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THE EFFECT OF SOIL-STRUCTURE INTERACTION ON A REINFORCED CONCRETE BRIDGE

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ABSTRACT: *This paper presents a numerical strategy to simulate the behaviour of a three-piles viaduct made of prestressed concrete. The viaduct was tested pseudodynamically in ELSA laboratory (JRC Ispra, Italy). During the experimental program, only the three piles were tested, whereas the behaviour of the deck was simulated using the finite element method. The first part of the paper presents a numerical model of the viaduct based on Timoshenko multifiber beam elements and non linear constitutive laws. Comparisons with the experimental results show the good performance of the approach. In the second part, a parametric study is carried out showing the influence of Soil-Structure Interaction (SSI). Various types of soils are considered using a recently developed macro-element representing a rigid shallow foundation. The macro-element is suitable for dynamic (seismic) loadings and it takes into account the plasticity of the soil, the uplift of the foundation, $P - \theta$ effects and the radiative damping. Finally, the numerical results are compared with the ones coming from a classical engineering approach using linear elastic springs at the base of the piles.*

1 INTRODUCTION

In engineering, boundary conditions have to be correctly reproduced in order to simulate numerically the behaviour of a structure. Soil-Structure Interaction (SSI) can not be neglected. This is particularly true for slender structures like tall buildings or bridge piers. Their behaviour is different whether the structure is embedded in a solid rock or on a soft soil.

This paper presents a simplified numerical strategy considering SSI to model a three-piles viaduct tested in ELSA (JRC Ispra, Italy). It is divided into two parts:

- Part I: The viaduct is considered embedded in the soil. It is simulated using multifiber Timoshenko beams and non linear constitutive laws based on damage mechanics and plasticity. Comparison with experimental results shows the performance of the approach.
- Part II: The influence of SSI is studied using a recently developed macro-element taking into account the plasticity of the soil, the uplift of the foundation, $P - \theta$ effects and the

radiative damping (Grange 2008), (Grange, Kotronis, & Mazars 2008a), (Grange, Kotronis, & Mazars 2008b). The macro-element approach consists in condensing all non-linearities into a finite domain and works with generalized variables (forces and displacements) at the centre of the foundation. This concept allows considerably decreasing the necessary degrees of freedom of the numerical model and the computational demand. The results of a classical engineering approach are finally provided, using linear elastic springs at the base of the piers. A comparison of the two approaches shows clearly the advantages of the new macro-element tool.

2 PART I: MODELING OF THE PSEUDO-DYNAMIC TEST

A partial $\frac{1}{2.5}$ scaled viaduct was recently studied in ELSA (JRC Ispra) (figure 1).

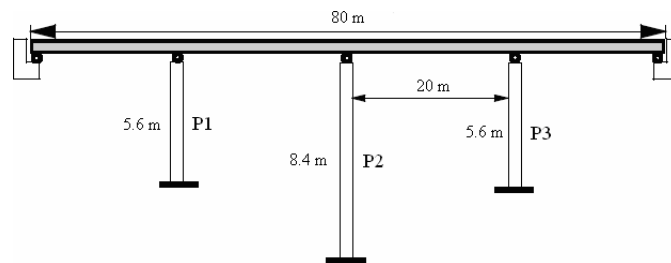


Fig. 1. Viaduct: plan view of the tested bridge in Ispra at scale $\frac{1}{2.5}$.

The viaduct was tested pseudo-dynamically. This kind of hybrid experiment allows testing only a part of the structure, while the rest is simulated with a finite element code. Thus, only the three piles embedded into the soil were tested the desk being simulated with the element finite code Cast3M (figure 2). During the test, the interaction between the piles and the desk was calculated in real time thanks to a numerical integration of the dynamic equations. Inertial forces were calculated and imposed to the specimen by applying the adequate displacements.

