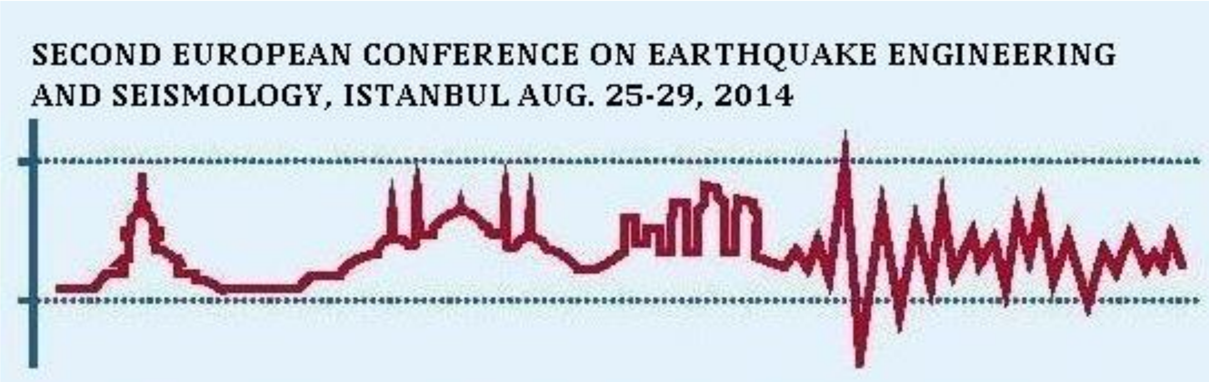

SEISMIC VULNERABILITY ASSESSMENT OF A SET OF PORTUGUESE VIADUCTS

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ABSTRACT

The main objective of the current work is to assess the seismic vulnerability of a set of viaducts, representing the structural behaviour of a typical Portuguese configuration. To this end, two different nonlinear modelling strategies were used: (i) a simplified bi-dimensional model, with lumped plasticity at the elements extremities (plastic hinge formulation); and (ii) a tri-dimensional model, with distributed plasticity along the elements length (fibre–based model).

A probabilistic seismic demand model that relates ground motion intensity measures to structural demand measures is considered in the present study towards the seismic vulnerability assessment of the aforementioned structures. Moreover, a comparison on the influence of the two modelling strategies is also herein ascertain.

The effectiveness of the proposed methodology is illustrated in the seismic vulnerability analysis of a set of Portuguese reinforced concrete viaducts designed with National code. Finally, the performances and solutions of the different methodologies are compared and discussed.

INTRODUCTION

Particularly significant damages, from personal, structural or even social nature, have always been associated to earthquakes. So it is recognized the importance and the need for a structure vulnerability analysis, in order to assure eventual adjustments in the seismic response capacity of existent or future structures.

Recent earthquake effects on reinforced concrete bridges have shown that many behave poorly and some possess very low levels of safety, to the extent that they are at risk of collapse, especially those built according to outdated seismic codes. Thus, efforts must be made to develop and apply accurate bridge assessment methodologies that will assist in the determination of failure probability in order to evaluate the need for retrofitting and to improve seismic safety levels.

The intention of this study was to evaluate the seismic safety of a set of reinforced concrete viaducts and make a comparative study between two methods of assessment, the fibre model (Seismostruct) and plastic hinges model (PNL). For this purpose, the model implemented in the calculation program Seismostruct was used for the numerical analysis, which uses a three-dimensional fibre model with distributed non-linearity.

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NUMERICAL MODELS

The laws of material behaviour are numerically established through a procedure based on a fibre model, knowing the geometric characteristics of the piers sections, reinforcement dimensions and location, and material characteristics. For the confined and unconfined concrete behaviour, two models were adopted: Kent and Park (1971) and Mander et.al. (1988). For the steel cyclic behaviour, the Meneggoto-Pinto (1973) model was considered.

