COMPDYN 2011
3rd ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis, V. Plevris (eds.)
Corfu, Greece, 25–28 May 2011

NUMERICAL SIMULATIONS OF THE WARTH BRIDGE SEISMIC RESPONSE

Pedro Delgado[1], **Nelson Vila Pouca**[2], **António Arêde**[2], **Rui Faria**[2], **Raimundo Delgado**[2]

[1] Polytechnic Institute of Viana do Castelo, Portugal
Avenida do Atlântico, 4900-348 Viana do Castelo
pdelgado@estg.ipvc.pt

[2] University of Porto, Civil Engineering Department
Rua Dr. Roberto Frias, s/n, 4200-465 Porto, Portugal
nelsonvp@fe.up.pt, aarede@fe.up.pt, rfaria@fe.up.pt, rdelgado@fe.up.pt

Keywords: Numerical structural modeling, Non-linear behavior, Seismic response, Bridge piers, asynchronous, synchronous

Abstract. *This work consists on the seismic analysis of the Talübergang Warth bridge studied within the framework of the European research project entitled VAB - Vulnerability Assessment of Bridges [1]. This case-study bridge was built in Austria during the 70's, designed to a very low seismic level, consisting of a seven span continuous deck supported on two abutments and six rectangular hollow section piers, the latter with some peculiar characteristics concerning the reinforcement detailing whose modeling is quite demanding for cyclic response simulation. Still in the VAB project context, a physical scaled model of the bridge was also experimental tested under pseudo-dynamic conditions at the JRC-Ispra [2] and the results were compared against numerical simulations carried out by the FEUP team involved in the project.*

The non-linear behavior is considered concentrated in the piers, which are discretized with (i) a refined constitutive model or (ii) a Plastic hinge type model for the nonlinear material behavior simulation. For the numerical prediction of the seismic performance of the Talübergang Warth bridge these methodologies were adopted with the seismic action taken as an asynchronous and synchronous ground motion induced along the transverse direction only.

The main results of the seismic analyses will be presented focusing on the essential role that the longitudinal reinforcement curtailment plays on macro-crack localization, which leads to a shift of the plastic hinge (usually at the base of piers) up to the elevation where a significant reduction of the longitudinal reinforcement takes place. From the comparison of the numerical predictions with the experimental results, as recorded during the pseudo-dynamic tests performed at the JRC, the capability from the damage model to provide accurate simulations of the seismic performance of the bridge was brought into evidence, even when the piers are difficult to simulate due to the concrete hollow section geometry and to the unusual reinforcement layout adopted in the design (as in this case).

1 INTRODUCTION

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 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, two numerical models were used:

(i) PNL [3] –non-linear behavior lumped in plastic hinges.

(ii) Damage Model [4] – refined constitutive model for the nonlinear material behavior.

The numerical prediction of the seismic performance of this bridge adopts a peculiar two-dimensional modeling of the entire bridge [3, 5] consisting of a simplified plane model with easy practical application and involving reduced calculation efforts while maintaining adequate accuracy. The bridge structural modeling is carried out with plane elements, bars or 2D finite elements, maintaining the essential features of the 3D transverse response through an appropriate plane structural simulation. This is achieved by considering the deck and piers in the same plane, while the top pier horizontal displacement and the correspondent horizontal transversal deck displacement are constrained by uniaxial tie elements in order to have the same value. 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, the performances and solutions of the different methodologies are compared and discussed.

2 DESCRIPTION OF STUDIED BRIDGE

The Talübergang Warth bridge was studied within the framework of the European research project entitled VAB - Vulnerability Assessment of Bridges [1, 2], and is illustrated in Figure 1. This bridge, at about 63 km to the south of Vienna, is formed by a deck with 7 spans and a total length of 459m, and it is supported by 6 piers and 2 abutments. Piers' nomenclature is also indicated in Figure 4, to ensure a clear identification.