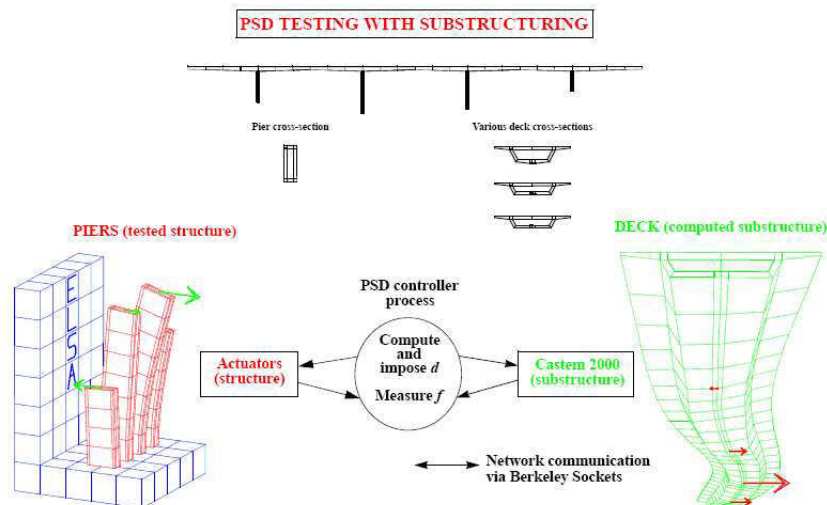


Fig. 2. Viaduct: Pseudo-dynamic scheme.

Piles were made of reinforced concrete with a hollow rectangular section shape. The desk was composed of hollow voussoirs made of prestressed concrete. Details of the geometrical characteristics of the section are given in table 1.

Table 1. Viaduct: geometrical characteristics of the section of the desk.

$A(m^2)$	$I_x(m^4)$	$I_y(m^4)$	$J(m^4)$	$E_b(Pa)$
1.11	0.13	2.26	2.39	25.10^9

2.1 Finite element mesh

A model using multifiber beams and concentrated masses is chosen to reproduce the structure (figure 3). The mass and rotational inertia details are given in table 2.

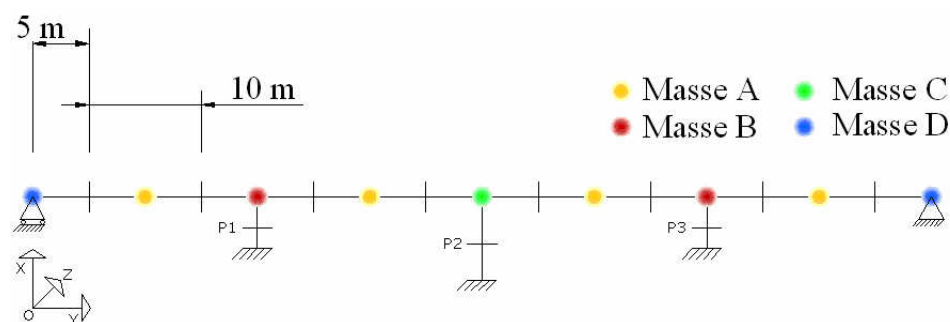


Fig. 3. Viaduct: model using multifiber beam elements and concentrated masses.

Table 2. Viaduct: masses and rotational inertias.

n	Mass M (kg)	Rotational inertia I_x (kg.m ²)	Rotational inertia I_z (kg.m ²)
A	27.5	285	234
B	32	287	271
C	34	288	322
D	13.75	143	117

Non-linear Timoshenko multifiber beam elements are used to reproduce the behaviour of the piles (Kotronis 2000), (Kotronis & Mazars 2005) and (Mazars, Kotronis, Ragueneau, & Casaux 2006). Six elements are used for the piles P1 and P3 and nine elements for the pile P2. Mesh is refined at the base of the piles where damage tends to be important. The desk being from prestressed concrete, its behaviour is assumed linear and it is discretised using linear beam elements. 40 concrete fibers and 80 steel fibers (representing the reinforcement bars at their actual position) are used in each section. Details of the fibers are provided in (Botrugno 2008).

2.2 Material parameters

The uniaxial version of the damage constitutive law (LaBorderie 1991) is used to reproduce the behaviour of concrete and the (Menegotto & Pinto 1973) law for steel. The parameters of

the damage constitutive law are fitted according to experimental concrete sample tests results. Details of the material parameters are provided in table 3.

Table 3. Viaduct: material data used for concrete and steel constitutive laws.