The deck of the bridge usually have no substantial efforts when compared to the piers, therefore the structural elements which represent the deck are considered with elastic behaviour. However, when subjected to cyclic loading, the piers develop several phenomena that cause the reduction of section capacity and that control the dissipation of hysteretic energy.

The difficulties of carry out analyses with methodologies that adopt hysteretic non-linear material behaviour increase significantly with the complexity degree of the model, involving a compromise between the accuracy and time computer consuming, and with the several parameters that is necessary to define.

In the current work a comparative study with two strategies to evaluate the seismic behaviour of bridges was intended to carry out. Therefore, two methods of assessment were used, the fibre model (Seismostruct) and plastic hinges model (PNL).

Fibre Model

The distributed plasticity analysis tool employed in the current work is SeismoStruct (2010), a fibre-modelling Finite Element program for seismic analysis of framed structures, which can be freely downloaded from the Internet. This package is capable of predicting the large displacement behavior of space frames under static or dynamic loading, taking into account both local (beam-column effect) and global (large displacements/rotations effects) geometric nonlinearities as well as material inelasticity. The spread of the latter along the member length and across the section area is explicitly represented through the employment of a fiber modelling approach, implicit in the formulation of the inelastic beam-column frame elements employed in the analyses. Full details on this computer package can be found in its accompanying manual.

Plastic Hinges Model

This numerical model adopt plane bar elements with elastic or inelastic behaviour, the latter constituted by an elastic central zone and two extreme zones with plastic characteristics. Thus, the non-linear material behaviour of the bar elements are concentrated on its extremities, since those are the critical zones where cracking occurs, being developed in a short extension, generally not exceeding the cross section height.

In order to obtain moderate calculation efforts, the dynamic analysis of the bridge in the perpendicular direction of the deck axis (transverse direction), a simplified structural model was used, which accurately simulates tri-dimensional behaviour through a bi-dimensional analysis when the bridge is nearly straight - a plane analysis suggested by Delgado et al (2002 and 2004).

For the cross sections of the bars elements with plastic hinges – adopted in the structural modelling - a global non-linear model for the sections must be used. Thus, the moment-curvature loops used in the idealization of the reinforced concrete are obtained by a modified Takeda model (Takeda et al., 1970; Duarte et al., 1990 and CEB, 1996). The characterization of these moment-curvature relationships is based on the initial cracking of the concrete and the yielding of the reinforcement that could be obtained from the monotonic material behaviour of the reinforced concrete element.

SEISMIC SAFETY ASSESSMENT

The safety assessment methodology proposed within the framework of this paper represents a quite general procedure, that can be used for any probability distribution of the seismic action and correspondent vulnerability functions with the desired polynomial approximation (including high order polynomial functions), as well as any probability distribution of the capacity.

In order to evaluate the structural safety with the proposed methodology, it is necessary to calculate the probability of collapse, given by the convolution of the probability distribution of demand with the probability distribution of capacity, Duarte et al (1990). To obtain the demand probability distribution it is necessary to know the probability distribution of the seismic action and define the so called vulnerability curve, a non-linear function that relates the seismic action with the action effect. Therefore, to establish the vulnerability curve, a seismic response of the bridge for increasing seismic intensities must be computed and, for each intensity level, the maximum response value obtained. This curve relates the seismic magnitude with the maximum value of the parameter chosen for describing the structural response, in this paper, the maximum ductility demand on the pier base. These values were adopted as the control parameters for the safety evaluation, bearing in mind that in these structural elements the higher strains and the greater non-linear incursions were developed specially in the zones close to the foundation, which significantly influence the bridge behaviour.

Several accelerograms, in order to take into account the stochastic characteristic of earthquakes, were synthesized, by a random manner, matching the response spectra that characterize the earthquake ground motion.

With these accelerograms, the average values of the ductility demanded for each magnitude of seismic action are obtained and polynomial functions have been adjusted through this series of points, thus obtaining the vulnerability functions of each pier, and, consequently, of the bridge, illustrated with the curve number 3 in Fig. 1.

