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RISK ASSESSEMENT OF AN EXISTING BRIDGE TAKING ACOUNT SOIL-STRUCTURE INTERACTION (SSI)

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ABSTRACT

A good knowledge of the behavior of structures under seismic loads is very important to assess structure's risk. This paper aims at providing a reliable simulation of seismic excitations and good reproduction of the behavior of structures.

Transient dynamic analysis is the most representative method of the complexity of civil engineering structures. This method consists in establishing a finite element model of a structure associated with nonlinear models of materials.

Numerical model of the an existing viaduct based on the Timoshenko multifibre beam elements and non-linear constitutive laws of concrete and steel compared with the experimental results show the good performance of the approach

In addition to the finite element model and the good knowledge of the dynamic's behavior of the materials, it will be necessary to quantify the influence of Sol-Structure Interaction (ISS) phenomena on the behavior of structures.

Several approaches exist to consider this phenomenon; we choose the "macro-element" concept for his ability to simulate the 3D behaviour of a rigid shallow foundation under static and dynamic loadings.

Finally, we establish the fragility curves, by monte-carlo method, of some types of concrete structures. Then quantify the degree of hypothetical damage that these structures could undergo at different levels of seismicity. However, we must ensure the compatibility of the real or artificial seismic record used with the geological profiles of the foundation ground. For that reason, we use a procedure for generation of spectrum-compatible time histories from real seismic groud motion records

Keywords: Fragility curves; Sol-Structure Interaction; non-linear constitutive laws; macro-element; spectrum-compatible.

1. INTRODUCTION

In recent years, different regions over the world: Mexico, Italy, China, Algeria… have been hit by devastating earthquake. Precisely, earthquakes appear to be one of the most harmful natural disaster. Indeed, the risk of humans and economics damages is steadily increasing on highly urbanized regions.
Unfortunately, there is no reliable way to predict where, when and with what intensity an earthquake will occur. Therefore, it seems obvious that is very important to define what the seismic risk depends on.

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Besides the fact the sociological aspect and the ability to decrease the seismic risk by the simple managing behavior of people during the earthquake; we can define the seismic risk as result of the combination of hazard and vulnerability.

So on one side; we require a good knowledge of hazard to fully understand the phenomenon. For that, it is necessary to develop technical tools that can simulate seismic action compatible with seismicity of local region and compatible with the geological profile. Moreover, on the other side, to estimate correctly the vulnerability of the buildings, we need to understand the dynamic behavior of buildings. The numerical modelling of civil engineering structures under seismic loading seems to be quite complicated.

The Non Linear Time History Analysis is considered as the most rigorous method for estimating the inelastic seismic demands of structures as shown below in Figure 1 (FEMA 440. 2004). Although transient analysis is very complicated and time-consuming, the fragility curves obtained using this procedure have better reliability.

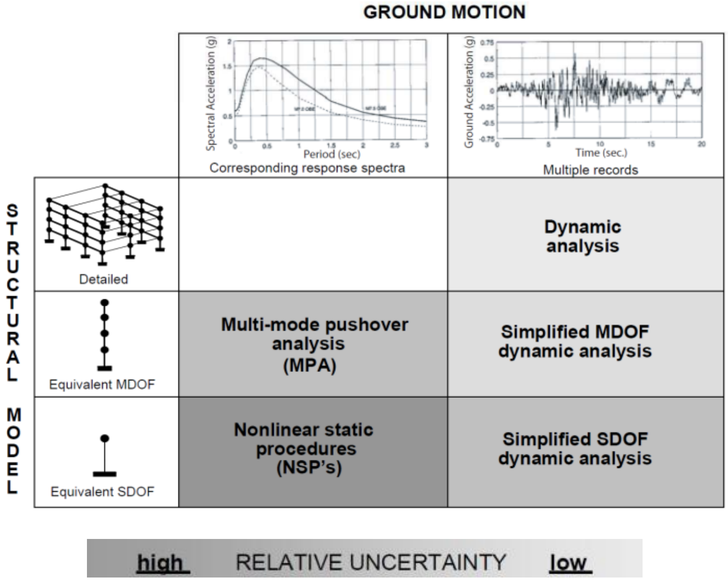

Figure 1. Nonlinear transient dynamic analysis FEMA 440 (2004).

This Non Linear Time History Analysis method, as in Figure 2, consists on establishing a finite element model of a structure associated with nonlinear models of materials. The seismic loading in this case will be represented by its temporal form expressed in terms of either displacements or acceleration time histories.

