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Using full bridge model to develop analytical fragility curves for typical concrete bridge piers

Authors:

Nadjib Hemaidi Zourgui, MSc. CE
High National School of Built and Ground Works, TPITE Laboratory (ENSTP), Algeria
National Earthquake Engineering Research Centre
nhemaidizourgui@cgs-dz.org

Assoc.Prof. Abderrahmane Kibboua, PhD. CE
High National School of Built and Ground Works, TPITE Laboratory (ENSTP), Algeria
National Earthquake Engineering Research Centre
akibboua@cgs-dz.org

Prof. Mohamed Taki, PhD. CE
High National School of Built and Ground Works, TPITE Laboratory (ENSTP), Algeria
atik_med@yahoo.fr

Subject review

Nadjib Hemaidi Zourgui, Abderrahmane Kibboua, Mohamed Taki

Using full bridge model to develop analytical fragility curves for typical concrete bridge piers

This study shows the effect of various numerical models on the development of analytical fragility curves for bridge piers. Two distinct models are compared: model with a single degree of freedom, and the proposed full bridge model. Bridge pier damage indexes are obtained by performing both dynamic and static nonlinear analyses (pushover and time history analysis), in order to develop fragility curves for this bridge pier. It was observed that capacity curves, ductility curves, and fragility curves, are sensitive to structural modelling.

Key words:

fragility curves, pushover analysis, seismic vulnerability, bridge pier model

Pregledni rad

Nadjib Hemaidi Zourgui, Abderrahmane Kibboua, Mohamed Taki

Primjena numeričkog modela cijelog mosta u analitičkom postupku izrade krivulja vjerojatnosti oštećenja

U radu se analizira utjecaj različitih numeričkih modela mosta na definiranje krivulja vjerojatnosti oštećenja njihovih stupova analitičkim postupkom. Uspoređuju se dva različita modela, model s jednim stupnjem slobode te predloženi model cijelog mosta. Za potrebe određivanja krivulja vjerojatnosti oštećenja promatranog stupa, određeni su njegovi indeksi oštećenja pomoću dinamičke i statičke nelinearne analize (metoda postupnog guranja i time-history analiza). Utvrđeno je da način modeliranja utječe na krivulje nosivosti, duktilnosti i vjerojatnosti oštećenja.

Ključne riječi:

krivulje vjerojatnosti oštećenja, metoda postupnog guranja, seizmička oštetljivost, model stupa mosta

Übersichtsarbeit

Nadjib Hemaidi Zourgui, Abderrahmane Kibboua, Mohamed Taki

Anwendung eines Vollbrückenmodells im analytischen Verfahren zur Erstellung der Wahrscheinlichkeitskurve der Beschädigungen

In der Abhandlung wird der Einfluss verschiedener nummerischer Brückenmodelle auf die Festlegung der Wahrscheinlichkeitskurve der Beschädigungen ihrer Pfeiler durch ein analytisches Verfahren analysiert. Verglichen werden zwei unterschiedliche Modelle, das Modell mit einem Freiheitsgrad sowie das vorgeschlagene Vollbrückenmodell. Zum Zweck der Festlegung der Wahrscheinlichkeitskurve der Beschädigung des betrachteten Pfeilers wurde sein Beschädigungsindex mithilfe der dynamischen und statischen nicht linearen Analyse (Methode der inelastischen statischen Untersuchung und Time-history-Analyse) festgelegt. Festgestellt wurde, dass die Art der Modellierung die Tragfähigkeits-, Duktilitäts- und Wahrscheinlichkeitskurven der Beschädigung beeinflusst.

Schlüsselwörter:

Wahrscheinlichkeitskurve der Beschädigung, Methode der inelastischen statischen Untersuchung, seismische Beschädigbarkeit

1. Introduction

Northern Algeria is one of the most seismically active regions in the western Mediterranean. It has experienced several large earthquakes during the last four decades, such as the El Asnam earthquake (1980) and the Boumerdes earthquake (2003). On each of these two occasions, the seismic action resulted in a lot of human casualties (3000 and 2300 persons, respectively), while also causing a financial loss to the Algerian economy of approximately 8 billion U.S. dollars [1-3].

Managing authorities are aware of the need to increase the awareness level regarding seismic assessment of structures. Thus, specialized national institutions, centres and researchers have been placing great efforts to study seismic vulnerability of existing structures.

A comprehensive study, focusing on evaluation of seismic vulnerability of buildings, has been conducted in Algeria [4-6]. However, just a few studies have been carried out for bridges [2, 7], although they constitute a key segment of transport infrastructure, highly important for daily life of residents and for economic development in general.

Algeria has around 12000 bridges, out of which more than 90 % are ordinary bridges. It is therefore highly important to preserve functionality of these structures and, in this respect, recommendations have been made to conduct research studies to improve their seismic resistance, and protect them from further degradation and collapse.

One of the topics of such studies is the development of fragility curves in the scope of assessment of seismic vulnerability of RC current bridges, which are quite frequently constructed in Algeria.

Due to importance of the topic, a considerable number of research studies, focusing on development of fragility curves for seismic risk assessment, have been published over the last decades. This activity can be traced back to 1975 when Whitman et al. [9] formalized the seismic risk assessment procedure [8].

Fragility curves express the probability of structural damage due to earthquakes as a function of ground motion indices [9]. They constitute a useful tool for evaluating seismic vulnerability of structures.

These curves can be developed either empirically, based on the damage reports from previous earthquakes [10, 11], or analytically [7, 12-16]. Some researchers have developed these curves experimentally. Vosooghi et al. [17] used data from 32 bridge column test models to develop fragility curves. Perrault et al. [18] used experimental data to evaluate the fragility of the BRD tower in Bucharest. Choi et al. [19] used RC column experimental results to develop nonlinear models. Nonlinear time history analyses were performed using ground motions and then fragility curves of the columns were constructed.

In Japan, United States, and other developed countries, several analytical fragility curves have been generated for the bridges [20-23].

Bhuiyan and Alam [24], Alam et al. [15], and Abbasi et al. [25], constructed fragility curves based on analytical simulation methods.