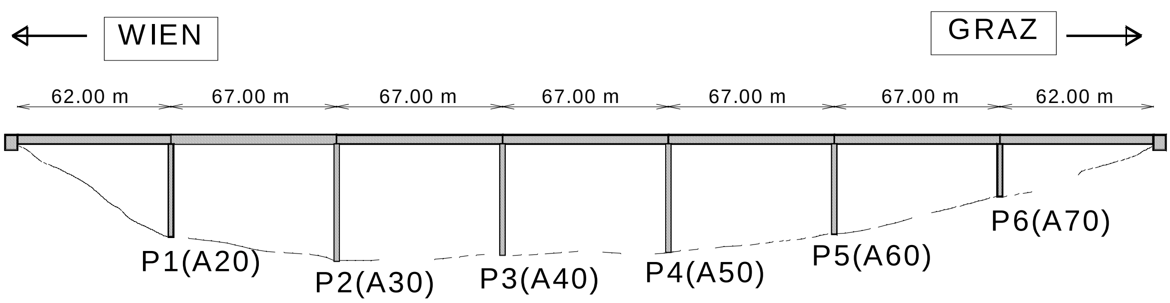


Figure 1: Talübergang Warth bridge and piers' nomenclature.

The geometry of the bridge piers was defined on the basis of the design drawings documented in Talübergang Warth Bridge Drawings (1975). Figure 2 and Table 1 reproduce the basic geometry of the piers, both for the concrete section and for the footings (piers height L refers to the distance between the top of the footing and the base of the deck bearings.). All the piers have $6.8{\times}2.5m^2$ concrete hollow sections.

In what concerns the curtailment of the longitudinal reinforcement, involving bars of different diameters, 4 regions are depicted in Figure 2. Details about the reinforcement layout are referred in previous publications [1, 6]. For piers P2, P3 and P4 the first interruption of the longitudinal rebars occurs close to the foundation, leading to a reduction of about 50% on the amount of steel reinforcement.

Transversal reinforcement is provided by a single rectangular stirrup on each wall of the hollow section [1, 6], according to the following distribution: *(i)* $A_{sw} = \phi12//0.20$m on the first 1m layer close to the piers footing, and *(ii)* $A_{sw} = \phi8//0.20$m elsewhere.