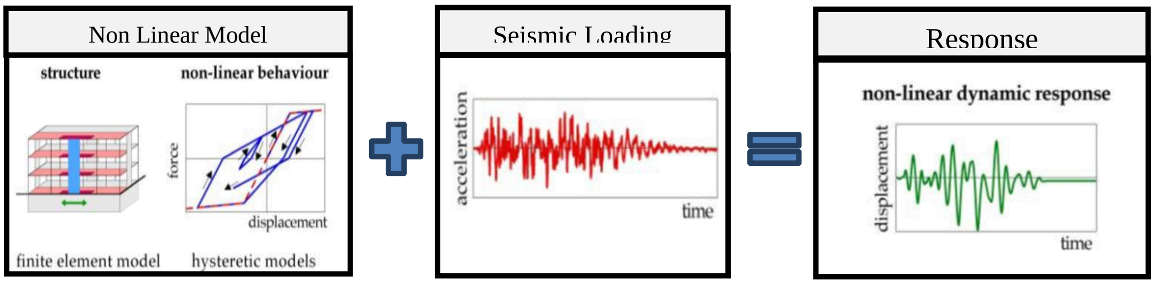

Figure 2. Nonlinear transient dynamic analysis.

In addition to the finite element model and the good knowledge of the dynamic's behavior of the materials, the boundary conditions, of the represented model, need also to be well known.

Widely known, it is necessary to quantify the influence of Sol-Structure Interaction (SSI) phenomena on the behavior of structures. Several approaches exist to consider this phenomenon. In the present

work, we choose the "macro-element" concept for its ability to simulate the 3D behavior of rigid shallow foundation under static and dynamic loadings (Grange et al. 2011).
Finally, most of the failure probability assessment methods used to determine the fragility curves are based on the assumption that the fragility curves take a lognormal distribution form. At least, lognormal law need only two parameters mean and standard deviation to identify those curves. A limit state is a criterion defined as the value of the structural demand that a system is unable to achieve at a specified level: damage or failure. The state of damage or failure can be specified through limits on any response parameter such as stresses, deformations, displacements, accelerations. Depending on the type of structure being studied, parameters corresponding to the following three ways can be used to characterize damage states.

2. THE PROPOSED APPROACH

Numerical model of a 1:2.5 scaled viaduct, as shown in Figure 3, which was tested pseudo-dynamically in ELSA (JRC Ispra), is performed with FEDEASLab (Filip Filippou FC and Constantinides M., 2004). Numerical and experimental results will be compared to show the good performance of the proposed approach.

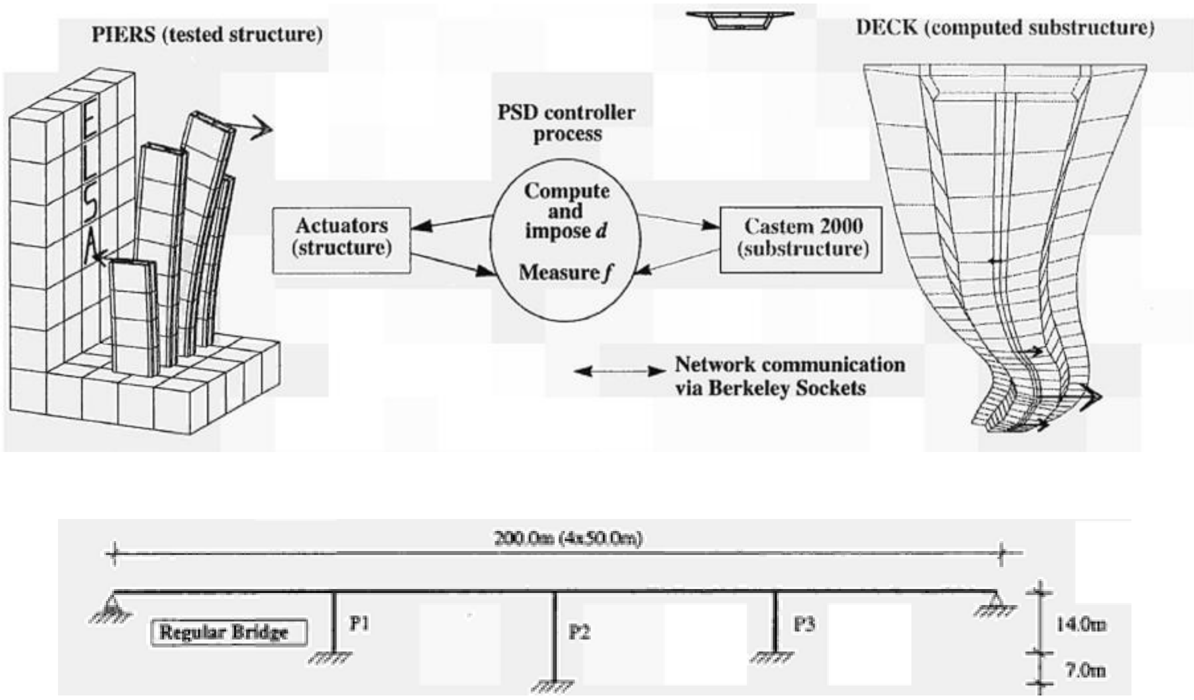

Figure 3. Pseudo-dynamic testing of a 1:2.5 Scaled R/C bridge: plan view of the tested R/C bridge ELSA, Ispra, Italy (Pinto et al 1996).