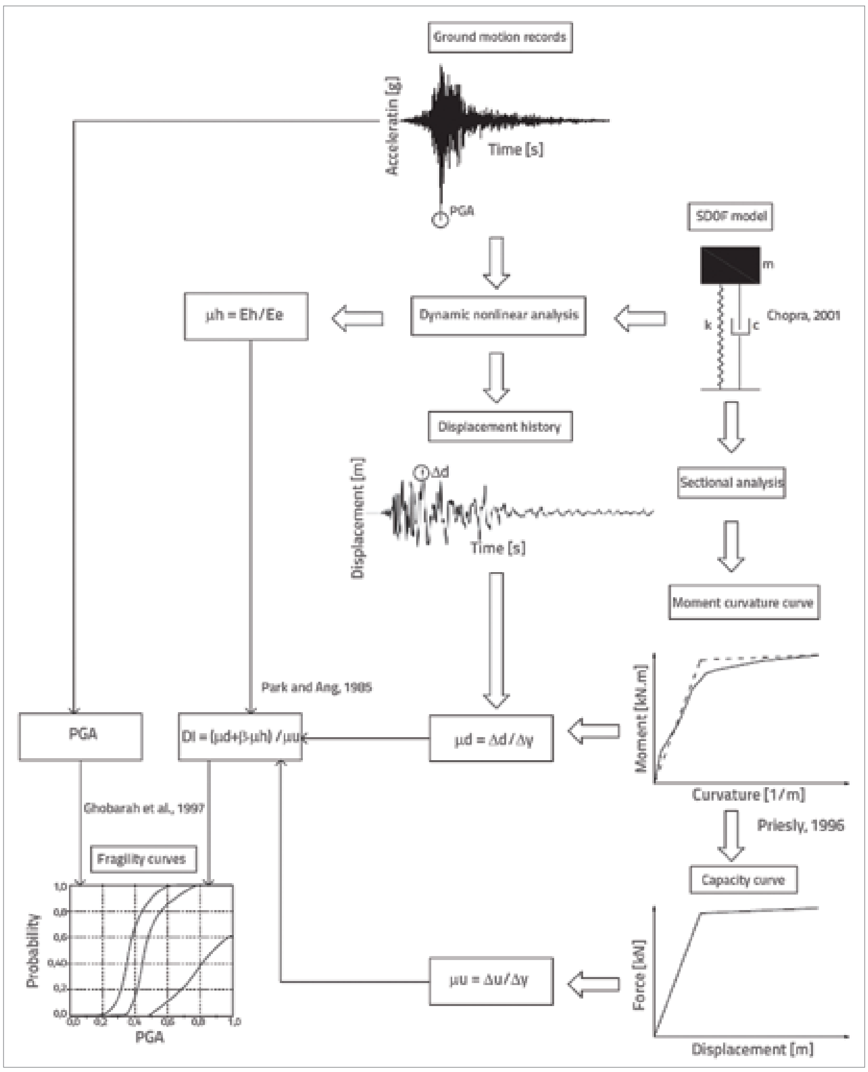

Figure 1. Flow chart describing procedure A

For 15 years, more than 50 % of research works have established the fragility of bridge systems based on the fragility of columns [26].

Karim and Yamazaki [9] used a simplified structural model to construct analytical fragility curves for bridge piers. In Algeria, Kibboua et al. [7] developed analytical fragility curves for typical reinforced concrete bridge piers using a simplified approach.

However, bridge components must absolutely be considered as a precondition for predicting real behaviour of such structures. Nielson and DesRoches [27] developed analytical fragility curves considering multiple vulnerable components of bridges. Ghotbi [28] considered the entire bridge system, while Choi et al. [29] modelled bearings in the bridge fragility assessments.

The main purpose of this research is to develop analytical fragility curves of a typical RC bridge in Algeria using two structural models (simplified model and full bridge model) and then comparing the results.

The bridge used in this study is located along the Bousmail-Cherchell highway in northern Algeria. Fifteen national and universal seismic records were employed to perform nonlinear time history analyses. Pushover analyses were then performed to develop fragility curves.

It was observed that the fragility curves are sensitive to the selected structural model.

2. Methodology

In this research, fragility curves were developed using an analytical approach with two procedures, A and B.

2.1. Procedure A

A single degree of freedom (SDOF) model was employed to perform static and dynamic nonlinear analyses. A total of 15 acceleration time histories were taken from Algerian and international earthquake events. The lumped mass, m, at the top of a multi-column bent, at the height h from the ground level, was modelled in this simplified method. The stiffness of the bridge pier was expressed as k. The following steps were used for developing the analytical fragility curves [7, 9], cf. Figure 1:

1. Select earthquake ground motion records
2. Scale ground motion records to different PGA values
3. Construct an analytical SDOF model of the bridge pier [30, 31]
4. Perform sectional analysis to obtain displacement ductility of the bridge pier using Xtract Software [32]
5. Perform non-linear dynamic analysis of the SDOF model using selected records to obtain the demand ductility and the cumulative energy ductility [33]
6. Obtain damage indices [34] of the pier for each excitation level
7. Construct fragility curves.

2.2. Procedure B

In this procedure, the most difficult step is to construct the full bridge model because of the nonlinear behaviour of its components (abutments, bearings, etc.), as shown in Figure 2. As stated above in procedure A, the development of fragility curves is based on a certain number of steps:

