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ISS modelling and its contribution to the seismic response of RC structures

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Abstract. This paper presents a simplified modelling strategy aiming at simulating the dynamic behaviour of reinforced concrete (RC) structures considering Soil-Structure Interaction (SSI). The presentation of a non linear interface element able to compute SSI by simulating a rigid shallow foundation is carried out. The new element is based on the “macro-element” concept. The foundation of the structure is supposed infinitely rigid and its movement is entirely described by a system of global variables (forces and displacements) defined in the foundation’s centre [Grange *et al.* , 2008], [Grange *et al.* , 2009]. This SSI element is available in various finite element codes (e.g. FEDEASLab or Cast3m). It can be coupled with multifiber Timoshenko beam elements to simulate the behaviour of the RC upper structure. The use of macro-elements combined with multifiber beam elements allows mitigating vulnerability of structures and evaluating the displacements that occur during an earthquake. Validation of the proposed modelling strategy is provided using experimental results.

keyword: foundation; plasticity; uplift; macro-element; soil structure interaction; RC walls.

1 Introduction

In civil engineering, boundary conditions have to be correctly modeled in order to reproduce numerically the non linear behavior 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 behavior 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 been recently developed.

The macro-element concept was applied for the first time in geomechanics in [Nova *et al.*, 1991]. It consists in condensing all nonlinearities into a finite domain and works with generalized variables (forces and displacements) that allow simulating the behaviour of shallow foundations in a simplified way. The macro-element developed in [Grange *et al.*, 2008], [Grange *et al.*, 2009] is inspired from [Crémer *et al.*, 2002] and it reproduces the behavior of a 3D shallow foundation of circular, rectangular or strip shape, submitted to cyclic or dynamic loadings. It takes into account the plasticity of the soil and the uplift of the foundation. A brief outline of the formulation of the macro-element follows.

2 Mathematical description of the macro-element

The associated generalized variables (displacement and force vectors) are dimensionless. They are defined hereafter (for any a , a' defines the corresponding dimensionless variable): vertical force V' , horizontal forces H'_x , H'_y and moments M'_x , M'_y , but also the corresponding displacements, vertical settlement u'_z , horizontal displacements u'_x , u'_y and rotations θ'_x , θ'_y . Torque moment (M'_z) is not taken into account (figure 1).

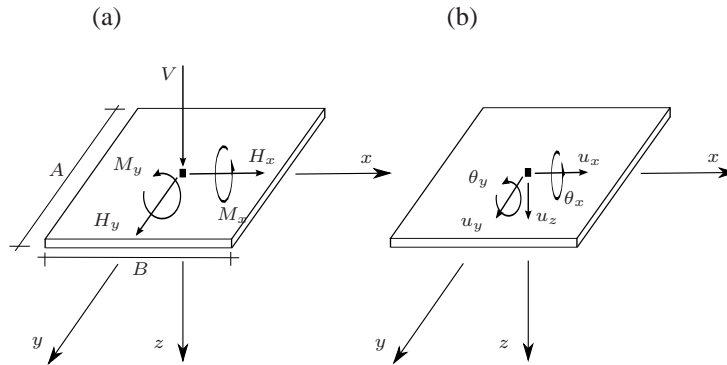


Figure 1: Generalised variables: (a) forces and (b) displacements for a rectangular foundation.

The elastic part of the constitutive law is defined as $\underline{\mathbf{F}} = \underline{\mathbf{K}}^{\text{el}} \underline{\mathbf{u}}^{\text{el}}$, where the displacement $\underline{\mathbf{u}}^{\text{el}}$ and force vectors $\underline{\mathbf{F}}$ are dimensionless. The elastic stiffness matrix $\underline{\mathbf{K}}^{\text{el}}$ is calculated using the real part of the static impedances. It is considered diagonal, i.e. there is not coupling between the different directions of the loading.