The intensity of an earthquake could be defined by the peak acceleration value in the horizontal direction, *a*, and the respective distribution of maximum peak accelerations can be identified as an extreme type I probability distribution (Costa, 1993 and Oliveira et al., 1999).

Thus, the seismic action, represented as curve number 1 in Fig. 1, can be characterised by the following equations, Eq (1 and 2):

$$f(a) = \alpha \times e^{y - e^{y}} \tag{1}$$

$$y = -\alpha(a - u) \tag{2}$$

where α and u are the parameters that characterise the extreme distribution, with α = 22.49 E^{-3} e u = 87.38, according to Borges et al. (1982).

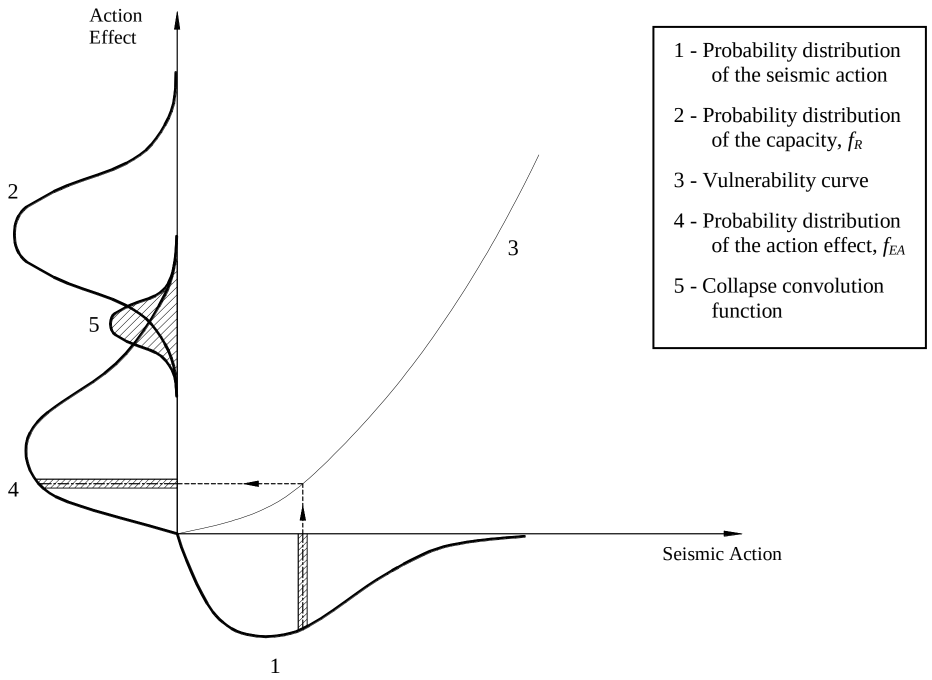


Figure 1. Graphic representation of safety assessment procedure

Finally, the collapse probability of the bridge is evaluated through the integration of the convolution function, Eq (3) and curve number 5 in Fig. 1, defined through the probability distribution function of the action effect, F_{EA} (expressed in maximum ductilities demanded in the bridge) and the capacity probability density function, f_R (expressed in available ductilities), (Borges et al., 1982 and Costa, 1993).

$$P_R = \int_{-\infty}^{+\infty} \left(1 - F_{EA)}\right) f_R \, dx \tag{3}$$

STUDIED VIADUCTS

Within the study it is aimed to assess the seismic safety of a set of viaducts and make a comparative study between the two numerical methods adopted, the model fibre (Seismostruct) model of plastic hinges (PNL).

Geometry

Four viaducts were analysed, shown in Fig.2, by applying the methodology proposed in the previous sections. These structures have 3 distinct deck sections, the same support conditions and different geometrical conditions, as the number of alignments, the number of piers for alignment, the lengths of the piers and spans. The viaducts were designated as V33, V31, V32 and V22, the first digit representing the number of spans and the second digit the number of pier in alignment (Fig. 2). The cross sections of the piers are illustrated in Fig. 3 and 4.