1. Select earthquake ground motion records
2. Scale ground motion records to different PGA values

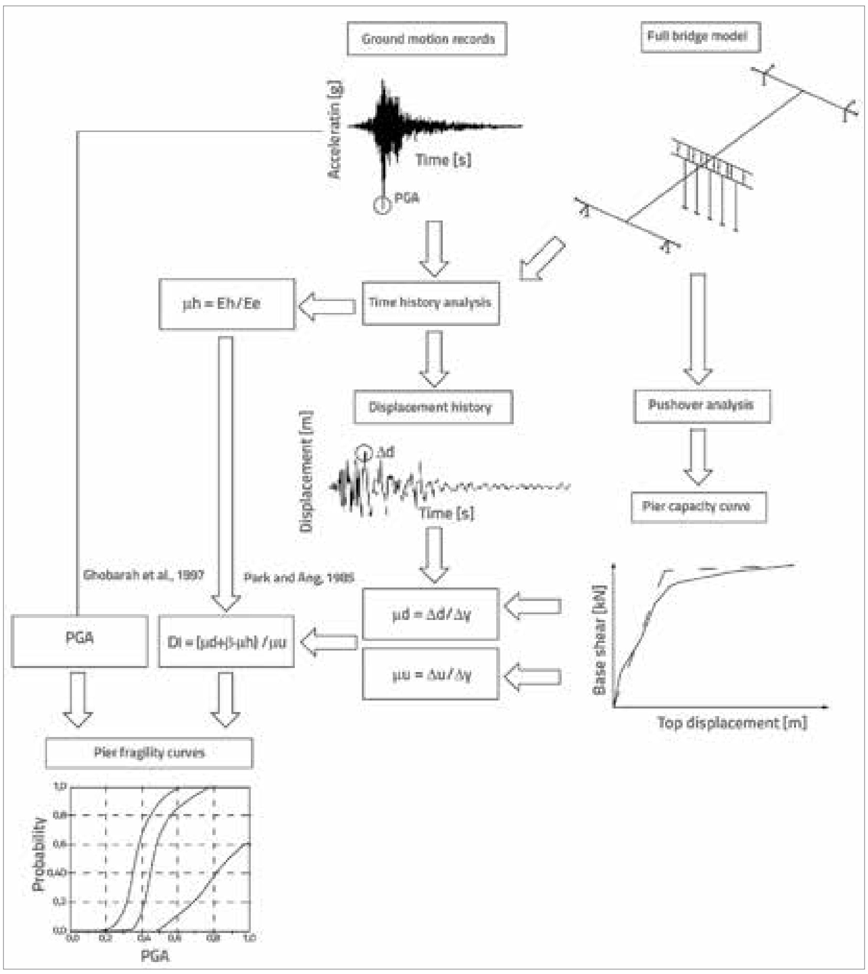

Figure 2. Flow chart describing procedure B

3. Construct the full bridge model
4. Perform Pushover analysis to obtain displacement ductility of the bridge pier
5. Perform nonlinear dynamic analysis of the full bridge model using selected records to obtain the demand ductility
6. Use an analytical model of the pier to calculate the cumulative energy ductility
7. Obtain damage indices of the pier for each excitation level
8. Construct fragility curves.

3. Ground motion selection

A suite of 15 earthquake ground motions with different range of PGAs were used to perform the nonlinear dynamic analyses. These records had to be representative of seismic characteristics of the bridge site [35]. Two kinds of records were employed. Local records were taken from the Boumerdes earthquake (which occurred in northern Algeria on 21 May 2003), as shown in Figure 3. These accelerometric data were recorded and monitored by our research centre (CGS) during and after the main shock of the Boumerdes earthquake. International records were obtained from the PEER Strong Motion Database, according to the response spectrum of the National Seismic Design Code RPOA 2008 [36], as shown in Figure 4. Selected ground motions are shown in Table 1.

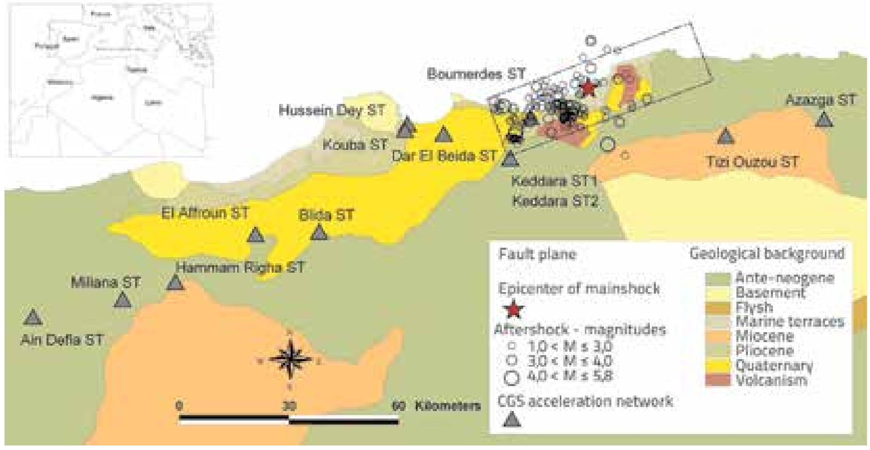

Figure 3. CGS Network of accelerograph stations located in central part of northern Algeria [37]

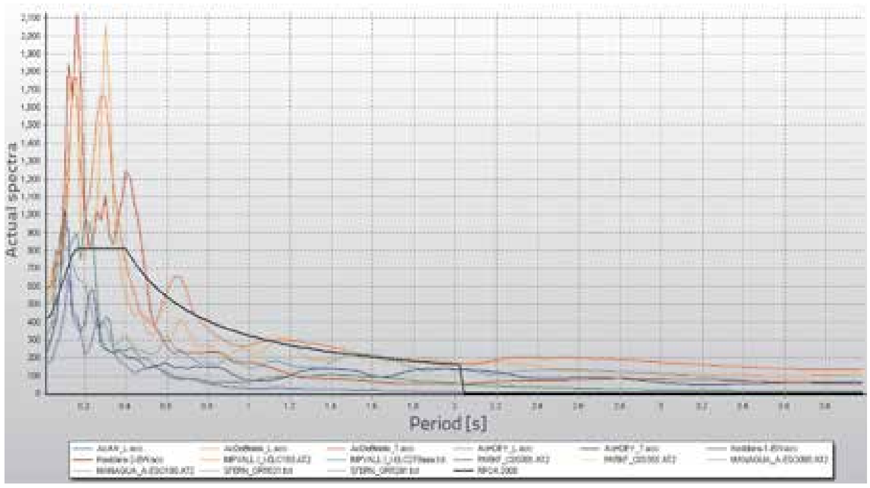

Figure 4. Acceleration response spectra with 5 % damping ratio of selected recorded ground motions and RPOA 2008 response spectrum