Figures 2 illustrate shape sections of the piers and the deck. The three piers of the bridge have a hollow rectangular section shape and they are made of reinforced concrete. Whereas the deck present a hollow ‘voussoirs’ shape and it’s made of prestressed concrete. So, we can consider that the deck has a linear elastic behavior and can be simulated with the element finite code Cast3M. The interaction between it and the piers will be integrated numerically during the test. Thus, only the piers of the bridge will be really tested.

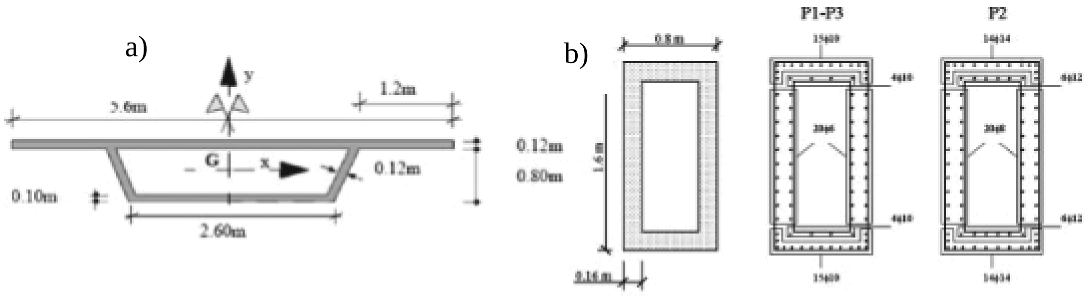

Figure 4. Large bridge: (a) scheme of the deck and (b) scheme of the piers (scale 1:2.5).(Grange et al 2011).

2.1 Finite Element Model

A lumped mass model, as illustrated in Figure5, combined with a finite element (FE) model based on a multifibre modelling strategy is considered to reproduce the behaviour of the viaduct. The details of the rotational masses and inertias used for the modelling are given in Table I.

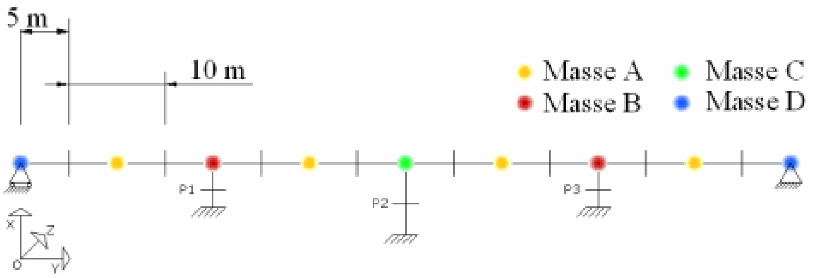

Figure 5. Model multifibre des piles. (Grange et al 2011).

Table 1. Masses and rotational inertia.

	Mass (Kg)	**Rotational inertia I_x (Kg.m²)**	**Rotational inertia I_z (Kg.m²)**
M_A	27.5	285	234
M_B	32	287	271
M_C	34	288	322
M_D	13.75	143	117

Piers are modelled with non-linear Timoshenko multifibre beam elements (Kotronis P and Mazars 2005); (Mazars et al 2006). Six elements are used for the piers P1 and P3 and nine elements for the pier P2. Mesh is refined at the base of the piers where damage is expected to be concentrated as shown in Figure 6. Details of the fibres used into the section for the piers P1–P3and P2 are given in Figure 6. The deck being made of prestressed concrete, its behaviour is assumed linear elastic and it is discretized using linear beam elements.

Each section of the piers is modelled with fibres. We use forty fibres for concrete while eighty steel fibres are used for the eighty reinforcement bars at their actual position.

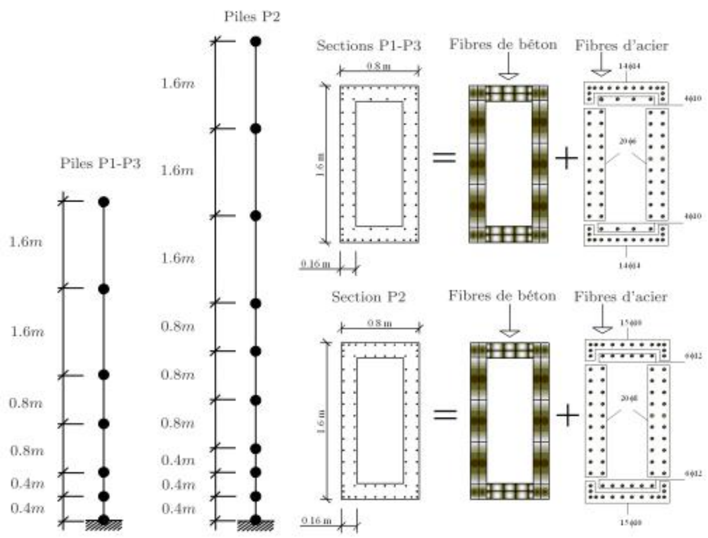

Figure 6. Piles meshing. (Grange et al 2011).