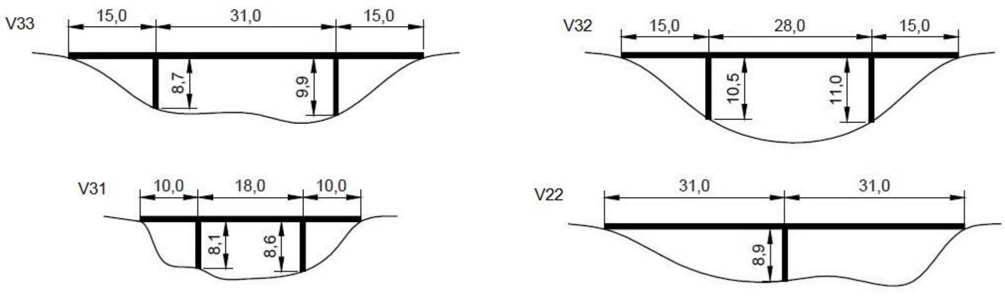


Figure 2. Viaducts configuration

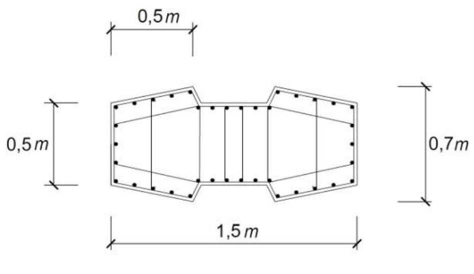


Figure 3. Piers cross section of viaducts V33, V31 e V32

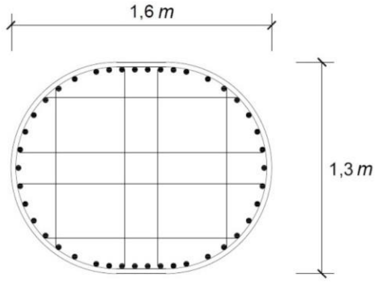


Figure 4. Piers cross section of viaduct V22

Material behaviour

The material behaviour is assessed through the following parameters: steel strength and concrete strength, both unconfined and confined. The concrete used in the viaducts piers corresponds to a C 25/30, except in viaduct V22 that was used the concrete C30/37. Table 1 shows the characteristics of the unconfined concrete, maximum compressive strength (f_c), elastic modulus (E_c) and peak strain (ε_0), and the parameter which characterizes the confinement of concrete (k).

Table 1. Concrete proprieties

	V33	V31	V32	V22
f_c (MPa)	33	33	33	35
E_c (GPa)	31	31	31	33
ε_0	0.002	0.002	0.002	0.002
k	1.1922	1.1922	1.1922	1.1276

For the steel (type A400), used in all the viaducts, was considered a bilinear curve obtained by the yield stress of 400 MPa and the elastic modulus of 200 GPa. Yielding plateau was assumed horizontal, due to the very small hardening.

The value of the piers axial force is obtained considering all permanent loads acting on the deck. Thus the values considered for specific weight of the concrete and asphalt, were 24 kN/m3 and 17 kN/m3, respectively.

Structural modelling

The cross sections of structural elements were adapted to equivalent sections of rectangular geometry keeping the area and flexural inertia, giving the dimensions shown in Table 2.

Table 2. Dimensions of the equivalent sections adopted

	V33		V31		V32		V22	
Element	b (m)	h (m)	b (m)	h (m)	b (m)	h (m)	b (m)	h (m)
Deck	6.81	1.88	2.50	1.70	4.25	1.04	3.93	1.76
Piers	1.50	0.60	1.50	0.60	1.50	0.60	1.60	1.30

The analysis was performed on the transverse direction of the viaduct, and the structures were discretized assuming that the nonlinear behaviour develops in the piers. The deck was discretized with one element in each span, with linear and elastic behavior. The piers, being responsible for the main structure energy dissipation due to its hysteretic behaviour for cyclic loads, were discretized in 8 elements with nonlinear behaviour distributed along its length.