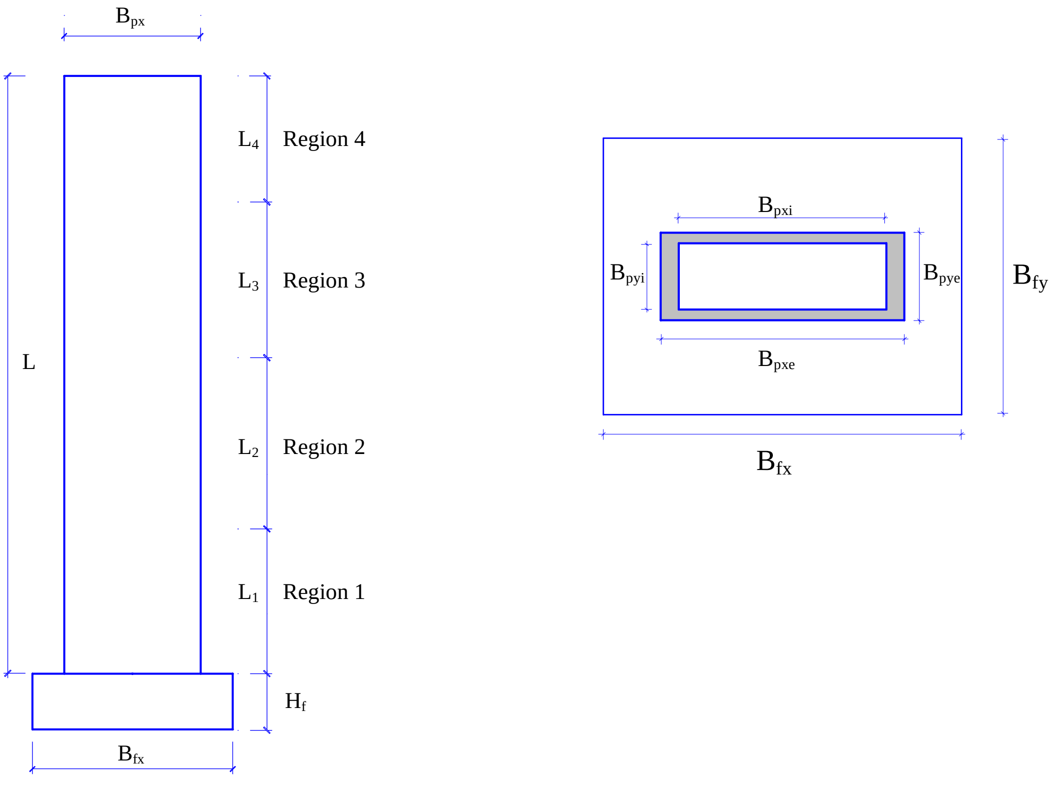


Figure 2: Basic geometry nomenclature.

Pier	L (m)	L/B_{pxe}	L/B_{pye}	Hollow section	Foundation		
					B_{fx}	B_{fy}	H_f
P1 (A20)	29.8	4.4	11.9	B_{pxe} = 6.8m B_{pxi} = 5.8m B_{pye} = 2.5m B_{pyi} = 1.9m	10.8	10.1	3.45
P2 (A30)	38.9	5.7	15.6		10.2	8.0	2.80
P3 (A40)	37.8	5.6	15.1		10.2	8.0	2.80
P4 (A50)	36.0	5.3	14.4		10.2	8.0	2.80
P5 (A60)	30.0	4.4	12.0		10.5	9.0	3.20
P6 (A70)	16.9	2.5	6.8		10.4	9.5	3.20

Table 1: Basic geometry.

Taking into consideration the stirrup arrangement documented in the design drawings, it is doubtful that concrete could benefit from any significant effect of confinement. Accordingly, for Talübergang Warth bridge piers the class B400 concrete was assumed as under unconfined conditions, with the material properties reproduced in Table 2 (average values).

E (GPa)	f_{co} (MPa)	ε_{co}	f_o^+ (MPa)	f_{cm} (MPa)	ε_{cm}	Z
33.5	43.0	2.0‰	3.1	–	–	100

Table 2: Concrete properties (class B400, unconfined).

Table 3 resumes the average material properties assumed for the longitudinal and transversal rebars, defined as corresponding to a class RT50 steel.

f_{sy} (MPa)	f_{su} (MPa)	ε_{su}	ε_{sh}	E_h/E	R_o	a_1	a_2
545	611	100‰	5.0‰	0.0034	20	18.5	0.15
f_{su} and ε_{su} are the ultimate strength and strain for the steel.							

Table 3: Steel properties (class RT50).

3 SIMULATION OF THE PSD TEST (ASYNCHRONOUS MOTION)

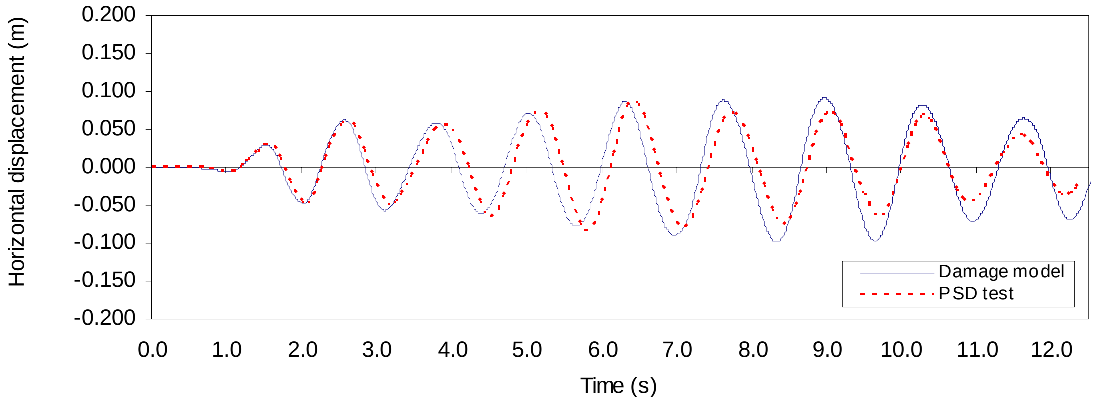
The Talübergang Warth bridge was tested on the ELSA Laboratory (Joint Research Centre, at Ispra) under pseudo-dynamic conditions and asynchronous ground motion induced along the transverse direction only, being all the experimental results described in the reference Pinto et al. [2].