Table 1. Ground motion records

Magnitude	PGA [g]	Earthquake name	Recording station and direction	Year
6.8	0.548	Boumerdes	Dar El Beida_L	2003
6.8	0.511	Boumerdes	Dar El Beida_T	2003
6.8	0.275	Boumerdes	H-Dey_L	2003
6.8	0.237	Boumerdes	H-Dey_T	2003
6.8	0.339	Boumerdes	Keddara_EW1	2003
6.8	0.588	Boumerdes	Keddara_EW2	2003
6.8	0.167	Boumerdes	El Affroun_EW	2003
6.24	0.372	Managua_ Nicaragua-01	Managua_ ESSO.90	1972
6.24	0.329	Managua_ Nicaragua-01	Managua_ ESSO.180	1972
6.61	0.320	San Fernando	Castaic - ORR021	1971
6.61	0.275	San Fernando	Castaic - ORR091	1971
6.19	0.368	Parkfield	Cholame - #5.C05355	1966
6.19	0.444	Parkfield	Cholame - #5.C05085	1966
6.95	0.254	Imperial Valley	El Centro Array #9. 180	1940
6.95	0.150	Imperial Valley	El Centro Array #9. 270	1940

4. Bridge description

As shown in Figure 5, the bridge is a typical post-tensioned girder bridge designed based on the RPOA-2008 [36]. This simple supported structure with two spans each measuring 25.7 m has a total length of 51.5 m (0.10 m of gap). The deck is formed of a 16.50 m wide reinforced concrete slab, supported by ten (10) I type girders, placed on 0.05 m thick elastomeric bearings measuring 0.20 m x 0.40 m in plan, as shown in Figure 6 and Figure 7.

The bent is composed of five circular columns (Diameter: Ø = 1.4 m, height: H = 6 m), and a cap beam of 16.00 m length with a section of 2.00 m x 1.00 m, as shown in Figure 8.

Figure 5. View of the studied bridge

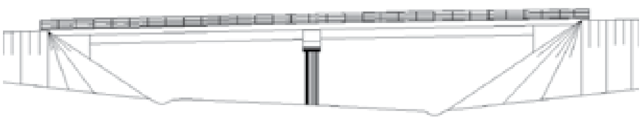

Figure 6. Elevation view

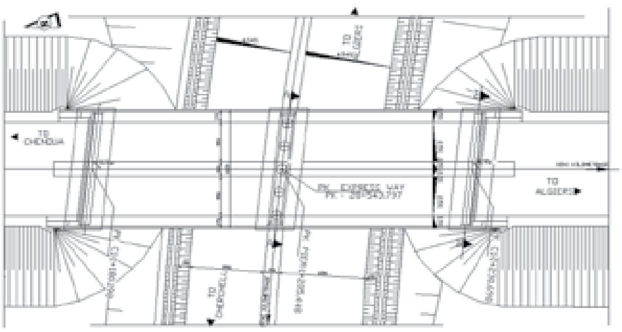

Figure 7. Plan view

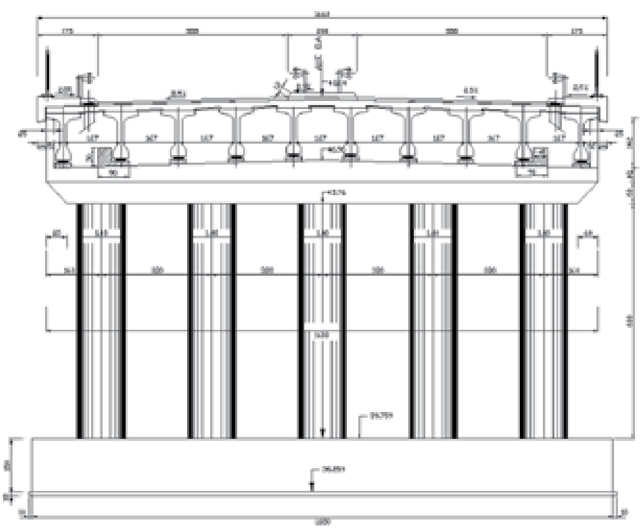

Figure 8. Cross-section of the bridge (section A-A)

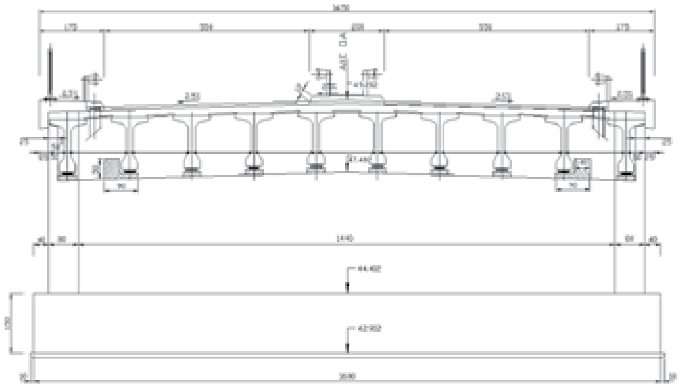

Figure 9. Cross-section of the bridge (section B-B).

Two rigid backfilled abutments have been constructed to support the deck and retain the embankment, as shown in Figure 9. The bent and abutments have been founded on rigid spread footings, as shown in Figure 8 and Figure 9.

5. Modelling details

The principal objective of modelling and analysis tools is the quantification of seismic response of bridges in terms of structural displacements and member forces and deformations [30]. Two bridge pier models were used in this study, a simplified model of the bridge pier using a bilinear model of Priestley et al. [30], as shown in Figure 10, and a full bridge model shown in Figure 12.

For the full bridge model, a software based on fibre modelling for the seismic analysis of various structures [38] has been used to perform both pushover and dynamic nonlinear analyses of the bridge, furthermore, to predict the behaviour of the bridge under seismic conditions.

The bridge was modelled in three dimensions taking into account material and geometric nonlinearities. All components of the structure were included, namely the columns, abutments, elastomeric bearings, shear keys and gaps (10 cm for longitudinal gap between the deck and abutments and 6 cm between shear keys and girders).

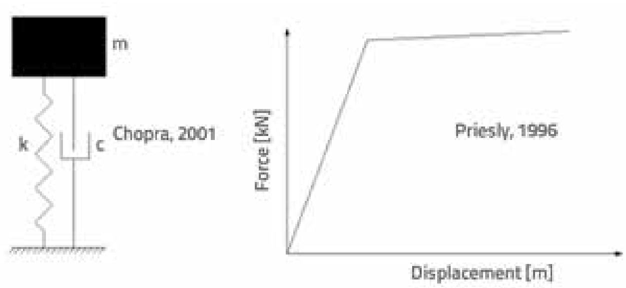

Figure 10. SDOF system

The zero-length and beam elements were used to integrate nonlinearities into the full bridge model so as to simulate real behaviour of the structure, cf. Figure 12. Spread footings were considered as rigid footings according to the SDC recommendations [39].

