Seismic vulnerability functions of existing reinforced concrete bridge piers

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SUMMARY:
During these last decades, several earthquakes of various intensities struck several countries and caused considerable loss in human, material and network systems. In order to reduce the seismic risk, fragility curves could be not only regarded as an important step in the seismic risk evaluation process but are essential for risk assessment of highway transportation networks exposed to seismic hazards. They describe the probability to reach or exceed a state of damage for a particular earthquake. In the present paper, the derivation of the fragility curves of two bridges located in a seismic prone area in Algeria is introduced. The methodology adopted in this respect, combines the Park and Ang approach as well as the dynamic non linear analysis method, where the earthquake ground motion are chosen from a worldwide earthquake database.

Key words: fragility curves, bridge piers, analytical method, earthquake data base.

1. INTRODUCTION

Recent earthquakes followed by catastrophic damages such as Northridge in America (1994), Kobe in Japan (1995), Izmit in Turkey (1999), Chi-Chi in Taiwan (1999) and Boumerdes in Algeria (2003), confirmed that the bridges could be very vulnerable structures under dynamic loadings (Kibboua et al, 2008), and damage to bridges can seriously disrupt the function of the traffic network. It takes long time to repair these structures. In addition to direct damage, the indirect damage, such as regional economic loss caused by disruption of the transportation network, is an important social issue (Shon et al., 2003; Seongkwan et al., 2007).

Bridge fragility curves, which express the probability of a bridge to reaching a certain damage state for a given ground motion parameter, are essential tools for assessing the vulnerability of a particular or a class of bridges, and play an important role in the overall seismic risk assessment of a transportation network (Padgett and Desroches, 2008; Moschonas et al., 2009). Because of the lack of strong ground motion records, fragility curves methodologies using analytical approaches have become widely adopted because they are more readily applied to bridge types and geographical regions where seismic bridge damage records are insufficient (Kibboua et al, 2011).

The purpose of this paper is to develop fragility curves for two typical Algerian reinforced concrete bridge piers based on a numerical approach taking into account, the structural parameters and the variation of the input ground motion. By using strong motion records, the damage indices as defined by Park and Ang (1985) are obtained through a non linear dynamic response analysis via the educational NONLIN software program (Charney, 1998). The obtained damage indices defined for five damage rank (Ghobarah et al., 1997) and the ground motion indices are then combined to derive the corresponding fragility curves.

2. METHODOLOGY FOR THE DEVELOPMENT OF ANALYTICAL FRAGILITY CURVES

This part describes the steps used to construct the analytical fragility curves for two specific Algerian reinforced concrete bridge piers, which were designed using the simplified seismic method for bridges in Algeria.

The yield stiffness of the piers was firstly obtained by performing a sectional static analysis by using XTRACT computer program (Chadwell et al., 2002). For the non linear dynamic response analysis, the piers were modeled as a SDOF system and subjected to 41 acceleration time histories taken from a worldwide earthquake database. The PGA of the selected records was normalized to different excitation level from 0.1g to 1.0g having 10 excitation levels with equal intervals. Using these acceleration time histories as an input motion, the Park-Ang damage indices of the bridge piers are obtained (Park and Ang, 1985). Finally, the obtained damage indices and the corresponding ground motion indices are combined to develop the analytical fragility curves for the RC bridge piers.

The schematic diagram for constructing the analytical fragility curves (Karim and Yamazaki, 2001) is shown in Figure 2.1.

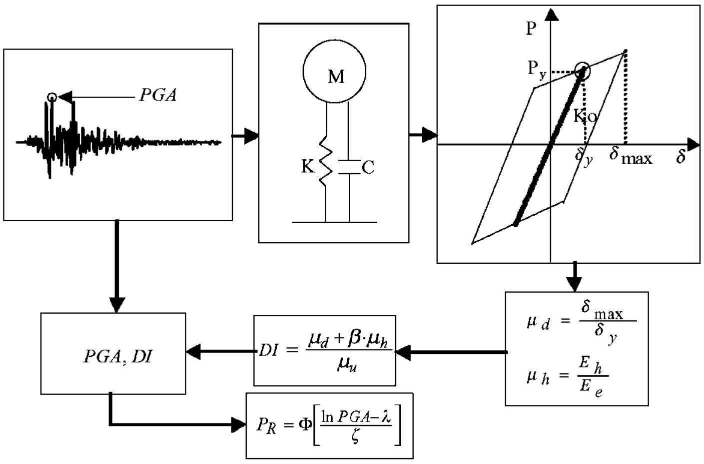

Figure 2.1: Schematic diagram for constructing the fragility curves for RC bridge piers

