

RESEARCH REPORT III

Pile foundations under lateral cyclic actions. FEM numerical modelling using 3D CESAR-LCPC

Doctorate domain: Civil Engineering and Installations
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Made by:
PhD Eng. Andrei-Valentin DRĂGUȘIN

Doctorate leader,
Dr. Eng. univ. prof. Loretta BATALI

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Research purpose

Using FEM numerical modelling for the design and analysis of civil engineering structures is a relatively recent approach method. These methods are also applicable in the geotechnical engineering domain, even for pile foundations submitted to lateral cyclic loading.

One can consider that these methods are offering complete solutions, thus an ideal situation in order to study such difficult problem. The FEM numerical modelling advantages are multiples, the most important being perhaps that one can model soil-pile interaction and consider complex constitutive model for soil behaviour. There are though some disadvantages when choosing such approach, as for instance the need of a large design practice experience or the long term use of certain FEM software.

The numerical modelling of pile-soil interface under cyclic loading is a difficult task in which, besides the fact that one shall consider the irreversibility of soil plastic strain, the numerical model must consider the accumulation of displacements on pile head with cycles. These cumulated cycles could lead to soil's failure by degrading its resistance, to high pile head displacements or to considerable bending moments in the pile.

An optimal numerical modelling is mainly influenced by soil behaviour, especially through the prism of complex parameters which one has to consider in the respective software. An efficient numerical modelling of such problem (pile foundations under lateral cyclic action) is in direct link with the choice of the soil constitutive model.

The purpose of the present research paper is to realize a FEM numerical modelling using the software CESAR-LCPC regarding the behavior of an isolated pile in sand submitted to lateral cyclic loading (which can simulate in reality for instance, an offshore windmill monopile submitted to cyclic action coming from waves of wind).

The license for CESAR-LCPC software has been made available through the amiability of Dr. Eng. David Remaud. The experimental results used for calibrating the numerical model were obtained from Dr. Eng. Luc Thorel – IFSTTAR Nantes.

The research report is structured in 3 main sections, namely:

Section I – 3D Numerical model calibration based on small-scale centrifuge tests

Section II – Parametrical studies regarding the cyclic parameters influence (amplitude and number of cycles)

Section III – Parametrical studies regarding the geotechnical parameters of the soil (volumic weight, internal friction angle and deformation modulus).

Section I

3D Numerical model calibration based on some experimental tests on small-scale centrifuged model

CHAPTER 1 – Short description of centrifuge tests used for calibration

Centrifuge tests performed by Rosquoët (*Rosquoët, 2004*) have been presented previously in research report II and from these tests some have been used for calibrating the numerical model.

The scale chosen for the tests was 1:40, implying a centrifugal acceleration of 40g. The main geometrical and mechanical characteristics for the prototype model are the following: (*Rosquoët, 2004*):

Prototype pile	Model pile
Tubular steel pile	Tubular aluminium pile
Pile length below soil surface $D = 12$ m	Pile length below soil surface $D = 300$ mm
Outer diameter $B = 0.72$ m	Outer diameter $B = 18$ mm
Inner diameter $b = 0.685$ m	Inner diameter $b = 15$ mm
Lateral force applied at 1.6 m above soil surface	Lateral force applied at 40 mm above soil surface
Young Modulus for steel $E_p = 2 \times 10^{11}$ Pa	Young Modulus for aluminium $E_m = 7.4 \times 10^{10}$ Pa
Inertial moment $I_p = 2.38 \times 10^{-3} \text{ m}^4$	Inertial moment $I_m = 2.67 \times 10^{-9} \text{ m}^4$
Bending stiffness $E_p I_p = 476 \text{ MNm}^2$	Bending stiffness $E_m I_m = 197.43 \text{ Nm}^2$

Table 1 – Main pile characteristics [*Rosquoët, 2004*]

For passing from prototype to model the similitude have to be followed, by working in multiple gravity and by introducing a scale factor. The scale factors for the physical parameters are the following:

Physical dimension	Scale factor
Length: L	$1/N$
Displacement: ξ	$1/N$
Deformation: ε	1
Effort: σ	1
Force: F	$1/N^2$
Bending moment: M	$1/N^3$
Bending stiffness: EI	$1/N^4$

Table 2 – Scale factors in order to pass from model to prototype and viceversa [*Popa, 2013*]

The maximum force that could be applied on pile head without having plastic strains has been determined on aluminium samples and was of 600 N.

The model pile has been instrumented with 20 pairs of transducers attached in semi-bridge along its length. On the pile head have been also attached 2 transducers for measuring the displacements. On the last 15 mm, in the tip area, the pile hasn't been instrumented and its lateral displacement has been considered to be 0. For this type of model pile there have been previously performed a series of specific tests for determining the lateral displacement in the tip area. *Remaud (1999)* has found that the lateral displacement of the tip is of max 0.08% of its diameter for 600 N force, therefore it can be neglected. Also, in order to have a zero bending moment for the pile head a special

attention has been paid on how the force has been applied so that no parasite moments appear.

The used soil for centrifuge tests was Fontainebleau sand for which the main geotechnical parameters have been determined by *MécaSol* (1996).

- $\rho_s = 2.65 \text{ g/cm}^3$;
- $\rho_{dmax} = 1.68 \text{ g/cm}^3$;
- $\rho_{dmin} = 1.41 \text{ g/cm}^3$;
- $e_{max} = 0.887$;
- $e_{min} = 0.581$.

Rosquoët (*Rosquoët, 2004*) choosed to work for his experiments with 3 values for the unit weight of $\gamma = 15.1, 16 \text{ și } 16.5 \text{ kN/m}^3$, corresponding to a relative density of $I_D = 53\%, 86\% \text{ și } 100\%$, according to the formula:

$$I_D = [\gamma_{dmax} (\gamma_d - \gamma_{dmin})] / [\gamma_d (\gamma_{dmax} - \gamma_{dmin})] \quad [1]$$

To be noted that the sand has been worked by pluviation in order to obtain the required relative density and an homogeneous mass.

From the more than 30 tests performed by Rosquoët (*Rosquoët, 2004*), for the calibration of the numerical model developed using CESAR-LCPC, only 3 have need retained so that having a variation for the amplitude, relative density, internal friction angle and deformation modulus. These results have been made available by IFSTTAR in January 2017. The main characteristics of these 3 tests are the following:

Parameter	P355	P354	P323
Maximum force F (N)	600	600	600
Number of cycles n (-)	15	15	15
Amplitude DF (N)	300	600	300
Unit weight (kN/m^3)	15.1	15.1	16.5
Relative density I_D (%)	53	53	100
Internal friction angle Φ (°)	32	32	42
Deformation modulus E (MPa)	30	30	40

Table 3 – Main characteristics of centrifuge experimental tests retained, [Rosquoët, 2004]

The data from Rosquoët regarding the geotechnical parameters of the reconstituted sand mass didn't comprise values of the internal friction angle or deformation modulus.

Therefore, the values for the internal friction angle have been approximated using Garnier's empirical formula (*Garnier, 2001*) determined based on more triaxial tests performed on Fontainebleau sand.

$$e \times \tan \Phi = 0.52, \text{ cu:} \quad [2]$$

e – void index, possible to be determined based on the relative density

$$I_D = (e_{\max} - e) / (e_{\max} - e_{\min}) \quad [3]$$

Therefore, for 100% relative density the internal friction angle resulted of $\phi = 42^\circ$, while for 53% relative density of $\phi = 32^\circ$, this latest value being confirmed by various other researchers (*Rakotonindriana, 2009 or Bourgeois, 2010*) for a relative density of approx. 50%

Regarding the deformation modulus E , its values have been chosen based on the same tests already mentioned: $E = 30$ MPa for a relative density of 53% and $E = 40$ MPa for the dense sand, respectively.

Least, but not last the values of these 2 geotechnical parameters are in good agreement with values proposed by Mississippi University (*seminar, GEOL 615, University of Mississippi*):

- * loose sand: $\Phi = 25^\circ - 30^\circ$;
- * medium-dense sand: $\Phi = 30^\circ - 35^\circ$;
- * dense sand: $\Phi = 35^\circ - 45^\circ$.
- * deformation modulus E : 10 – 45 MPa (from loosest to densest).

Anyhow, the numerical modelling performed after, have shown that the single influence of the deformation modulus E (all the other parameters being kept constant) (with values from 30 MPa to 35 MPa for example) is negligible both on displacements and bending moments for the calibration of the numerical model, if its value is within the proposed interval. On the other hand, the single influence of the internal friction angle (all other parameters being kept constant) is somehow more accentuated on displacements (when ϕ is varied from 30° to 35°) but it is of max. 5%, while for bending moments it is still negligible. These are valid for displacements and bending moments at the end of all cycles.

CHAPTER 2 – Description of the numerical model

CESAR-LCPC software (*ver. 6.0, itech 2014*) is a finite element software allowing to solve difficult geotechnical problems by using a series of complex constitutive, non-linear, plastic laws as Nova, Vermeer, Hardening soil, Drucker Prager, Tresca, Prevost-Hoeg etc.), by considering the soil – structure interaction and staged-construction. All these aspects are indispensable when analysing piles under cyclic lateral actions.

2.1. Choice of constitutive law for the soil

Cyclic lateral actions such as wind or waves represent, probably, the most complex way of action on a foundation, therefore an adequate constitutive law for the soil is required. Besides the fact that this model has to simulate the plastic behaviour of the soil, the most difficult aspect is certainly to simulate and to obtain the accumulation of displacements in the pile, named „effet de rochet”

Therefore, after a detailed study about this subject in the specialty literature, for the numerical modelling it has been chosen the Drucker-Prager law associated with a kinematic hardening law. The simple Drucker-Prager law, without associating a kinematic hardening law couldn't simulate the displacement accumulation, the bending moment increasing or the degradation of soil strength with the cycles.

It is proposed to write the plasticity criterion as following:

$$f(\underline{\sigma}) = F(\underline{\sigma} - \underline{X}), \text{ where:} \quad [4]$$

$F(\underline{\sigma})$ – initial plastic criterion of Drucker-Prager;

$\underline{X}$ – tensor variable which evolves along the cyclic action and whose role is to take into account of the plastic displacement accumulation

$f(\underline{\sigma})$ – Drucker – Prager criterion associated with kinematic hardening.

The plastic potential defining the yield law has the function considered by CESAR-LCPC software, as following

$$F(\underline{\sigma}) = (j_2)^{0.5} + \alpha \times I_1 - k, \text{ where:} \quad [5]$$

j_2 – 2nd order invariant of the deviatoric tensor, q ;

$$j_2 = I_1^2/3 - I_2, \text{ where:} \quad [6]$$

I_1 – 1st order invariant of the spheric tensor, σ ;

$$I_1 = \sigma_1 + \sigma_2 + \sigma_3, \text{ where:} \quad [7]$$

$\sigma_1, \sigma_2, \sigma_3$ – main normal stresses developed in the soil following the 3 directions, (kPa);

I_2 – 2nd order invariant of the tensor, σ ;

$$I_2 = \sigma_1\sigma_2 + \sigma_2\sigma_3 + \sigma_3\sigma_1;$$

$$\alpha = \text{tg}\Phi / (9+12\text{tg}^2\Phi)^{0.5} \text{ and } k = 3c / (9+12\text{tg}^2\Phi)^{0.5} - \text{material constants;} \quad [8]$$

Φ° and c (kPa) – shear strength parameters of the soil;

Equation [5] expressing the plasticity criterion is in agreement with the general law of the Drucker – Prager criterion:

$$(j_2)^{0.5} = A + B \times I_1, \text{ where:} \quad [9]$$

A and B – are 2 material constants similar with α and k, if Drucker – Prager surface is inscribed in the Mohr-Coulomb one.

The main problem in this moment is to determine the tensor variable X. For this, the simplest choice is a law proposed by Armstrong and Frederick (*Armstrong and Frederick, 1966*), attributed later to Chaboche (*Chaboche, 1977*), inserted into CESAR software under this name. This law proposed to write the following

$$\underline{\underline{X}} = 0.67 \times C \times \underline{\underline{\epsilon}}^p - D \times \underline{\underline{X}} \times \xi, \text{ where:} \quad [10]$$

C (kPa) – first parameter of the kinematic hardening law;

D (-) – second parameter of the kinematic hardening law;

$\underline{\underline{\epsilon}}^p$ – tensor of plastic strains;

ξ – absolute value of the rate of plastic deformation accumulation, always positive, allowing the progressive accumulation of plastic deformations.