The failure criterion for the plasticity mechanism is defined for an overturning mechanism with uplift [Salençon *et al.*, 1995]. The adaptation in 3D is done by adding the two terms related with the horizontal force and moment according to the other axis and assuming axial symmetry. One finally obtains the following 5D surface:

$$f_c(\underline{\mathbf{E}}, \underline{\mathcal{I}}, \rho, \gamma) \equiv \left(\frac{H'_x}{\rho a V'^c (\gamma - V')^d} - \frac{\alpha_1}{\rho} \right)^2 + \left(\frac{M'_y}{\rho b V'^e (\gamma - V')^f} - \frac{\alpha_2}{\rho} \right)^2 + \left(\frac{H'_y}{\rho a V'^c (\gamma - V')^d} - \frac{\alpha_3}{\rho} \right)^2 + \left(\frac{M'_x}{\rho b V'^e (\gamma - V')^f} - \frac{\alpha_4}{\rho} \right)^2 - 1 = 0 \quad (1)$$

The coefficients a, b define the size of the surface in the planes $(H' - M')$. c, d, e and f define the parabolic shape of the surface in the planes $(V' - M')$ and $(V' - H')$. These parameters can be fitted to different experimental results found in the literature. $\underline{\mathcal{I}} = [\alpha_1, \alpha_2, \alpha_3, \alpha_4]$ is the kinematics hardening vector composed of 4 kinematics hardening variables and ρ the isotropic hardening variable. The variable γ is chosen to parametrize the second intersection point of the loading surface with the V' axis and its evolution along the V' axis. The evolutions of the hardening variables are obtained considering experimental results and numerical simulations of foundations under cyclic loadings from the model of Crémer [Crémer *et al.*, 2002].

An evolution of this variables linked to the distance Δ between the loading point and the failure criterion leads to a perfect tangency of the loading surface and the failure criterion without any interpenetrations between the surfaces (see figure 2 where, for instance: $H^* = \frac{H'}{a V'^c (1 - V')^d}$).

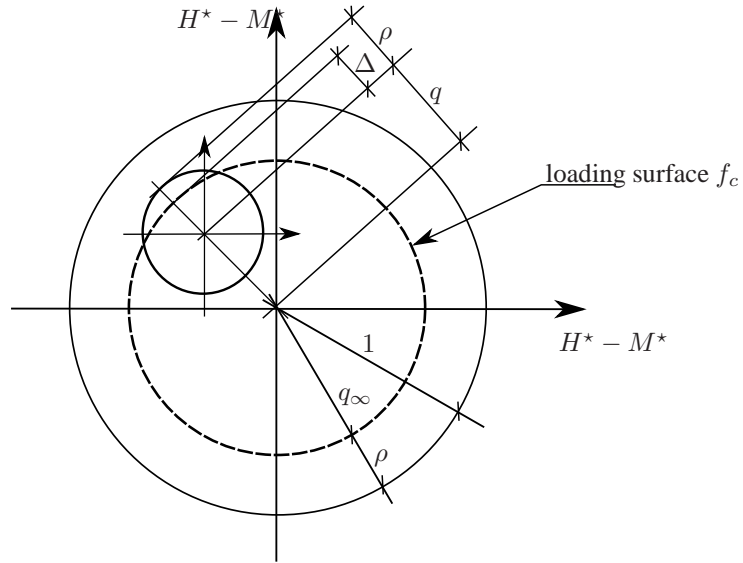


Figure 2: Definition of the distance Δ in hyperplan $(H_x^*, M_y^*, H_y^*, M_x^*)$.

The failure criterion is given by equation (1) considering $(\alpha_1, \alpha_2, \alpha_3, \alpha_4, \rho, \gamma) = (0, 0, 0, 0, 1, 1)$.

The flow rule used is associated in the (H'_x, M'_y, H'_y, M'_x) hyperplane and non associated in the (H'_x, V') , (M'_y, V') , (H'_y, V') , (M'_x, V') planes.

Uplift behavior can be treated with a non-linear elastic mechanism following the $M - \theta$ relationship proposed by Crémer. A Newton algorithm solver is computed in order to solve the three non-linear equations system governing the uplift mechanism. Let's remind that uplift mechanism is essential for computing a negative vertical displacement of the center of the foundation. For further details see [Grange *et al.* , 2009].

3 Numerical results

A model using Timoshenko multifiber beams [Kotronis *et al.* , 2005], [Mazars *et al.* , 2006] and concentrated masses is chosen to reproduce the structure (3). Six beam elements are used for the piles P1 and P3 and nine elements for the pile P2. The mesh is refined at the base of the piles where damage tends to be important. 40 concrete fibers and 80 steel fibers (representing the reinforcement bars at their actual position) are used in each section. Details of the model are provided in [Grange *et al.* , 2010]. Concrete is simulated using a uniaxial damage mechanics law [La Borderie , 1991] and steel with the classical Menegoto Pinto model [Menegoto *et al.*, 1973].The desk being from prestressed concrete, its behaviour is assumed linear and it is discretised using linear beam elements. The loading is applied according to the z axis. Calculations are made with FEDEASLab, a finite element MATLAB toolbox [Filippou *et al.* , 2004].