2.2 Nonlinear behaviour laws

Figure 7 a,b, include constitutive model for both concrete and steelThe uniaxial concrete damage law developed by. La-Borderie model (1991) is used to reproduce the behavior of concrete while the law of Menegoto and Pinto (1973) is used to reproduce the behavior of steel. Table 2 summarize the different material parameters for the both models.

Table 2. Material parameters details for concrete and steel Constitutive laws.

	Concrete Parameters		**Steel Parameters**
E	29.4 GPa	E	200 GPa
ν	0.175	f_y	450 Mpa
Y_{01}	1000 Pa	f_{su}	710 Mpa
Y_{02}	0.0001 MPa	ε_{sh}	0.0060
A_1	7000 MPa^{-1}	ε_{su}	0.10
A_2	6.0 MPa^{-1}		
B_1	1.0		
B_2	1.3		
β_1	0.5 MPa		
B_2	-19 MPa		
σ_f	3.0 MPa		

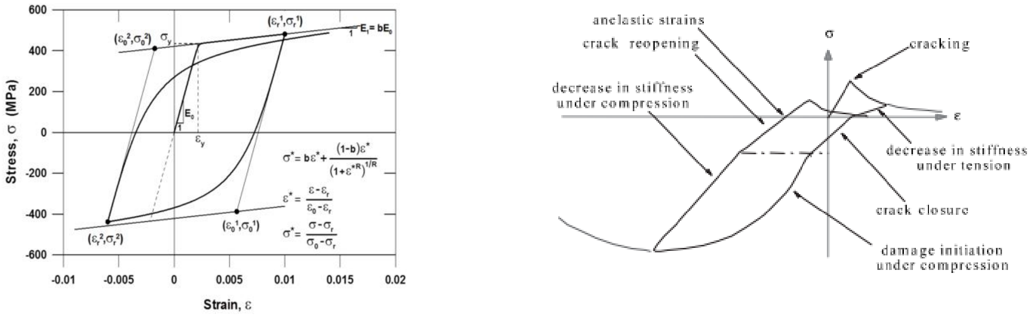

Figure 7. Nonlinear models of materials : a. Cylical behaviour of steel Menegoto and Pinto, 1973. b. Cyclical model of concrete La Borderie, 1991.

2.3 Seismic loading

For the transient nonlinear analyses, we need ground motion time histories. Therefore, we have use the same seismic excitation used during the pseudo-dynamic test. The latter, represented below in figure 8, is simply a synthetic accelerogram compatible with the 5% damping design spectrum of Eurocode 8 for B soil type.

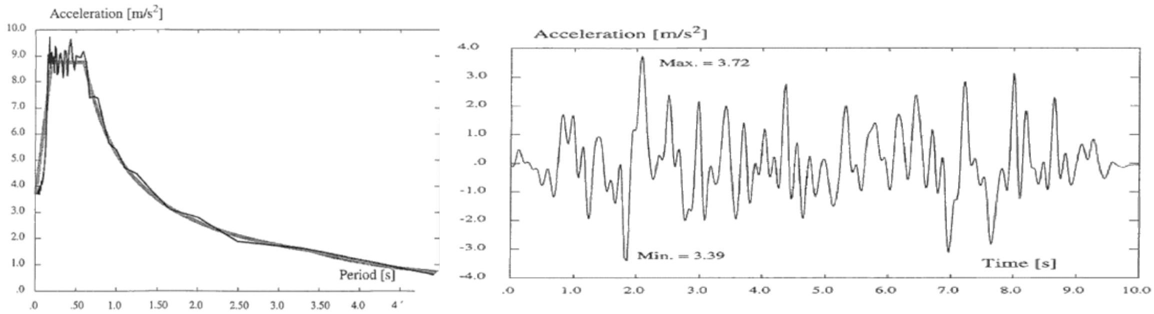

Figure 8. Synthetic accelerogram compatible with the 5% damping regulatory spectrum of Eurocode 8 for B class soil. Pinto et al, 1996.

In order to respect the similitude laws this accelerogram will be modified. So the ground motion acceleration will be amplified of order 2.5 and the time scale divided by 2.5. At the end, we have an accelerogram represented in Figure 9.