The deck was modelled using an elastic linear beam element with the mass distributed along the superstructure's centerline. It was calculated based on the equivalent section of the deck (slab and girders).The connection between the slab and the girders was taken using rigid links. A spring element was used to simulate behaviour of elastomeric bearings. Circular columns were modelled using the discretized fibre section, cf. Figure 11. The confined and unconfined concrete properties were assigned to the core fibres and the cover concrete, respectively. Twenty-seven longitudinal bars 32 mm in diameter, and spirals 6 mm in diameter for transverse reinforcement, were introduced into the column section.

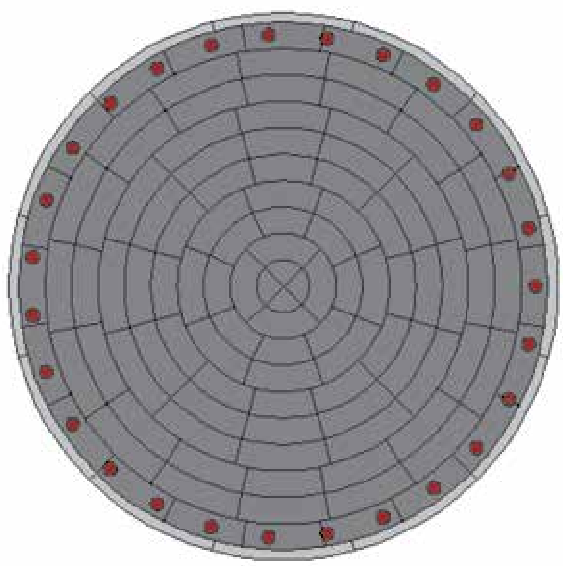

Figure 11. Discretized column section

The nonlinear concrete model proposed by Mander et al. [40] was used to define the core and cover of the concrete, while the bilinear steel model by Menegotto–Pinto [41] was selected for the steel reinforcement bars.

The inelastic force-based frame element type infrmFB was used to simulate nonlinear behaviour of the columns, with 5 integration sections and 150 section fibres.

The cap beam was modelled as a reinforced concrete elastic linear beam element, connected with columns by rigid links. In the longitudinal and transverse directions, the elastomeric bearings were modelled with an effective stiffness (Kbear = 2160 kN/m) and the rotational stiffness, Kq, amounted to 5000 kN/rad.

Table 2. Material properties defined in Seismostruct

Material parameters	Values
Concrete	
Compressive strength	27 MPa
Tensile strength	2.1 MPa
Modulus of elasticity	33000 MPa
Strain at peak stress	0.002
Specific weight	25 kN/m^3
Steel reinforcement	
Modulus of elasticity	200000 MPa
Yield strength	400 MPa
Strength hardening parameter	0.005
Specific weight	78 kN/m^3

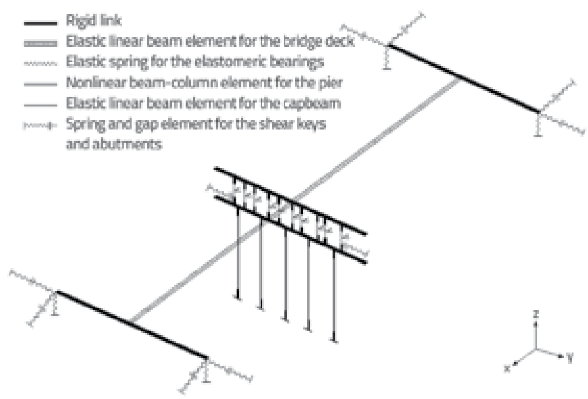

Figure 12. Proposed full bridge model

In this study, the abutments were modelled using springs in the longitudinal axis of the superstructure [42]. The model consists of spring elements connected to the deck. It was assumed that shear keys contribute to stiffness in transverse direction, after transverse displacement reaches the gap value. Shear keys were sacrificial and were designed to remain elastic under seismic excitations (Kskey = 500000 kN/m at the pier and Kskey/2 at the abutments) [43].

An elastic-perfectly-plastic backbone curve, shown in Figure 13, with abutment stiffness (K_{abut}), see Eq. (2), and ultimate strength (Pbw), as mentioned in Eq. (3), was obtained according to the SDC [39] recommendations, which were used for this model of abutment [42].

$$K_{abut} = K_i \cdot w \cdot (h_{bw}/1.7) \tag{2}$$

$$P_{bw} = A_e \cdot 239 \cdot (h_{bw}/1.7) \tag{3}$$

where:

K_{abut} - Initial abutment stiffness adjusted to backwall height

K_i - Initial abutment stiffness based on test results (11.5 kN/mm/m), (the value of K_i = 14.35 kN/mm/m is recommended in SDC Version 1 dated 7 April 2013)

w - backwall width
h_{bw} - backwall height
P_{bw} - Maximum passive pressure force
A_e - Effective abutment area
Δ_{gap} - Distance between abutment and deck (0.10 m)

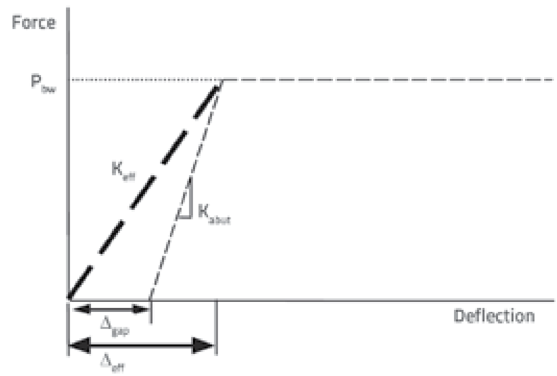

Figure 13. Effective abutment stiffness for seat type [39]

6. Moment curvature and pushover analysis

The moment curvature analysis for column section is used in order to determine the yield. The data is obtained from a computer program and calculated according to Priestley et al. [30]. In this paper, the cross section of the pier column is discretized with a fibre model. The moment curvature diagram is determined using XTRACT [32], see Figure 14.