3. STATIC ANALYSIS

The sectional analysis is carried out for two reasons: (1) to find out the two possible structural failure modes, i.e.: the shear or the flexural failure modes of the bridge piers and (2) to obtain the force-displacement relationships at the top of the bridge piers. The displacement at the top of the bridge pier is given by the following equation:

$$\delta = \sum_{i=1}^{N} \left(\phi_i \times dy \times d_i + \gamma_i \times dy \right) \tag{3.1}$$

Where δ is the displacement at the top of the bridge pier, N is the number of cross-sections, ϕ_i is the curvature of the section i, dy is the width of each cross-section of the pier, d_i the distance from the top of the pier to the centre of gravity of each cross section and, γ_i is the shear strain.

4. DYNAMIC ANALYSIS

To perform the dynamic response analysis, the piers are modeled as a single-degree-of freedom (SDOF) system using a bilinear model (Priestley et al. 1996). The damage assessment of the bridge piers is carried out using the Park-Ang damage index *D* expressed as:

$$DI = \frac{\mu_d + \beta\mu_h}{\mu_u} \tag{4.1}$$

where μ_d is the displacement ductility, μ_u is the ultimate ductility of the bridge piers, β is the cyclic loading factor taken as 0.15 and μ_h is the cumulative energy ductility defined as:

$$\mu_h = {E_h}/{E_e} \tag{4.2}$$

where E_h and E_e denote the cumulative hysteretic (obtained from dynamic analysis) and the elastic energy (obtained from elastic analysis) of the bridge piers respectively. The damage indices of the bridge piers are obtained using Eq. 4.1, and then calibrated for each given input ground motion to get the relationship between the damage index (DI) and the damage rank (DR). This calibration is performed using the Ghobarah et al. (1997) proposed method. Table 4.1 shows the relationship between the damage index and the damage rank. As it can be seen, each DR has a certain range of DI varying from slight to complete. Using the relationship between DI and DR, the number of occurrence of each damage rank is obtained. These numbers are then used to obtain the damage ratio for each damage rank.

Table 4.1. Relationship between the damage index (DI) and damage rank (DR)

Damage index (DI)	Damage rank (DR)	Definition
$0.00 < DI \leq 0.14$	D	No damage
$0.14 < DI \leq 0.40$	C	Slight damage
$0.40 < DI \leq 0.60$	B	Moderate damage
$0.60 < DI \leq 1.00$	A	Extensive damage
$1.00 \leq DI$	As	Complete damage

5. DETERMINATION OF BRIDGE PIERS FRAGILITY

As it deals with piers that are not designed according to the 2008 new Algerian seismic design code for bridges (RPOA-2008), it is assumed that only the size and the reinforcement of the piers can be changed with other conditions such as their height, the length and the weight of the superstructure. The two sample bridges used to perform the analysis are shown in Figures 5.1 and 5.2.

- Bridge 1 is a multi-span simply supported (MSSS) concrete girder bridge with four spans and an overall length of 116.80m. The superstructure consists of a longitudinally reinforced concrete deck slab of 10m wide and it is supported by three sets of columns and by an abutment at each end. Each set has three columns with a circular cross section of 1.20m diameter.

- Bridge 2 has an overall length of 64.20m with two spans. It is supported by a wall pier type of a rectangular cross section having 8.61m x 0.80m dimensions and 6.805m height. The deck width is 10.05m.