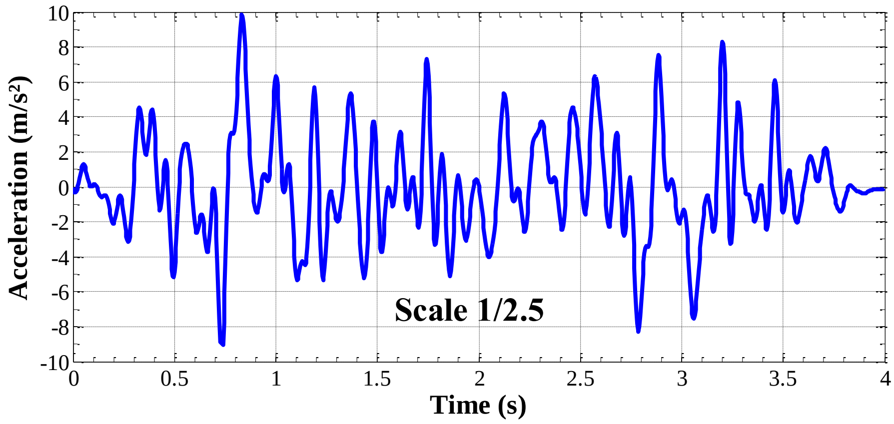

Figure 9. Modified Synthetic accelerogram.

3. MODEL VALIDATION

As shown in Figure 10, the numerical model with multifiber elements seems to reproduce correctly the behavior of the structure for the three piers. The displacement peaks are correctly reproduced, as well as the frequency content of the response.

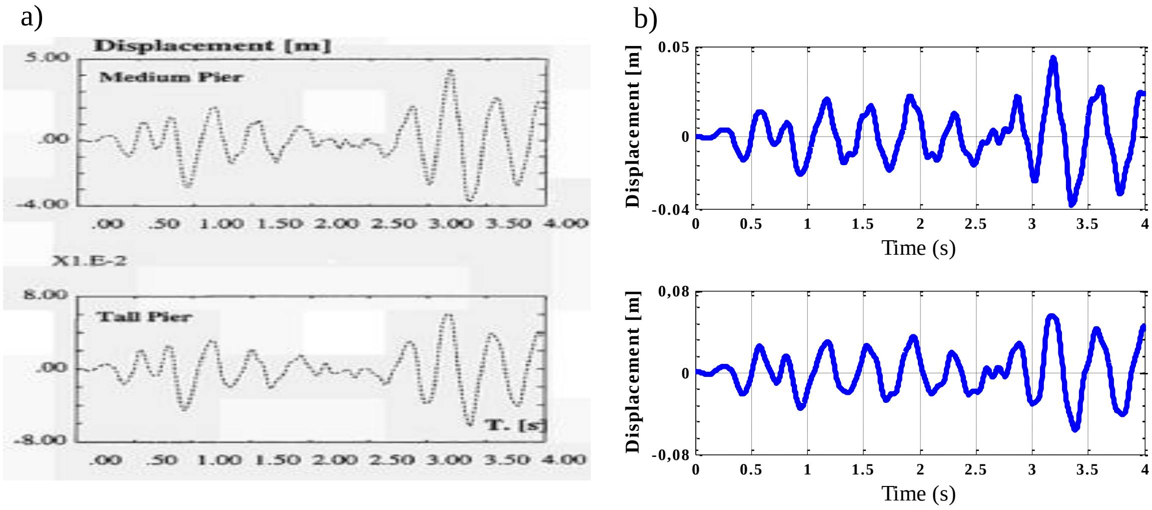

Figure 10. Displacement at the top of piers. a) Experimental displacement from pseudo dynamic test Pinto et al, 1996; b) numerical displacement.

4. SOIL STRUCTURE INTERACTION INFLUENCE

In this part, we will try to quantify the influence of the soil structure interaction on the nonlinear response of the pile of the viaduct model. To this end, we will use the same numerical model previously validated in this article but the bases of the viaduct piles are equipped now with macroelements, and the foundation soil is considered as a C soil type.
The macro-elements as previously demonstrated by Grange & al 2011 seems to be a very good tool for modelling the soil structure interaction for their ability to reproduce geometric and material nonlinearity, the ability to replicate the repayment of the foundation and its great ability to reduce the calculation times.

4.1 Eurocode response spectrum compatibility

The numerical model has to be done for a C soil type. So, the use of the previous accelerations used becomes more possible. In order not to involve other speakers who can change or influence the nonlinear dynamic response, and to keep the same frequency content of the seismic signal, we will to artificially generate a second accelerogram.

