



DIFFERENT STRATEGIES FOR SEISMIC ASSESSMENT OF BRIDGES – COMPARATIVE STUDIES

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SUMMARY

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possess very low levels of safety, to the extent that they are at risk of collapse, especially those built according to outdated seismic codes.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.

Within the framework of the current work, seismic assessment analyses have been undertaken using three different numerical methodologies: (i) simplified 2D concentrated plasticity modelling, (ii) 3D concentrated plasticity modelling and (iii) 3D distributed inelasticity fiber modelling. Structural safety is then evaluated by means of a probabilistic methodology of seismic failure quantification involving a non-linear transformation of the seismic action into structural effects.

The effectiveness of the proposed methodology is illustrated in the seismic vulnerability analysis of a reinforced concrete bridge constructed in Warth (Austria) and designed to a very low seismic level. Finally, the performances and solutions of the different methodologies are compared and discussed.

INTRODUCTION

The difficulties of carry out analyses with methodologies that adopt hysteretic non-linear material behavior 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 several strategies to evaluate the seismic behavior of bridges was intended to carry out. Therefore, three numerical models were used:

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- (i) PNL (Delgado [1]) – simplified bi-dimensional structural modelation, simulating the transversal behavior of the bridge (tri-dimensional analysis) through a plane model, and non-linear behavior lumped in plastic hinges.
- (ii) Ruaumoko (Carr [2]) – tri-dimensional structural modelation with non-linear behavior lumped in plastic hinges.
- (iii) Seismostruct (Seismosoft [3]) – tri-dimensional fiber model with distributed inelasticity.

A static analysis of the piers was carried out, imposing cyclic displacements, in order to calibrate the hysteretic parameters that characterize the energy dissipation of the bridge piers. These parameters are used in the seismic analysis, characterizing the piers properties for the structural modelling of the bridge, where the non-linear behavior is considered.

The comparative analyses were carried out with increasing seismic action level, in order to show the sensibility of the results with the different non-linear levels. Finally, with these obtained results the correspondents vulnerability curves are computed and structural safety of the bridge evaluated.

SIMPLIFIED 2D PLASTIC-HINGE MODELLING

This numerical model, developed at the Faculty of Engineering of University of Porto, adopt a structural modelling that use plane bar elements with elastic or inelastic behavior, the latter constituted by an elastic central zone and two extreme zones with plastic characteristics. Thus, the non-linear material behavior 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 - a plane analysis suggested by Delgado et al, [1] and [4] - was used, which accurately simulates tri-dimensional behavior through a bi-dimensional analysis when the bridge is nearly straight, as is the case. So, with this model illustrated in Figure 1 it is possible to simulate, in a 2-D domain, the links between piers and deck through ties that insure compatibility of displacements, allowing independent rotations. The piers are oriented to have their larger dimension in the plane direction and an axial load equal to the vertical action of the deck must be considered.

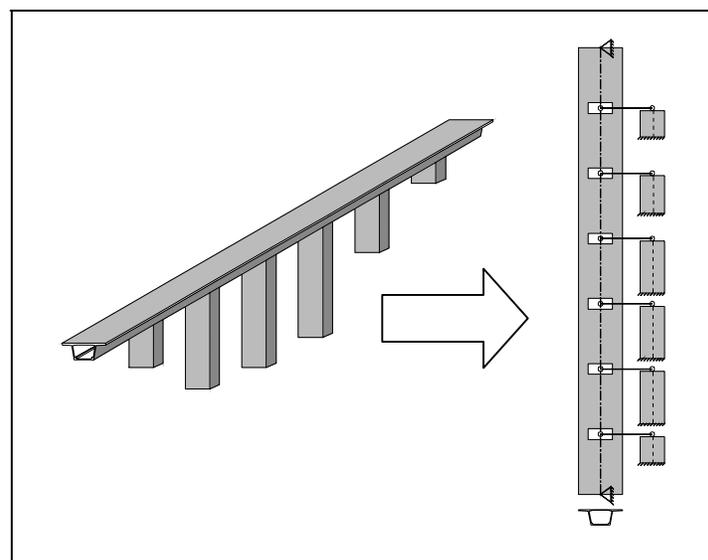


Figure 1: Bridge structural modelling.

This plane model is extremely economic in terms of calculation effort, especially when non-linear effects must be considered, but it allows the main aspects of the structural behavior of the bridge for horizontal actions in the transverse direction of the deck axis to be captured in an accurate way. As mentioned above, only the eventual torsion stiffness of the deck mobilized with the rotation of the top extremity of the piers is not included.