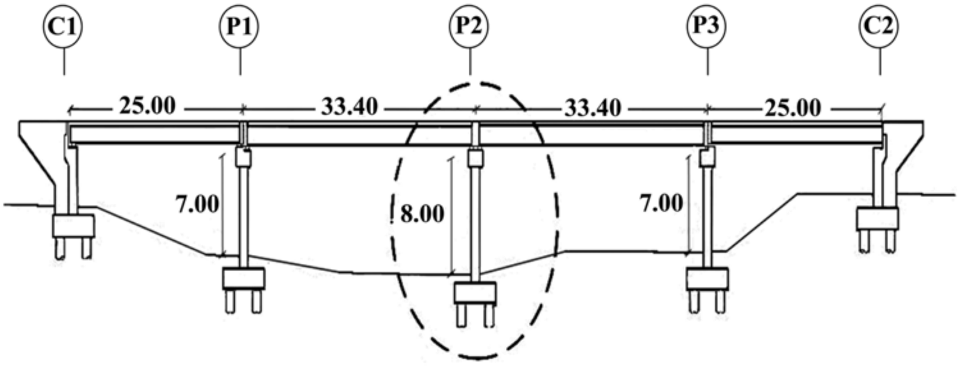

Figure 5.1: Elevation of a sample bridge 1 with a circular pier type

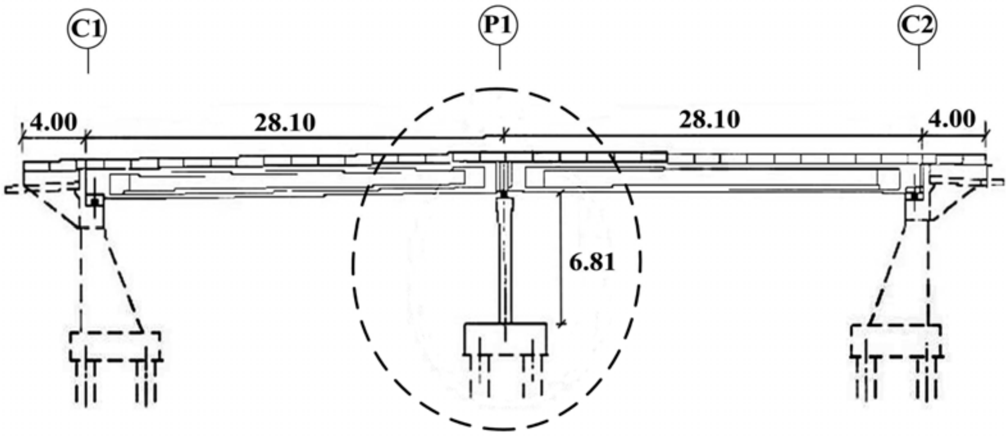

Figure 5.2: Elevation of a sample bridge 2 with a wall pier type

6. MOMENT CURVATURE CURVES FOR LATERAL DIRECTION

The sectional analysis of the bridge pier is carried out to get the moment curvature relationship necessary for the non linear analysis. In this respect, the cross sectional dimension of the pier bridge, the yield strength of steel σ_{sy}, the compressive strength of concrete σ'_c, the diameter of the longitudinal reinforcement bars as well as the tie reinforcement bars are taken as input parameters.

Figures 6.1 and 6.2 show the cross sections and the deduced moment rotational curves of the bridge piers. For the sectional analysis, the height of the pier bridge taken into consideration is: 8m and 6.81m respectively for bridge 1 and 2. It is found that in most cases, the flexural failure governs the failure mode.

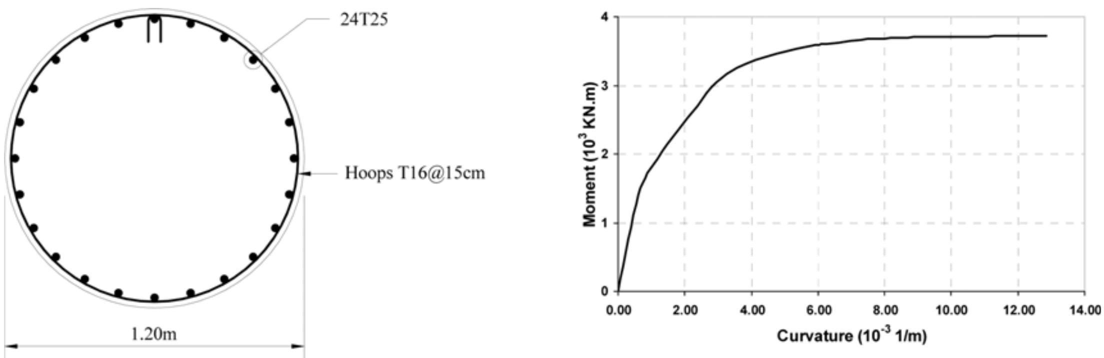

Figure 6.1: Cross section and its moment curvature curve for the Sample bridge 1 with a circular pier type