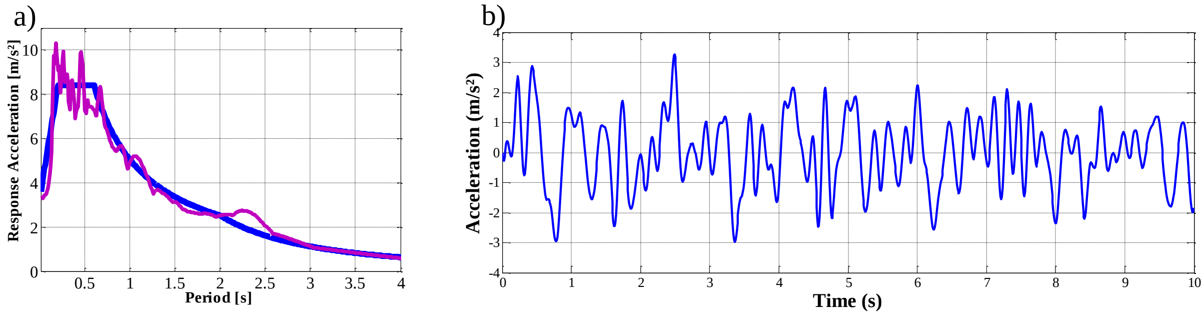

Figure 11. (a) C soil class design spectra coming from Eurocode 8 for a 5% damping and synthetical accelerogram spectra; (b) synthetical accelerogram spectra

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The conditional generation approach of seismic signals compatible with a target response spectrum of Rachedi et al 2015 is used to generate the second accelerogram. This approach based on the Deodatis G (1996) and Benmansour N (2013) approaches makes it possible to preserve the frequency content of the original signal while guaranteeing PGAs compatible with the soil profile. The generated signal is represented in Figure 11 with a comparison of its response spectrum with that one of Eurocode (type C). Otherwise, on Figure. 12, a comparison is made between the artificial accelerograms for the two types of ground. The PGA for the new artificial seismic signal is 0.30g a little smaller than the original one, which was 0.35g.

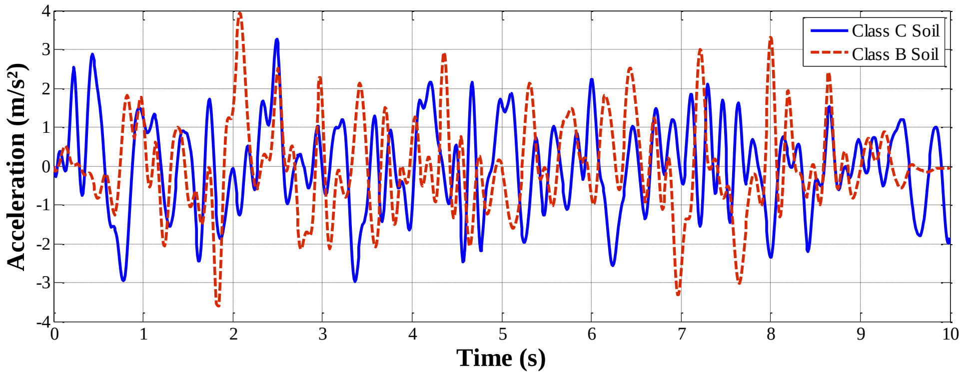

Figure 12. Comparison between synthetic accelerograms compatible with B and C soil type design response spectrum of the EUROCODE.

4.2 Numerical results considering SSI

Despite the fact that the intensity of the seismic excitation applied now at the base of the piles is little smaller than that applied for the first model with a fixed base. It is clearly shown in figure 13, that the displacement at the top of the piles obtained when taking into account the SSI are more important to those of the model with fixed base.

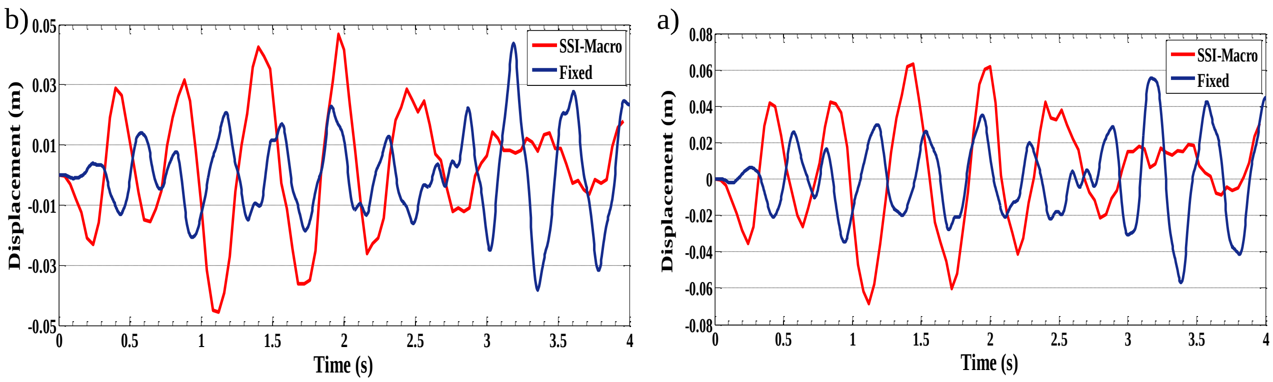

Figure 13. SSI effect: comparison of the response displacements at the top of piers. a) Displacement of piers P1 and P3 , b) Displacements od central piers P2.