Concrete parameters		Steel parameters	
E	29.4GPa	E	200GPa
ν	0.175	f_y	450MPa
Y_{01}	1000Pa	f_{su}	710MPa
Y_{02}	0.0001MPa	ϵ_{sh}	0.0060
A_1	7000MPa ⁻¹	ϵ_{su}	0.10
A_2	6.0MPa ⁻¹		
B_1	1.0		
B_2	1.3		
β_1	0.5MPa		
β_2	-19MPa		
σ_f	3.0MPa		

Note: The test being pseudo-dynamic, the Rayleigh damping coefficient used in the numerical model has to be small. Its value has been chosen equal to 0.5%.

2.3 Experimental versus numerical results: modal analysis

Natural frequencies for the embedded structure are given in table 4. Modal deformed shapes are given in figure 4.

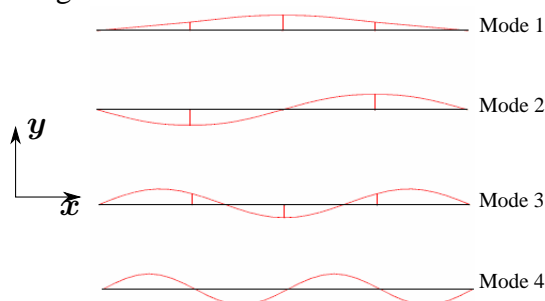


Fig. 4. Viaduct: deformed modal shapes for the modes 1 to 4 of the bridge.

mode	frequency (Hz)	
	Test	Simulation
1	4.2	4.2
2	6.6	6.8
3	9.8	10.4
4	16.1	16.8

Table 4. Viaduct: comparisons between the experimental and the numerical modal frequencies.

The numerical model is able to provide accurately the first four natural frequencies of the bridge. It is thus obvious that it reproduces correctly the masses, the stiffness and the boundary conditions of the experiment.

2.4 Accelerations imposed to the structure

The accelerations imposed to the structure are coming from a synthetic accelerogram with a 5% spectral response fitted according to Eurocode 8 for a soil of class B (EC8 2005). The figure 5(a) shows the comparison between the synthetic spectral response and the design spectra of the Eurocode 8.

The synthetic accelerogram is presented in figure 5(b). The peak of accelerations is situated at 0.35g (“low” earthquake). A second similar accelerogram (dilated, non represented here), is also imposed at the base of the structure. Its peak of acceleration is equal to 0.7g (“strong” earthquake).

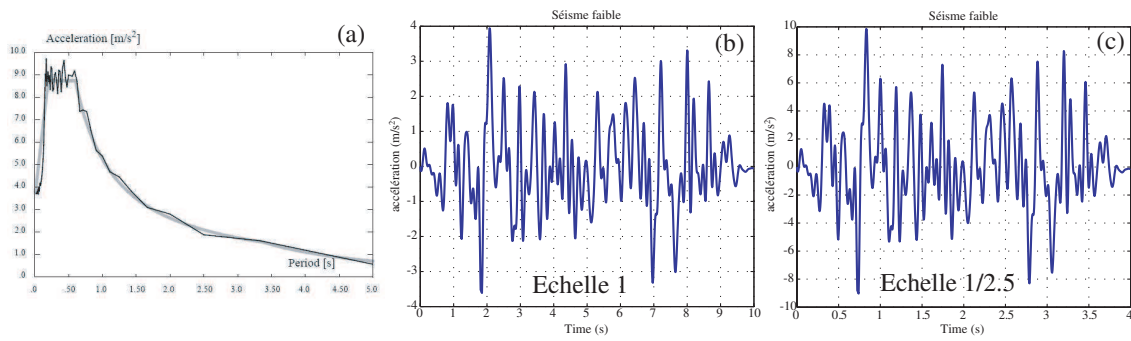


Fig. 5. Viaduct: (a) Design spectra coming from the Eurocode 8 for a 5% damping and synthetical accelerogram spectra, (b) low level earthquake signal for the scale 1 structure and (c) for the scale $\frac{1}{2.5}$ structure.

Note: The specimen being at $\frac{1}{2.5}$ scale, accelerograms have to be modified in order to respect the similitude laws. Accelerations are thus multiplied by 2.5 and time is divided by 2.5. The low earthquake ($0.35g$ at scale 1) is provided in figure 5(c) at scale $\frac{1}{2.5}$.