The external connections are made through fixed supports at the piers base, the connection between piers and deck with continuity of moments and abutments considered as pinned supported with restricted rotation along the deck axis, and it was considered 2% elastic damping for the structure.

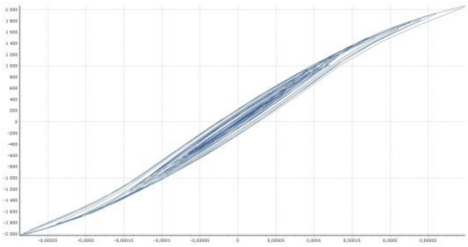
Seismic analysis

For seismic analysis of viaducts under study, the numerical models previously described was used. Thus, it was considered a non-linear dynamic analysis of the viaduct in the perpendicular direction to the deck axis.

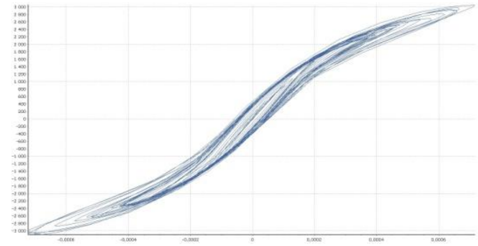
The main results of that study focus on the piers top displacement over the time and the respective diagrams of moment-curvature, allowing to assess energy dissipation and the maximum ductility required, essential aspects for the characterization of the piers vulnerability functions.

To evaluate the viaducts seismic response, non-linear dynamic analysis were used with a set of five different artificial accelerograms generated for seismic action of type 1 and with a duration of 10 seconds. For maximum peak acceleration 270 cm/s^2 has been adopted (as proposed by EC8) and one soil type B was considered. The bridges are located in different locations, most located in earthquake zones B and D, but it was chosen to standardize the analysis for zone A, which corresponds to more severe intensity zone.

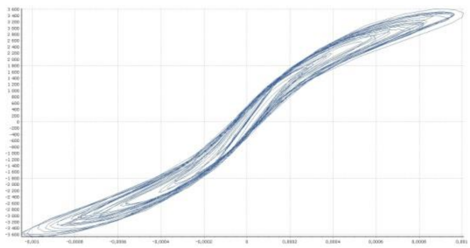
From the results of the seismic analysis, responses of hysteretic moment-curvature at the piers base were obtained, for each considered earthquake and respective intensities, with the aim to obtain the vulnerability function associated with the respective pair of values, acceleration and ductility.



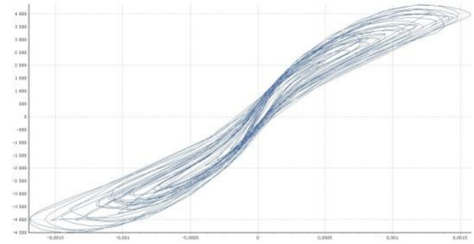
a) 135 cm/s^2



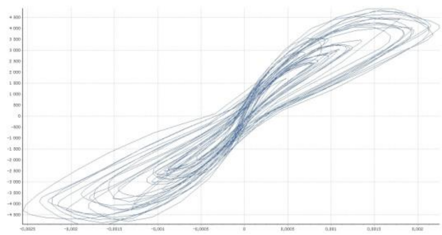
b) 270 cm/s^2



c) 405 cm/s^2



d) 540 cm/s^2



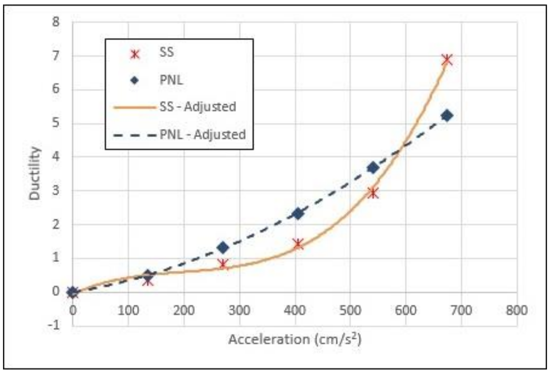
e) 675 cm/s^2

Figure 5. Cyclic moments-curvature diagrams for pier P1 of the viaduct V33, for different peak accelerations

