The effects of Soil–Structure Interaction on a reinforced concrete viaduct

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SUMMARY

This paper presents a numerical strategy to model a three-pier viaduct made of prestressed concrete. The viaduct was tested pseudodynamically in ELSA laboratory (JRC Ispra, Italy). During the experimental campaign, only the three piers 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 the Timoshenko multifibre 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 piers. This comparison shows that SSI is a complex phenomenon inducing displacements and internal forces in the structure that are difficult to predict with the linear approach. Based on the results obtained in this paper, it seems now possible to use this approach to investigate numerically the behaviour of a wider variety of configurations. Copyright © 2010 John Wiley & Sons, Ltd.

Received 8 September 2009; Revised 28 May 2010; Accepted 28 May 2010

KEY WORDS: Soil–Structure Interaction; foundation; macro-element; Timoshenko beam; viaduct

1. INTRODUCTION

In civil engineering, boundary conditions have to be correctly modelled in order to reproduce numerically the non-linear behaviour of a structure. Soil–Structure Interaction (SSI) cannot be neglected. This is particularly true for slender structures like tall buildings or bridge piers. Their behaviour is different whether the structure is on a solid rock or on a soft soil.

However, simulating SSI often necessitates the use of detailed and complex 3D finite element models for the soil and the structure, leading to a great number of degrees of freedom and thus to significant computational costs. This is the reason why various simplified modelling strategies have recently been developed. Multifibre beam elements coupled with non-linear constitutive laws allow reproducing the non-linear behaviour of the structure. The ‘macro-element’ approach helps reproducing the non-linear behaviour of the foundations considering material and/or geometric non-linearities.

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This work deals with a simplified numerical strategy based on multifibre beams and macro-elements to investigate the effects of SSI on a reinforced concrete viaduct. The three-pier viaduct made of prestressed concrete was tested experimentally in ELSA laboratory (JRC Ispra, Italy, [1]). The tests were pseudodynamic, i.e. only the three piers were tested. As for the deck, it was simulated using the finite element code Cast3M [2]. During the experiments, the three viaduct piers were fixed at the base (both displacements and rotations are prevented). The paper is divided into the following two parts:

- Part I: The viaduct is considered to be fixed at its base. It is simulated using multifibre Timoshenko beams [3, 4] and non-linear constitutive laws based on damage mechanics [5] and plasticity [6]. A 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 inelastic behaviour of the soil, the uplift of the foundation, $P-\theta$ effects and the radiative damping [7–11]. The results of a classical engineering approach are finally provided, using linear elastic springs at the base of the piers having an equivalent initial stiffness calibrated using an energy criterion. A comparison of the two approaches shows clearly the advantages of the new macro-element approach.

2. PART I: MODELLING OF THE PSEUDO-DYNAMIC TEST

A 1:2.5 scaled viaduct was recently tested pseudo-dynamically in ELSA (JRC Ispra) (Figure 1, [1]). This kind of a hybrid experiment allows testing only a part of the structure, whereas the rest is simulated with a finite element code. Thus, only the three piers fixed at their bases were tested, the deck being simulated with the element finite code Cast3M [2]. During the test, the interaction between the piers and the deck was calculated in real time by numerically integrating the dynamic

![Figure 1. Viaduct: plan view of the tested viaduct in Ispra, (scale 1:2.5) [1].](image1)

![Figure 2. Viaduct: (a) scheme of the deck (modelled with Cast3M during the experiments) and (b) scheme of the piers (scale 1:2.5).](image2)
Equations of motion in time. Inertial forces were calculated and imposed to the model piers by applying the adequate displacements.

Details of the deck and piers are given (scaled) in the elevation and section plans of Figures 2(a) and (b). Piers are made of reinforced concrete and present a hollow rectangular section shape. The deck is composed of hollow ‘voussoirs’ made of prestressed concrete. Its behaviour can thus be considered linear. Details of the geometrical characteristics of the section are given in Table I.

### 2.1. Finite element (FE) mesh

A finite element (FE) model using multifibre beams and concentrated masses is chosen to reproduce the structure (Figure 3). The mass and rotational inertia details are given in Table II.

Non-linear Timoshenko multifibre beam elements are used to reproduce the behaviour of the piers, see References [3, 4]. 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 tends to be concentrated (Figure 4). Forty concrete fibres and eighty steel fibres (representing the reinforcement bars at their actual position) are used in each section. Details of the fibres used into the section for the piers P1–P3 and P2 are given, respectively, in Figure 4 and Reference [12]. The deck being made of prestressed concrete, its behaviour is assumed to be linear and it is discretized using linear beam elements. Calculations are made with FEDEASLab, a finite element MATLAB toolbox [13].

