical analyses of the Kavala Bridge, it was found that its fundamental period is T = 1,46 s in the y and x directions. It is known that in this period range, the EAK2003-compatible spectrum is expected to stress the bridge much heavier than the one derived by the sample of Greek earthquakes [12]. The proper choice of the demand spectrum is thus essential for the derivation of fragility curves that reliably predict the vulnerability of the bridge under examination.

From the obtained results it also becomes obvious that this particular bridge is more vulnerable in the x (longitudinal) direction, a conclusion not immediately evident at the onset of the investigations.

4. Further Applications of the Proposed Methodology to Other Egnatia Highway Bridges

The proposed methodology has been further applied to other Egnatia Highway bridges, in the frame of ASPROGE project [14].

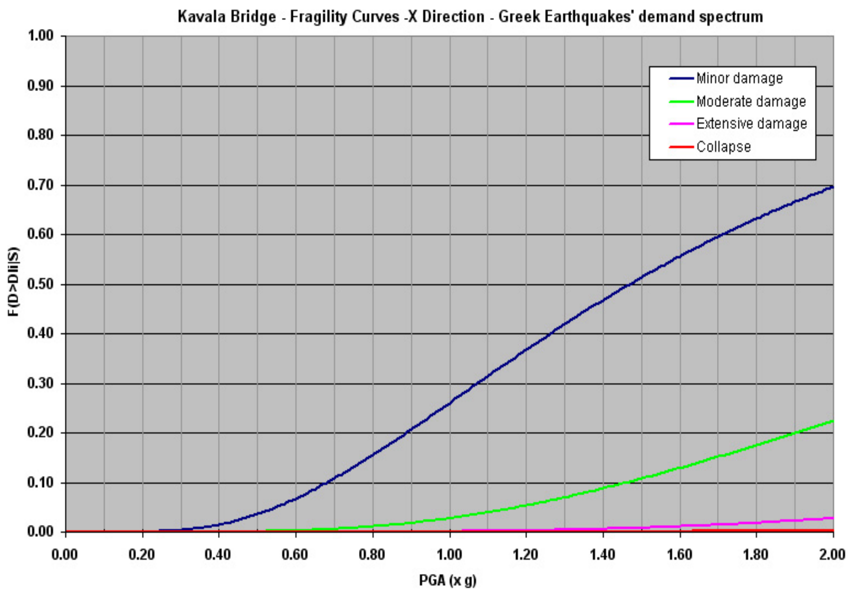


Figure 7. Fragility curves for Kavala Bridge (x-longitudinal - direction, mean demand spectrum from sample of Greek earthquakes)

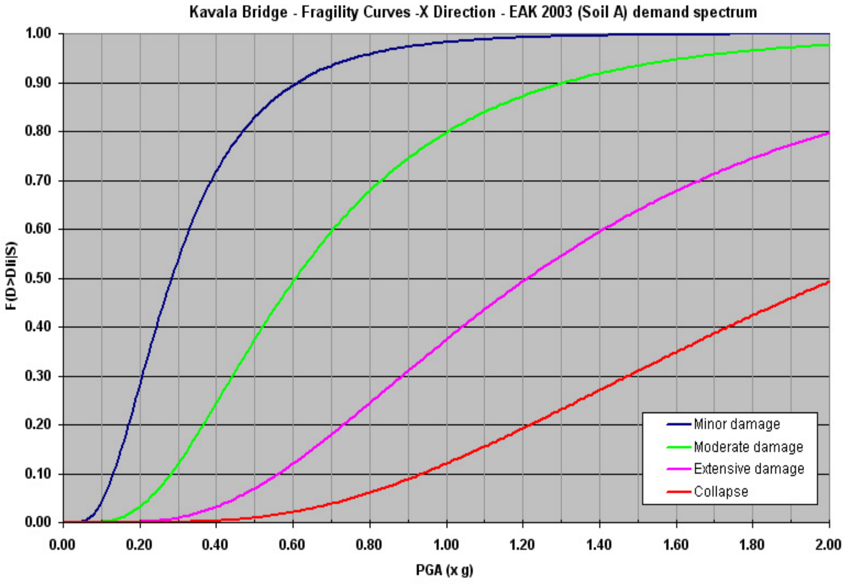


Figure 8: Fragility curves for Kavala Bridge (x-longitudinal - direction, EAK2003 elastic demand spectrum

In Tab. 2 the main characteristics of some Egnatia Highway bridges are shown, and in Fig. 9 the main characteristics of the Egnatia bridges, whose fragility curves were determined, are also shown.

