8th Conference of ACES
GEOTECHNICS IN CIVIL ENGINEERING
13 – 15 November 2019, Vrnjačka Banja, Serbia

A STOCHASTIC APPROACH FOR DYNAMIC SOIL-PILE INTERACTION UNDER ENVIRONMENTAL EFFECTS

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ABSTRACT. The paper deals with a computational stochastic approach for the unilateral contact problem of dynamic soil-pile interaction. Unilateral contact effects due to tensionless soil capacity, soil elastoplastic-fracturing behaviour and gapping are strictly taken into account, as well as environmental effects under uncertainty decreasing the soil resistance. The proposed methodology concerns the treatment of both, the deterministic and the probabilistic problem. The numerical approach concerning the deterministic problem is based on a double discretization, in space by the Finite Element Method combined with Boundary Element Method, and in time, and on nonconvex optimization. Uncertainties concerning the input parameter values are treated by the Monte Carlo method in the probabilistic problem section. Finally, the proposed methodology is applied for a practical case of dynamic soil-pile interaction.

KEY WORDS. Dynamic soil-structure interaction, unilateral contact, environmental degradation, numerical and stochastic geotechnical engineering.

1. INTRODUCTION

Uncertainty concerning input parameters in seismic soil-structure interaction is a crucial problem in geotechnical engineering. As concerns the problem mathematical formulation of the dynamic soil-pile interaction, this involves equalities as well as inequalities [1-3]. Indeed, for the case of the general dynamic soil-structure interaction, see e.g. [1], the interaction stresses in the transmitting interface between the structure and the soil are of compressive type only. Moreover, due to in general nonlinear, elastoplastic, tensionless, fracturing etc. soil behavior, gaps can be created between the soil and the structure. Thus, during strong earthquakes, separation and uplift phenomena are often appeared, as the praxis has shown [1,2,9-14].

Due to above inequality conditions, the pile-soil interaction can be considered as one of the so-called inequality problems of structural and geotechnical engineering [4-7]. The mathematical treatment of the so-formulated inequality problems can be obtained by the variational or hemivariational inequality approach [4,5]. Numerical approaches for some inequality problems of structural elastoplasticity and earthquake engineering have been also presented, see e.g. [1,3-8].

In the present paper, a stochastic numerical approach for the inequality dynamic problem of soil-pile interaction under uncertain input parameters is presented. Environmental degradation for the soil and second-order geometric effects for the pile behaviour due to preexisting compressive loads are taken into account. The proposed numerical approach consists of solving first the deterministic problem and next the probabilistic problem. The numerical method for the treatment of the deterministic problem is described in details in [1]. This approach is based on a double discretization and on methods of nonlinear programming. So, in space the finite element method (FEM) coupled with the boundary element method (BEM), and in time a step-by-step method for the treatment of convolutional conditions are used. In each time-step a non-convex linear complementarity problem is solved with reduced number of unknowns. The probabilistic numerical approach uses the Monte Carlo simulation [15-18] for the treatment of uncertain input parameters. Finally, the presented procedure is applied to an example problem of dynamic pile-soil interaction, and some concluding remarks useful for the Civil Engineering praxis are discussed.

2. THE PROBABILISTIC COMPUTATIONAL APPROACH

The probabilistic approach for the dynamic soil-pile interaction can be obtained through Monte Carlo simulations. As well-known, see e.g. [15-18], Monte Carlo simulation is simply a repeated process of generating deterministic solutions to a given problem. Each solution corresponds to a set of deterministic input values of the underlying random variables. A statistical analysis of the so obtained simulated solutions is then performed. Thus the computational methodology consists of solving first the deterministic problem for each set of the random input variables and finally realizing a statistical analysis.

Details of the methodology concerning the deterministic problem and the probabilistic aspects are given in the next sections.

2.1. Numerical Treatment of the Deterministic Problem

First, a discretization in space by combining the finite element method (FEM) with the boundary element one (BEM) is used for the soil-pile system [1,2,5,9-14]. The pile is discretized into frame-beam finite elements. Each pipeline node is considered as connected to the associate soil nodes on both sides through two *unilateral* (interface) elements. Every such *u-element* consists of an elastoplastic softening spring and a dashpot, connected in parallel (see e.g. the Figure 1a), and appears a compressive force r(t) only at the time-moments t when the pipeline node comes in contact with the corresponding soil node. Let v(t) denote the relative retirement displacement between the soil-node and the pipe-node, g(t) the existing gap and $w_g(t)$ the soil displacement induced by moving sources of the type described in the Introduction. Then the piece-wise linearized unilateral contact behaviour of the soil-pipeline interaction is expressed in the compact form of the following linear complementarity conditions:

$$v+g+w_g \geq 0, \quad r \geq 0, \quad r.(v+g+w_g) = 0. \tag{1}$$

Further, the *u-element* compressive force is in convolutional form [2,13]

$$r = S(t)*y(t), \quad y = w - (g + v), \tag{2a,b}$$

or in form used in Foundation Analysis [14]