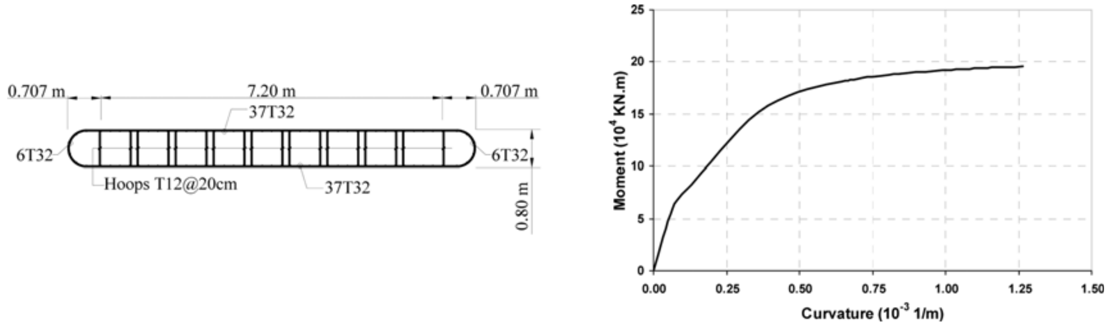

Figure 6.1: Cross section and its moment curvature curve for the Sample bridge 2 with a wall pier type

7. FRAGILITY CURVES

Established fragility curves are constructed with respect to PGA. The damage ratio for each damage rank at each excitation level is obtained by calibrating the DI using Table 4.1. Based on this data, fragility curves for the bridge piers are derived assuming a lognormal distribution. The cumulative probability of occurrence P_R of a damage equal or higher than rank R is given as:

$$P_R = \Phi\left[\frac{\ln X - \lambda}{\zeta}\right] \tag{7.1}$$

Where Φ is the standard normal distribution, X is the ground motion indices in term of PGA, The two parameters of the distribution λ and ζ are the mean and the standard deviation of ln X. The log-normal distribution has a probability density function:

$$f(x,\mu,\sigma) = \frac{1}{x\sigma\sqrt{2\pi}} e^{-\left(\frac{(\ln(x)-\mu)^2}{2\sigma^2}\right)} \tag{7.2}$$

Where x is the value at which the function is evaluated, μ is the median value of the PGA and σ is the log-standard deviation.

The cumulative log-normal distribution is obtained by integration of the area below the density function shown in Eq. 7.3.

$$f(x,\mu,\sigma) = \frac{1}{x\sigma\sqrt{2\pi}} \int_0^x \frac{e^{-\left(\frac{(\ln(x)-\mu)^2}{2\sigma^2}\right)}}{t} dt \tag{7.3}$$

In order to obtain the two parameters that define the log-normal distribution (μ, σ), the Microsoft Excel Solver tool was used. Microsoft Excel applies the Generalized Reduced Gradient Nonlinear Optimization Code.

Define a preliminary value for the median and standard deviation (μ, σ);

i. Plot the values obtained from the data ;
ii. Calculate the cumulative log-normal distribution using the two preliminary values of μ and σ ;
iii. Calculate the sum of the difference between the probability found from the lognormal probability plot constructed in step (iii) and the probability plot constructed in step (ii) ;
iv. Perform the optimization code included in Microsoft Excel ;

v. Repeat this procedure for each damage state.

Figures 7.1 and 7.2 show the fragility curves, for each damage state and for the entire sample pier bridges.