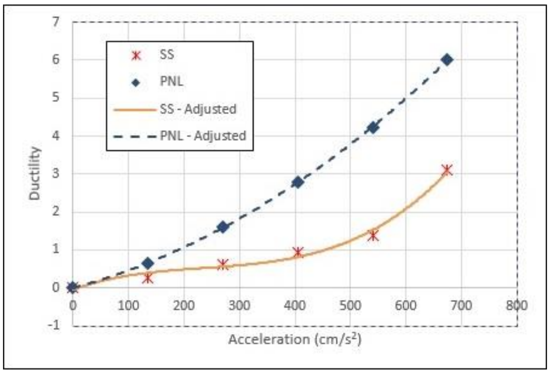
In this work the vulnerability function points were obtained from the average value of the demanded ductilities of five accelerograms, for each peak acceleration intensity. Considering the referred earthquake with 270 cm/s^2 peak acceleration, five intensities were increasingly applied to the bridge (from 0.5 to 2.5 times). In Fig. 5, the moment-curvature diagrams of the pier P1 of viaduct V33 are shown, for the different peak accelerations corresponding to one of the five series of earthquakes considered.

Seismic safety evaluation

Using the least-squares method, polynomial functions of 3rd order were adjusted through a series of points of the demanded ductilities for each level of seismic action, obtaining relationships between action and action effects that characterize the vulnerability functions of each pier of the four viaducts, as represented in Fig. 6 to 9.