The non-linear behavior is considered concentrated in the piers, which are discretized with a refined constitutive model for the nonlinear material behavior simulation. The refined numerical model combines a 2D plane-stress finite element discretization for the concrete with 2-noded truss elements to include the steel reinforcement. The concrete behavior is simulated resorting to a constitutive model based on Continuum Damage Mechanics, involving two independent scalar damage variables to account for degradation under tensile and compressive stress conditions. The cyclic response of steel is simulated via the Giuffré-Menegotto-Pinto model [7].

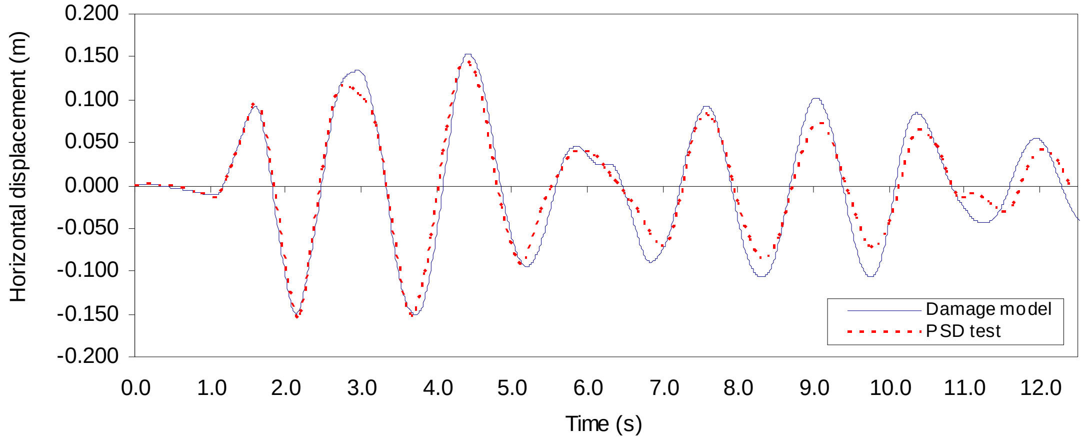
The numerical analysis were performed in a sequential way, where the accelerograms were inputted according to increasing intensities, in order to better reproduce the experimental tests; when a given seismic action is considered the structure has already been modified due to the non-linear effects induced by the previous one.

Figure 3 illustrates the comparison of the pier P3 top displacement responses obtained numerically (using the Damage Model) and experimentally (pseudo-dynamic test performed at the JRC), for three different intensity level earthquakes. These three responses are in general very close as much in frequency as in maximum values, except for the high level earthquake where some differences can be found on the response last part.

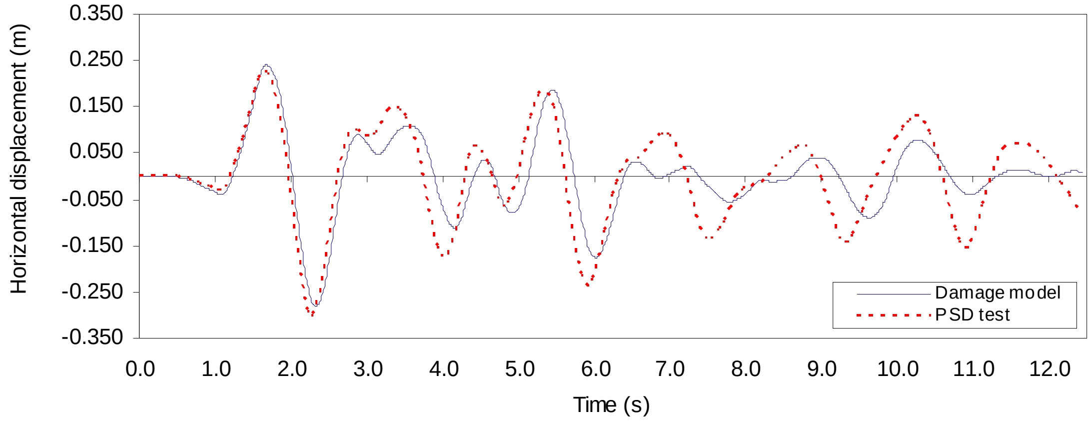
From the comparison of the numerical simulations with the experimental results, the capability from the damage model to provide accurate simulations of the seismic performance of the bridge was brought into evidence, even when the piers are difficult to simulate due to the concrete hollow section geometry and to the unusual reinforcement layout adopted in the design. In fact, the longitudinal reinforcement curtailment represents an important influence on macro-crack localization, which leads to a shift of the plastic hinge up to the elevation where a significant reduction of the longitudinal reinforcement takes place.