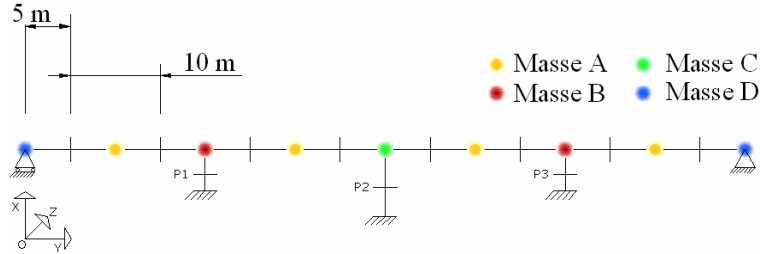


Figure 3: Viaduct - SSI: model using multifiber beam elements and concentrated masses.

Rectangular shallow foundations are numerically introduced at the base of each pile in two different ways, via macro-elements or linear springs. The dimensions of the foundations are: $L_z = 4.2m$, $L_y = 2.1m$. The stiffness of the springs is such that they accumulate the same energy as the non-linear SSI macro-element. A class B soil is considered according to the Eurocode 8 classification. Its characteristics are given in table 1.

Numerical results are presented in 4. Three types of boundary conditions are considered: linear springs (EL), macro-element (ME) and embedded (Fixed). Results are similar in terms of internal forces. Nevertheless, maximum displacements at the top of the piles are found significantly increased (multiplied by 2) for the cases considering SSI (EL and

Table 1: Viaduct - SSI: Characteristics of the class B soil.

| soil | Shear modulus G_0 velocity V_s | cohesion c and friction angle ϕ | Stiffness and damping | ultimate bearing stress q_{max} |
|-----------------|--|--|--|---|
| Class B soil | $V_s = 360\text{m/s}$ $G_0 = 259.2\text{MPa}$ | $c_u = 370\text{kPa}$ $\phi_u = 0$ | $K_{\theta\theta} = 144484.1\text{MNm/rad}$ $K_{zz} = 1845.5\text{MN/m}$ $K_{hh} = 2260.2\text{MN/m}$ $C_{\theta\theta} = 23.17\text{MNms/rad}$ $C_{zz} = 3.61\text{MN s/m}$ $C_{hh} = 2.95\text{MN s/m}$ | $q_{max} = 2100\text{MPa}$ |

ME). Non-linearities are concentrated principally at the base of each pile. In other words, the resisting moment of concrete guides the behavior 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.

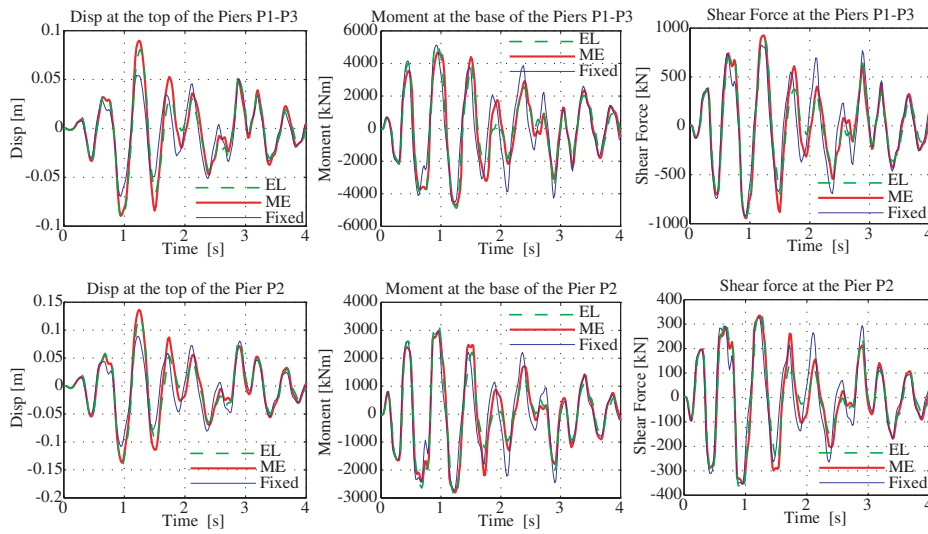


Figure 4: Viaduct - SSI: comparisons of the displacements, moments, shear forces for a class B soil.

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