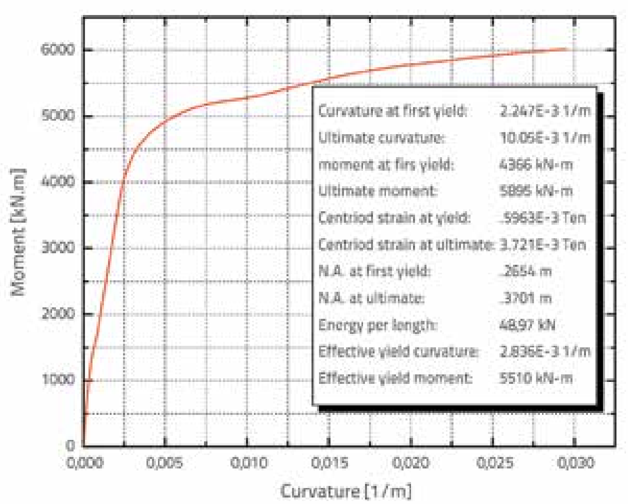

Figure 14. Moment/curvature curve for one column (SDOF)

The stiffness of the simplified model was determined using the moment/curvature curve, while the total mass of the bridge was used to evaluate characteristics of the model, ($K_{elastic}$ = 183667 kN/m and $K_{plastic}$ = 13948.91 kN/m).

The Pier capacity of the full bridge model was represented using the pushover curve, as shown in Figure 15. This curve can be obtained by applying a monotonically increasing horizontal load, representing the ground motion excitation. Analytical procedures based on pushover analysis have now gained popularity due to efficiency and reliable results [44]. Many researchers [45, 46] have been using this method for developing analytical fragility curves for bridges.

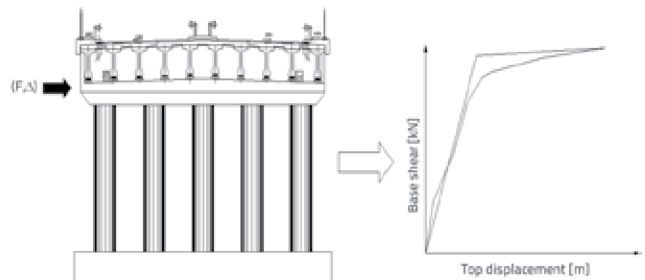

Figure 15. Pushover curve procedure

The pushover analysis results are shown in Figure 16, where the nonlinear force-displacement relationship of the pier for the full bridge model is presented in terms of the base shear and top displacement. The above analysis was performed for the transverse direction (Figure 15) using the first translational eigenmode for the shape of lateral loads, Figure 16.

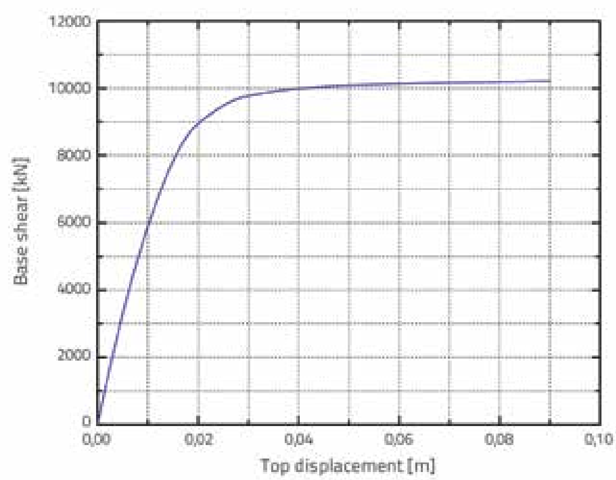

Figure 16. Pushover curve_Y direction (full bridge)

Two capacity curves were plotted on the same Figure, as shown in Figure 17, in order to illustrate the influence of the structural model on the capacity curve. The first curve was calculated using the moment curvature analysis [30, 39] while the second curve was obtained via the pushover analysis. The transverse direction was considered to be the most critical.

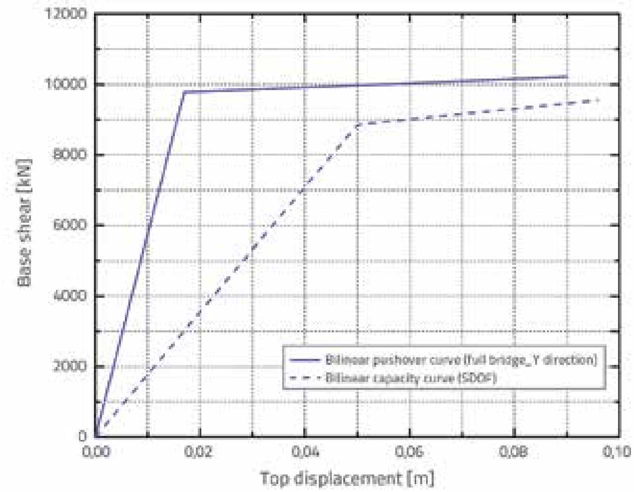

Figure 17. Comparison of capacity curves

Figure 17 shows a significant decrease in yield displacement (from 0.05 cm to 0.017 cm), and a 9 % increase in yield force (from 8900 KN to 9800 KN) due to the underestimation of the bridge elements stiffness, such as the abutment stiffness, elastomeric bearing stiffness, and shear keys stiffness.

The comparison shows a 6 % increase of ultimate force from the simplified model to the full bridge model.

7. Nonlinear time history analysis

The time history analysis is the most accurate method for analysing structures and predicting their nonlinear inelastic response to seismic load.

This analysis is the most sophisticated method in earthquake engineering assessment [47].

It takes into account nonlinearity of members using the step by step integration procedure, which is the most effective technique for this kind of analysis [48].

To apply the nonlinear time history analyses to the bridge, the two models were analysed using a suite of 15 scaled records (cf. Table 1). A total of more than 300 nonlinear analyses (running and results post-processed for each record) were conducted to produce analytical fragility curves by this numerical simulation, and to evaluate seismic vulnerability of the bridge [16, 22, 27].