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 ([5]; [6] and [7]). 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 behavior of the reinforced concrete element.

The laws of monotonic material behavior are numerically established through a procedure based on a cross section fiber model, [8], knowing the geometric characteristics of the piers sections, reinforcement dimensions and location, as well as material characteristics. This fiber model also allows the separation of confined and unconfined concrete regions.

The definition of the cyclic stress-strain response can be evaluated through the monotonic stress-strain curve of the structural element, associated with adequate rules that control the response evolution, in load-unload cycles, load inversion and the pinching effect, [6] and [7].

PLASTIC-HINGE MODELLING

This numerical model developed by Athol J. Carr, [2], at the Canterbury University, New Zealand, use a structural modelation quite similar to the model referred before, adopting plane bar elements with elastic or inelastic behavior, the latter constituted also by extreme zones with plastic characteristics, therefore being an numerical model with plastic hinges. However, the dynamic analysis of the bridge at the perpendicular direction of the deck is carried out through a tri-dimensional structural modelling.

As well as the PNL model, for the plastic hinges of the bar elements adopted in the structural modelation a non-linear global model based in the modified Takeda model is used. In this work, bi-linear laws at the piers cross sections with non-linear behavior were adopted for the characterization of the moment-curvature curves, based in the monotonic laws of the RC elements, more precisely on the initial stiffness and the stiffness after the reinforcement yielding.

DISTRIBUTED INELASTICITY MODELLING

The distributed plasticity analysis tool employed in the current work is SeismoStruct [3], 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.

SAFETY ASSESSMENT

To evaluate the structural safety 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 [7]. 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 return periods (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 behavior.

For the reached ductilities at each return period (seismic level), polynomial functions have been adjusted through this series of points, thus obtaining the vulnerability functions of each pier, and, consequently, of the bridge.

The seismic action could be characterized by a extreme distribution, Gumbel type 1, of the maximum accelerations, a , (Campos Costa [9]), represented by Eq. (1) and with the following parameters, α e u :

$$f(a) = \alpha \cdot \exp(y - \exp(y)) \quad ; \quad y = -\alpha(a - u) \quad (1)$$

The characteristic value of the peak acceleration, a_k , for the referred extreme distribution, to 50 years of the structure life time and to 475 years of the return period, could be adopted as the 0,90 fractile, as demonstrated by Eq. (2).

$$F(a_k) = \left(1 - \frac{1}{475}\right)^{50} = 0.90 \quad (2)$$

In this case, for the extreme distribution the parameters $\alpha = 22,5E-3$ and $u = 87.4\text{cm/s}^2$ were obtained, considering the characteristic value of the peak acceleration equal to, $a_k = 218\text{cm/s}^2$ and adopting a variation coefficient equal to 0,5.

A transformation of this action through the vulnerability function is carried out, and the action effect in the bridge obtained, more precisely in each of its piers, once a vulnerability function for each had been defined through the bridge response. Thus the probability density function of the action effect, expressed in demanded ductilities is obtained and the corresponding probability distribution of maximum ductility demand, F_{EA} , is achieved.

For the characterisation of the “strength” of pier sections - in this case their available ductility - a probability density function, f_R , was considered. This function was defined through the mean ductility, μ_d , and its variation coefficient, VC, by means of a simulation method, the Latin Hypercubic Sampling, as employed by Delgado et al [10].

Finally, the collapse probability of the bridge is evaluated through the integration of the convolution function, Eq (3), defined through the probability distribution function of the action effect, F_{EA} - expressed

in maximum ductilities demanded in the bridge - and of the strength probability density function, f_R - expressed in available ductilities -, Costa [11].

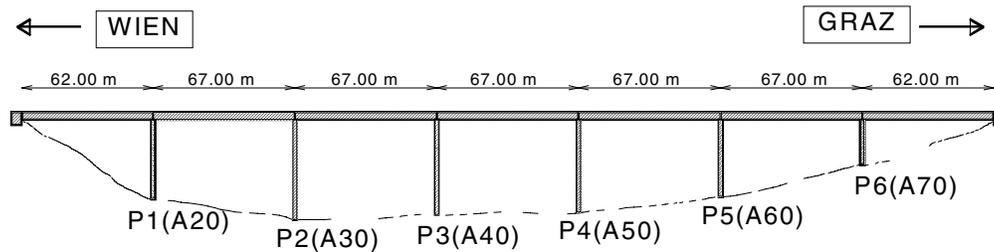
$$C(\mu) = (1 - F_{EA}) \cdot f_R \quad (3)$$

TALÜBERGANG WARTH BRIDGE (VAB PROJECT)

In this work the Talübergang Warth's bridge, built in the seventies in Vienna and designed for a low seismic level, was adopted. This bridge was the main study purpose on the scope of a European project, Vulnerability Assessment of Motorway Bridges (VAB), in which the Faculty of Engineering of the University of Porto participated.

