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Macroelement modeling of SSI effects on offshore wind turbines subject to large number of loading cycles

Diana Salciarini[a,]*, Martina Frizza[a], Claudio Tamagnini[a], Marcos Arroyo[b], David Abadìas[b]

[a]*Department of Civil and Environmental Engineering, University of Perugia, via G. Duranti 93, 06125 (PG), Italy*
[b]*Department of Civil and Environmental Engineering, UPC Barcelonatech, Campus Nord C Jordi Girona 1-3, 08034, Barcelona*

Abstract

In this paper, the hypoplastic macroelement formulation proposed by [1] has been modified in order to extend its range of applicability to offshore structures subject to cyclic loads with very high number of cycles, with particular attention to fatigue phenomena and cyclic displacement accumulation. A series of FE analysis has been performed to model the soil–foundation interaction processes of a prototype of offshore wind turbine, for which the geometrical characteristic of the superstructure and foundation, the soil conditions and the predicted environmental (wave and wind) loads were known. The study, carried out in parametric form, has allowed to better understand the role played by the modified cyclic part of the macroelement model in reproducing the shake–down effects as observed in small–scale model tests.

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Keywords: Offshore structure; FE analysis; macroelement; SSI analysis

1. Introduction

For shallow isolated foundations, a substantial progress towards an efficient and reliable approach for the analysis of SSI (Soil–Structure Interaction) problems has been achieved by the development of the so–called macroelements

* Corresponding author. Tel.: +39-075-585-3957; fax: +39-075-585-3756.
E-mail address: diana.salciarini@unipg.it

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for describing the overall behavior of the foundation-soil system.

Approaches based on macroelements include the behavior of shallow isolated footings and of the interacting soil mass in a single (non–linear, inelastic and history–dependent) constitutive equation, formulated in the generalized loading space. For their accuracy in reproducing the main features of the foundation–soil system under complex loading paths and for their high computational efficiency, macroelements appear particularly well suited in the SSI analyses of special structures, such as high–rise towers, bridge piers or offshore platforms.

Macroelements, originally proposed by [2] in the framework of the theory of elastoplasticity, have been recently extended in the framework of the hypoplastic theory [1,3,4].

In this paper, the 6 degrees–of–freedom macroelement proposed by [3], implemented in the Finite Element code Abaqus v6.10 for modeling SSI problems, has been extended to deal with practical cases where a large number of cycles in the loading conditions are applied to the structure. The presented case study is based on a prototype with well–defined geometrical characteristics. The study has been carried out in parametric form to understand the role played by the modified cyclic part of the macroelement in reproducing the shake–down effects.

2. The hypoplastic macroelement

2.1. Basic model

In the macroelement approach, the mechanical response of the foundation–soil system under general 6–dimensional loading conditions is described by means of a constitutive equation relating the generalized load vector **t** to the generalized displacement vector **u**:

$$\mathbf{t} = \frac{1}{V_0}\mathbf{T} = \frac{1}{V_0}\left\{\mathbf{V}, \mathbf{H}_x, \frac{\mathbf{M}_x}{d}, \mathbf{H}_y, \frac{\mathbf{M}_y}{d}, \mathbf{Q}\right\} \qquad \mathbf{u} = \frac{1}{V_0}\mathbf{U} = \frac{1}{V_0}\left\{\mathbf{U}_z, \mathbf{U}_x, \boldsymbol{\theta}_y d, \mathbf{U}_y d, \boldsymbol{\theta}_x d, \boldsymbol{\Omega}\right\} \tag{1}$$

where **V**, $\mathbf{H}_x$, $\mathbf{H}_y$, $\mathbf{M}_x$, $\mathbf{M}_y$ and **Q** are the resultant forces and moment acting on the foundation; **U**x, **U**y, **U**z, θ_x, θ_y, and Ω are the conjugated displacements and rotations; *d* is a characteristic length introduced for dimensional consistency (foundation diameter) and V_0 is the bearing capacity of the foundation under centered vertical loading.