A partial derivate equation can be written regarding the deformation tensor $\underline{\underline{\epsilon}}^p$:

$$\underline{\underline{\epsilon}}^p = \lambda \times \frac{\partial F}{\partial \sigma} (\underline{\underline{\sigma}} - \underline{\underline{X}}), \text{ where:} \quad [11]$$

λ – takes into account the multiplication of plastic deformations;

Solving this equation together with generalized Hooke's law leads to the expressions for the longitudinal deformation ϵ_1 and radial deformation ϵ_3 , respectively, function of the deviatoric stress q:

$$\epsilon_1 = -q/E + (\ln(1+(q_{el} - q) \times D/C)) / D \quad [12]$$

$$\epsilon_3 = q \times v/E - (\ln(1-(q_{el} - q) \times D/C)) / 2D \quad [13]$$

with v – Poisson's coefficient.

From the last 2 equations one can determine the maximum value of the deviatoric stress q:

$$q_{\max,ult} = q_{el} + C/D. \quad [14]$$

$$C/D = q_{\max,ult} - q_{el}. \quad [15]$$

In conclusion, it can be said that C/D ratio represents the difference between the deviatoric stress at rupture and its value at the end of elastic zone. A series of triaxial tests performed by Rakotonindriana (*Rakotonindriana, 2009*) or (*Bourgeois, 2010*) on a medium dense sand ($\gamma = 15.1 - 15.5 \text{ kN/m}^3$, $\Phi = 30^\circ - 34^\circ$, $E = 30 \text{ MPa}$) led to a deviatoric stress at rupture $q_{\max, \text{ult}}$ of 444 kPa and to a deviatoric stress at the end of elastic zone of $q_{\text{el}} = 222 \text{ kPa}$ ($q_{\text{el}} \approx 50\% \times q_{\max, \text{ult}}$). Therefore a ratio C/D of approx. 0.25 MPa resulted for a medium dense sand.

By using other triaxial tests at our disposal, made available by IFSTTAR and performed by Gaudin on the Fontainebleau sand (*Gaudin, 2002*), a value of 43 MPa for the deformation modulus and 40° for the internal friction angle were obtained, parameters confirming a very dense sand. The maximum deviatoric stress $q_{\max, \text{ult}}$ resulted to be equal to 542 kPa and if considering $q_{\text{el}} = 50\% \times q_{\max, \text{ult}}$, it results $q_{\text{el}} = 271 \text{ kPa}$ and a ratio C/D = 0.27 MPa for a very dense sand.

One can note that both for very dense and medium dense sand, the ratio C/D is around 0.25 MPa. Taking into account that this value resulted from a relative small number of tests, it cannot be concluded that C/D ratio is always equal to 0.25 MPa and this should be verified by many triaxial tests and considering various situations and parameters.

For the numerical modelling using CESAR it has been considered a value of the ratio C/D = 0.25 MPa for both relative densities considered. The choice of the effective values for these two parameters of the kinematic hardening law has been made such as the pile deformation at the end of static loading is identical in 2 different situations:

- in the first situation has been considered as constitutive model for the soil a classic Drucker-Prager criterion (static loading);
- in the second situation has been considered a Drucker-Prager constitutive law associated with Chaboche's kinematic hardening law (same static loading).

Thus, by a perfect superposition of the static deformation of the pile for these 2 situations, it resulted the following values of C and D:

$C = 5 \text{ MPa}$ and $D = 20$,

with $C/D = 5/20 = 0.25 \text{ MPa}$.

The later modelling showed that the effective values of C and D don't have an important influence on the displacement accumulation rate on pile head, therefore on the cyclic sequence, but it is the ratio C/D which is influencing this aspect.

As well, from the triaxial test at our disposal was possible to determine the Poisson's coefficient $\nu = 0.3 - 0.35$ (corresponding to the recommended value for sand in literature).

2.2. Geometry of the numerical model

Before establishing the geometry of the numerical model, a general study regarding the influence of its size and mesh density has been performed.

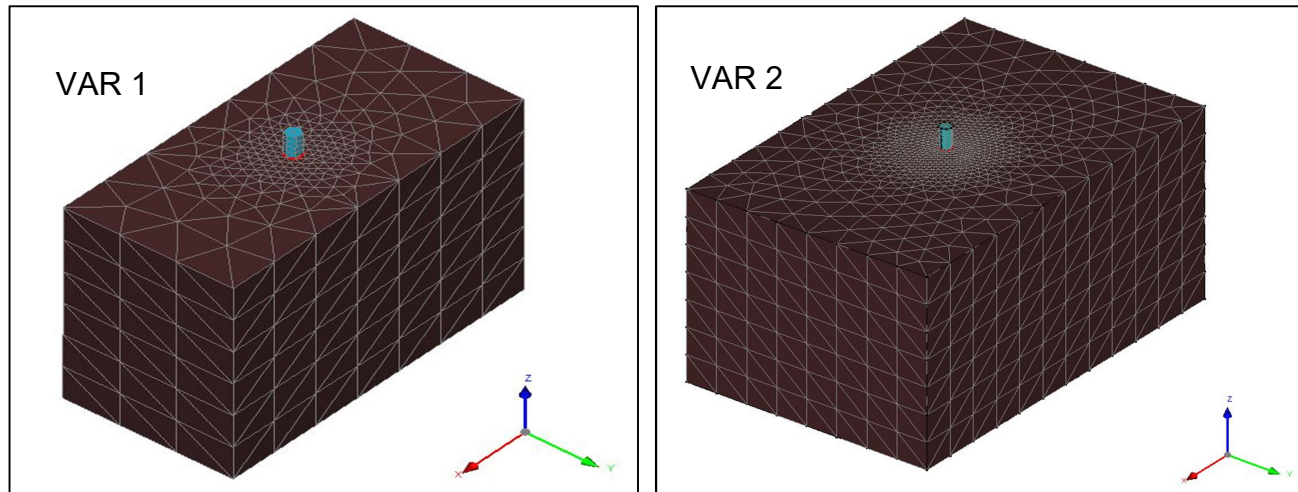


Figure 1 – 3D numerical model - [CESAR-LCPC]

Two situations have been considered:

VAR 1:

- * size of soil mass: 400 x 200 x 400 mm (xyz);
- * mesh density for the soil: 60 mm;
- * mesh density for pile and interface: 10 mm;
- * function of meshing: cubic with quadratic interpolation;
- * mesh type: TETGEN + Native;
- * total number of nodes: 31171.

VAR 2:

- * size of soil mass: 600 x 400 x 400 mm (xyz);
- * mesh density for the soil: 50 mm;
- * mesh density for pile and interface: 7 mm;
- * function of meshing: cubic with quadratic interpolation;

* mesh type: TETGEN + Native;

* total number of nodes: 85249.

In order to analyse the influence of these two situations on the results (displacements and bending moments for the pile), the following parameters have been considered:

* medium dense sand with $\Phi = 32^\circ$ și $E = 30 \text{ MPa}$;

* maximum force applied on pile head $F = 600 \text{ N}$ (small-scale value);

* amplitude $DF = 300 \text{ N}$ (small-scale value);

* 15 cycles.

The numerical analysis led to negligible differences between the results, of maximum 1% - displacements and bending moments.

Therefore, situation 1 has been chosen, meaning a numerical model 3D with 31171 nodes in order to obtain a significant reduction of the analysis time. The second situation, with 85249 nodes is obviously unjustified as it is leading to very long analysis time.

2.3. Properties of the numerical model

The numerical model has been created for the model pile. The elastic parameters of the model are the following:

* unit weight: $\gamma = 27 \text{ kN/m}^3$;

* Young's modulus: $E = 3.83 \times 10^{10} \text{ Pa}$;

* Poisson's coefficient: $\nu = 0.3$.

Young's modulus considered for the pile (aluminium) was different than the one considered in experiments, because in the numerical model the pile is a complete circular section, while in the centrifuge tests a hollow cylinder has been used.

$$I = \pi B^4 / 64 \text{ (inertia momentum for circular section)} \quad [16]$$

$$I = (\pi(B^4 - b^4) / 64) \text{ (inertia momentum for hollow section)} \quad [17]$$

with B – outer diameter of the pile (mm) and b – inner diameter of the pile (mm).

The fundamental condition is that the bending stiffness is the same in the numerical simulation and in experiments $EI = 197.43 \text{ Nm}^2$. Thus, keeping the same bending stiffness,

but changing the inertia momentum, the resulted Young's modulus for the pile in the numerical model is 3.83×10^{10} Pa.

When studying such a complex problem as pile foundation under lateral cyclic loading, soil-structure interaction is very important. Therefore, the choice of a contact element between pile and soil is very important. The chosen element was „perfect sliding”-type. For this type of interface the parameters required by CESAR software are:

- * the lowest deformation modulus between the elements in contact (soil).
- * traction strength (nil).

Regarding the properties of the soil mas, these were considered as following, according to tests P355, P354 and P323 (*IFSTTAR, Rosquoët, 2004*):

Geotechnical parameter	P355	P354	P323
Unit weight γ (kN/m ³)	15.1	15.1	16.5
Deformation modulus E (MPa)	30	30	40
Poisson's ration ν	0.33	0.33	0.33
Cohesion c (kPa)	1	1	1
Internal friction angle Φ (°)	32	32	42
1 st parameter of the hardening law C (MPa)	5	5	5
2 nd parameter of the hardening law D (-)	20	20	20

Table 4 – Sandy soil properties used in the numerical model

All 3 analyses were performed based on 15 non-alternate cycles (the same as for the experimental tests).

In general, for the first stage the pile has been submitted to a lateral monotonic load in six increments, until reaching the maximum value $F = 600$ N.

Afterwards, 30 loading – unloading cycles were applied (15 unloading and 15 loading cycles).

The numerical analysis has been carried out using a specific module of CESAR-LCPC (solving a certain mechanical problem using a non-linear behaviour - MCNL).

In addition, in order to take into account the cycles effect and to allow the accumulation of displacements on the pile head and body, a particular module of MCNL module implemented in the software has been used.

In the following figure are shown the limit conditions of the numerical model, as well as the deformations at the end of the monotonic loading stage.

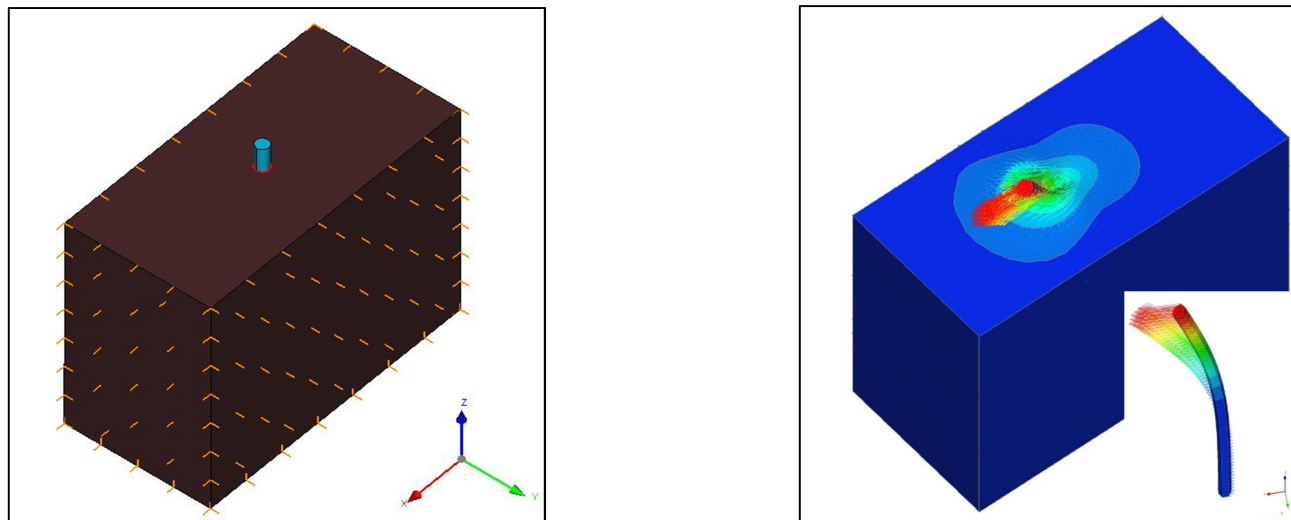


Figure 2 – **Boundary limits (left) and deformed mode (right)** - [CESAR-LCPC]

2.4. Obtained results

Here below are presented the results obtained for the 3 analyses corresponding to the 3 centrifuge tests (P355, P354 and P323) and the numerical vs. experimental values are compared. The comparison is made in terms of pile head displacement, pile deformation on its length and bending moments, especially the maximum bending moment and its depth. Of course, the soil reaction and the cyclic P-y curves are also very important for this problem as the soil – structure interaction has to be studied, but a special chapter in the final PhD thesis will be devoted to this aspect.