2.5 Experimental versus numerical results: dynamic analysis

Experimental and numerical results of the dynamic analysis for the embedded structure are presented in figures 6 and 7. The two earthquakes are considered. These figures show the evolution of the shear forces at the base and the lateral displacements at the top of the piles P1, P2 and P3 according to time.

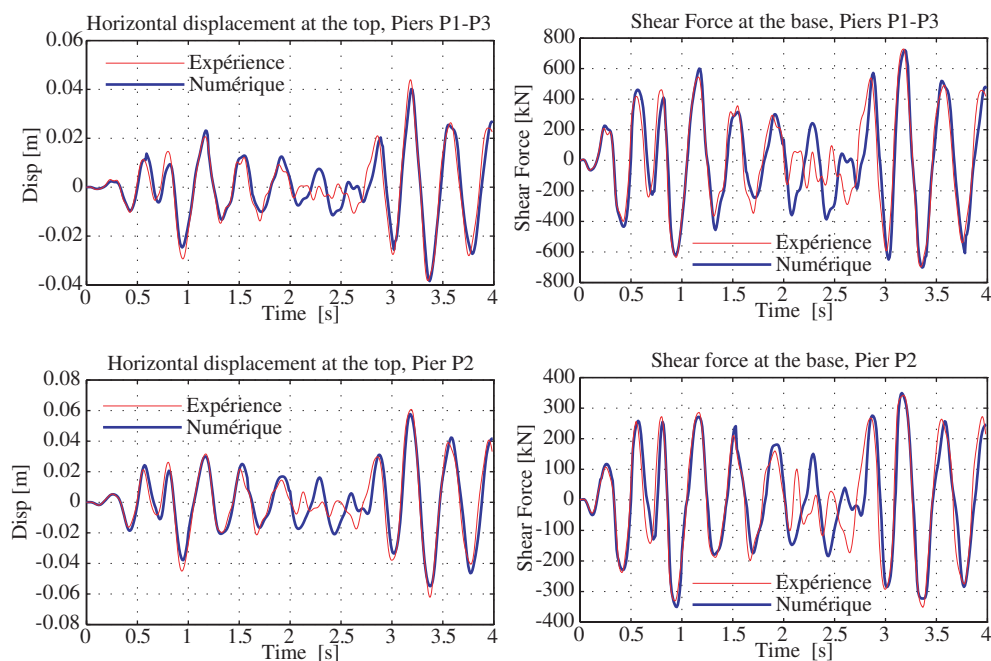


Fig. 6. Viaduct: comparison between the experimental and numerical displacements and shear forces for the low earthquake.

The non linear behaviour of the bridge is correctly reproduced. Not only the the peaks are well simulated in the two directions but also the frequency contents of the response.