In order to reproduce correctly some important features of the experimentally observed behavior of the foundation–soil system (such as nonlinearity, irreversibility and dependence from past loading history), the constitutive equation for the macroelement must be formulated in the following rate–form :

$$\dot{\mathbf{t}} = \mathbf{K}\left(\mathbf{t}, \mathbf{q}, \boldsymbol{\eta}\right)\dot{\mathbf{u}} \tag{2}$$

where $\dot{\mathbf{u}}$ is the generalized velocity vector; **K** is the tangent stiffness of the system, depending on the current stress state **t** of the system; **q** is a pseudo–vector of internal variables accounting for the effects of previous loading history; and $\boldsymbol{\eta} = \dot{\mathbf{u}} / \left\|\dot{\mathbf{u}}\right\|$ is the direction of the generalized velocity. In the hypoplastic macrolement developed by [1] the tangent stiffness tensor **K** appearing in Eq. (2) has the following basic structure:

$$\mathbf{K} = \left(\mathbf{L}\boldsymbol{\eta}\right)\boldsymbol{\eta}^T + \mathbf{N}\boldsymbol{\eta}^T \tag{3}$$

and, differently from elastoplasticity, **K** varies continuously with the direction of the generalized velocity $\boldsymbol{\eta}$. This property is known as “incremental nonlinearity”, and it allows the macroelement to model the irreversible response. Therefore, the basic version of the hypoplastic macroelelement can be recast in the following equivalent form:

$$\dot{\mathbf{t}} = \mathbf{L}\dot{\mathbf{u}} + \mathbf{N}\left\|\dot{\mathbf{u}}\right\| \qquad \Rightarrow \quad \mathbf{N}\left(\mathbf{t}, V_f\right) = -Y\left(\mathbf{t}, V_f\right) \cdot \mathbf{L} \cdot \mathbf{m}\left(\mathbf{t}\right) \tag{4}$$

where Y and $\mathbf{m}$ are respectively defined as the loading function and the flow direction vector (see [1,3] for further details).

2.2. Extension to cyclic loading paths

An extended version of the basic model suitable for cyclic loading conditions can be obtained by including in the set of the internal variables, the so–called "internal displacement vector" $\boldsymbol{\delta}$, which mimics the concept of the "intergranular strain" introduced by [5] for continuum hypoplasticity. Under cyclic load paths the constitutive equation of the enhanced version of the macroelement is given by:

$$\dot{\mathbf{t}} = \mathbf{K}^{hp}\left(\mathbf{t},\mathbf{d},\mathbf{q}\right)\dot{\mathbf{u}} \qquad \mathbf{K}^{hp}\left(\mathbf{t},\mathbf{d},\mathbf{q}\right) = \left[\rho^{\chi}\mathbf{m}_T + \left(1-\rho^{\chi}\mathbf{m}_R\right)\right]\mathbf{L} + \tilde{\mathbf{K}}\left(\mathbf{t},\mathbf{d},\mathbf{q}\right) \tag{5}$$

where $\mathbf{m}_R$, $\mathbf{m}_T$ and χ are model constant. The evolution equation for the internal displacement is given by:

$$\dot{\boldsymbol{\delta}} = \begin{cases} \left(\mathbf{I} - \rho^{\beta_r}\boldsymbol{\eta}_\delta\boldsymbol{\eta}_\delta^T\right)\dot{\mathbf{u}} & \left(\boldsymbol{\eta}^T \cdot \boldsymbol{\eta}_\delta > 0\right) \\ \dot{\mathbf{u}} & \left(\boldsymbol{\eta}^T \cdot \boldsymbol{\eta}_\delta > 0\right) \end{cases} \tag{6}$$

where the unit vector $\boldsymbol{\eta}_{\boldsymbol{\delta}}$ gives the direction of the internal displacement vector; $\mathbf{I}$ is the identity matrix; β_r is a model constant that controls the velocity with which the internal displacement decreases with the increase of the scalar ρ [0; 1]; and ρ is a normalized measure of the internal displacement magnitude defined as:

$$\rho = \frac{\left\|\boldsymbol{\delta}\right\|\mathbf{M}}{R} = \frac{\sqrt{\delta^T \cdot \mathbf{M} \cdot \boldsymbol{\delta}}}{R} \qquad \mathbf{M} = diag\left\{1; w_H; w_M; w_H; w_M; w_Q\right\} \tag{7}$$

where $\mathbf{M}$ is a diagonal matrix of coefficients that account for the geometry of the failure surface; R represents the size of the region in which the behavior of the system is quasi–linear and is defined "elastic nucleus radius".