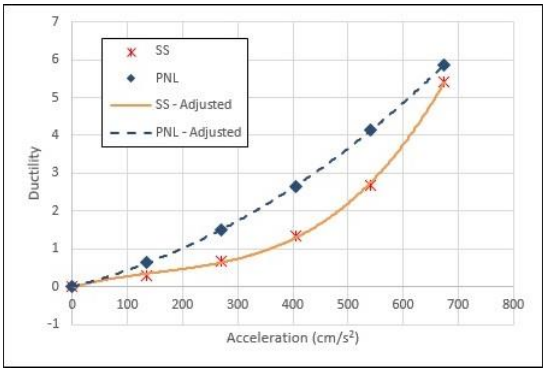


a) P1

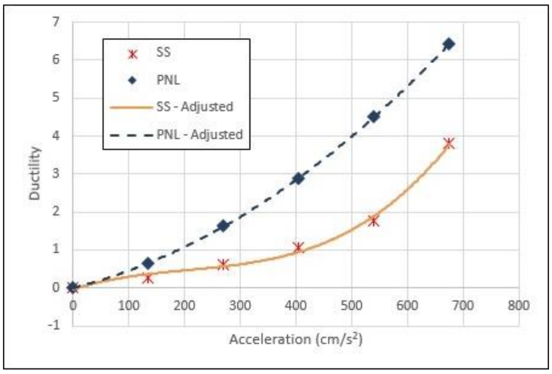


b) P2

Figure 6. Vulnerability functions for piers P1 and P2 of the viaduct V33

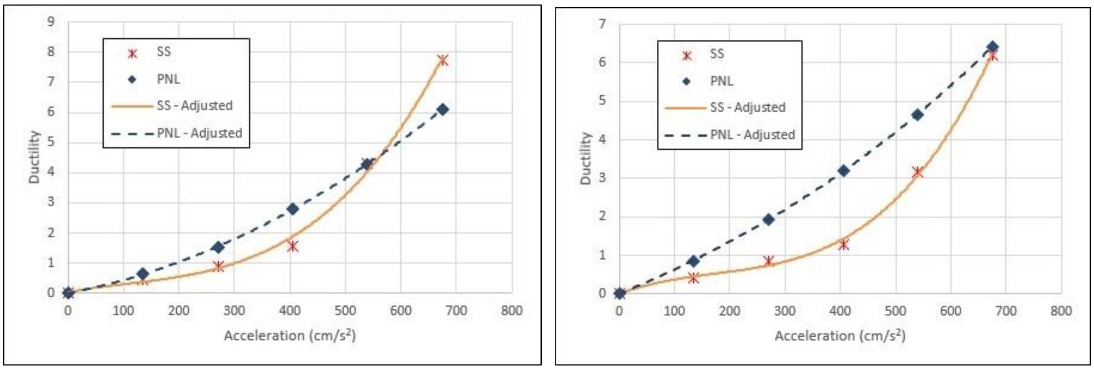


a) P1



b) P2

Figure 7. Vulnerability functions for piers P1 and P2 of the viaduct V31



a) P1 b) P2

Figure 8. Vulnerability functions for piers P1 and P2 of the viaduct V32

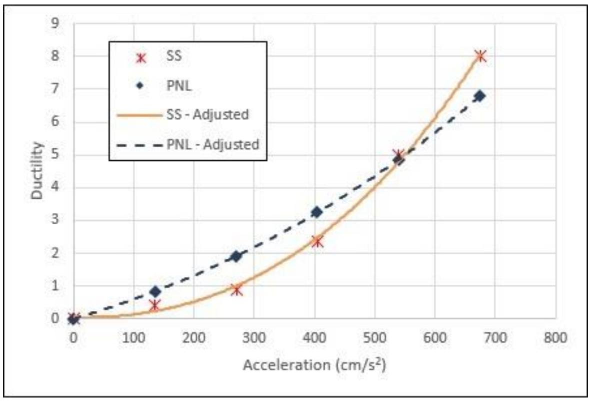


Figure 9. Vulnerability functions for pier P1 of the viaduct V22

The vulnerability functions obtained show a slight difference in their development, and it was found generally lower ductility demands in SS modelling compared with PNL modeling. This difference is obtained due to the consideration of a monolithic structure in the SS modeling, thus having a higher structure frequency when compared to the PNL modeling, which is considered with a pinned connection between pier and deck.

In order to obtain the failure probability, a Gumbel distribution function for the seismic action was used and for the “strength”, a gauss distribution function was adopted. Through the integration of the convolution function the collapse probability is obtained, with the two models referred above, for the more vulnerable pier in each viaduct (see Table 3).

Table 3. Collapse probabilities for the two study methodologies

Viaduct	PNL	SS
V33	6.3560E-06	2.4413E-06
V31	4.8954E-06	1.5425E-06
V32	1.0450E-05	2.1352E-06
V22	1.7800E-05	7.8988E-06

From observation of Table 3 it was verified that the collapse probability obtained with the Seismostruct model (SS) are lower than those obtained with the PNL model (Marques et al., 2005).

The collapse probability of the viaduct is determined assuming that the pier failure is obtained due to lack of available ductility in flexural behavior, so it must be ensured that is not achieved by another collapse mechanism, being therefore important to verify the shear capacity of the piers, considering for this purpose the methodology proposed by Priestley et al. (1996).

It was possible to verify that viaducts, V33 and V22, do not have enough shear capacity to allow that the pier failure occurs by achieving the maximum ductility in flexure. Therefore the collapse probability can be higher than calculated, being necessary to compute the collapse probability with another methodology. So, in the scenario of this research it would be necessary to perform the pier strengthening of viaducts V33 and V22 in order to guarantee a sufficient seismic safety.