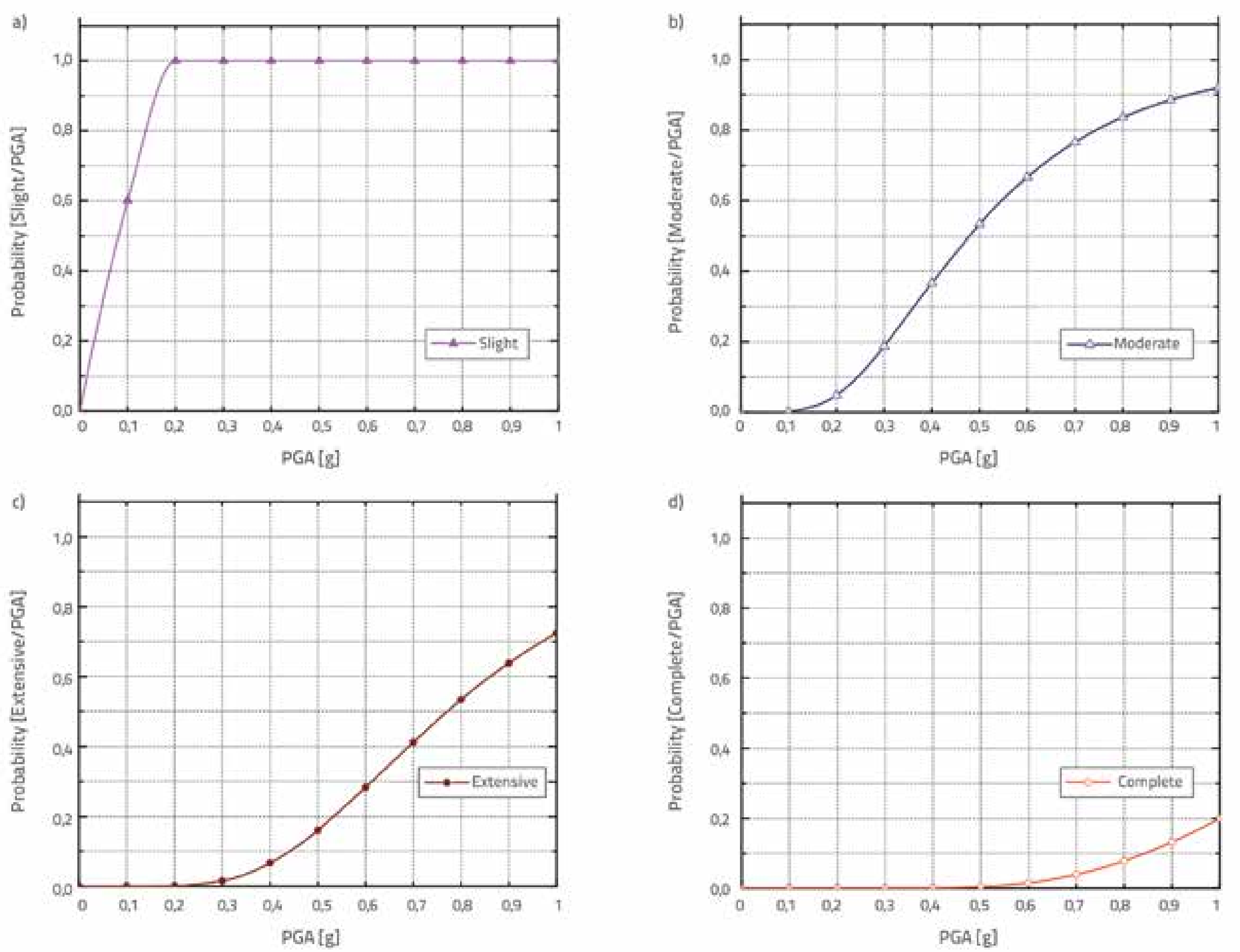

Figure 18. Fragility curves of bridge pier (SDOF model): a) Slight damage; b) Moderate damage; c) Extensive damage; d) Complete damage