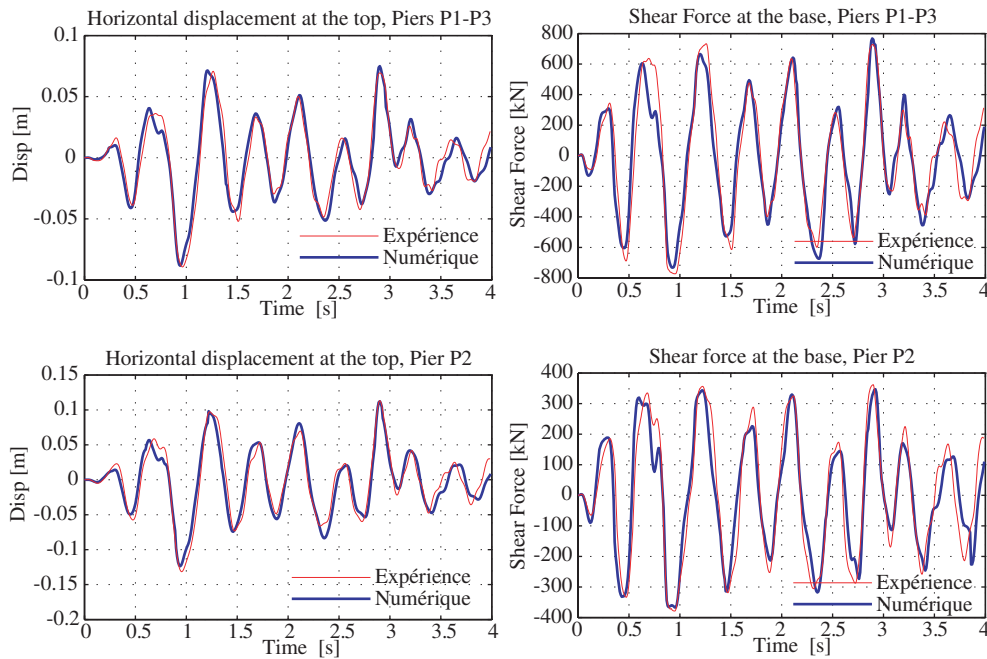


Fig. 7. Viaduct: comparison between the experimental and numerical displacements and shear forces for the strong earthquake.

3 PART II: INFLUENCE OF SOIL-STRUCTURE INTERACTION (SSI)

In this second part of the article, two modelling strategies are proposed to take into account SSI. The first uses a recently developed macro-element considering material and geometrical non linearities. The second is based on linear elastic springs at the base of each pile. Parametric studies taking into account two types of soils and comparisons between the different modelling strategies are detailed hereafter.

3.1 Types of soils

The first soil belongs to class B, according to the Eurocode 8 classification (EC8 2005). A rectangular shallow foundation is considered at each pile with the following dimensions: $L_x = 4.2m$ and $L_y = 2.1m$. The second one is a class C soil. In order to amplify the influence of SSI, the foundations are also taken smaller ($L_x = 3.2m$ et $L_y = 1.6m$). The characteristics of both soils are given in the table 5.

3.2 Calibration of the linear springs

For the second model and in order to reproduce the behaviour of the soils, the stiffness of the linear springs have to be calibrated. The following energy criterion is chosen: the stiffness of the springs is such that they accumulate the same energy as the non-linear SSI macro-element (figure 8).

Note: The dissipated energy produced by the macro-element within the hysteresis loops is not taken into account.

Table 5. Viaduct - SSI: Characteristics of the soils used for the parametric study.

soil	Shear modulus G_0 velocity V_s	cohesion c and friction angle ϕ	Stiffness and damping (Gazetas 1991)	ultimate bearing stress q_{max}
Class C soil	$V_s = 150\text{m/s}$ $G_0 = 45\text{MPa}$	$c_u = 200\text{kPa}$ $\phi_u = 0$	$K_{\theta\theta} = 1112.5\text{MNm/rad}$ $K_{zz} = 298.68\text{MN/m}$ $K_{hh} = 244.36\text{MN/m}$ $C_{\theta\theta} = 4.34\text{MNms/rad}$ $C_{zz} = 1.17\text{MN s/m}$ $C_{hh} = 1.00\text{MN s/m}$	$q_{max} = 1.1\text{MPa}$
Class B soil	$V_s = 360\text{m/s}$ $G_0 = 259.2\text{MPa}$	$c_u = 370\text{kPa}$ $\phi_u = 0$	$K_{\theta\theta} = 144484.1\text{MNm/rad}$ $K_{zz} = 1845.5\text{MN/m}$ $K_{hh} = 2260.2\text{MN/m}$ $C_{\theta\theta} = 23.17\text{MNms/rad}$ $C_{zz} = 3.61\text{MN s/m}$ $C_{hh} = 2.95\text{MN s/m}$	$q_{max} = 2100\text{MPa}$

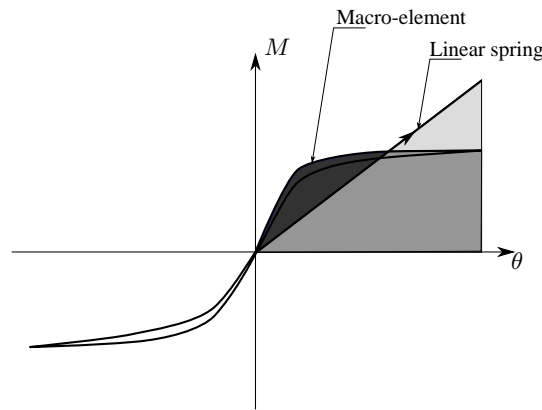


Fig. 8. Viaduct - SSI: calibrating the stiffness of the elastic linear springs.