CONCLUSIONS

The development of this work has shown that there is a good approximation between the two studied models, regarding the values obtained for the vulnerability functions and for the collapse probability, although slightly lower with the Seismostruct modeling.

The collapse probability obtained with the Seismostruct model are all bellow the value considered as the reasonable limit for bridges (≈10-5), indicating that the viaducts generally has a good safety margin due to the seismic action, as in PNL modeling.

The study conducted on these viaducts allowed to demonstrate that the Seismostruct and PNL models can provide, with reasonable simplicity and accuracy, the evaluation of the seismic response and collapse probabilities of reinforced concrete viaducts.

Finally, in the scenario of this research it would be necessary to perform the pier strengthening of viaducts V33 and V22 in order to guarantee a sufficient seismic safety.

REFERENCES

Borges JF and Castanheta M (1982) "Structural Safety"; Curso 101, 3ª Edition, LNEC, Lisbon

Costa A (1993) "A Acção dos Sismos e o Comportamento das Estruturas" ; PhD Thesis, FEUP, Porto (in Portuguese)

CEB (1996) "RC Frames Under Earthquake Loading"; *Comité Euro-International du Béton*, Bulletin nº231

Delgado, P; Marques, M.; Monteiro, R.; Delgado, R and Costa, A (2006) "Two ways of assessing the seismic vulnerability e seismic vulnerability of bridges", *Proceedings 13th European Conference on Earthquake Engineering*, Geneva, Switzerland, 3-8 September

Delgado P, Costa A, Delgado R (2002) "A Simple Methodology for Seismic Safety Assessment of Bridges", *12ECEE - The Twelfth European Conference on Earthquake Engineering*, London, UK, 9-13 September

Delgado P, Costa A, Pinho R, Delgado R (2004) "Different Strategies for Seismic Assessment of Bridges – Comparative studies", *Proceedings of the 13th World Conference on Earthquake Engineering*, Vancouver, Canada, 1-6 August

Duarte RT, Oliveira CS, Costa AC, Costa AG (1990) "A Non-linear Model for Seismic Analysis, Design and Safety Assessment of Reinforced Concrete Buildings"; *Earthquake Damage Evaluation & Vulnerability Analysis of Building Structures*, edited by A.Koridze, OMEGA Scientific

Kent DC and Park R (1971) "Flexural Members with Confined Concrete." *Journal of the Structural Division*, ASCE, July, vol. 97, No. ST7

Menegotto M and Pinto PE (1973) "Method of Analysis for Cyclically Loaded R.C. Plane Frames Including Changes in Geometry and Non-Elastic Behavior of Elements Under Combined Normal Force and Bending" *Symposium on Resistance and Ultimate Deformability of Structures Acted On by Well Defined Repeated Loads*, IABS Reports Vol. 13. Lisbon.

Mander JB, Priestley MJN, Park R (1988) "Theoretical Stress-Strain Model for Confined Concrete." *Journal of Structural Engineering*, Vol.114, No.8, pp.1804-1826

Marques M, Monteiro R, Curreli D (2005) "Avaliação da Vulnerabilidade Sísmica de Pontes", Relatório da unidade curricular de Seminário de Estruturas do curso de Engenharia Civil, FEUP, Porto (in Portuguese)

Oliveira CS, Sousa ML, Costa AC (1999) "Definition of Seismic Action in the Context of EC-8. Topics for Discussion", *12th World Conference on Earthquake Engineering*, Auckland, New Zealand

Priestley MJN, Seible F, Calvi GM (1996) "Seismic Design and Retrofit of Bridges"; John Wiley & Sons Inc., New York

SeismoStruct (2010) “A computer program for static and dynamic nonlinear analysis of framed structures” by SeismoSoft [online], Available from URL: http://www.seismosoft.com

Takeda T, Sozen MA, Nielsen NN (1970) ”Reinforced concrete response to simulated earthquakes“; *Journal of the Structural Mechanics Division* of the ASCE, ST12, 96