$$r = c_s . (dy/dt) + p(y). \tag{2c}$$

Here c_s is the soil damping coefficient, w = w(t) the pile-node lateral displacement, y = y(t) the shortening deformation of the soil-element, and p(y) the spring force. By * is denoted the convolution operation. S(t) is the dynamic stiffness coefficient for the soil and can be computed by the BEM [2]. Function p(y) is mathematically defined by the following, in general nonconvex and nonmonotone constitutive relation:

$$p(y) \in C_g P_g (y), \tag{2d}$$

where C_g is Clarke's generalized gradient and P_g ( ) the symbol of superpotential nonconvex functions [4-5,8]. So, eq. (2d) expresses in general the elastoplastic-softening soil behaviour, where unloading-reloading, gapping, degrading, fracturing etc. effects are included.

For the herein numerical treatment, p(y) is piece-wise linearized in terms of non-negative multipliers as in plasticity [1,7]. So, the dynamic equilibrium conditions for the assembled soil-pile system are written in matrix form as follows:

$$\underline{M}\, \underline{\ddot{u}}(t) + \underline{C}\, \underline{\acute{u}}(t) + \underline{K}\, \underline{u}(t) = \underline{f}(t) + \underline{A}^T \underline{r}(t), \tag{3}$$

$$\underline{h} = \underline{B}^T\underline{r} - \underline{H}\, \underline{z} - \underline{k}, \quad \underline{h} \leq \underline{0}, \quad \underline{z} \geq \underline{0}, \quad \underline{z}^T.\underline{h} = 0. \tag{4}$$

Here, eq. (3) is the dynamic matrix equilibrium condition and eqs. (4) include the unilateral and the piece-wise linearized constitutive relations. Dots over symbols denote, as usually, time-derivatives. $\underline{M}$, $\underline{C}$ and $\underline{K}$ are the mass, damping and stiffness matrix, respectively; $\underline{u}$, $\underline{f}$ are the displacement and the force vectors, respectively; $\underline{A}$, $\underline{B}$ are kinematic transformation matrices; $\underline{z}$, $\underline{k}$ are the nonnegative multiplier and the unilateral capacity vectors; and $\underline{H}$ is the unilateral interaction square matrix, symmetric and positive semidefinite for the elastoplastic soil case. But in the case of soil softening, some diagonal entries of $\underline{H}$ are nonpositive [7]. Finally, the force vector $\underline{f}$ includes the effects due to high-speed moving sources in the surrounding soil along the pile-line.

Thus the so-formulated problem is to find ($\underline{u}$,$\underline{r}$,$\underline{g}$,$\underline{z}$) satisfying (1)-(4) when $\underline{f}$ and suitable initial conditions are given.

Assuming that the unilateral quantities $\underline{z}$ and $\underline{h}$ include all local nonlinearities and unilateral behaviour quantities, applying the central-difference time discretization, and after suitable elimination of some unknowns, we arrive eventually at

$$\underline{h}_n = \underline{D}\,\underline{z}_n + \underline{d}_n, \quad \underline{z}_n \geq \underline{0}, \quad \underline{h}_n \leq \underline{0}, \quad \underline{z}_n^T.\underline{h}_n = 0. \tag{5}$$

Thus, at every time-moment $t_n = n.\Delta t$, where Δt is the time step, the problem of rels. (5) is to be treated. This problem is a *Non-Convex Linear Complementarity Problem* (NCLCP), can be treated as an hemivariational one and is solved by available methods and computer codes of nonconvex optimization [1, 4-8]. So, in each time-step Δt we compute which of the unilateral constraints are active and which are not. Due to soil softening, the matrix $\underline{D}$ is not a strictly positive definite one in general. But as numerical experiments have shown, in most civil engineering applications of soil-pile interaction this matrix is P-copositive, and thus the existence of a solution is assured [7].

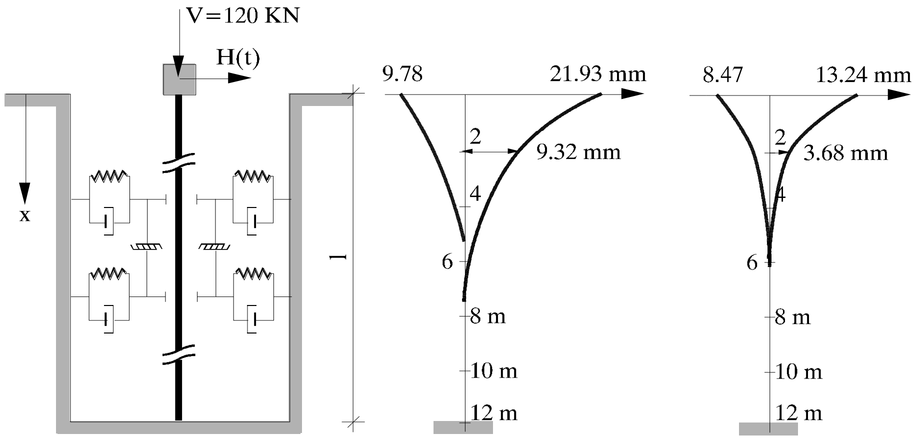
2.2. Numerical Treatment of the Probabilistic Problem

In order to calculate the random characteristics of the response of the considered soil-pile system, the Monte Carlo simulation is used [15-17]. As mentioned, the·main element of a Monte Carlo simulation procedure is the generation of random numbers from a specified distribution. Systematic and efficient methods for generating such random numbers from several common probability distributions are available. The random variable simulation is implemented using the technique of Latin Hypercube Sampling (LHS) [18]. The LHS is a selective sample technique by which, for a desirable accuracy level, the number of the sample size is significantly smaller than the direct Monte Carlo simulation.