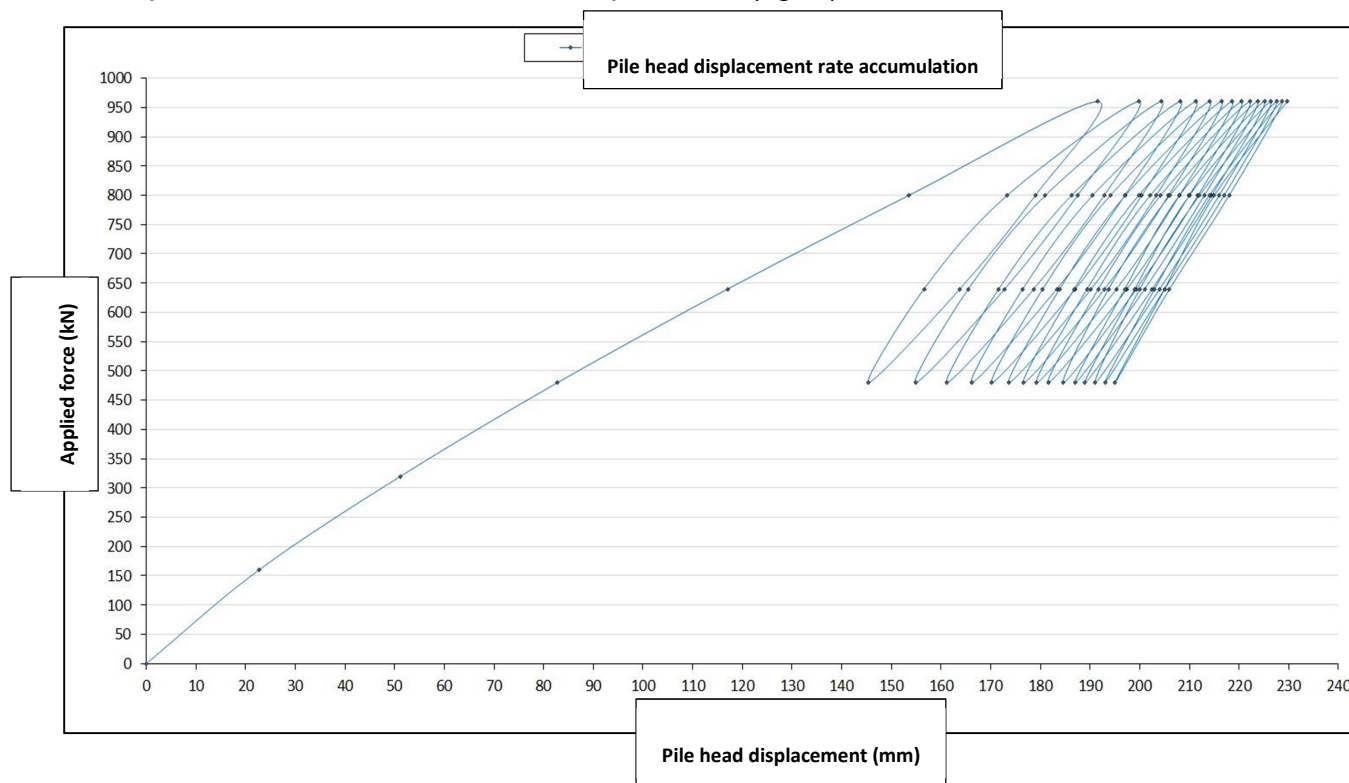
2.4.1. Test P355

Here below is presented a synthesis of the 3D numerical modelling using CESAR-LCPC:

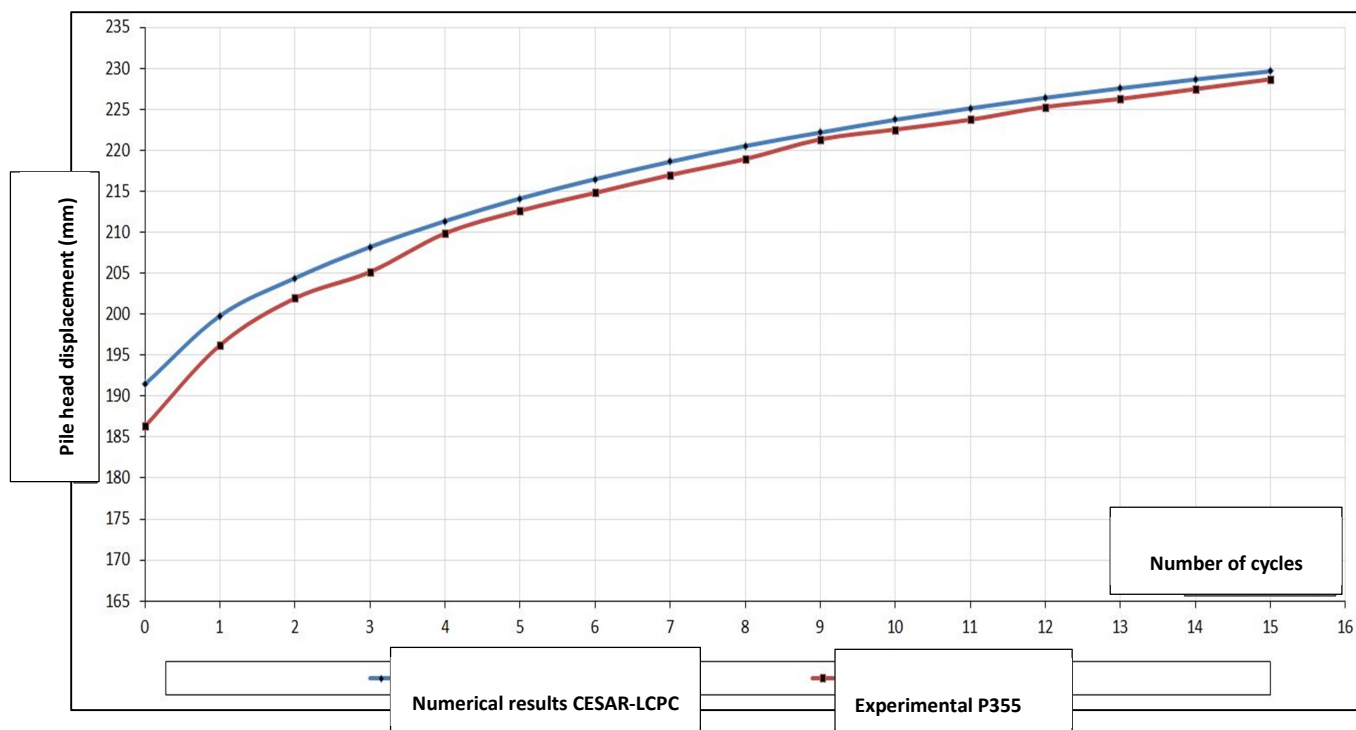
Cyclic parameters		Soil geotechnical parameters		Pile elastic parameters	
Maximum lateral force F (N)	600	Unit weight γ (kN/m ³)	15.1	Unit weight γ (kN/m ³)	27
Amplitude DF (N)	300	Deformation modulus E (MPa)	30	Deformation modulus E (Pa)	3.83×10^{10}
Number of cycles n	15	Poisson's ratio ν	0.33	Poisson's ratio	0.3
Cyclic loading type	Nonalt.	Cohesion (kPa)	1		
		Internal friction angle Φ (°)	32		
		1 st parameter of the hardening law C (MPa)	5		
		2 nd parameter of the hardening law D (-)	20		

Table 5 – **Numerical simulation P355**

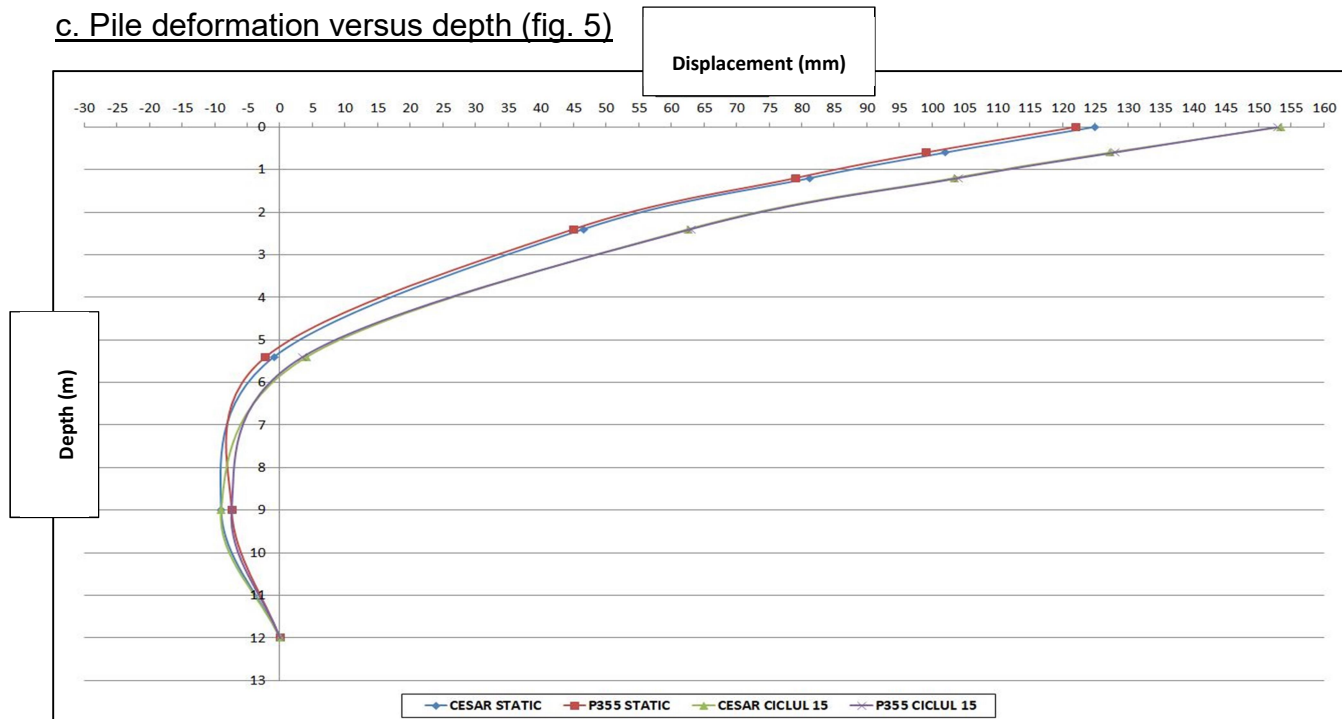
a. Displacement accumulation rate on pile head (fig. 3)



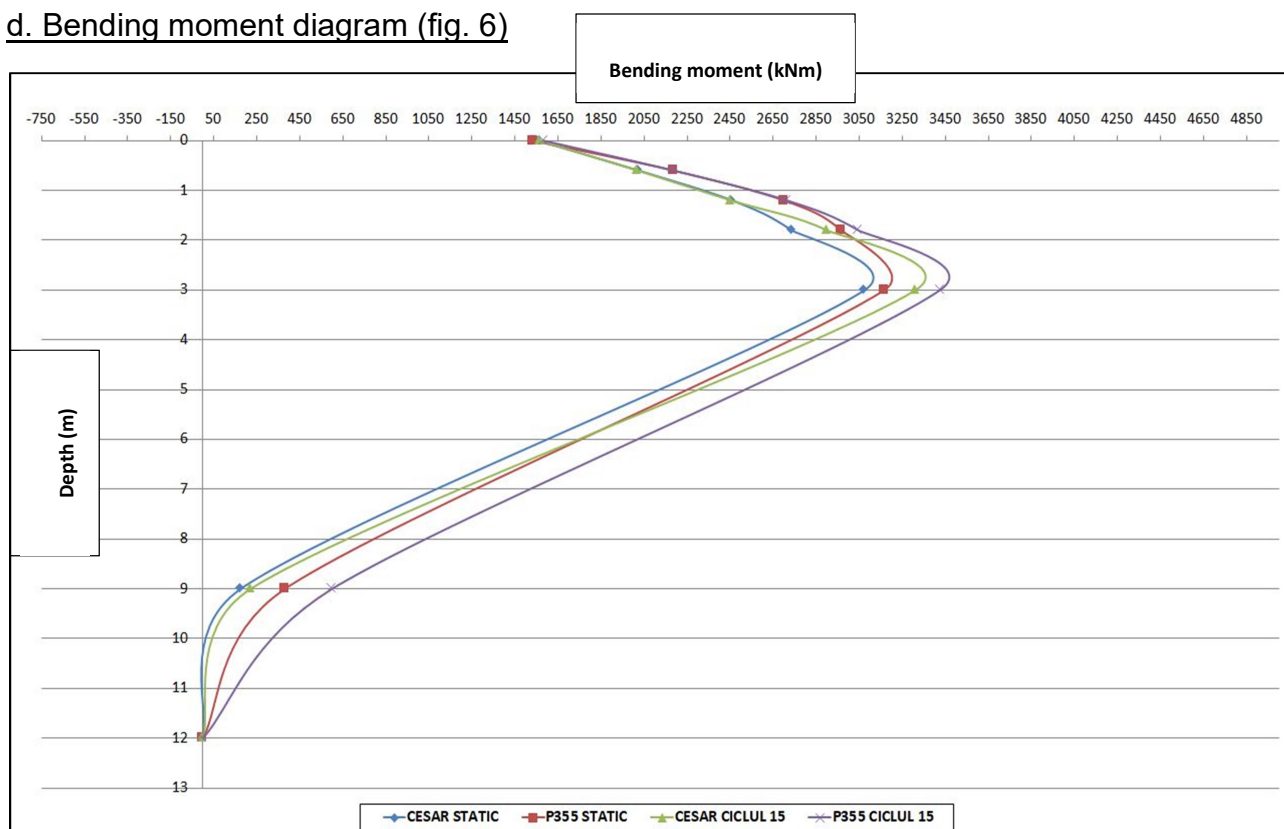
b. Displacement of pile head along the cycles (fig. 4)



c. Pile deformation versus depth (fig. 5)



d. Bending moment diagram (fig. 6)