### 2.2. Material parameters

Constitutive model for concrete under cyclic loading ought to take into account some observed phenomena such as decrease in material stiffness due to cracking, stiffness recovery which occurs at crack closure and inelastic strains concomitant to damage. To simulate this behaviour we use a damage model with two scalars variables, one in compression and one in tension [5]. Unilateral
effect and stiffness recovery (damage deactivation) are also included. Inelastic strains are taken into account, thanks to an isotropic tensor (Figure 5). The total strain is given by

\[
\epsilon = \epsilon^e + \epsilon^{in}
\]

\[
\epsilon^e = \frac{\langle \sigma \rangle_+}{E(1-D_1)} + \frac{\langle \sigma \rangle_-}{E(1-D_2)} + \frac{\nu}{E}(\sigma - \text{Tr}(\sigma)\mathbf{I})
\]

\[
\epsilon^{in} = \frac{\beta_1 D_1}{E(1-D_1)} \frac{\partial f(\sigma)}{\partial \sigma} + \frac{\beta_2 D_2}{E(1-D_2)} \mathbf{I}
\]

where

\[
\text{Tr}(\sigma) \in [0, +\infty) \rightarrow \frac{\partial f(\sigma)}{\partial \sigma} = \mathbf{I}
\]

\[
\text{Tr}(\sigma) \in [-\sigma_f, 0] \rightarrow \frac{\partial f(\sigma)}{\partial \sigma} = \left(1 - \frac{\text{Tr}(\sigma)}{\sigma_f}\right) \mathbf{I}
\]

\[
\text{Tr}(\sigma) \in (-\infty, -\sigma_f] \rightarrow \frac{\partial f(\sigma)}{\partial \sigma} = 0.\mathbf{I}
\]

with \(\epsilon^e\) the elastic strains, \(\epsilon^{in}\) the inelastic strains and \(\sigma\) the stress tensor, \(\mathbf{I}\) denotes the unit tensor, \(\text{Tr}(\sigma) = \sigma_{ij}\), \(f\) is the crack closure function and \(\sigma_f\) the crack closure stress. \(\langle . \rangle_+\) denotes the positive and \(\langle . \rangle_-\) the negative part of the tensor. \(E\) is the initial Young’s modulus and \(\nu\) the Poisson ratio. \(D_1\) and \(D_2\) are, respectively, the damage variables for tension and compression, \(\beta_1\) and \(\beta_2\) are material constants. Damage criteria are expressed as \(f_i = Y_i - Z_i\) (\(i = 1\) for tension or \(2\) for...
Figure 5. Cyclic response of the La Borderie model for concrete.

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

| Concrete parameters | Steel parameters |
|---------------------|------------------|
| $E$                 | 29.4 GPa         | $E$                 | 200 GPa           |
| $\nu$              | 0.175            | $f_y$              | 450 MPa           |
| $Y_{01}$           | 1000 Pa          | $f_{su}$           | 710 MPa           |
| $Y_{02}$           | 0.0001 MPa       | $\varepsilon_{sh}$ | 0.0060            |
| $A_1$              | 7000 MPa$^{-1}$  | $\varepsilon_{su}$ | 0.10              |
| $A_2$              | 6.0 MPa$^{-1}$   |                   |                   |
| $B_1$              | 1.0              |                   |                   |
| $B_2$              | 1.3              |                   |                   |
| $\beta_1$          | 0.5 MPa          |                   |                   |
| $\beta_2$          | $-19$ MPa        |                   |                   |
| $\sigma_f$         | 3.0 MPa          |                   |                   |

Figure 6. Cyclic response of the Menegotto–Pinto model for steel.

compression, $Y_i$ is the associated force to the damage variable $Z_i$ and $Z_i$ is a threshold dependent on the hardening variables). The evolution laws for the damage variables $Z_i$ are written as

$$D_i = 1 - \frac{1}{1 + [A_i(Y_i - Y_{0i})]^B_i}$$

where $Y_{0i}$ is the initial elastic threshold ($Y_{0i} = Z_i(D_i = 0)$) and $A_i$, $B_i$ are material constants. For the calculations presented hereafter, the uniaxial version of the damage model is used (shear is considered to be linear). Figure 5 gives the stress–strain response of the model for an uniaxial tension–compression.

A modified version of the classical Menegotto–Pinto model [6] with an isotropic hardening is used for steel. $f_y$ and $f_{su}$ are the yield and the maximum stresses for the steel associated with the strains $\varepsilon_{sh}$ and $\varepsilon_{su}$, respectively. Figure 6 gives the stress–strain response of the model. Table III gives material parameters used for the concrete and steel constitutive laws.
Table IV. Viaduct: comparisons between experimental and computed modal frequencies.

| Mode | Test (Hz) | Simulation (Hz) |
|------|-----------|-----------------|
| 1    | 4.2       | 4.2             |
| 2    | 6.6       | 6.8             |
| 3    | 9.8       | 10.4            |
| 4    | 16.1      | 16.8            |

It is worth noting that as the tests are pseudo-dynamic, the damping coefficient adopted in the numerical simulations has to be small. A 0.5% coefficient is used, calibrated on the first and fifth natural modes.