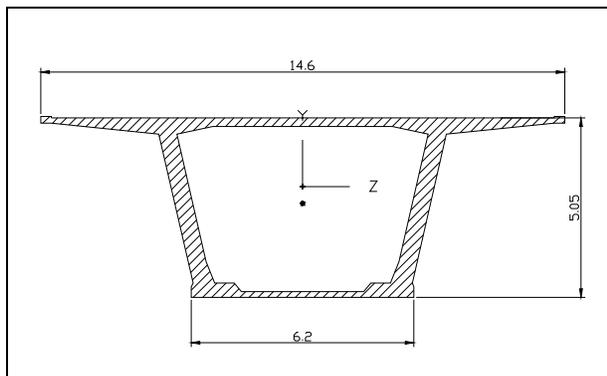
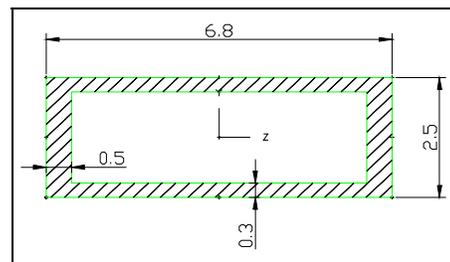
Bridge Geometry

The reinforced concrete Talübergang Warth bridge is located approximately 63 km to south of Vienna and it is constituted by a continuous deck, with five central spans of 67m and two extreme spans of 62m, supported by piers with rectangular hollow section, pinned in their connection to the deck and fully-fixed in the base, where is concentrated the largest seismic effects. This bridge is constituted by six piers with heights between 16.9 and 38.9 meters and identical rectangular hollow sections in concrete, being the external dimensions of 2.5x6.8m² with the largest dimension in the transverse direction of the deck axis, and with a walls thickness of 0.3m along the largest dimension of the section of the pier and 0.5m along the smallest. In Figure 2 the geometry of the bridge is illustrated, with the further on adopted designation for the piers, and in Table 1 the piers height are presented, measured from the top of the foundation to the deck base.



Deck section properties:

- Area : 10.02062 m²
- Prin. moment of inertia Y : 124.3786 m⁴
- Prin. moment of inertia Z : 34.77017 m⁴
- Torsional constant : 52.84362 m⁴



Pier section properties:

- Area : 5.98 m²
- Prin. moment of inertia Y : 34.61394 m⁴
- Prin. moment of inertia Z : 5.538984 m⁴
- Shear ratio Y : 3.40178
- Shear ratio Z : 1.736134
- Torsional constant : 15.12595 m⁴

Figure 2: Bridge Geometry.

Table 1: Piers height

Pier	P1 (A20)	P2 (A30)	P3 (A40)	P4 (A50)	P5 (A60)	P6 (A70)
L (m)	29.8	38.9	37.8	36.0	30.0	16.9

The structural modelling was carried out with one element piers with inelastic behavior at the extremities, 3.4 m length of plastic hinge. The deck is constituted by 16 elements and the connecting ties with one element each, with linear and elastic behavior.

Piers Sections

All the piers have the same transverse hollow section in concrete, with the external dimensions of 2.5x6.8m² and the interiors, corresponding to the hollow interior zone, of 1.9x5.8m². The piers longitudinal reinforcement is suppressed along its height in three regions, but it is distributed evenly in the transverse section. For all the piers the transverse reinforcement spacing is of 20 cm, constituted by a single rectangular stirrup in each of the four walls of the hollow section, with an 8 mm diameter for the current section and a 12 mm diameter in a zone of 1m close to the basis of each pier. For all the piers, a 4cm thick cover concrete was used.