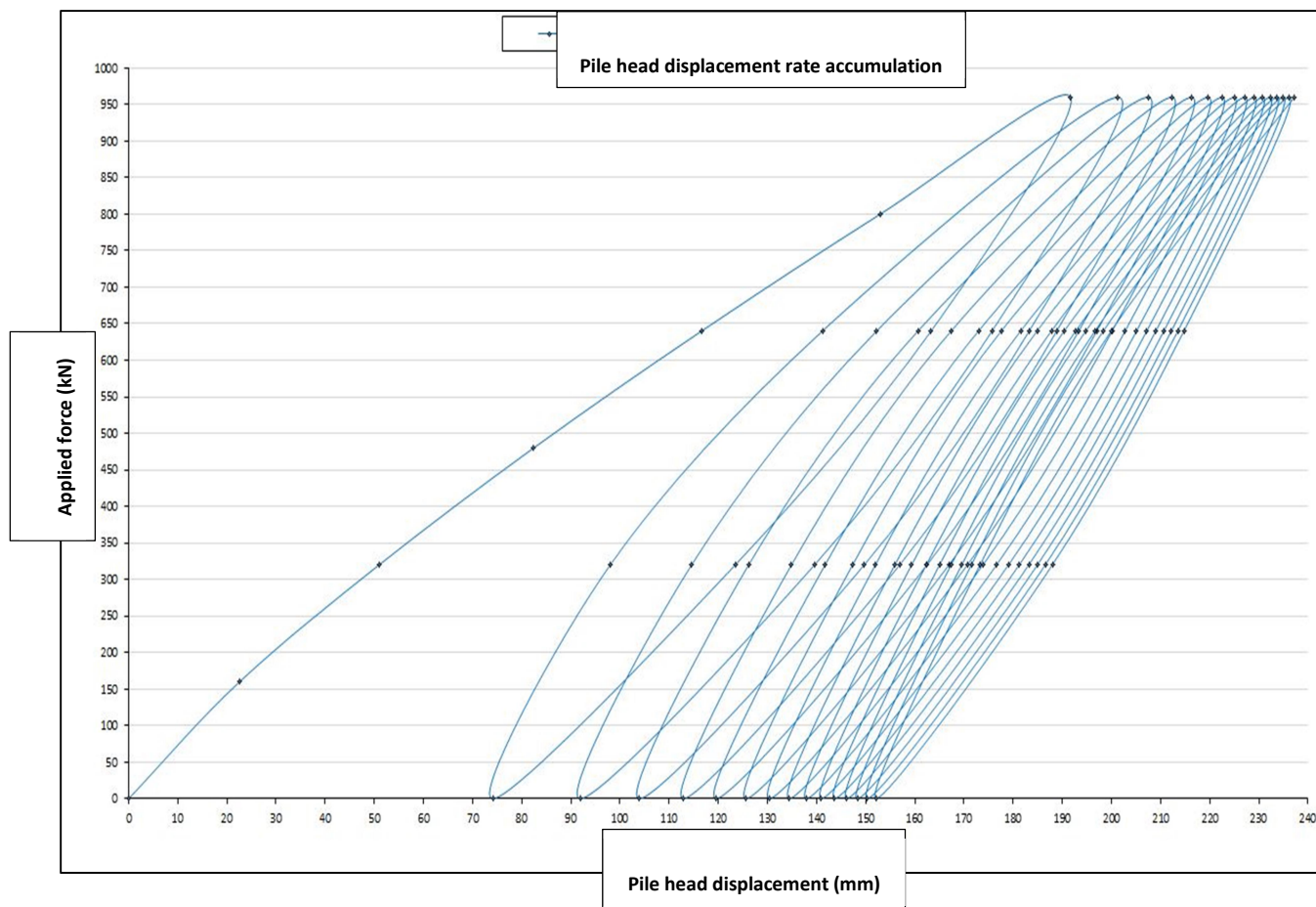
2.4.2. Test P354

Here below is presented a synthesis of the 3D numerical modelling using CESAR-LCPC:

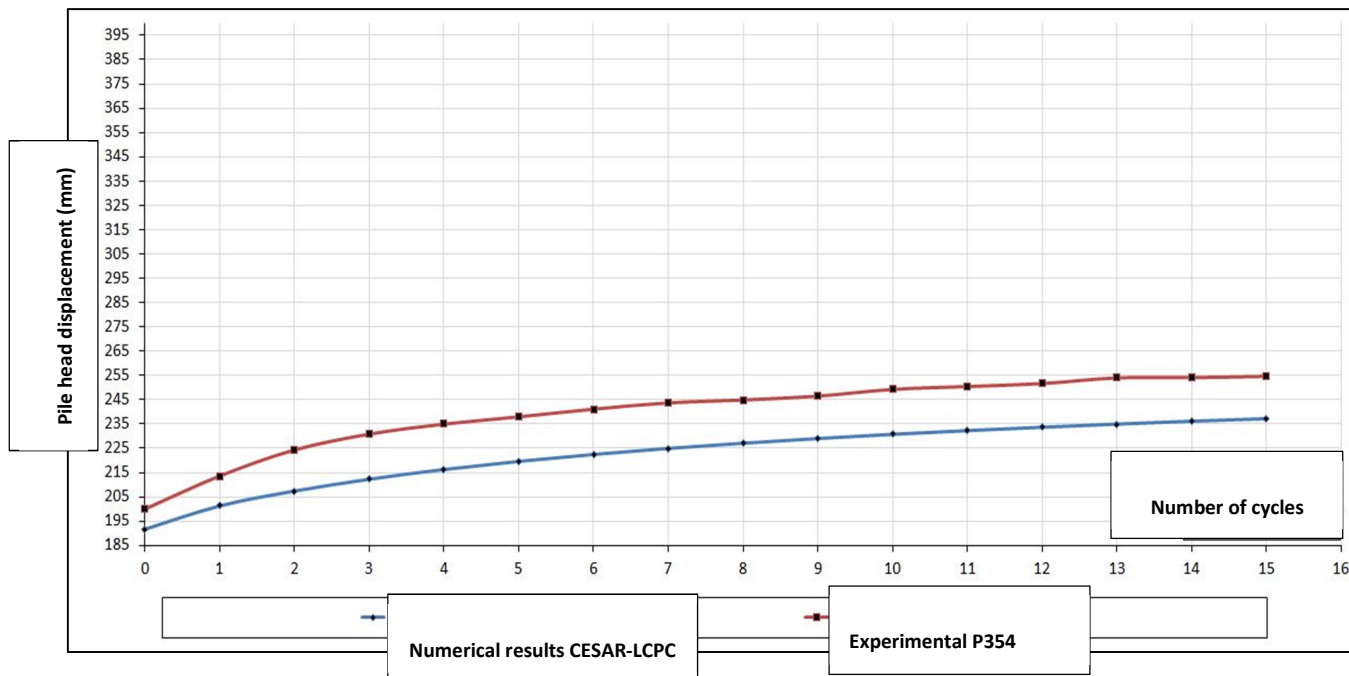
Cyclic parameters		Soil geotechnical parameters		Pile elastic parameters	
Maximum lateral force F (N)	600	Unit weight γ (kN/m ³)	15.1	Unit weight γ (kN/m ³)	27
Amplitude DF (N)	600	Deformation modulus E (MPa)	30	Deformation modulus E (Pa)	3.83×10^{10}
Number of cycles n	15	Poisson's ratio ν	0.33	Poisson's ratio	0.3
Cyclic loading type	Nonalt.	Cohesion (kPa)	1		
		Internal friction angle Φ (°)	32		
		1 st parameter of the hardening law C (MPa)	5		
		2 nd parameter of the hardening law D (-)	20		

Table 6 – Numerical simulation P354

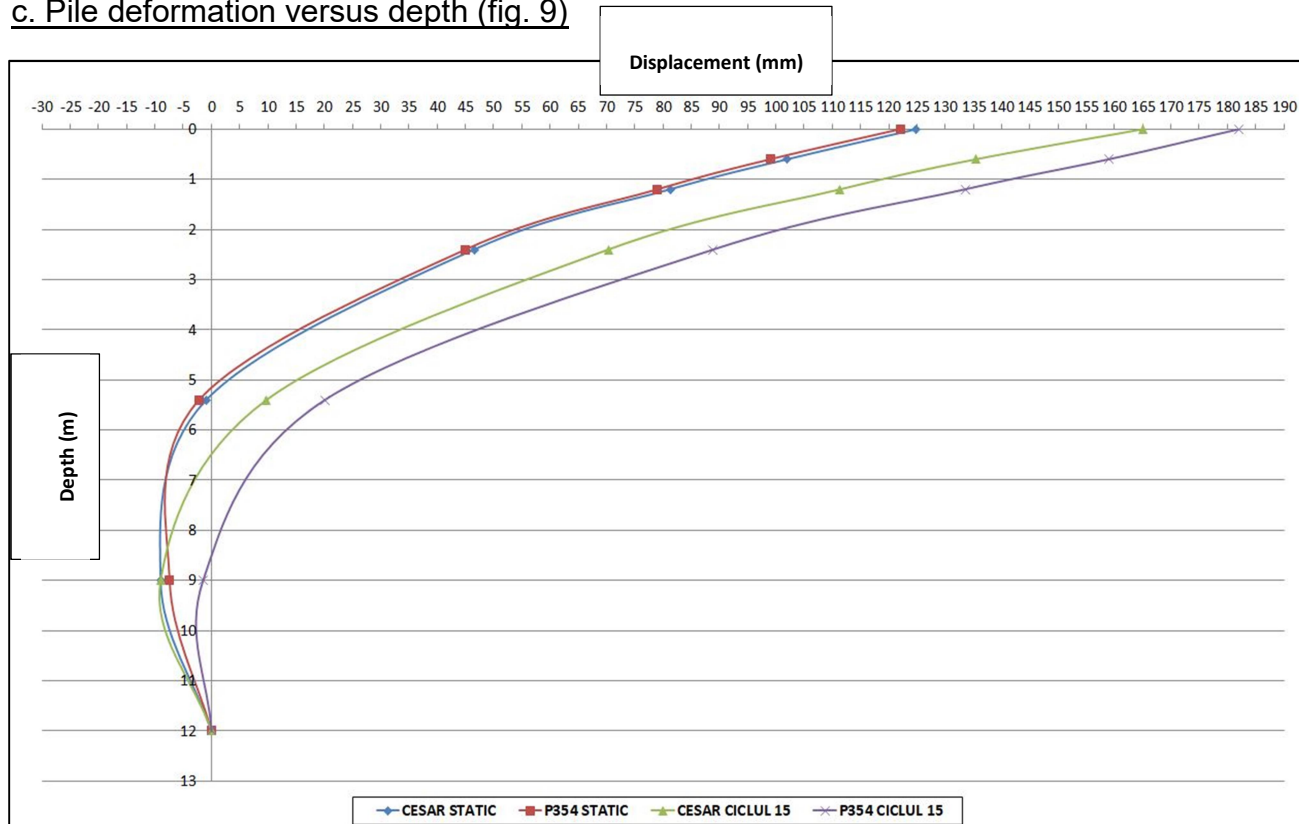
a. Displacement accumulation rate for pile head (fig. 7)