2.3. Experimental versus numerical results: modal analysis

Natural frequencies for the structure rigidly constrained at the base are given in Table IV. Modal deformed shapes of the viaduct are shown in Figure 7.

2.4. Loading sequence

The accelerations imposed at the base of the structure derive from a synthetic accelerogram consistent with a 5% damping response spectrum selected according to Eurocode 8 for a soil of class B [14]. Figure 8(a) shows the comparison between the synthetic spectral response and the design spectra of the Eurocode 8. The synthetic accelerogram is presented in Figure 8(b). The peak of accelerations is situated at 0.35g (‘weak’ earthquake). A second similar accelerogram (dilated, not represented here) is also imposed at the base of the structure. Its peak of acceleration is equal to 0.7g (‘strong’ earthquake).

It is worth noting that as the small-scale model of the pier is reduced in size by a factor of 2.5, 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 weak-level earthquake (0.35g at scale 1) is provided in Figure 8(c) at scale 1:2.5.

2.5. Experimental versus numerical results: dynamic analysis

Figures 9 and 10 show the comparison between the experimental and the numerical results of the dynamic analysis for the fixed structure. The two earthquakes (weak and strong) are considered. The figures show the evolution with time of the shear forces at the base and the lateral displacements at the top of the piers P1, P2 and P3.

One can clearly see that despite the small number of degrees of freedom of the finite element model, the non-linear behaviour of the viaduct is reproduced quite satisfactorily. Not only the peaks
Figure 8. Viaduct: (a) design spectra coming from Eurocode 8 for a 5% damping and synthetical accelerogram spectra; (b) weak-level earthquake signal for the scale 1 structure; and (c) for the scale 1:2.5 structure.

Figure 9. Viaduct: comparison between experimental and numerical displacements and shear forces for the weak-level earthquake.

in both directions are well reproduced but also the frequency content of the response is correctly matched.

3. PART II: INFLUENCE OF SSI

In this second part, 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 applied at the base of each pier. Parametric studies taking into account two types of soils and comparisons between the two different modelling strategies are detailed.
3.1. Description of the macro-element

The macro-element approach consists of condensing all non-linearities into a finite domain and works with generalized variables (forces and displacements) defined at the centre of the foundation (Figure 11). It thus allows a significant reduction of the degrees of freedom. Several macro-elements can be found in the literature, e.g. [15–20]. The macro-element used in this paper takes into account three different mechanisms: elasticity, plasticity of the soil and uplift of the foundation. The total displacement is thus decomposed as a sum of an elastic, plastic and uplift part. Plasticity and uplift are described according to the classical theory of plasticity.

The general formulation of the macro-element is detailed in [8] for the plasticity model, [9] for the uplift mechanism, [10] for the coupling of the two mechanics and [11] for the dynamic behaviour.

3.2. Types of soils

In order to investigate the influence of SSI on the reinforced concrete viaduct, two types of soils are considered (according to the Eurocode 8 classification [14]): a class B and a class C soil. Both soils are considered as purely cohesive. The values of the parameters defining the shape of the failure locus for both soils are given according to [16], and are listed in Table V. See [8] for more details.

3.3. Geometry of the foundations

For the class B soil, a rectangular shallow foundation is considered at each pier. The foundation has the dimensions: $L_x = 4.2$ m and $L_y = 2.1$ m. For the class C soil and in order to amplify the influence of SSI a smaller foundation is considered, with $L_x = 3.2$ m and $L_y = 1.6$ m.
3.4. Calibration of the linear springs

For the second model and in order to reproduce the behaviour of the soils seen in Table V, the stiffness of the linear springs has 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 12). The energy dissipated by the macro-element in the hysteresis loops is not taken into account.

The maximum rotation taken into account for calibrating the stiffness of the linear springs is the maximum rotation for the weak motion for soils B and C. Figure 13 shows the Moment–Rotation curves for the two soils and gives a comparison between the results of the macro-element and the linear springs (EL).

3.5. Numerical results considering SSI

The influence of SSI effects on the dynamic response of the viaduct has been evaluated by comparing—for the two soils considered—the predictions obtained by: (i) modelling the foundations with the macro-element, (ii) modelling the foundations with equivalent linear springs, (iii) considering the piers fixed at the base.