Table 2. Main characteristics of some Egnatia bridges

Region	**Bridge Name**	**Carriageway length / span (m)**	**Height (m)**
Epirus	Aracthos	1.000/142	80
Thrace	Nestos	450/40	10
Macedonia	Greveniotikos	920/100	40
Epirus	Krystallopigi	850/55	30
Epirus	Metsovitikos	540/235	100
Epirus	Votonosi	490/230	53
Epirus	Megalorema	480/45	28
Macedonia	G12 (section Polymylos-Lefkopetra)	465/110	90
Thrace	Lissos	450/45	15
Epirus	Mesovouni	260/100	30

5. Concluding remarks

The calculation of the vulnerability curves of bridges with seismic stoppers can be obtained by the presented simplified methodology. This methodology is based on a modal pushover nonlinear static analysis using the Finite Element Method and on a capacity demand spectrum approach. So, a time consuming non-linear dynamic based vulnerability analysis can be overcome. Using the aforementioned approach, fragility curves were developed for a characteristic ravine bridge constructed in the Kavala section of Egnatia Motorway, in the county of East Macedonia, Northern Greece, and for other Egnatia Motorway bridges. The relative obtained results show the applicability of the methodology to Earthquake Civil Engineering in combination with the regional aseismic codes.

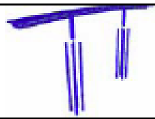
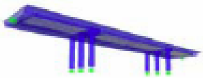
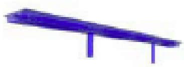
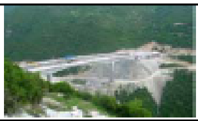
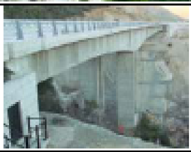
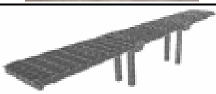
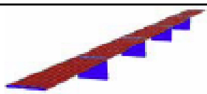
Structural configuration	Bridge name and class*	No. of spans	Span length	Total length	Pier-to-deck connection	Curva-ture	Founda-tion
	Pedini Bridge 111	3	19.0+ 32.0+ 19.0	70.0	monolithic	in height	pile groups
	Siatista Bridge 311	3	16.25+ 30.5+ 16.25	63.0	monolithic	minor curvature in plan	pile groups
	T7 (Section 14.1.2) bridge 121	3	27.0+ 45.0+ 27.0	99.0	monolithic	no	footings
	G11 bridge (right branch) 221	3	64.3+ 118.6+ 64.3	247.2	monolithic	in plan	caissons
	G9 (Section 5.1) Bridge 421	2	85.0	170.0	monolithic	in plan	caissons
	Eirini Bridge 122	4	45.0	180.0	through bearings	no	pile groups
	Lissos River Bridge 422	11	1x29.56+ 3x37.05+ 6x44.35+ 1x26.50	433.31	through bearings	no	pile groups
	2nd Kavala Ravine Bridge 232	4	42.0+ 2x43.5+ 42.0	180.0	through bearings	no	caissons
	G2 (Section 1.1.6) Bridge 332	3	30.7+ 31.7+ 30.7	93.1	through bearings	no	pile groups
	Kossynthos River Bridge 432	5	35.0+ 3x36.0+ 35.0	178.0	through bearings	no	pile groups
	Krystallopigi Bridge 223	12	44.17+ 10x54.98+ 44.17	638.19	monolithic/ through bearings	in plan	pile groups

Figure 9. Main characteristics of the Egnatia bridges whose fragility curves were determined [14]

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