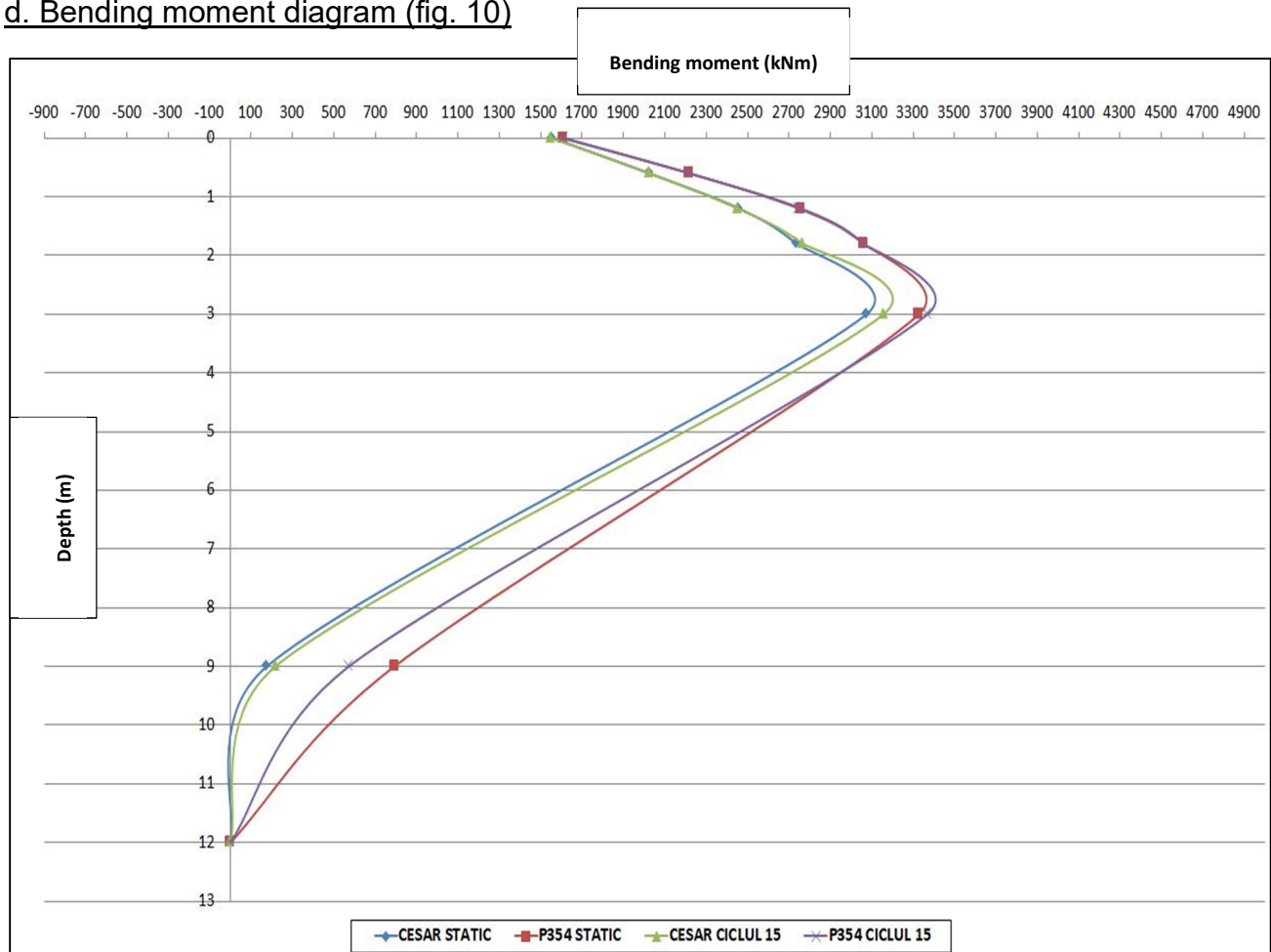
b. Displacement evolution for pile head (fig. 8)



c. Pile deformation versus depth (fig. 9)



d. Bending moment diagram (fig. 10)



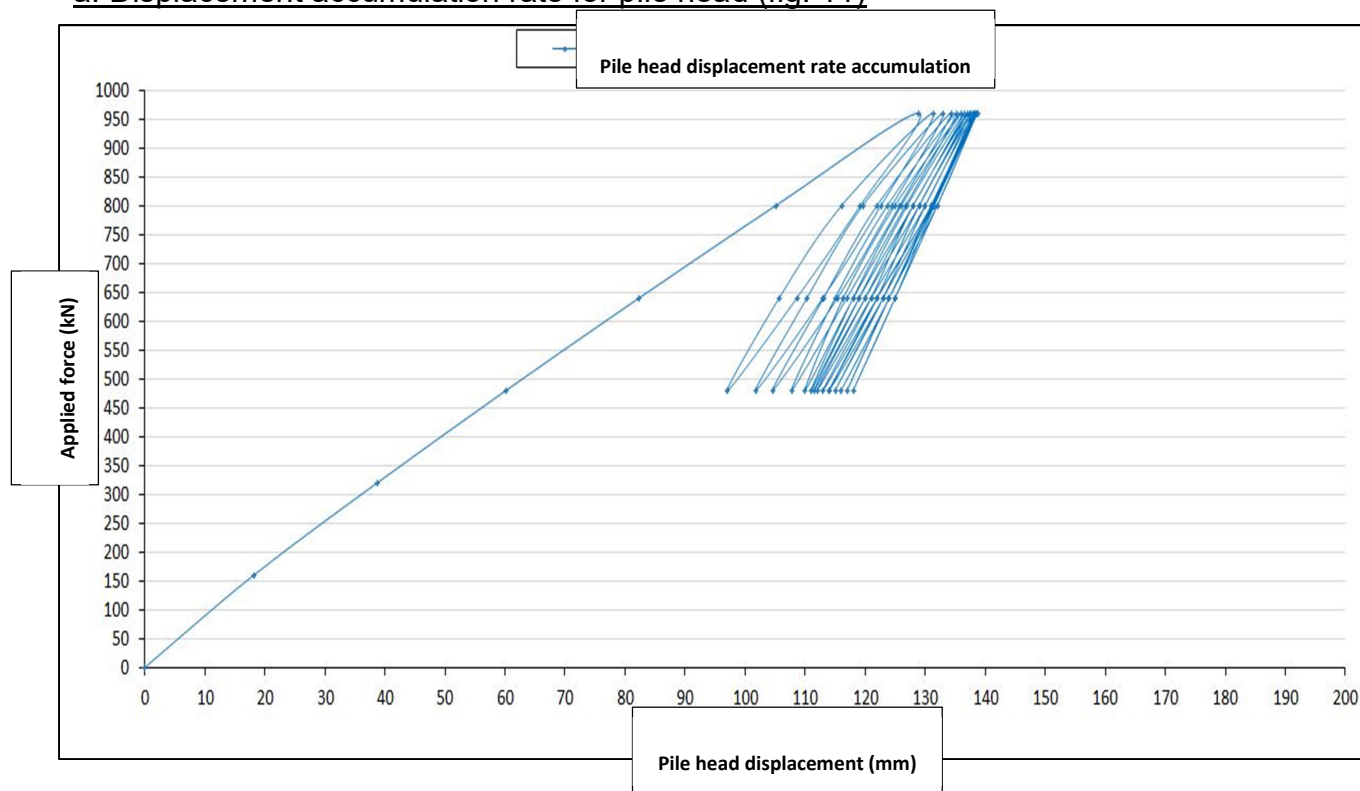
2.4.3. Test P323

Here below is presented a synthesis of the 3D numerical modelling using CESAR-LCPC:

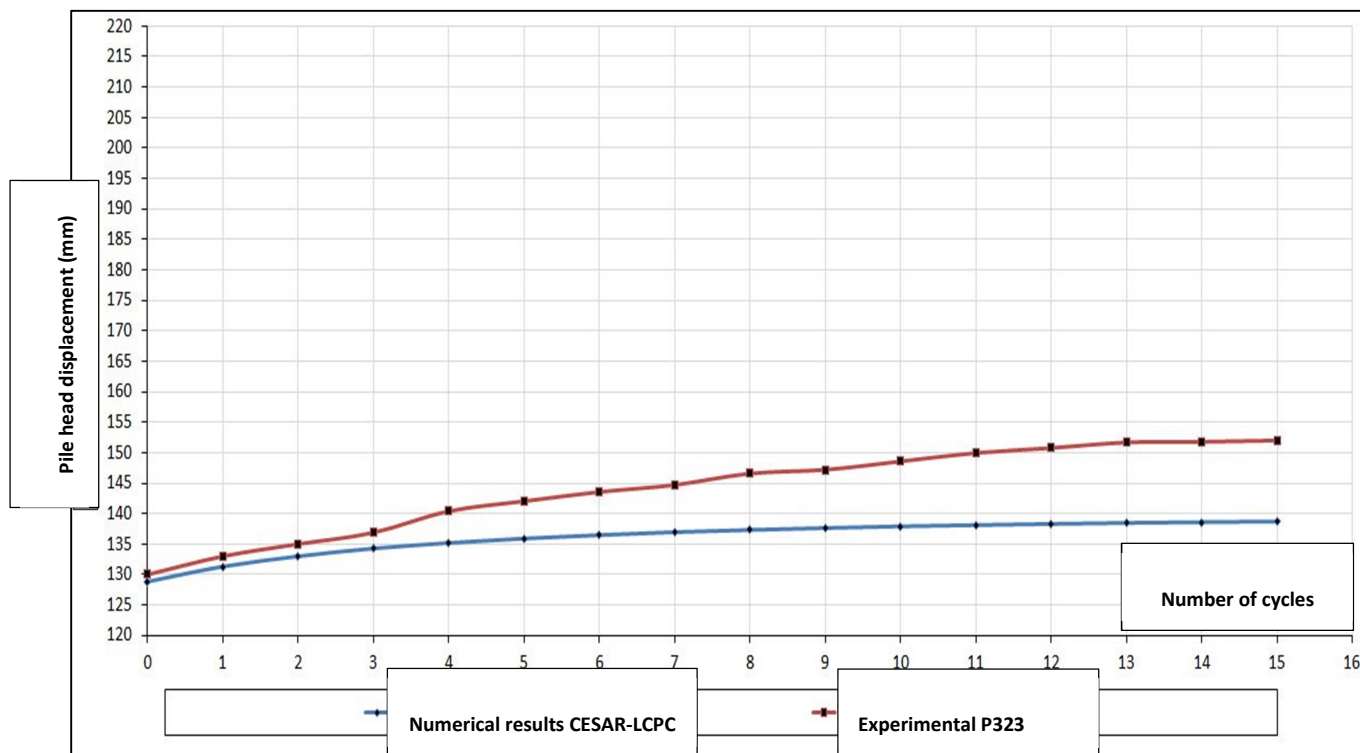
Cyclic parameters		Soil geotechnical parameters		Pile elastic parameters	
Maximum lateral force F (N)	600	Unit weight γ (kN/m ³)	16.5	Unit weight γ (kN/m ³)	27
Amplitude DF (N)	300	Deformation modulus E (MPa)	40	Deformation modulus E (Pa)	3.83×10^{10}
Number of cycles n	15	Poisson's ratio ν	0.33	Poisson's ratio	0.3
Cyclic loading type	Nonalt.	Cohesion (kPa)	1		
		Internal friction angle Φ (°)	42		
		1 st parameter of the hardening law C (MPa)	5		
		2 nd parameter of the hardening law D (-)	20		

Tabelul 7 – Numerical simulation P323

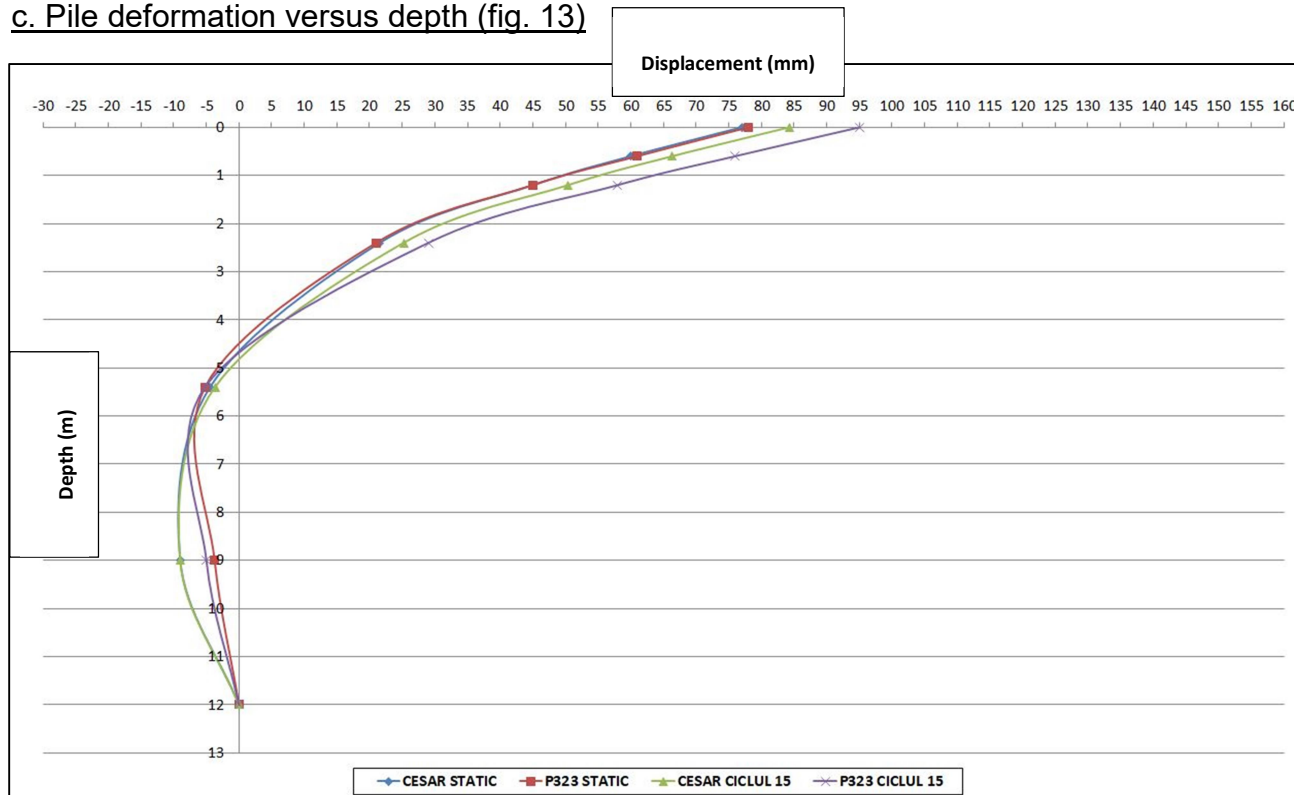
a. Displacement accumulation rate for pile head (fig. 11)