a) Low level earthquake



b) Moderate level earthquake



c) High level earthquake

Figure 3: Top horizontal displacement history of pier P3.

4 SYNCHRONOUS NUMERICAL ANALYSIS

In this section, the numerical analyses were carried out assuming synchronous ground motion in the transverse direction, being considered the pier P3 accelerogram as the reference.

Again the numerical analysis were performed in a sequential way, where the accelerograms were inputted according to increasing intensities, therefore; when a given seismic action is considered the structure has already been modified due to the non-linear effects induced by the previous one. The non-linear behavior was considered concentrated in the piers, which are discretized with (i) the Damage model, a refined constitutive model or (ii) the PNL model [3, 8], a plastic hinge type model for the nonlinear material behavior simulation.

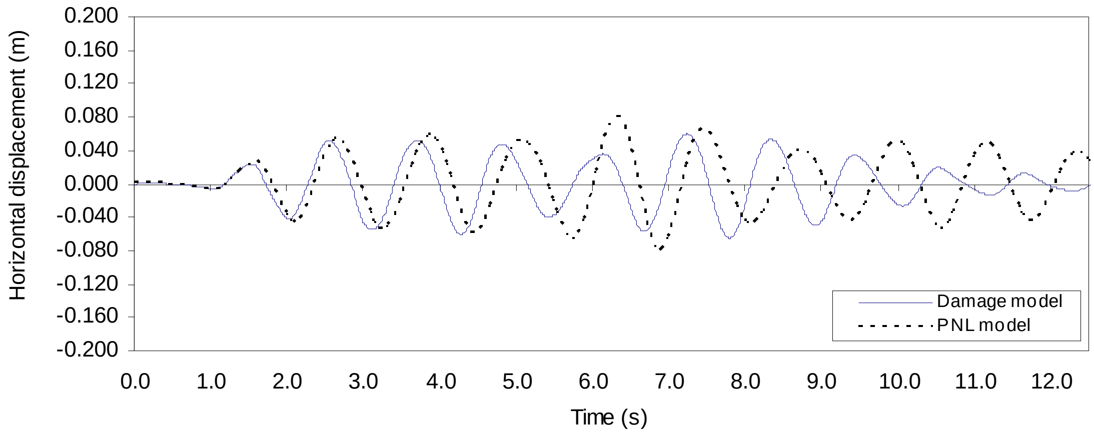
Once the capability from the damage model to provide accurate simulations of the bridge seismic performance was brought into evidence from previous asynchronous responses, it is reasonable to assume that accurate response simulations are expected for synchronous input motion.

Figure 4 illustrates the comparison of the pier P3 top displacement responses obtained numerically, using the Damage Model and the plastic hinge model, for three different intensity level earthquakes. The first response, for low level earthquake, has similar results for both models in the initial seconds, but after half time the frequencies become different and higher maximum displacement values were achieved for the plastic hinge model. For subsequence responses of the numerical simulations, it was difficult to achieve accurate results for the plastic hinge model due to the unusual reinforcement layout adopted in the design, the longitudinal reinforcement curtailment represents an important influence on crack localization and plastic hinge formation, which leads to a shift of the plastic hinge up to the elevation where a significant reduction of the longitudinal reinforcement takes place.