2.3. Modeling the shake-down effect for a large number of cycles

Slender structures, such as offshore wind turbine, subject to external repeated loads may suffer a crisis for accumulation of unlimited plastic deformations (incremental collapse, ratcheting) and for the recurrence of plastic deformation of opposite sign. However, it has been experimentally observed that such kind of structures, after the development of plastic deformations with the first cycles of load, tend to exhibit a more stable behavior in the following, due to the so–called *shakedown* phenomenon.

To model such plastic adaptive behavior, avoiding excessive ratcheting and reproducing the *shakedown* effects, a change in the mathematical formulation of the macroelement has been introduced. Specifically, in the considered hypoplastic macroelement proposed by [3], a new definition for the parameter representing the elastic nucleus R has been introduced, allowing R to progressively enlarges during the repetition of cycles, following the law:

$$R(n) = R_{\min} + \left(R_{\max} - R_{\min}\right)\left[1 - \exp\left(-\frac{n-1}{\lambda}\right)\right] \tag{8}$$

where: n is the number of cycles; $R_{\min}$ is the initial value of R (for n = 1); $R_{\max}$ is the maximum asymptotic value of R; and λ is a model constant that describes the decay of the exponential function. We call β the ratio of $R_{\max}$ over $R_{\min}$ and it provides a measure of how the elastic nucleus radius is modified by the number of the cycles.

3. The considered case study

3.1. Characteristics of the wind turbine

The case study considered in this paper uses a well–known reference multi–megawatt turbine defined by NREL. This is a horizontal axis turbine with three blades and an upwind rotor with a power class of 5 MW [6]. The foundation adopted here for the NREL 5MW is a Gravity Base Structure (GBS), designed according the model proposed for the Thorthon Bank wind farm [7]. The GBS has a very simple configuration of a concrete formwork with a thickness of 0.5 m, ballasted with coarse filling material. The base diameter of the foundation is 20 m, and the embedment is 2 m under the ground surface.

The adopted soil properties were obtained from the survey conducted for the construction of the Kriegers Flak windfarm, located in the Baltic Sea between Germany, Denmark and Sweden [8]. The geotechnical investigations revealed a stratigraphic profile consisting in a homogeneous dense sand layer of very large thickness, with a friction angle of 35°, a dynamic shear stiffness G_0 = 107.3 MPa, and a Poisson's ratio ν = 0.3.

3.2. Loading conditions

In general, an offshore wind turbine is subjected to: (*i*) the actions of self–weight (superstructure and foundation), responsible for the vertical component V of the load, (*ii*) the wind, and (*iii*) the wave loads. All these loads are dynamic in nature and due to the non–linear servo–aroelastic response are typically evaluated in time domain. In order to obtain a servo–aerodynamic response, the wind action on the tower and on the flexible part of the concrete GBS has been evaluated by the open source software FAST [9], assuming fixity at the interface with the rigid part of the GBS. Wave loading is separately computed by a MATLAB subroutine that, following the IEC 61400-3 [10], generates a stochastic sea state in agreement with the Jonswap spectrum and computes hydrodynamic forces on the foundation using Morison's formula with Mc-Camy Fuchs correction [11]. The superposition of the history of the two loads during time has been computed at the soil–structure interface. Given the design load case of IEC 61400-3, namely DLC 1.6, which describes a situation of power production with a severe sea state (1 year return period), three wind scenarios have been chosen: (i) reference wind speed 8.0 m/s, wave height 3.6 m; (ii) reference wind speed 11.4 m/s, wave height 3.6 m; and, (iii) reference wind speed 15.0 m/s, wave height 3.6 m (Fig. 1).

4. Numerical simulation

Given the geometry of the foundation of the considered case study (D = 20 m), the macroelement is completely defined by constants that can be subdivided into 4 main groups: (i) 7 constants which define the elastic behavior (k_v, k_h, k_m, k_q, k_c, m_R and m_T) [note that no torsion has been considered in this application, thus they reduce to 6]; (ii) 7 constants which define the shape of the boundary surface (V_0, h_0, m_0, q_0, β_1, β_2, and a); (iii) 5 constants which define the law of the hypoplastic flow $\mathbf{m}$ (α_h, α_m, α_q, β_3 and β_4) [note that no torsion has been considered in this application, thus they reduce to 4]; (iv) 5 constants which define the hardening law for V_0 (k_1, w_1, w_2, c_1, c_2, c_3) [note that no hardening for V_0 has been considered in this application, thus they reduce to 0]; and, (v) 4 constants that control the non-linearity degree and the evolution law for (k, R, β_r and χ).