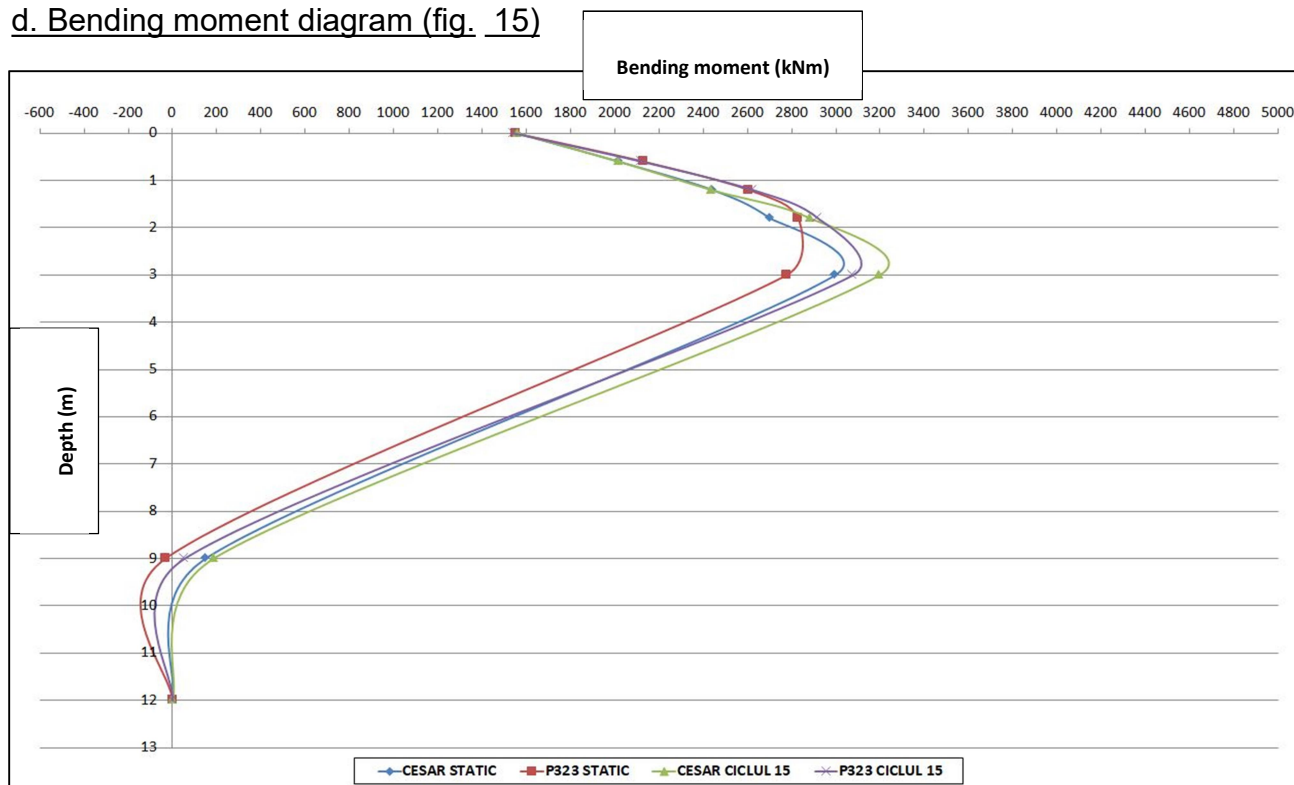
b. Displacement evolution for pile head (fig. 12)



c. Pile deformation versus depth (fig. 13)



d. Bending moment diagram (fig. 15)



Analysing the above graphs, one can note that differences between the numerical and experimental results are very small for all studied elements, of less than 10%, which is an acceptable error, especially for such a complex problem.

It can be affirmed that the numerical model is in good agreement with the experimental tests that it is calibrated and, therefore can be used for performing parametric studies regarding the different parameters of influence.

Partial conclusions

The partial conclusions that can be drawn based on the analysis of the above mentioned graphs are the following:

- * Regardless the situation, the 15 considered cycles have as effect the displacement accumulation on the pile head and the increase of the maximum bending moment
- * Regarding the displacement accumulation, for all 3 analysed situations the hysteretic loops describing the cyclic loading are bug for the first 5 cycles, while their surface are decreasing with the cycles. This is in direct connection with the rate of pile head displacement accumulation, which is faster for the first 5 cycles and it is followed by a slight stabilization. The most clear stabilization of the displacement accumulation is for test P323 (dense sand), which is quite logical. On the contrary, the most unfavourable situation regarding displacement accumulation and stabilization is found for test P354 due to high amplitude $DF = 600$ N and to the large surface of the hysteretic loops, which is translated into a higher energy dissipation.
- * Regarding the maximum bending moment, this appears around 2.5 m depth in all 3 considered cases. The increase of the maximum bending moment compared to the monotonic one is more pronounced for 300 N amplitude ($0.5 \times F$) than for 600 N. The most plausible explanation would be a densification of the sand along the cycles for 300 N (P323 and P355). For 600 N amplitude (P354) the maximum bending moment at the end of the monotonic loading is in fact the same as the one at the end of 5 cycles. In this latest situation the explanation could be the fact that the sand cannot densify as the loading – reloading phases are quite slow due to the amplitudes which equals the maximum lateral force applied on pile head.
- * Confirmation or infirmation of these preliminary hypotheses will be obtained after performing a detailed parametric study regarding the main influence parameters.

These influence parameters can be classified into two categories:

1. Cyclic loading parameters (amplitude and number of cycles);
2. Geotechnical parameters of the soil (relative density, deformation modulus and internal friction angle).

Section II

Parametric studies regarding the influence of cyclic parameters (amplitude and number of cycles) on the behaviour of a single pile in sand

Introduction

Taking into account the results obtained until now, it can be observed that the number of cycles is playing an important role on the main sizing elements for a laterally loaded pile, besides which also the amplitude is having an influence.

In order to analyse the influence of the amplitude DF on the displacements and bending moments for a single pile laterally loaded with 15 cycles, 4 numerical modelling were carried out for the following values of the amplitude: 150 N, 300 N, 450 N and $DF = 600$ N. The maximum lateral force was $F = 600$ N – small-scale value.

These values have been chosen such as to be in agreement with the ratio DF/F considered in the various centrifuge tests (*Rosquoët, 2004* or *Rakotonindriana, 2009*). The respective authors proposed for $DF/F = 0.25, 0.5, 0.75$ and 1 , respectively. On the other hand, model LISM – "Linearly increasing soil reaction modulus" (*Long and Vanneste, 1994*) proposes an empirical formula introducing a coefficient taking into account, among others, the ratio DF/F (considered also to be $0.25, 0.5$ and 1 , respectively).

The geotechnical parameters of the soil used for these numerical modelling aiming to study the influence of amplitude, DF were the following:

- Medium dense sand ($I_D = 53\%$ and $\gamma = 15.1 \text{ kN/m}^3$);
- Deformation modulus $E = 30 \text{ MPa}$;
- Internal friction angle $\Phi = 30^\circ$.

CHAPTER 1 – Numerical results

The numerical results obtained using CESAR-LCPC software were the following:

1.1. Displacement accumulation on pile head

Analysing the figure below one can immediately note that amplitude DF is playing an important role on the accumulation rate

Generally speaking, in all cases, whatever the considered amplitude, the displacement accumulation rate is maximum for the first 5 cycles, decreasing afterwards along the cycles. Higher the amplitude, larger the surface of hysteretic loops, therefore also higher the rate of displacement accumulation. As well, for low amplitudes, the hysteretic loops are smaller and the rate of displacement accumulation is much low. In the same time, for low amplitudes ($DF = 150 \text{ N}$) it can be observed a relative stabilization of the displacements, while for higher amplitudes the displacement accumulation is still relatively pronounced even for cycle 15. Therefore, it can be affirmed that the most unfavourable situation regarding the displacement accumulation on pile head is the one for high amplitude ($DF = 600 \text{ N}$) and a ratio $DF/F = 1$, respectively.

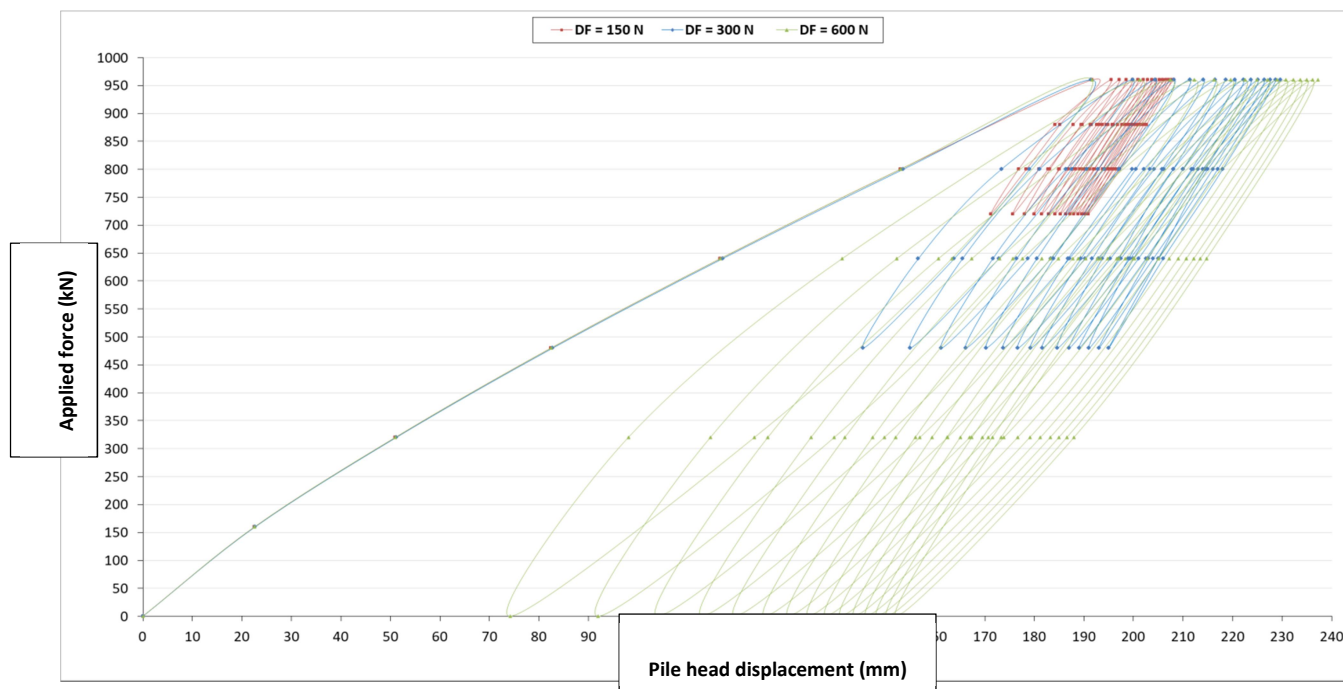


Figure 16 – Pile head displacement accumulation

1.2. Displacement evolution on the pile head

The graph below is in direct connection with the displacement accumulation on pile head, one being able to notice that the slope for 150 N amplitude is less steep than for higher amplitudes, indicating a faster stabilization of displacements.