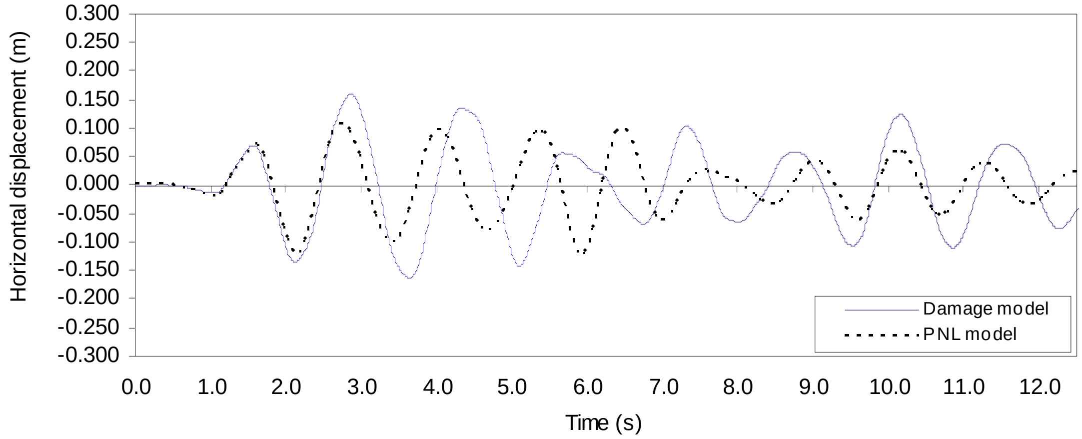
For this assumption of consider the bridge synchronous ground motion as the pier P3 accelerogram (used in asynchronous analysis), the comparison between the asynchronous and synchronous response obtained with the damage model, Figure 3 and Figure 4, allows to verify that maximum peak displacement is reasonable similar, for moderate and high level earthquake. Both responses are similar for initial seconds, however with some differences for final seconds of the accelerograms.

After these first results and conclusions obtained with the plastic hinge modeling, a second set of simulations was carried out assuming the longitudinal reinforcement reduction at the pier base and therefore the corresponding decrease on the yielding moment, but keeping the plastic hinges in the same place (near the foundation), although the referred longitudinal reinforcement curtailment takes place in region 2 (see Figure 2). Figure 5 illustrates the comparison of the pier P3 top displacement responses obtained numerically with the referred plastic hinge model modifications, for three different intensity level earthquakes.

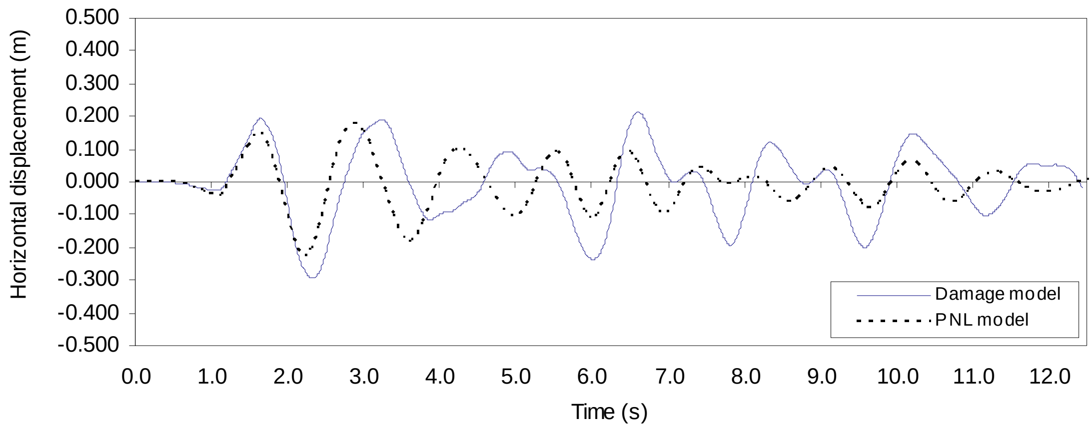
Generically, as is possible to see from Figure 5, the results for the plastic hinge model considering the longitudinal reinforcement reduction improve the quality of the responses, once both damage model and PNL model are now more similar responses. In fact, regarding frequencies and maximum displacement values, both responses are now significantly closer.