Non-linear material behavior of the piers sections

The concrete of the piers bridge is the B400, with a maximum compression tension of 43.0 MPa and a Young modulus of 33.5 GPa. The concrete confinement depends on the efficiency of the stirrups. In this case, very small values are adopted, due to the much reduced confinement obtained with the transverse reinforcement, since only the four corners of the piers are truly confined. The steel is the type TR50 according to the supplied design project, and a bilinear curve was adopted, defined through a Young modulus of 200 GPa and a yielding tension of 545.0 MPa, being considered that no hardening exists, therefore with a horizontal yielding.

The axial force resulting from the deck and the piers weights are the following: 26460kN for the pier P1, 28633kN for the pier P2, 28473kN for the pier P3, 28218kN for the pier P4, 27353kN for the pier P5 and 24610kN for the pier P6.

In Figure 3 the moment-curvature curves computed at the base of piers P3 and P6 are illustrated, with several structural models.

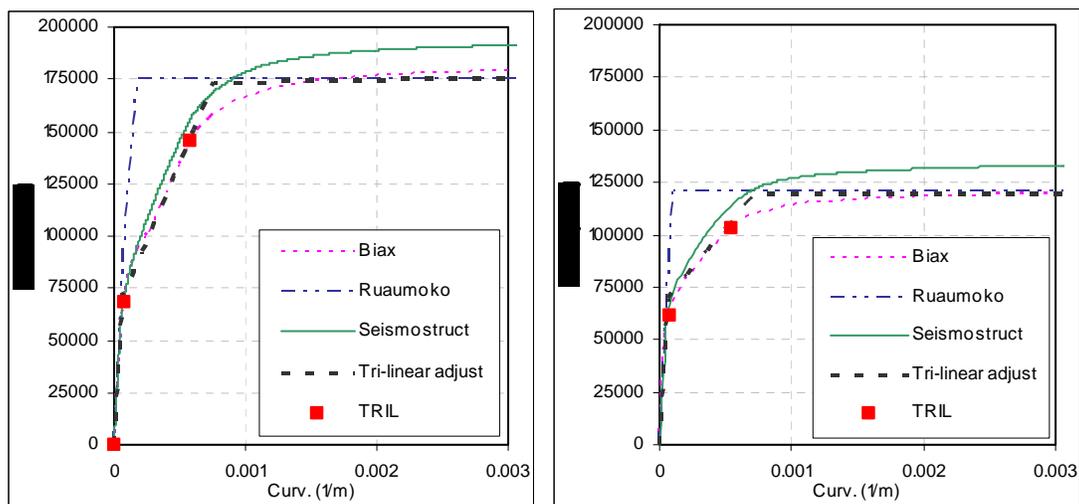


Figure 3: Moment-curvature curves at the base of piers P3 (left) e P6 (right).

Using the cross section fiber model (Biax, [8]) the moment-curvature curves with dot line were obtained, computing the section moment for a given curvature and axial force applied to the pier. Based on this curves, tri-linear laws were adjusted for the monotonic loading, ruling the piers non-linear behaviour in the PNL program. For Ruaumoko, bi-linear laws were defined with the section initial stiffness and after yielding stiffness (bold dot line). With Seismostruct numerical model the moment-curvature monotonic curve (solid line) at the piers base was obtained through a pushover analysis. The concrete first crack and the steel first yielding in the cross section at the piers base were also computed with the analytical equations programmed by Arêde (TRIL, [12]) and illustrated (in Figure 3) with square points.

As it can be seen, the curves obtained with the Biax fiber model and Seismostruct model are reasonably close, but the ultimate moment value is higher in the Seismostruct program. Due to the bi-linear law adopted for the analysis with Ruaumoko program and the tri-linear law for the PNL program, there are significant differences at the yielding curvatures. This will be the main cause for the differences at the bridge seismic response for low seismic actions, as it will be seen ahead.

For the plastic hinges models (PNL e Ruaumoko), these monotonic laws are the basis for the characterization of the piers cyclic behavior, involving alterations to their initial characteristics, caused by the cyclic loads, simulating the strength and stiffness degradation, as well as the pinching effect.

Pier response to imposed top displacements

The analysis of the piers for cyclic loads is presented, corresponding to a horizontal displacement law applied to their top extremities, and considering an independent operation in each of them. The displacement is prescribed in the free extremity of the pier (P3 e P6) and in the largest transverse direction, therefore, in the perpendicular direction of the deck axis.

The displacement time history imposed on the piers P3 and P6, illustrated in Figure 4, corresponds to the application of twelve semi-cycles of growing intensity.