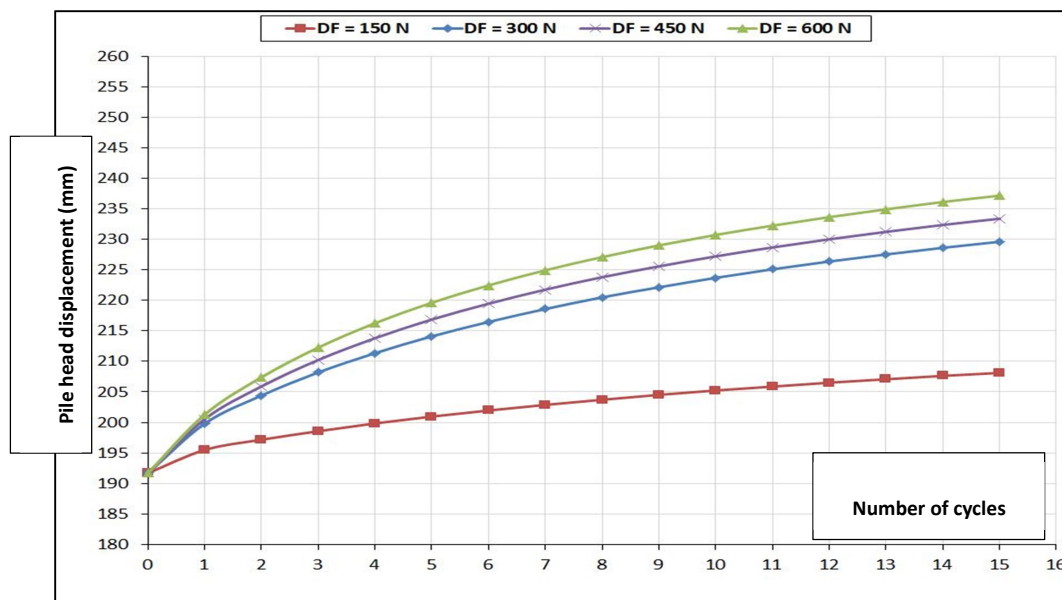


Figure 17 – Displacement evolution on pile head with cycles

Differences between maximum displacement of pile head for cycle 15 for amplitudes $DF = 150$ N and 600 N are quite large, of around 15% ($d_{pc_{max, cyc15, DF=150N}} \approx 208$ mm and $d_{pc_{max, cyc15, DF=600N}} \approx 237$ mm).

1.3. Pile deformation

Figure below presents the pile deformation after 15 cycles and this is in good agreement with the previous graphs, meaning that the lateral displacements of the pile at higher depths are lower for 150 N amplitude than for the other higher amplitudes ($DF=300$, 450 and 600 N).

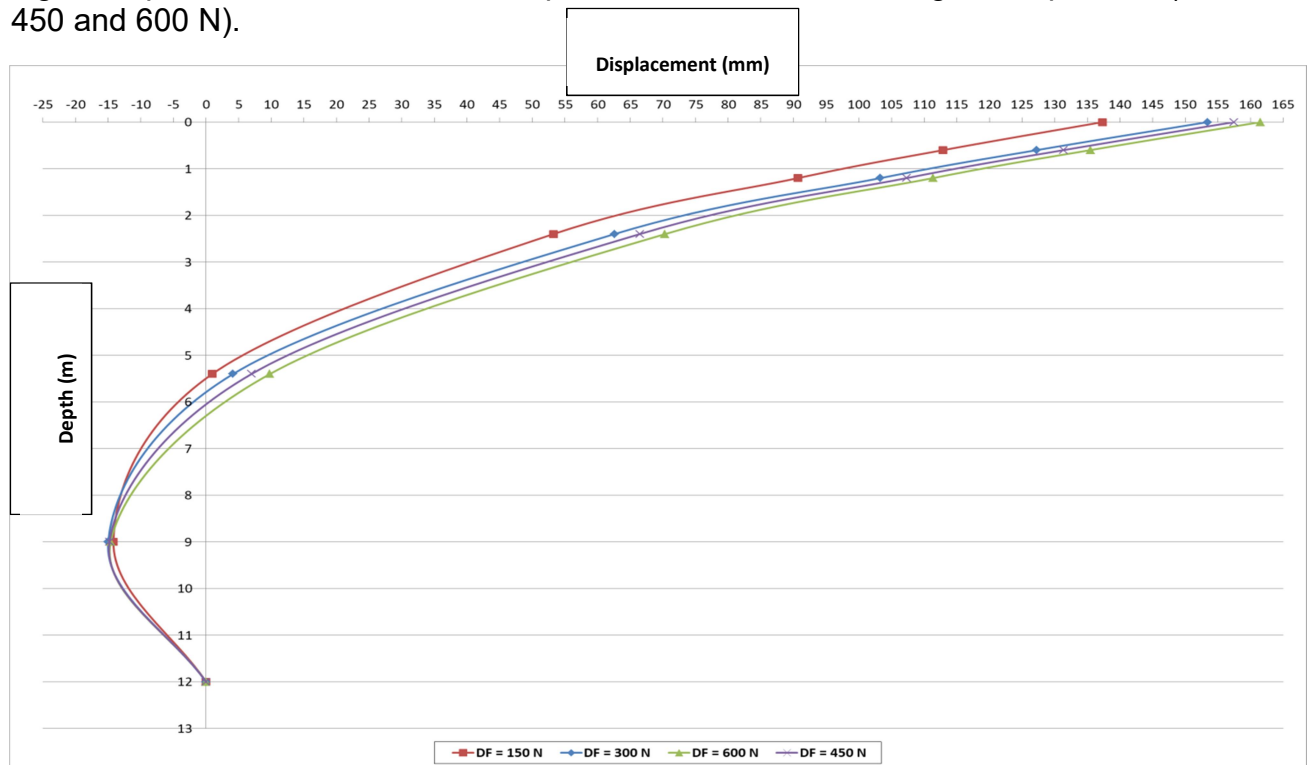


Figure 18 – Pile deformed versus depth at the end of the 15 considered cycles

1.4. Bending moment

Next figure presents the bending moment diagrams at the end of the 15 cycles. The portion $0 - (-2$ m) represents the free length of the pile, while the portion $(0-12$ m) is the pile length below the ground level.

One can note that the difference between the 4 situations (amplitudes 150 , 300 , 450 and 600 N) is mainly in the maximum bending moment area, thus around 2.5 m depth.

The maximum bending moment developed in the pile is obtained this time for the lowest amplitude $DF = 150$ N, while the lowest bending moment is recorded for the highest amplitude $DF = 600$ N. The most logical explanation could also be the densification of the

sand for low amplitudes, resulting in a better soil and, therefore, a higher bending moment. For high amplitudes it is obvious that the sand cannot densify, resulting in a maximum bending moment at the end of the 15 cycles almost equal to the one at the end of monotonic stage.

However, between 150 N amplitude and 600 N one, the differences in maximum bending moment at cycle 15 are relatively small, of approx. 3-4 % ($M_{\max, \text{cyc}15, \text{DF}=150\text{N}} \approx 3300$ N and $M_{\max, \text{cyc}15, \text{DF}=600\text{N}} \approx 3200$ N), in comparison with the differences recorded in case of displacement of pile head (15%).

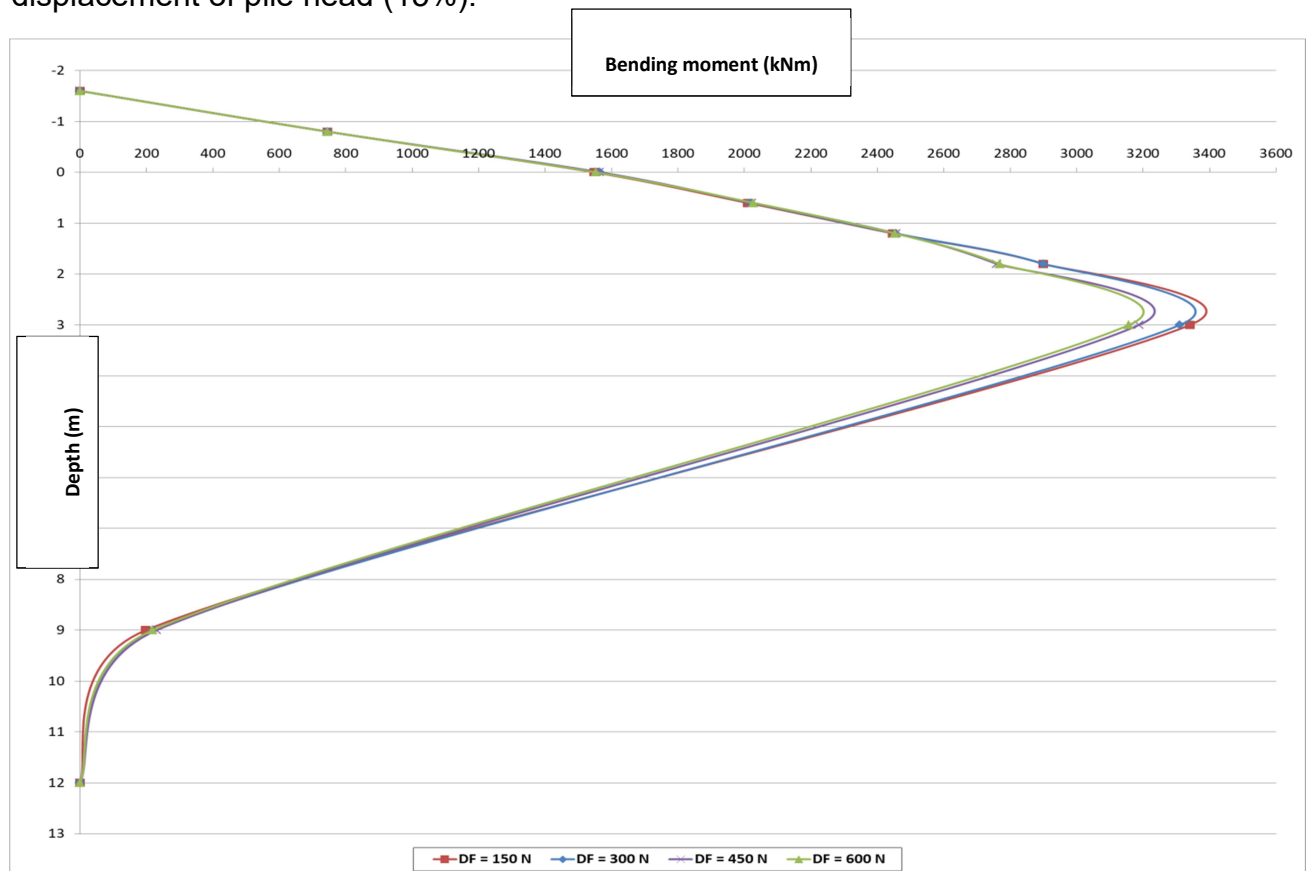


Figura 19 – Diagrama de moment încovoietor în pilot la sfârșitul celor 15 cicluri considerate

1.5. Bending moment evolution along the cycles

The next graph shows the evolution of the maximum bending moment along the cycles for the 4 considered amplitudes. It can be noted a higher increase of the maximum bending moment for the lowest amplitude (DF = 150 N), compared to the slower evolution of this one in case of maximum amplitude considered (DF = 600 N).

Anyhow, for all 4 situations it can be observed a slight stabilization of maximum moment after cycle 14.