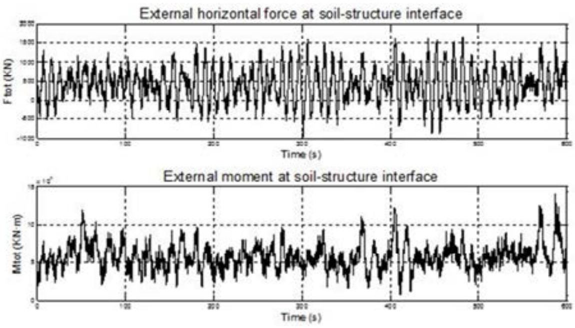

Fig. 1. External horizontal force and moment at the soil–structure interface, for a wind speed of 15.0 m/s.

The numerical values for the 21 model constants adopted in the numerical simulations are given in Table 1, obtained as deeply discussed in previous papers by [1], [3], [4] and [12].

Table 1. Model constants adopted in the numerical simulations.

k_v (-)	k_h (-)	k_m (-)	k_c (-)	m_R (kPa)	m_T (kPa)	V_0 (kN)	h_0 (-)	m_0 (-)	q_0 (-)	β_1 (-)
2.857	2.353	0.476	0.0	3.0	1.5	1145.3	0.14	0.095	0.14	1.0
β_2 (-)	a (-)	α_h (-)	α_m (-)	β_3 (-)	β_4 (-)	k (-)	R (m)	β_r (-)	χ (-)	
0.95	0	1.25	3.467	1.0	0.95	1.4	5.0e-4	1.0	1.5	

4.1. Results

A more complete description of the research is presented in [13], where all the considered combinations of loading conditions are shown, alongside a sensitivity analysis on the variation of the parameters controlling the cyclic behavior of the system (k, R, β_r and χ). In this paper, the discussion of the results is limited to the effect produced by the newly implemented shakedown effect on the model response.

To this aim, two sensitivity analyses by varying λ and β have been carried out. In the first analysis, λ has been kept constant, and β has been varied between 1 and 100. Figure 2 shows the results for the maximum value of the load acting, with a reference wind speed equal to 15.0 m/s.

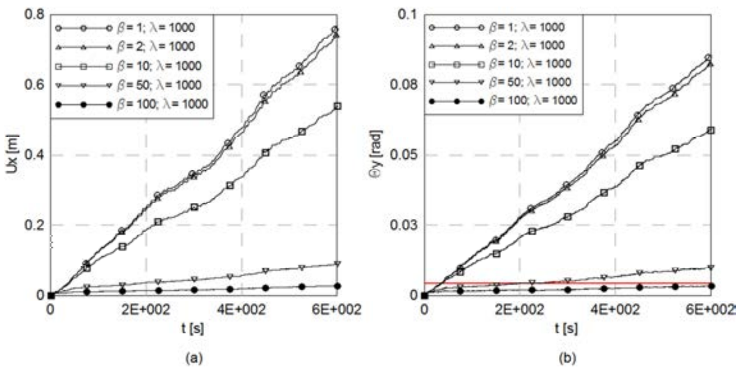

Fig. 2. Model response varying β, considering wind velocity of 15.0 m/s and wave height of 3.6 m: (a) horizontal displacements; (b) rotations.

It is evident that, for $\beta = 1$ (no shakedown), the system gradually accumulates very high horizontal displacements U_x and rotations θ_y with a strong ratcheting effect. Increasing the values of β the system shows a gradual reduction of horizontal displacements U_x and rotations θ_y accumulated over time. Indeed, the curves show a progressive reduction of their slope and an overall trend that tends to stabilize for increasing number of cycles. In serviceability conditions the maximum allowable rotation of the foundation is in a range between 0.25–0.50 radians (red line in Fig. 2b). In the second parametric analysis the parameter β has been kept constant, while λ has been varied form 500 to 2000. Figure 3 shows the results for the maximum value of the load acting with a reference wind speed equal to 15.0 m/s. In this case, for a fixed value of β equal to 100, the increase of λ has an effect only in the first load cycles. Greater values of λ induce a greater accumulation of horizontal displacements U_x and rotations θ_y, after which the system provides a constant rate of accumulation and tends to stabilize.