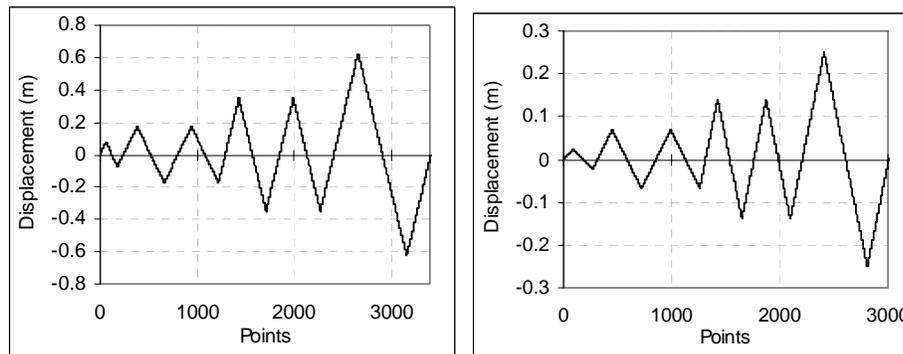


Figure 4: Displacement time history imposed on the pier P3 (left) and pier P6 (right).

These experimental cyclic tests were carried out at Ispra laboratory, Italy, giving a good basis for comparing the results obtained by the different numerical models. It was also possible to compare the results with a damage model carried out by Vila Pouca, [13], a much more refined (and accurate) methodology based on finite elements. Figure 5 and 6 represents the force-displacement relationships obtained experimentally and numerically (PNL, Seismostruct and Damage Model) on the top of the piers, due to the displacement time history imposed on the piers. The force values and the dissipated energy of these numerical results are very close to the experimental results of Ispra, Pinto A. et al, [14] and Gago et al, [15], showing the good performance of the numerical model used for simulating the inelastic behavior of the piers under cyclic loads.

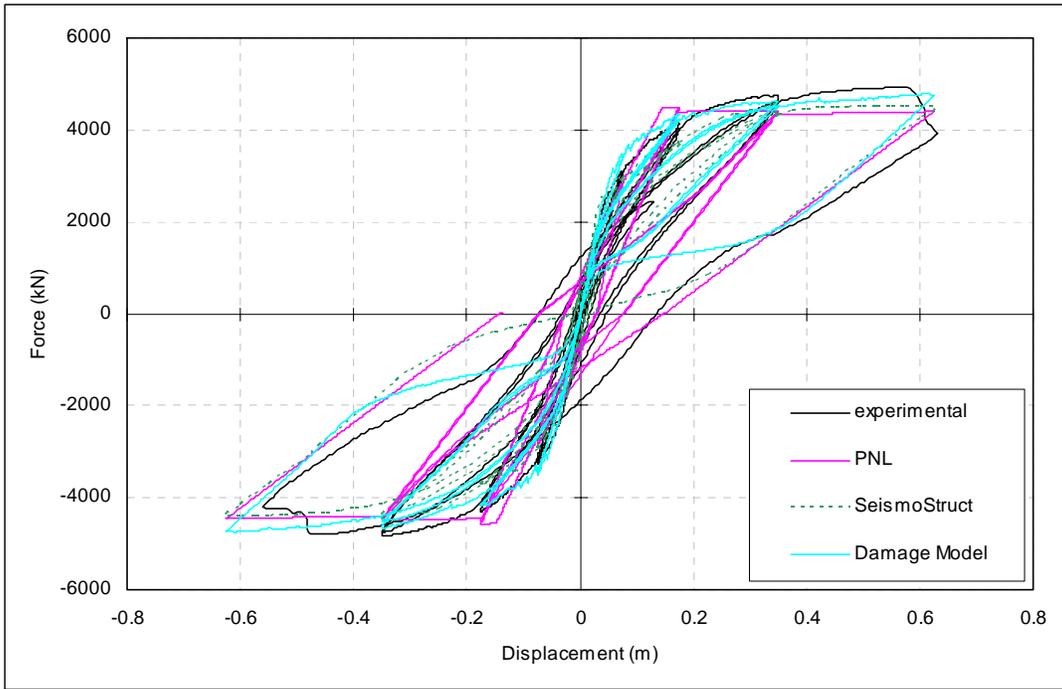


Figure 5: Force-displacement curves for the pier P3.