a) Low level earthquake

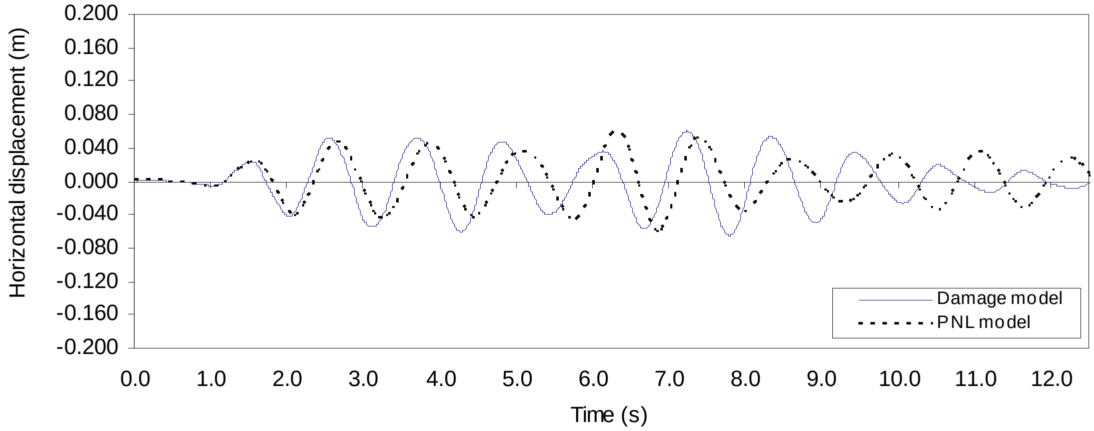


b) Moderate level earthquake

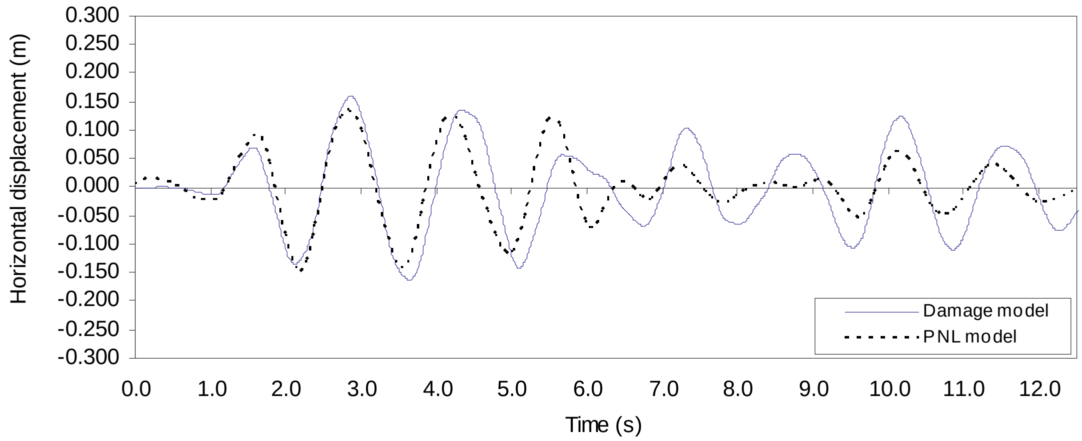


c) High level earthquake

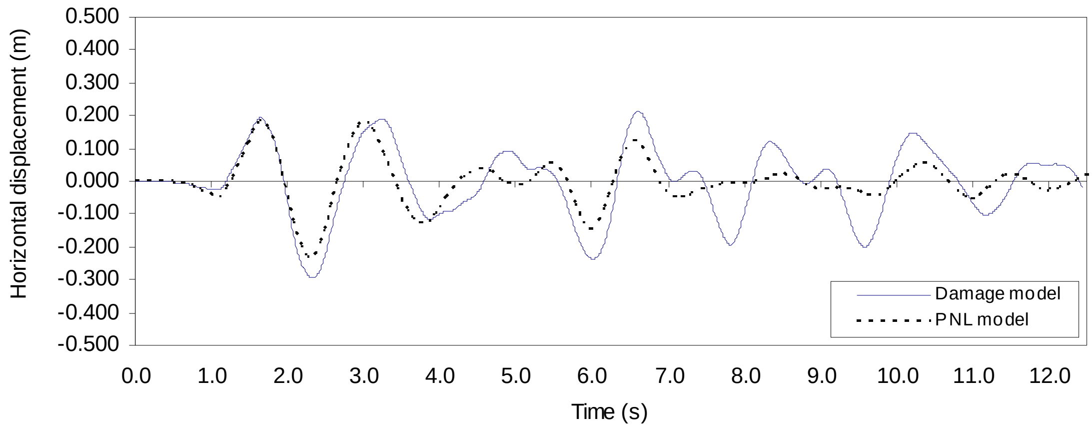
Figure 4: Top horizontal displacement history of pier P3.