3.3 Numerical results considering SSI.

The results presented hereafter show the influence of SSI on the viaduct and this for two different types of soils (see table 5). Each time the numerical results are also compared with the behaviour of the bridge considered fixed (embedded in the soil).

SSI - class B soil Only the results for the strong earthquake are presented in this section (figure 9). Three different boundary conditions are considered: embedded (Fixed), with the non-linear macro-element (ME), and with the linear springs (EL). Results are similar in terms of internal forces. Nevertheless, the maximum displacements at the top of the piles are significantly increased (multiplied by 2) for the cases considering SSI (ME and EL).

SSI - class C soil Due to numerical problems, only the results for the low motion are presented in figure 10 for the class C soil. This time, the behaviour of the viaduct is different depending on the boundary conditions. The displacements are strongly amplified (multiplied by 4) in the case of the structure lying on the macro-element and on the linear elastic springs. In terms of internal forces at the base of the piles (moments and shear forces), they are reduced significantly only for the case of the macro-element. They are almost equal for the embedded structure and for the one on the elastic linear springs.

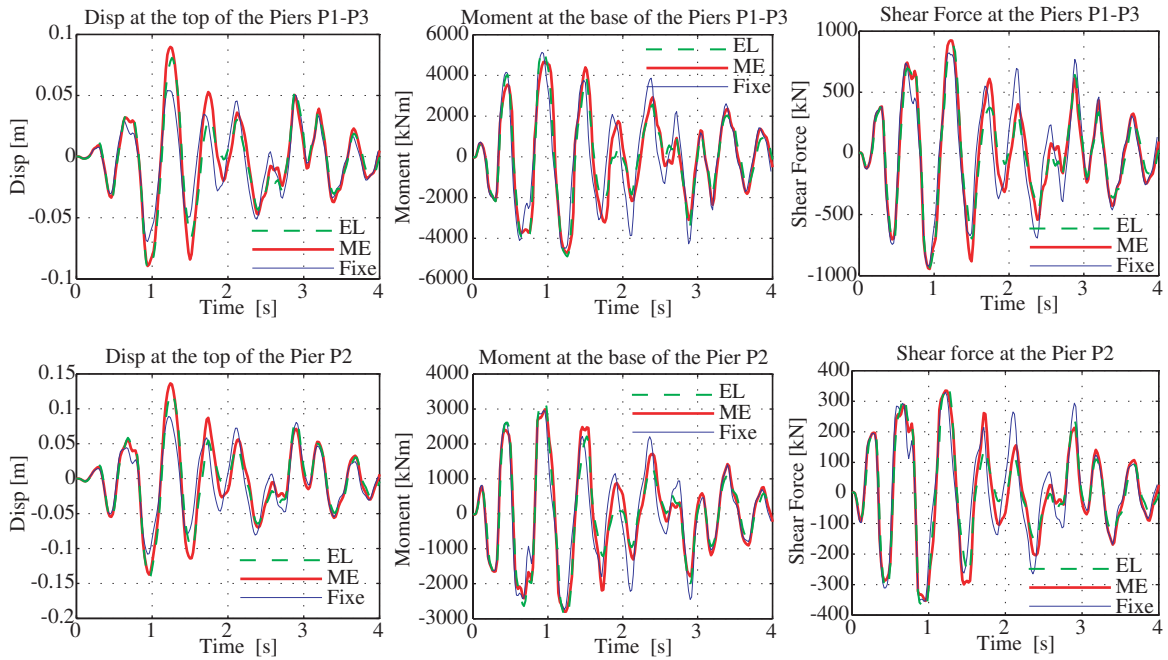


Fig. 9. Viaduct - SSI: comparisons of the displacements, moments, shear forces for the strong motion and the class B soil.