Figure 13 shows a comparison between soil B and C in terms of Moment–Rotation curves. It is clear that uplift is the principal mechanism for soil B (S-shape of the loops), whereas plasticity is predominant for soil C (size of the loops). For both soils, rotations and moments are bigger in the case of the strong earthquake.
Figure 12. Viaduct—SSI: calibrating the stiffness of the elastic linear springs.

Figure 13. Viaduct—SSI: comparison of the Moment-Rotations curves for (a) soil B and (b) soil C for the weak and strong motion, considering the macro-element and the linear springs (EL).

3.5.1. SSI—class B soil. Only the results for the strong earthquake are presented in this section (Figure 14). The three types of boundary conditions are denominated as follows: linear springs (EL), macro-element (ME) and fixed (Fixed). The results are similar in terms of internal forces. Nevertheless, maximum displacements at the top of the piers are found to be significantly increased (multiplied by 1.5) for the cases considering SSI (EL and ME).

For the weak earthquake, the computed response of the structure is the same independent of the assumed boundary conditions (the maximum internal forces and the level of damage are similar). Indeed for a stiff soil, damage and non-linearities are concentrated in the lower portion of the piers, close to the foundations. It is the bending strength of the pier that controls the internal forces in the structure.

3.5.2. SSI—class C soil. The results for the weak earthquake are presented in Figure 15 for the class C soil. This time, the behaviour of the viaduct is different depending on the assumed boundary conditions. The displacements are strongly amplified, multiplied by 3 or 4, for the case of the structure resting on the macro-element and on the linear elastic springs, respectively. The results are, however, more pronounced for the internal forces at the base of the piers (moments and shear
forces). In fact, loads on the structure are significantly reduced for the case of the macro-element, due to the bearing moment and force capacity that are reached into the soil. The results obtained with the elastic linear springs do not have such limits and can be similar to the ones found for the fixed piers.

For the strong earthquake, conclusions about the computed response of the structure are similar. Forces are limited by the bearing capacity of the foundation and displacements are amplified compared with the weak motion.

4. CONCLUSIONS

In this work, we propose a new simplified numerical strategy to analyse dynamic SSI problems, based on the macro-element approach. The structural part of the numerical model, employing the Timoshenko multifibre beam elements with complex non-linear constitutive laws, is validated by comparing the predicted response with the experimental results obtained on a small-scale, three-pier reinforced concrete viaduct, tested pseudo-dynamically at the ELSA laboratory.

Two groups of numerical simulations are presented: In the first part of the work, the structural part of the numerical model, employing Timoshenko multifibre beam elements with complex non-linear constitutive laws, is validated by comparing the predicted response with the experimental results obtained on the reinforced concrete viaduct at the ELSA laboratory. In the FE simulations, the piers are fixed at the base and no SSI effect is considered. In the second part of the work, the effects of SSI interaction have been evaluated considering two different types of soils (class B and C). The influence of SSI is quantified by modelling the foundation–soil system with a single kinematic hardening elastoplastic macro-element, placed at the bottom of each pier. It is found that SSI isolates the structure as global forces and damages are significantly reduced in this case. Finally, comparison with an engineering approach based on linear elastic springs with an equivalent elastic stiffness calibrated using an energy criterion shows clearly the advantages of the
new macro-element. More specifically:

- for a class C soil (of poor mechanical properties), internal forces at the base of the piers are smaller compared with the ones provided considering the piers fixed at the base. The displacements at the top of the structure are strongly amplified (multiplied by 4).
- for a class B soil (of good mechanical properties), non-linearities are concentrated principally at the base of each pier. In other words, the resisting moment of the pier section controls the behaviour of the structure. The maximum structural strength of the pier is reached before the collapse of the foundation–soil system. The response of the system is similar as if the piers were fixed at the base, except for the displacements that are significantly amplified (multiplied by 2).

Finally, the limits of the classical engineering approach based on elastic linear springs are evident. The results obtained with this approach are conservative (particularly for the case of a class C soil). Internal forces and displacements are higher than the ones obtained using the macro-element approach, which allows a more appropriate description of the non-linear behaviour of the foundation–soil system. Moreover, in spite of the appealing simplicity of this approach, the a priori calibration of the spring constants remains a quite difficult task.

Based on the results obtained in this work, it appears now possible to use the macro-element approach in both research and current design practice, to investigate numerically the behaviour of a wider variety of configurations that is practically impossible to study experimentally. However, more experimental results under dynamic loadings are crucially needed in order to validate the performance of the macro-element.

ACKNOWLEDGEMENTS

Part of this work has been done within the European Contract LESSLOSS, Risk Mitigation for Earthquakes and Landslides, Sixth Framework Programme (Project No.: GOCE-CT-2003-505488), http://www.lessloss.org.
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