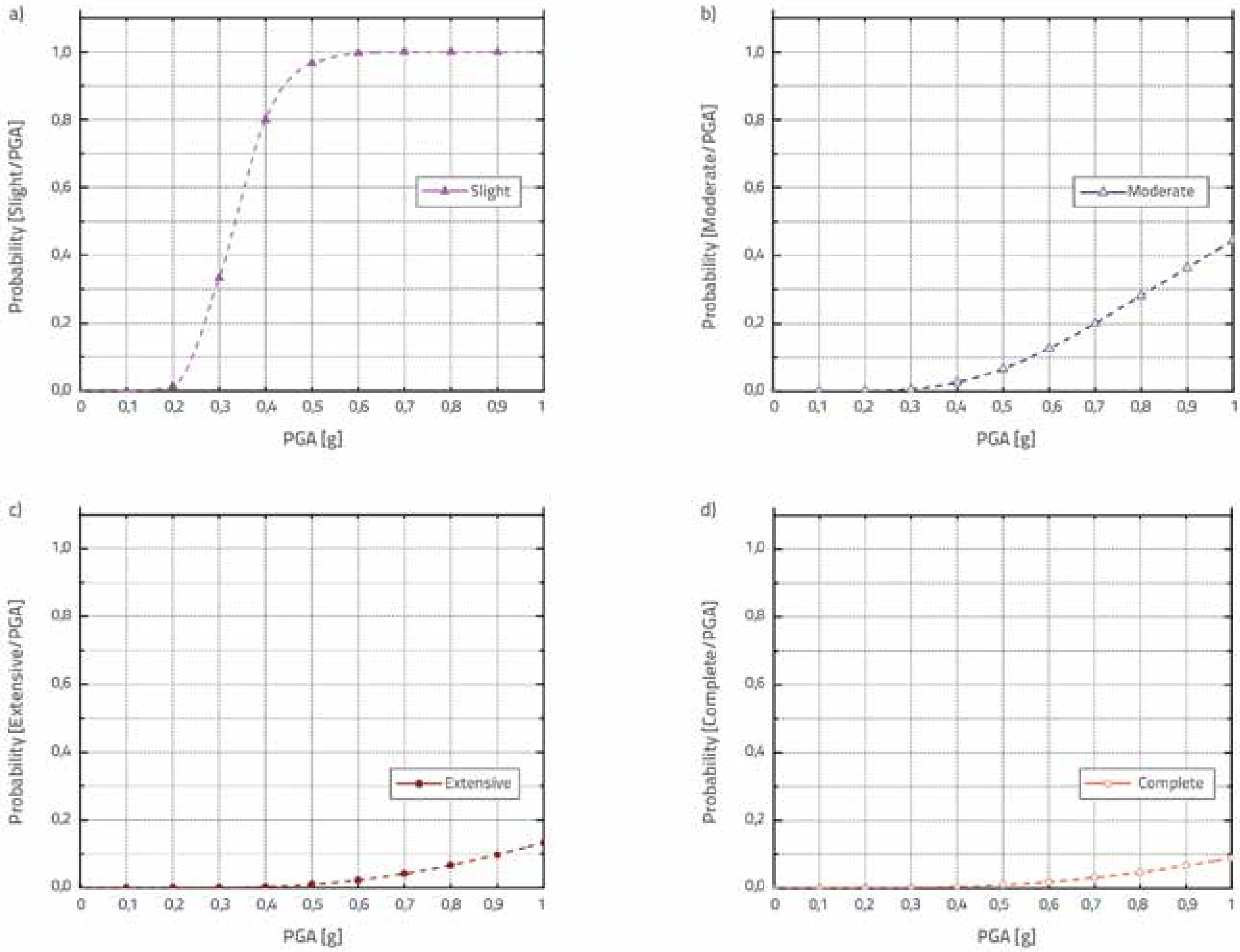

Figure 19. Fragility curves of bridge pier (full bridge model): a) Slight damage; b) Moderate damage; c) Extensive damage; d) Complete damage

8. Fragility curves

This work focuses on the seismic vulnerability assessment of the existing typical bridges in Algeria (multi span and simply supported prestressed girder bridges) using analytical fragility curves developed based on pushover and time history analyses. The Federal Emergency Management Agency (FEMA) and the Applied Technology Council (ATC) contributed actively in the development of fragility functions and vulnerability assessment procedures after 1975 when Whitman et al, (1975) formalized the seismic risk assessment technique [8]. They displayed the conditional probability that the structural demand exceeds the structural capacity [49], and considered this as one of the most useful tools for assessing existing bridges under seismic events [50, 51], which is critical in pre-earthquake planning and post-earthquake inspections.

Fragility curves for the bridge pier, as shown in Figure18 and Figure 19, were calculated as log normally-distributed functions. The cumulative probability of occurrence of damage P_R equal or higher than rank R is given as:

$$P_R = \Phi\left[\frac{\ln X - \lambda}{\zeta}\right] \quad (4)$$

where

Φ - the standard normal distribution

X - the ground motion index in terms of PGA.

The two distribution parameters, λ and ζ, are the mean and the standard deviation of ln X. See more detail about the functions in the research done by Karim and Yamazaki [9, 22], Kibboua et al. [7], and Kibboua [52].

They obtained the relation between the damage index DI created by Park and Ang [34] and a number of damage ranks, using the cumulative energy ductility, displacement, and ultimate ductility.

9. Discussion of results

Most researchers use four (04) limit states of damage namely: slight, moderate, extensive and complete. The results of the analysis of the simplified system show lower damage estimation for the slight, moderate and extensive damage. However, the fragility curve of complete damage shows good correlation with the full bridge model fragility curves, as shown in Figure 20.

The probability of exceeding the slight damage was reached for the simplified model at the PGA of 0.2 g. However, only 0.01 of this probability was recorded at the same PGA for the full bridge model. For this damage state the comparison between the developed fragility curves shows a difference ranging from 20 % at 0.4 g of PGA to 99 % at 0.2 g of PGA. Performing analysis after 0.5 g of PGA will give the same fragility curves for both current bridge models for this damage state.

By analysing fragility curves of the second damage state (moderate damage), the difference in consequences of probability of exceeding this state varies from 5 % at 0.2 g of PGA to 55 % at 0.7 g of PGA. Values of 5 % at 0.4 g to 55 % at 0.9 g of PGA were recorded for extensive damage, and a maximum of 10 % was recorded at the PGA of 1 g for complete damage.

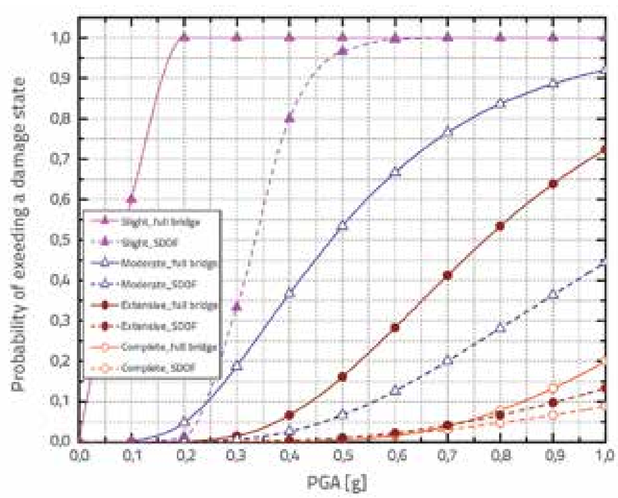

Figure 20. Comparison of fragility curves

After comparing the simplified system with the full bridge model, a significant effect on the fragility curves was established due to the selected structural model. The simplified system was modelled as an SDOF system, neglecting a number of bridge components such as elastomeric bearings, abutments, shear keys, and gaps.

The comparison proves that the effective stiffness of this system greatly influences the capacity of the bridge pier, and hence also the fragility curves.

10. Conclusion

This research focuses on the effect the structural model has on the seismic vulnerability assessment of bridge piers, using analytical fragility curves. To reach the objective, static nonlinear and nonlinear time history analyses were conducted using the Siesmostruct 2015 software.

A probabilistic procedure was used to generate fragility curves for a prestressed concrete bridge pier. This paper presents the development of analytical fragility curves for one of the most common bridge typologies in Algeria.

Based on comparison of capacity curves and fragility curves, it was established that the structural model can play a significant role in the evaluation of seismic performance of bridges. The capacity curve of the bridge pier is underestimated when the simplified system is used. However, the probability of exceeding damage state is overestimated, as stiffness of bridge components is neglected. Full bridge modelling presents a highly realistic evaluation of seismic performance of bridges. In this case, the probability of exceeding extensive and moderate damage is less than 0.45 (45 %).

Both structural models show that there is a small probability for complete damage of this type of current bridges.

An important conclusion is that simplified models are not always the ideal structural system for fragility analyses, as perceived by some of the previous researchers. In fact, each approach has its own advantages and disadvantages.

Research studies on seismic assessment of bridges are still very topical in all parts of the world. This work is a contribution to this field based on the Algerian database.

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