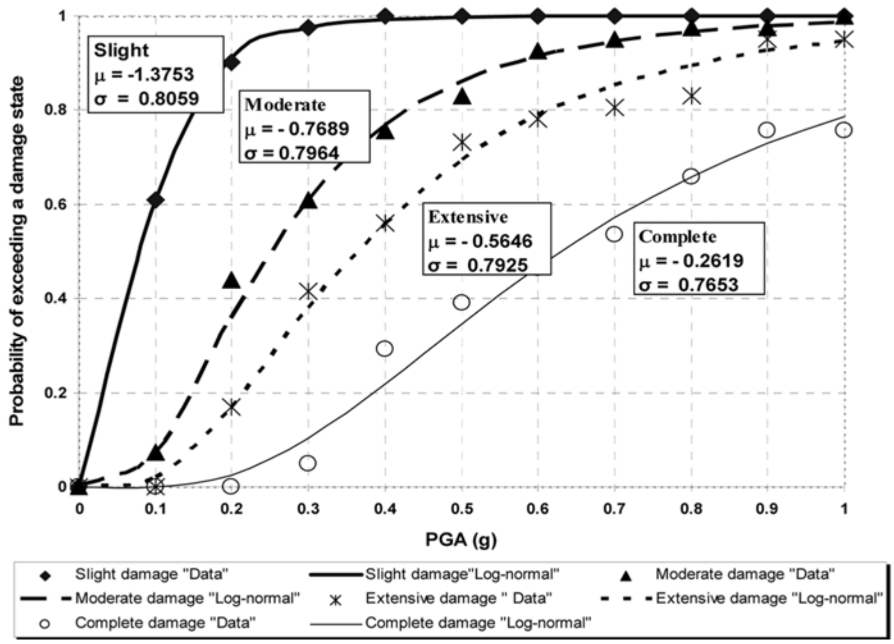

Figure 7.1: Fragility curves for all damage states: Bridge pier's sample 1

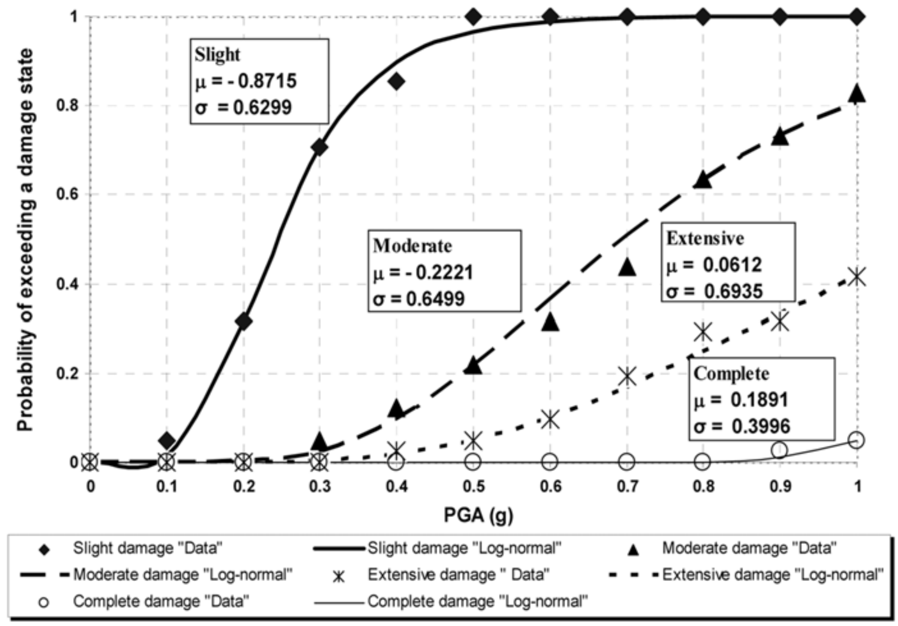

Figure 7.2: Fragility curves for all damage states: Bridge pier's sample 2

Figure 7.3, 7.4, 7.5 and 7.6 show the comparison between slight, moderate, extensive and complete damages for the two typical RC bridge piers.