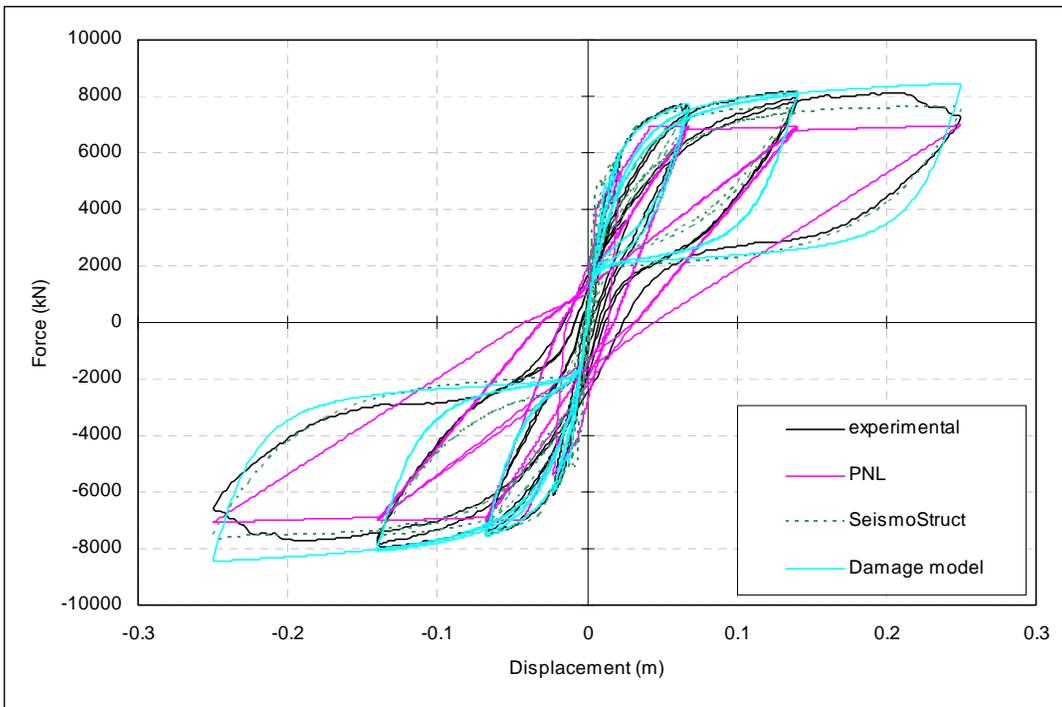


Figure 6: Force-displacement curves for the pier P6.

Seismic response

For the dynamic and non-linear analysis of the bridge a synthesized artificial accelerogram compatible with the response spectrum of the EC8 was considered, for medium soil (soil B), with 15 seconds duration and peak acceleration related with the return period. For the accelerogram with return period of 2000 years, the peak acceleration is 3,728 m/s², according to seismicity studies carried out at the bridge location Gago et al, [15].

Due to the hysteretic material behavior when subjected to cyclic loadings, the plastic hinges carry the main responsibility for energy dissipation in the structure, so a low elastic structural damping proportional to the stiffness and equal to 1.5% was considered.

Figures 7 to 10 illustrates the comparison of the pier P3 top displacement responses obtained numerically, using the several procedures presented (Seismostruct, PNL and Ruaumoko), for four different return periods: 73, 475, 2000 e 9980 years. These four responses are in general very close as much in frequency as in maximum values, excepted the Ruaumoko response for the 73 years return period, which have higher displacement values, due to the adopted bi-linear law remains elastic until the yielding is reached and so without hysteretic damping. It also can be noticed that the PNL program, for return periods of 2000 and 9980 years, have displacement values, at the last part of the response, slightly higher than the others two models.

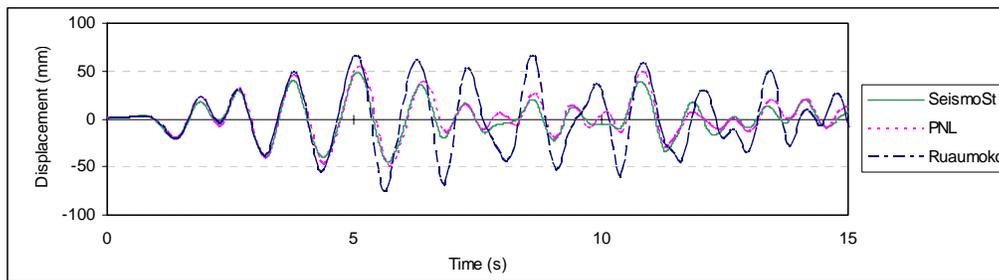


Figure 7: Pier P3 response for 73 years return period.