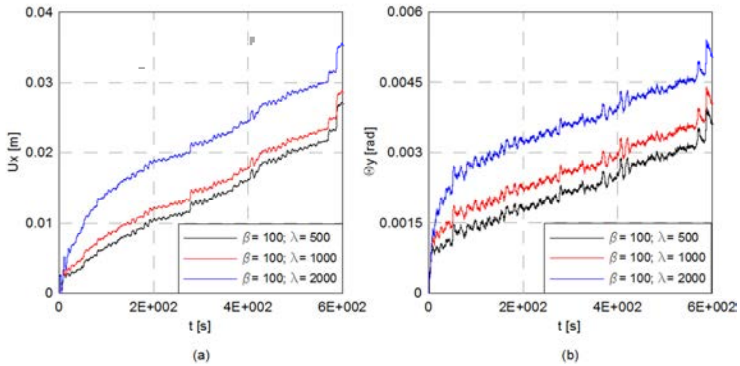

Fig. 3. Model response varying λ, considering wind velocity of 15.0 m/s and wave height of 3.6 m: (a) horizontal displacements; (b) rotations.

5. Conclusion

This work has illustrated the potential of an extended macro-element formulation to deal with the complex dynamic load history imposed on the foundation by offshore-wind structures. This opens the way for the integration of soil-structure interaction in the design process of such structures. The apparent complexity of the model should be put into perspective, as the dominant parameters may be calibrated back-analyzing structural performance data.

References

[1] D. Salciarini, C. Tamagnini. A hypoplastic macroelement model for shallow foundations under monotonic and cyclic loads. Acta Geotechnica 4 (2009) 163-176.

[2] R. Nova, L. Montrasio. Settlements of shallow foundations on sand. Geotechnique 41 (1991) 243–256.

[3] C. Tamagnini, D. Salciarini, R. Ragni. Implementation of a 6-DOF macroelement in a finite element code. In Proc. of International Conference on Computational Geomechanics - Comgeo III, at Cracovia (Poland). 2014

[4] D. Salciarini, B. Bienen, C. Tamagnini. Incorporating scale effects in shallow footings in a hypoplastic macroelement model. In Proc. of Numerical Methods in Geotechnical Engineering – Hicks, Brinkgreve & Rohe (Eds), Taylor & Francis Group, London (2014) 397-402.

[5]A. Niemunis and I. Herle. Hypoplastic model for cohesionless soils with elastic strain range. Mechanics of Cohesive-Frictional Materials, 2(4) 279-299 1997.

[6] S. Butterfield, W. Musial, G. Scott. Definition of a 5-MW reference wind turbine for offshore system development. 2009.

[7] K. Peire, H. Nonneman, E. Bosschem. Gravity based foundations for Thornton Bank Offshore Wind Farm. Terra et Aqua, 115 (2009) 19-29

[8] L. Bülow, L. Jorgensen, H Gravessen. Kriegers Flak Offshore Wind Farm. Basic data for conceptual design of foundations. Vattenfall Vindkraft AB. 2009

[9] J.M. Jonkman, L. Marshall Buhl Jr., FAST User's Guide, USA, NREL/EL-500-38230, Golden, CO: National Renewable Energy Lab. 2005.

[10] IEC 61400-3 (2009) International standard Wind turbines–Part 3: Design requirements for offshore wind turbines. International Electrotechnical Comission

[11] N. Hogben. Wave Loads on Structure, Behaviour of Offshore Structures (BOSS), Norwegian Institute of Technology, Oslo. 1976.

[12] G. Buscarnera, R. Nova, M. Vecchiotti, C. Tamagnini, D. Salciarini. Settlement analysis of wind turbines. In R. P. Orense, N. Chouw, & M. J. Pender (Eds.), Proc. Int. Workshop on Soil–Foundation–Structure Interaction (SFSI 09), Auckland, New Zealand. CRC press. 2010.

[13] D. Salciarini, M. Frizza, C. Tamagnini, M. Arroyo, D. Abadìas. Extending the macroelement formulation to reproduce the SSI effects in structures subject to large number of loading cycles. (in prep.)