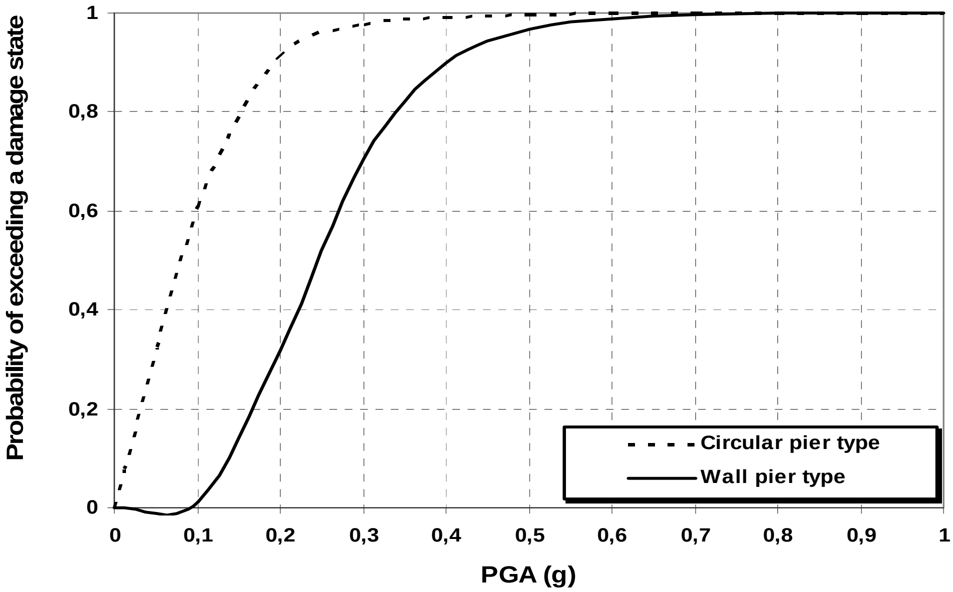

Figure 7.3: Fragility curves between slight damage for the two typical bridge piers

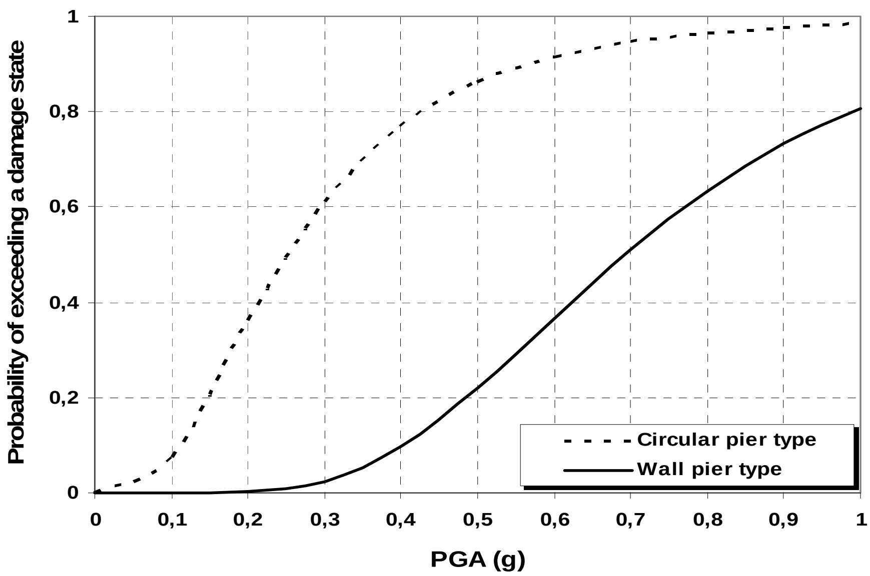

Figure 7.4: Fragility curves between moderate damage for the two typical bridge piers