The non-linear dynamic behavior of the viaduct is completely modified when taking into account the interaction of the foundations of the piles with the ground. This influence of the SSI must be taken into account when estimating the risk as it amplifies the displacements and internal forces of the bridge piers.

5. FRAGILITY CURVES

As mentioned in the introduction, the objective of the present paper is to establish fragility curves of the studied bridge. In the literature we can find several methods that describe how to obtain the fragility curves, (a good revue is given by C.T Dang (2014)). These curves could be established by using actual structural damage obtained empirically by F. Yamazaki et al. (1999) M. Shinozuka et al. (2000). Others methods are based on expert opinion as mentioned in ATC 1985 and the guide proposed by D. Wakefield et al (2003). Otherwise, we can use a numerical model results to obtain analytical fragility curves. These analytic fragility curves are constructed from statistical derivation of simulated damage distributions using structural models and increasing earthquake intensity.
The Monte Carlo simulation method is considered as the most efficient analytic method to establish "exact" fragility curves but a large number of simulations is required. The following equation (Equation 1) is used to define clearly this method. In this case, the seismic fragility of a structure is defined as the conditional probability of failure for a corresponding intensity of ground motion. The evaluation of the probability of failure for each level of intensity of the ground motion allows obtaining, at the end of the process, all the points of the fragility curve Fr (a).

$$F_r(a) = P[G(Z) \leq 0 | A = a] \cong \frac{\sum 1[G(Z) \leq 0 | A = a]}{N_{S_a}} \quad (1)$$

The indicator function 1 […] used in the last equation is a function which is equal to 1 if there is failure, otherwise 0. So, $\sum 1[G(Z) \leq 0 | A = a]$ is actually the number of intensity earthquakes causing failure. Finally the number Nsa is the total number of ground motion used with the same seismic intensity PGA equal to “a”. Consequently, more we use a large number of the groud motion more we improve the quality of the approximation done by the Monte Carlo simulation method.

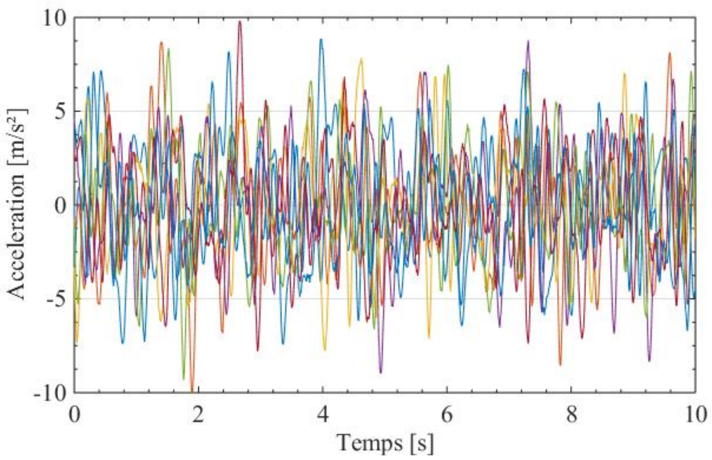

Figure 14. Compatible EUROCODE (Type C of soil) design response spectrum synthetic accelerograms

In order to establish fragility curves of the viaduct ELSA presented above, a set of transient nonlinear dynamic analyses using the multifibre model associated with the macro element were performed. The seismic loading is expressed by a set of EUROCODE C response spectrum compatible synthetic time histories as shown in Figure 14.
At the end of each nonlinear analysis, we estimate the damage index of the most solicited section of the viaduct piers (linked in particular to a damage state), which is permitted by the use of multifibre model Figure 15. Statistical analysis evidence of damage leads to the evaluation of the probabilities of the different states of damage and thus fragility were established.

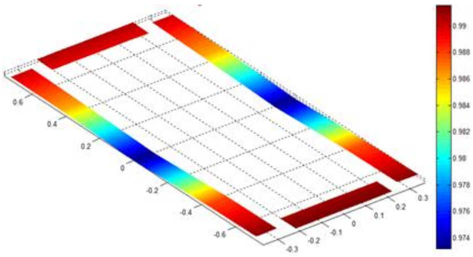

Figure 15. Damage index for bottom section of piers P2.