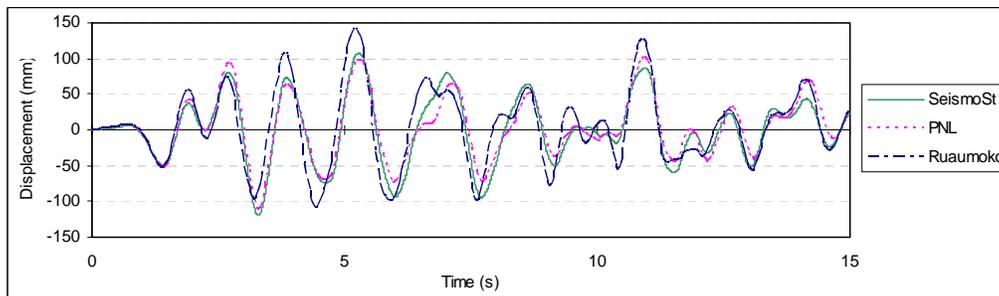


Figure 8: Pier P3 response for 475 years return period.

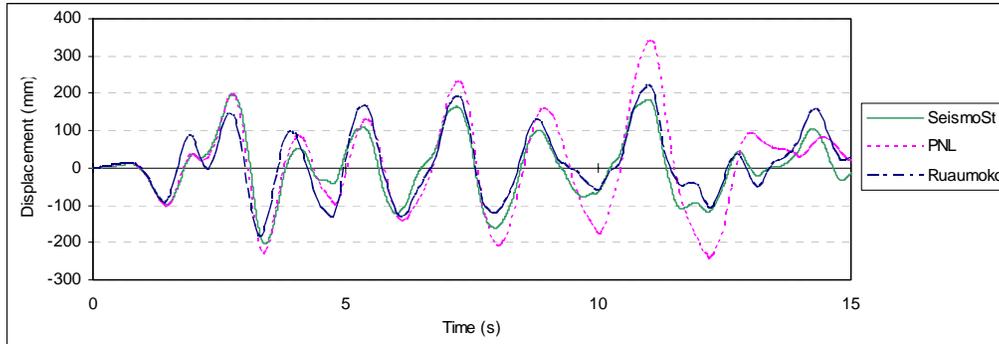


Figure 9: Pier P3 response for 2000 years return period.

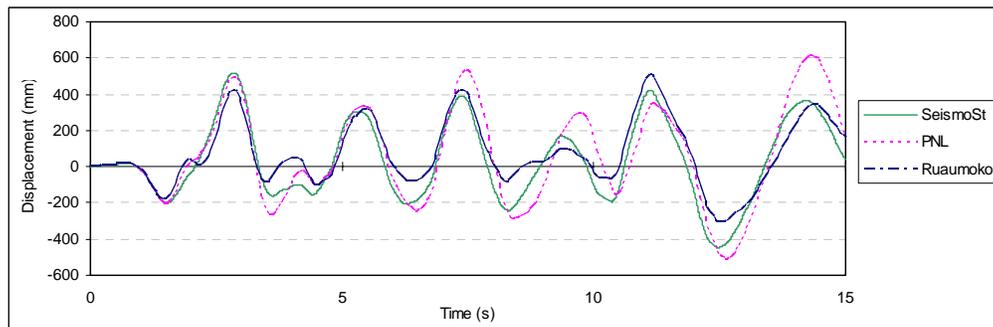


Figure 10: Pier P3 response for 9980 years return period.

Safety assessment

The seismic safety assessment of the bridge was performed by the evaluation of the probability of the bridge failure through the structural vulnerability curve, taking into account the probability density functions of the seismic action and the selected “structural strength” - the available ductility in these analyses - [1] and [7].

In order to obtain the vulnerability curves of the piers, different return periods of the referred earthquake were increasingly applied to the bridge, being these values related with the peak accelerations. Therefore, from each return period value the corresponding moment-curvature result was obtained, allowing the evaluation of the maximum ductility demanded at the piers base, the values adopted as the control parameters to evaluate the safety assessment of the piers.

Using the least-squares method, polynomial functions of 2nd 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, as represented in Figure 11 for the piers P3 and P6. This procedure was carried out for the 2D numerical model with concentrated plasticity (PNL) and for the tri-dimensional fiber model with distributed inelasticity. The differences verified at the vulnerability curves for the two modelations are significant, about 15% at pier P3 and about 45% at the pier P6, dues to the higher values obtained by the PNL model near the end of the response.