a) Low level earthquake



b) Moderate level earthquake



c) High level earthquake

Figure 5: Top horizontal displacement history of pier P3 (with long. reinf. reduction).

5 CONCLUSIONS

- The Talübergang Warth bridge, studied in the VAB project context, was experimental tested under asynchronous pseudo-dynamic conditions at the JRC and the results were compared against numerical simulations carried out with the damage model.
- Displacement responses obtained numerically and experimentally, for three different intensity level earthquakes, are in general very close as much in frequency as in maximum values, being highlighted the capability from the damage model to provide accurate simulations of the seismic performance of the bridge even when the longitudinal reinforcement curtailment represents an important influence on macro-crack localization.
- Once the capability from the damage model to provide accurate simulations of the bridge seismic performance was brought into evidence from asynchronous responses, it is reasonable to assume that accurate response simulations are expected for synchronous input motion.
- The comparison between the asynchronous and synchronous response obtained with the damage model allows to conclude that maximum peak displacement is reasonable similar, for moderate and high level earthquake.
- For the synchronous numerical simulations with damage model and a plastic hinge model, it was difficult to achieve accurate results for the plastic hinge model due to the longitudinal reinforcement curtailment, which leads to a shift of the plastic hinge up to the elevation where a significant reduction of the longitudinal reinforcement takes place.
- The results for the plastic hinge model considering the longitudinal reinforcement reduction improve the quality of the responses, once both damage model and PNL model are now closer responses. Therefore, a significant importance must be given to the simulations with plastic hinge model regarding the aspects and singularities of the structures that are likely to affect the seismic response.

REFERENCES

[1] Faria R, Vila Pouca N, Delgado R., Numerical Models to Predict the Nonlinear Behaviour of Bridge Piers Under Severe Earthquake Loading, Deliverable N. 6, Task 3, Report N. 3/2 (VAB Project), FEUP, Porto, 2001.

[2] Pinto A., Pegon P., Magonette G., Molina J., Buchet P., Tsionis G., Pseudodynamic tests on a large-scale model of an existing RC bridge using non-linear substructuring and asynchronous motion, EUR 20525 EN, EC, Joint Research Centre, ISIS, 2002.

[3] Delgado, P., Costa, A., Delgado, R., A Simple Methodology For Seismic Safety Assessment Of Bridges, *Proceedings of the 12th ECEE*, London, 2002.

[4] Faria, R., Oliver, J., Cervera, M. A strain based plastic viscous damage model for massive concrete structures, *International Journal of Solids and Structures* 35(14), 1533-1558, 1998.

[5] Vila Pouca, N., Faria, R., Delgado, R., Numerical Simulation of the Seismic Behaviour of Talübergang Warth Bridge, *Proceedings of the 12th ECEE*, London, 2002.

[6] Vila Pouca, N. Simulação Numérica da Resposta Sísmica de Elementos Laminares em Betão Armado, *Ph.D. thesis*, FEUP, 2001 (in Portuguese), (http://ncrep.fe.up.pt/web/artigos/NVilaPouca_PhD_Thesis.pdf).

[7] Giuffrè, A., Pinto, P. Il comportamento del cemento armato per sollecitazione ciclice di forte intensitá, *Giornale del Genio Civile,* 1970.

[8] Delgado, P., Costa, A., Pinho, R., Delgado, R. Different Strategies for Seismic Assessment of Bridges – Comparative studies, *Proceedings of the 13th World Conference on Earthquake Engineering* – 13WCEE, Vancouver, Canada, 2004.