The lognormal cumulative law model, defined below by the equation 2, is the statistical model most frequently used to represent the fragility curves of a construction.
In this model, the probability of exceeding a state of damage or failure is defined by two parameters of the cumulative probability function (Am, β):

$$F_r(a) = \Phi\left[\frac{\ln(a/A_m}{\beta}\right] \qquad (2)$$

Where Φ is the probability distribution function of the reduced normal centred law, Am is the median and β is the standard deviation of the natural logarithm of the seismic intensity A.

The fragility curves established for our studied case, viaduct of ELSA (Ispra, Italy), for various damage states according to Table 3, are detailed in Figure 16.

Table 3. Structural Damage Category Definition.

Damage Degrees	**Damage States**	**Damage Index**
0	None	DI<0,14
1	Light	0,14<DI<0,40
2	Moderate	0,4<DI<0,6
3	Extensive	0,6<DI<1
4	Collapse	DI=1

We can clearly notice that there is a large probability to have a collapse on bottom piers, slightly greater than 80%, for strong groud motion intensity (PGA>0,9g), whereas there is low probability of collapse from moderate ones. Otherwise, the piers are lightly damaged when the seismic loading PGA exceeds the 0,5g.

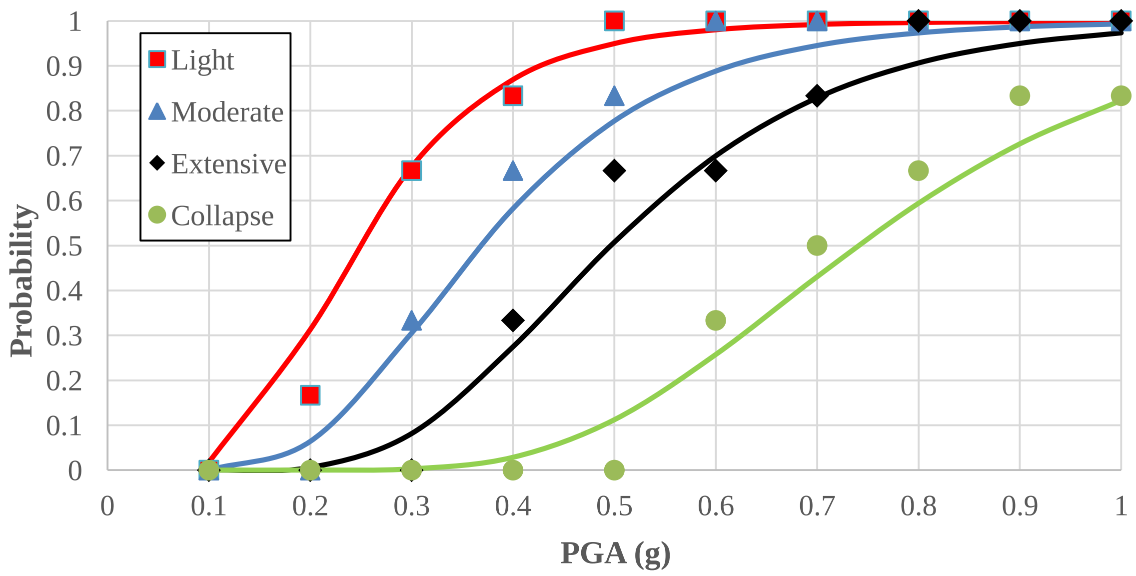

Figure 16. Fragility curves of Ispra viaduct.

6. CONCLUSION

In this work, we evaluate the piers's seismic risk of a viaduct pseudo-dynamically tested in ELSA (JRC Ispra) considering SSI. The reason why we established fragility curves based on analytical approach.

These analytic fragility curves are constructed from statistical derivation of simulated damage distributions using structural models and increasing earthquake intensity.

Firstly, the structural part of the numerical model, with fixed piers at the base, employing the Timoshenko multifibre beam elements with complex non-linear constitutive laws, is validated by comparing the numerical response with the experimental results obtained from the pseudo-dynamic test on a viaduct at the ELSA laboratory. Furthermore, the effects of SSI interaction is introduced by modelling the foundation–soil system with macroelement developed by Grange et al 2011 at the base of piers. In this case, a soil class C is considered to quantify the effect of SSI. So, we proceed to the modification of the original synthetic seismic excitation used in the pseudo-dynamic experimental test, in order to be compatible with EUROCODE design response spectrum for soil class "C". Despite the fact that the seismic intensity of the current synthetic signal is less than the original one, the predicted displacements at the top of the structure are strongly amplified compared to the fixed model.

Finally, based on the multifibre model we choose the piers section's damage index as a structural limit state (failure index). After, the lognormal cumulative law model is chosen to represent de distribution of the result of Monte Carlo approach. Fragility Curves established, we notice that a strong probability of collapse of piers with strong seismic excitation occurs. At least, the piers are certainly lightly to highly extensive damaged when they are excited with moderate strong motion.

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