In more details, a set of values of the basic design input variables can be generated according to their corresponding probability distributions by using statistical sampling techniques. The generated basic design variables are treated as a sample of experimental observations and used for the system deterministic analysis to obtain a simulated solution as in subsection 2.1. is described. As the generation of the basic design variables is repeated, more simulated solutions can be determined. Finally, statistical analysis of the simulated solutions is then performed. The results obtained from the Monte Carlo simulation method depend on the number of the generated basic design variables used.

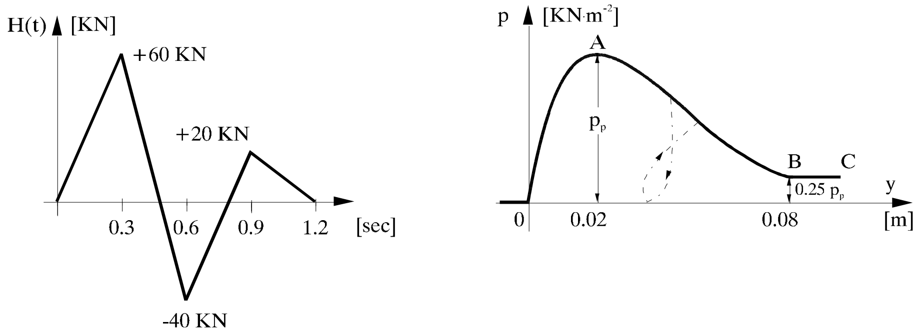
3. NUMERICAL EXAMPLE

The steel IPB300 H-pile depicted in Figure 1(a) has a length L = 12 m and is fully embedded into a clay deposit. The pile has a stiffness EI=52857 KN.m^2, is fixed at the bottom and free at the top. The effects of the over structural framing are approximated by a lumped mass 2 KN.m^{-1}.sec^{2} and a rotational inertia 2 KN.m.sec^{2}.



(a) (b) (c)

Figure 1: The numerical example: (a) The soil-pile system model, (b) Mean values of maximum horizontal pile displacements, (c) Mean values of final soil-pile gaps.



(a) (b)

Figure 2: The numerical example: (a) Dynamic loading diagramme, (b) Diagramme (p-y) of the soil behaviour.

The pile is subjected to a vertical constant top force V of 120 KN and to a dynamic horizontal top force H(t) with the time history shown in Figure 2(a).

Denoting by x the axis along the pile-see Fig. 1(a)-the elastoplastic-softening soil behavior according to eqs. (2) is shown in Figure 2(b)-diagramme (p-y)- where:

i. For the branch OA holds the exponential form $p(x,y) = p_p.[1\text{-}\exp(-ay)]$, where $p_p = b.[1\text{-}0.5\exp(-cx)]$,
ii. For the branch AB holds $p(x,y) = 0.75\ p_p.\ (-3\xi^2+2\xi^3)+p_p$, where $\xi = (y\text{-}0.02)/0.06$. For unloading-reloading paths the inclination is ap_u.

In the above equations, the involved input parameters to be estimated are: a in units [m^{-1}], b in [kN/m^2] and c in [m^{-1}]. Based on experimental investigations and on in-situ results, the lower and upper bound estimates for these uncertain parameters in the examined example are as follows:

$$97 \leq a \leq 103, \quad 360 \leq b \leq 390, \quad 0.40 \leq c \leq 0.60 \qquad (6)$$

According to above values-ranges, the mean values are a_m=100 m^{-1}, b_m=375 kN/m^2 and c_m=0.55 m^{-1}.

The developed numerical procedure is applied by using 200 Monte Carlo samples based on eqs.(6). Some response results from the ones obtained are indicatively reported in Figure 1(b) and 1(c). So, the mean values of the maximum pile horizontal displacements and of the final gaps along the pipeline due to permanent soil deformations are shown in Figures 1(b) and 1(c), respectively.

3. CONCLUDING REMARKS

In the case of uncertain input parameters, the herein presented stochastic procedure provides a numerical tool for the probabilistic soil-pile interaction dynamic analysis. The representative results of the numerical example show that unilateral contact effects due to tensionless soil capacity, reduced by environmental effects, and due to gapping, may be significant and have to be taken into account for the dynamic soil-pile interaction. So the herein presented stochastic procedure can be useful in the geotechnical praxis for the earthquake resistant construction, design and control of piles.

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