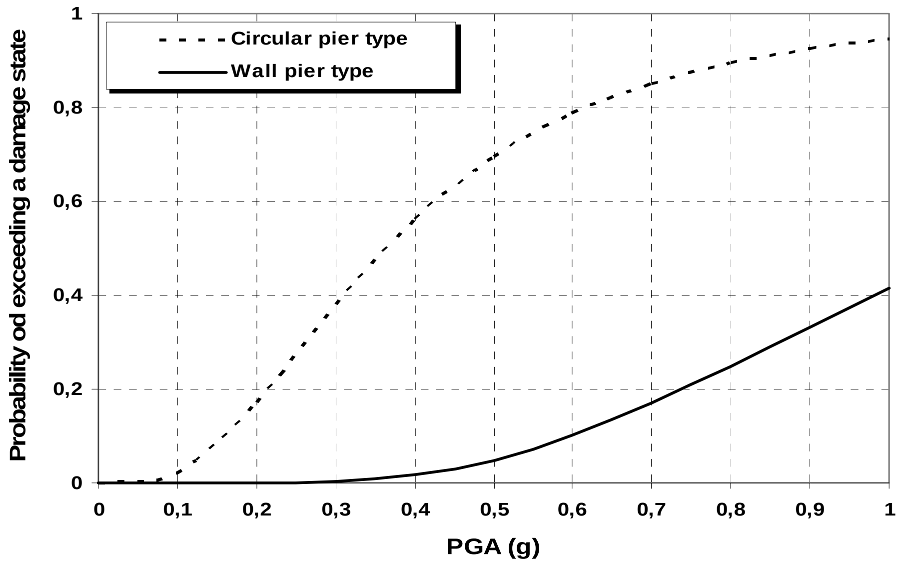

Figure 7.5: Fragility curves between extensive damage for the two typical bridge piers

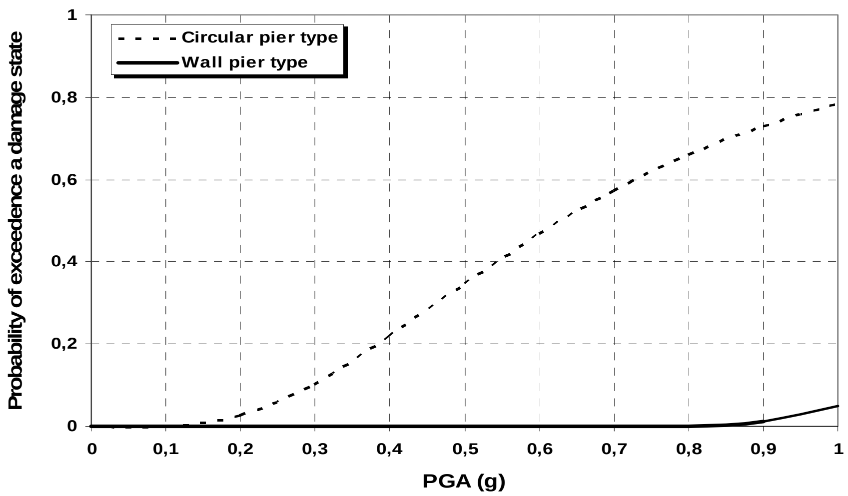

Figure 7.6: Fragility curves between complete damage for the two typical bridge piers

8. RESULTS INTERPRETATION

Analytical fragility curves for two typical Algerian reinforced concrete bridge piers having different structural properties (Fig. 6.1 and Fig. 6.2) were obtained with respect to the peak ground acceleration based on numerical simulation using 41 worldwide accelerometer records assuming a lognormal distribution.

It was found that there is a significant effect on the fragility curves due to the variation of structural parameters in terms of the cross section shapes, the longitudinal reinforcement and the tie reinforcement.

The level of damage probability in the cases of *slight, moderate, extensive* and *complete damage* is not the same for bridge type 1 (circular pier type) and bridge type 2 (wall pier type).

The bridge type 2 (wall pier type) has a lower level of damage probability than the Bridge type 1 (circular pier type). It implies that the bridge type 2 which is supported by a wall pier type performs better against seismic forces than the bridge type 1 which is supported by a circular pier type.

8. CONCLUSION

The vulnerability assessment of bridges is useful for seismic retrofitting decisions, disaster response planning, estimation of direct monetary loss, and evaluation of loss of functionality of highway transportation systems. This paper illustrates the results of the seismic vulnerability study aimed to develop the analytical fragility curves for typical Algerian bridge piers based on numerical simulations.

Bridge piers designed with the simplified seismic design method for bridges in Algeria are analyzed, and a large number of worldwide accelerometer records from which, Algerian strong motion records and earthquake records from some major event, e.g., the 1995 Kobe, the 1994 Northridge were selected in order to get a wide range of the variation of input ground motions. The fragility curves for the bridge piers are then developed by performing both, the static and the non linear time history analyses and following the same numerical approach that is described in chapter 2.

One pier model has been selected as a representative of all other piers for a particular bridge structure. It can be seen that the analytical fragility curves for the two bridge piers show a very different level of damage probability with respect to PGA. This difference is due to the shape of the cross section and the percentage of the longitudinal and tie reinforcements. The wall pier type shows the best seismic performance while compared to the circular pier type.

It is hoped that fragility curves developed here will play an important role as a basis for assessing the socio economic impacts of disrupted economic activities at a regional or a national scale of a country.

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