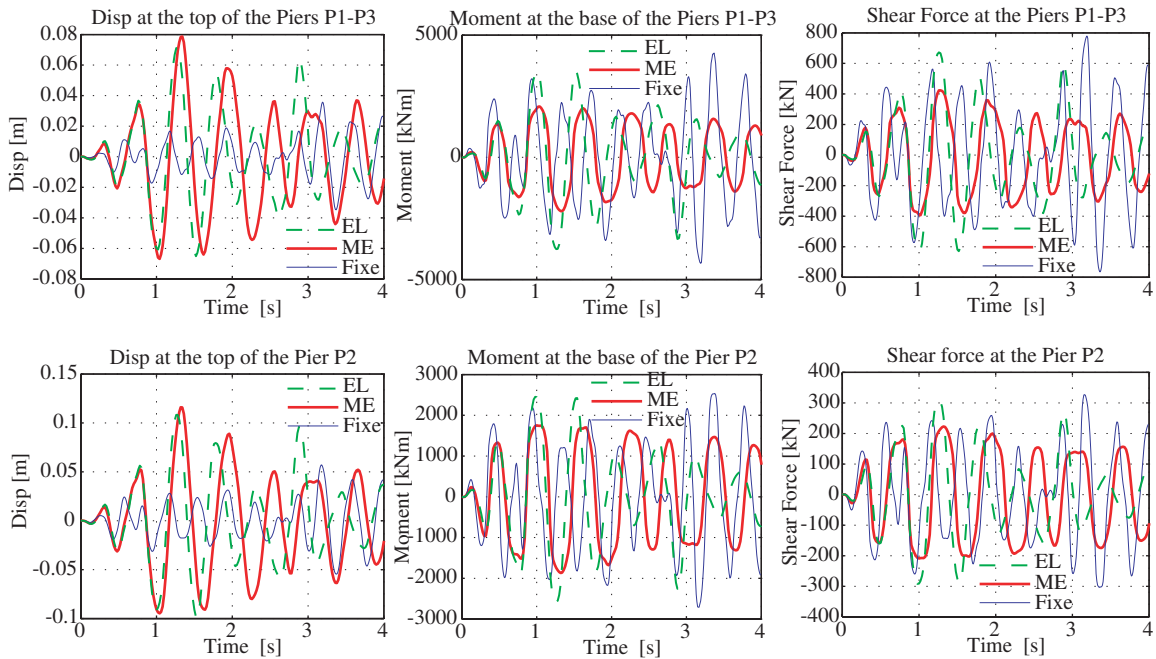


Fig. 10. Viaduct - SSI: comparison of the displacements, moments and shear forces for the low motion and for the class C soil.

4 CONCLUSION

In this article we propose a new simplified numerical strategy to reproduce dynamic Soil-Structure Interaction (SSI). Based on the macro-element approach, it is validated using a three-pile reinforced concrete viaduct tested pseudo-dynamically in ELSA laboratory.

Two simulations are presented: The first one considering the structure embedded in the soil and the second one taking into account SSI and this for two types of soils (class B and C). After validating the numerical model with the experimental results of the viaduct considered embedded in the soil, the influence of SSI is quantified introducing the macro-element at the bottom of each pile. It is found that SSI isolates the structure as global forces and damage are found significantly reduced. Finally, comparison with an engineering approach based on linear elastic springs calibrated using an energy criterion shows clearly the advantages of the new macro-element. More specifically when considering SSI:

- for a class C soil (low mechanical characteristics), internal forces at the base of the piles are found reduced compared to the ones provided considering the piles fixed. The displacements at the top of the structure are strongly amplified (multiplied by 4).
- for a class B soil (higher mechanical characteristics), non-linearities are concentrated principally at the base of each pile. In other words, the resisting moment of concrete guides the behaviour of the structure. The resisting force of the pile is reached before the resisting moment of the system {soil+foundation}. The procedure is similar as if the piles were embedded in the soil, except for the displacements that are amplified (multiplied by 2).

Finally, the limits of the classical engineering approach based on elastic linear springs are evident. The results are found conservative. Internal forces and displacements are higher from the ones coming from the macro-element which reproduces better the non-linear behaviour of the viaduct. Moreover, the calibration of the stiffness of the elastic springs in the engineering approach is a quite difficult task.

Based on the results obtained within this work, it appears now possible to use the macro-element tool in design offices and research groups to investigate numerically the behaviour of a wider variety of configurations that is practically impossible to study experimentally.

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