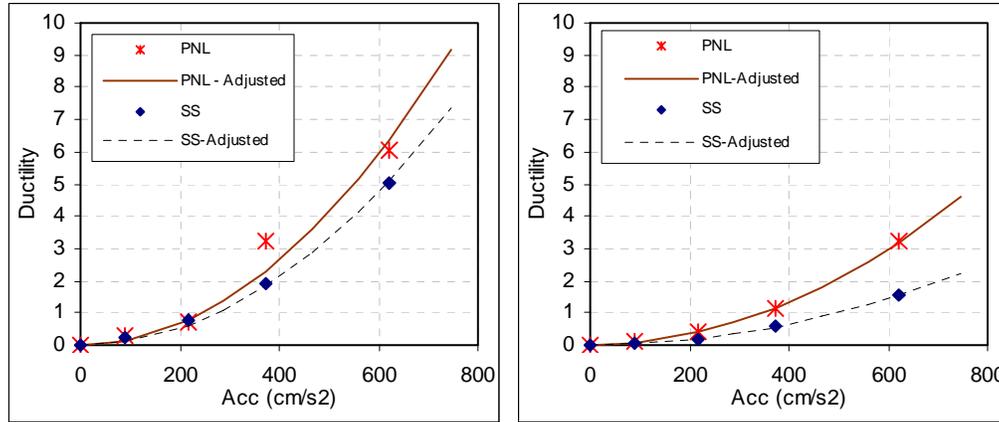


Figure 11: Vulnerability curves of the piers P3 (left) e P6 (right).

In order to obtain the failure probability, a Gumbel distribution function for the seismic action was used. For the “strength”, a gauss distribution function was adopted with a 4.4 mean ductility value and a 0.21 variation coefficient value, [16]. Through the integration of the convolution function the failure probability is obtained, with the two models referred above, for pier P3 (see Table 2), the more vulnerable in this bridge.

Table 2: Collapse probabilities.

Pier	PNL		Seismostruct	
	$\mu_d = 2.2$	$\mu_d = 4.4$	$\mu_d = 2.2$	$\mu_d = 4.4$
P3	0.000204	0.000030	0.000075	0.000010

The collapse probability obtained with the Seismostruct model correspond to a value considered as the reasonable limit for bridges ($\approx 10^{-5}$), and for PNL model the collapse probability value is above this limit. However, if the available mean ductility value for the section was reduced to half, the collapse probabilities increase drastically.

This achieved collapse probabilities to the bridge, computed through the failure probability of this piers, assume that the collapse of the bridge is verified due to a ductility failure mechanism. Therefore, the guarantee that no other collapse mechanism of the bridge will be reached before is needed, mainly the strength capacity verification of the structural elements for the maximum forces obtained.

For the piers the shear strength capacity is needed to verify, mainly to the pier P3 and P6, which have a maximum shear force demanded of about 5000kN and 8000kN, respectively. For the shear strength capacity the Priestley et al Model [17] was used. The concrete component (V_c) – assuming an reasonable high curvature ductility of 9 –, transverse reinforcement component (V_w) and axial load component (V_p) are represented in the Table 3 and also the shear capacity supplied (V) at each pier which is enough to prevent brittle shear failure mechanism.

Table 3: Shear strength capacity.

Pier	V_c	V_w	V_p	V (kN)
P3	3660	2771	2546	7631
P6	3129	2771	4922	9199

CONCLUSIONS

In this work is was presented a comparative study of different methodologies for the seismic response and seismic safety evaluation of a reinforced concrete bridge build in Vienna and experimentally study in Ispra. The methodologies shows good accuracy for the structural assessment analyses of bridges subjected to static and cyclic displacement laws, as well as seismic actions, confirmed by the excellent approach of obtained results. To the four different return periods: 73, 475, 2000 e 9980 years, the seismic results have presented in a general way a very close course, except at the Ruaumoko result for a low level of seismic action where the linear behavior have predominance, and the PNL results for high seismic actions, that seems to present a slightly low damping than the other two models.

Therefore, this models show adequacy for the safety evaluation studies, where a great number of analysis is needed. Adopting as the control parameter the demanded curvature ductility at the piers base the safety of the bridge with two hypotheses of numerical modelations was evaluated. The obtained collapse probability has a value close to the limit considered as reasonable, having available an almost null safety coefficient for the news codes seismic action predicted in the bridge site.

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