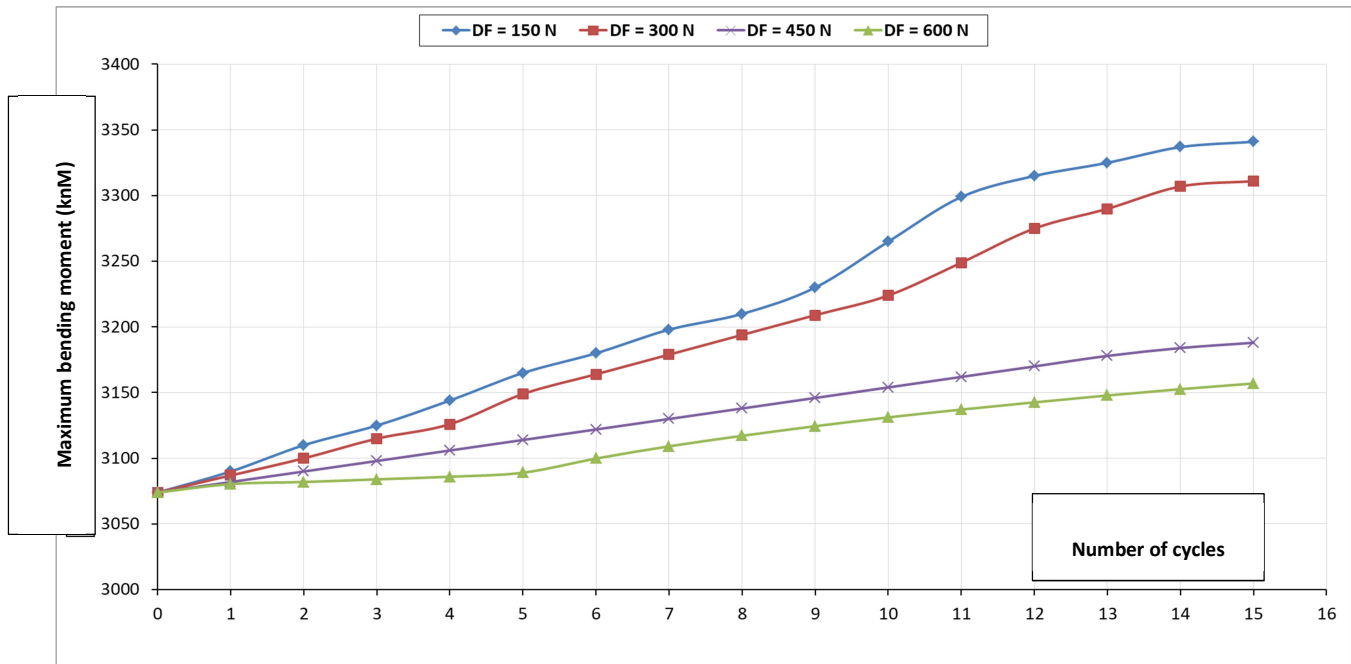


Figure 20 – Maximum bending moment evolution in pile along the cycles

As well, for higher amplitudes (450 and 600 N) the evolution of the maximum bending moment is almost linear. On the contrary, for lower amplitudes (150 and 300 N) the bending moment increasing is more irregular, especially for 150 N amplitude. This could indicate a modification of the relative density of the sand. Maximum bending moment has a sudden increase from 9th to 11th cycle (for 150 N and 300 N amplitudes), which could emphasize the accentuated densification of the sand at that moment (plausible from physical point of view, as for the first cycles the sand cannot densify suddenly, more cycles being required for this phenomenon).

CHAPTER 2 – Considering a large number of cycles

Taking into account that the foundation of an offshore structure, as for example a sea windmill, is submitted during its lifetime to millions of cycles of lateral loading – unloading from wind and/or waves it is inappropriate to consider only 15 cycles in calculation. Centrifuge experimental tests for a large number of cycles performed by Rakotonindriana (*Rakotonindriana, 2009*) for a single pile in sand showed that neither the maximum displacement on pile head, nor the maximum bending moment is not stabilised after 75000 cycles.

The 15 cycles analysed serve to determine some interpolation/extrapolation functions in order to allow further analysis for high number of cycles. Functions have been determined using CurveExpert software (*CurveExpert, version 1.4*).

The aim of this study is to analyse the influence of the amplitude DF through:

- Determination of the number of cycles required for the stabilisation of the pile head displacement;
- Determination of the number of cycles after which the maximum bending moment stabilises;
- Determination of the maximum displacement of the pile head for cycle „n” of stabilisation;
- Determination of the maximum bending moment for cycle „n” of stabilisation;
- The increasing of maximum displacement on pile head after cycle „n” of stabilisation compared to the monotonic one;
- The increasing of maximum bending moment after cycle „n” of stabilisation compared to the monotonic one.

2.1. Evolution of maximum displacement on pile head

By implementing the data in Curve Expert software for the 15 cycles the following interpolation functions have been obtained:

a. DF = 150 N

$$y = (ab+cx^d) / (b+x^d), \text{ with:} \quad [18]$$

$$a = 191.68 \quad b = 17.88 \quad c = 260.16 \quad d = 0.641$$

b. DF = 300 N

$$y = (ab+cx^d) / (b+x^d), \text{ with:} \quad [19]$$

$$a = 191.68 \quad b = 8.524 \quad c = 267.11 \quad d = 0.797$$

c. DF = 450 N

$$y = (ab+cx^d) / (b+x^d), \text{ with:} \quad [20]$$

$$a = 191.68 \quad b = 7.47 \quad c = 265.52 \quad d = 0.84$$

d. DF = 600 N

$$y = (ab+cx^d) / (b+x^d), \text{ with:} \quad [21]$$

$$a = 191.68 \quad b = 6.836 \quad c = 266.52 \quad d = 0.877$$

The results are presented in the table below:

DF (N)	y_{static} (mm)	y_{15} (mm)	y_{100} (mm)	y_{1000} (mm)	y_{10000} (mm)	y_{100000} (mm)	$y_{1000000}$ (mm)	Percentage Increasing (%)
150	191.68	208.12	227.13	248.15	257	259.43	259.44	35.3%
300	191.68	229.71	253.68	264.59	266.71	266.8	266.82	39%
450	191.68	233.46	255.56	263.89	265.28	267.2	267.25	39.4%
600	191.68	237.43	258.39	265.17	266.12	268.4	268.5	40%

Table 8 – Results synthetises – pile head displacement evolution with a high number of cycles

Analysing this table one can note that the maximum displacement on pile head for all 4 considered amplitudes is stabilising at cycle 100000. Between cycle 100000 and 1000000 the differences are negligible.

When looking to the percentage increase from monotonic to cycle 100000 in terms of maximum displacement, one can say that the average for all 4 amplitudes is around 38%.

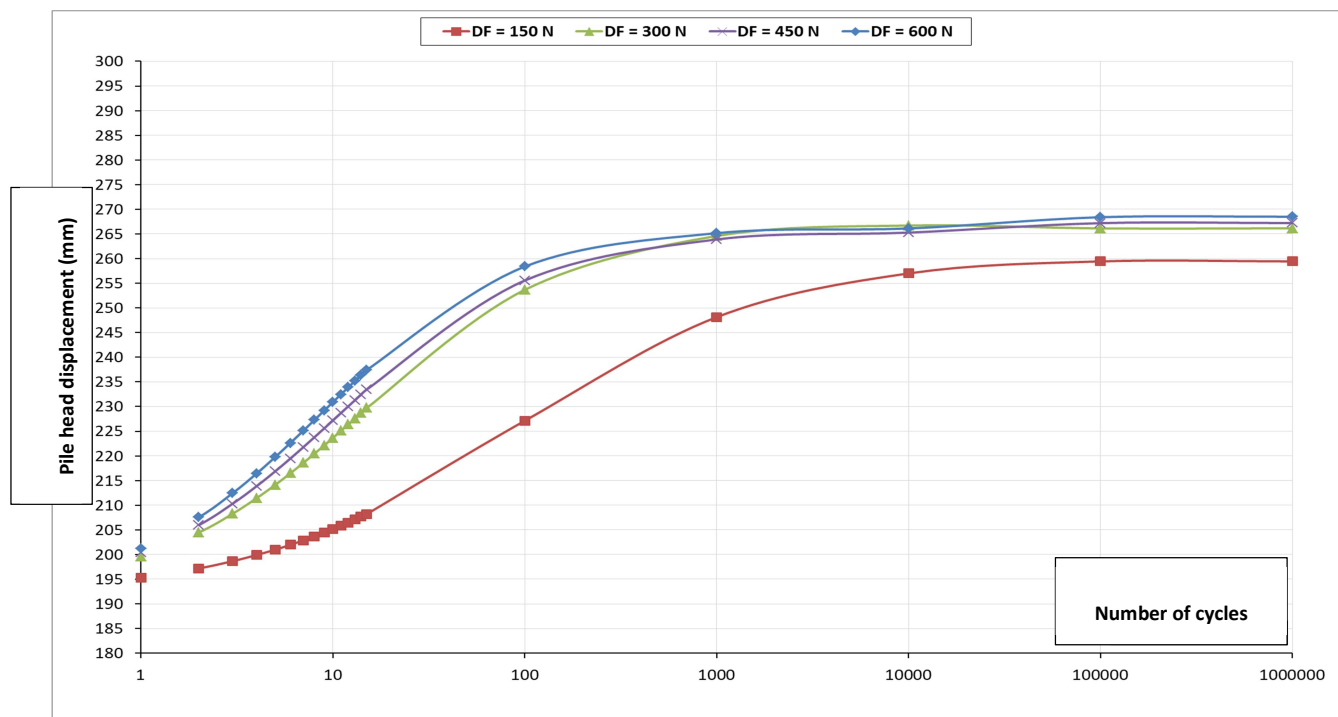


Figura 21 – Pile head displacement evolution with 100000 cycles

The figure here above at logarithmic scale shows the evolution of maximum displacement on pile head along the cycles and emphasizes its stabilisation starting with cycle 100000, regardless the amplitude. It can also be noted that cycles have a maximum importance for the first 100 cycles and this decreases after cycle 1000.

If for the first 100 cycles the differences in terms of displacement on pile head between the lowest amplitude (150 N) and the highest (600 N) are considerable, while after the 100000 cycle the differences are small, of only 3%.

The function and graphs describing the evolution of the maximum displacement of pile head after 100000 cycles for the 4 considered amplitudes and a maximum lateral force on the model of 600 N (or 960 kN prototype-value) and for a sand with the characteristics ($E = 30$ MPa and $\Phi = 30^\circ$) is the following:

$$y = a - be^{-cx^d}, \text{ with:} \quad [22]$$

$$a = 268.2; b = 16000; c = 1.73; d = 0.296$$

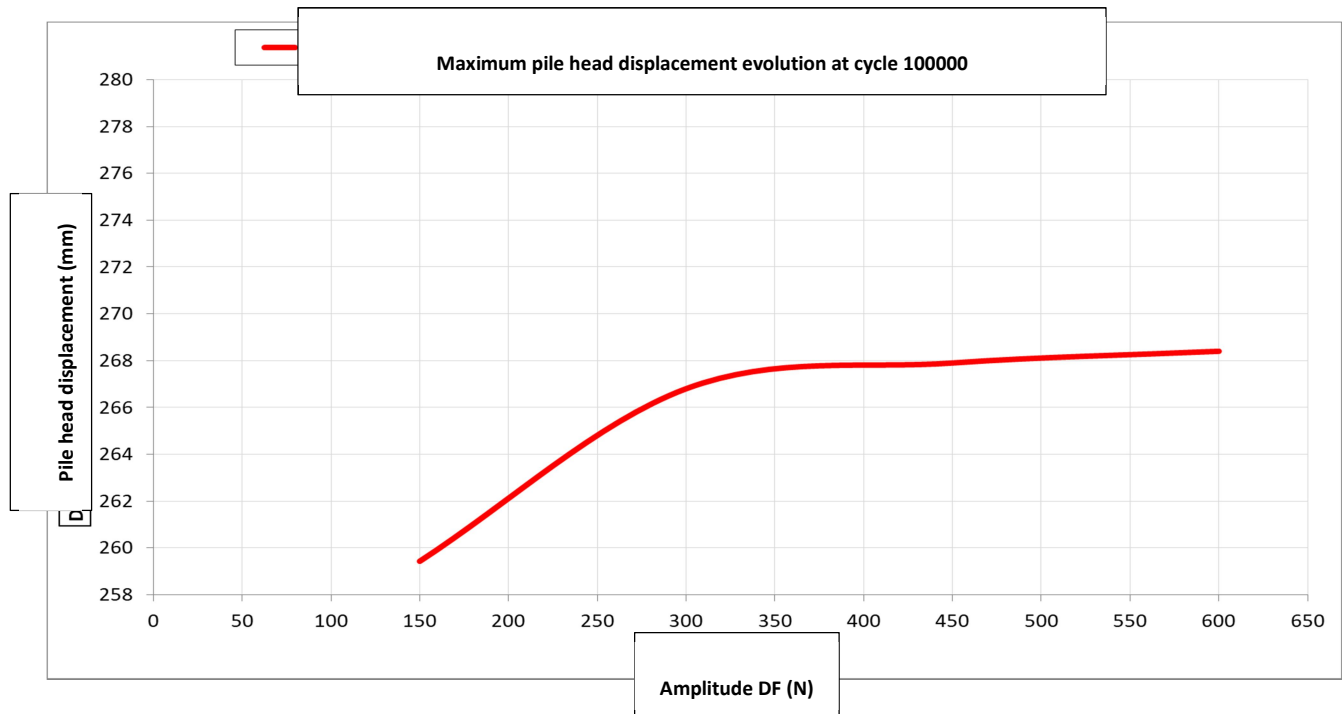


Figure 22 – Maximum pile head displacement evolution at cycle 100000 of stabilisation

2.2. Evolution of maximum bending moment

After using CurveExpert software for 15 cycles, the following interpolation functions were obtained:

a. DF = 150 N

$$y = (ab+cx^d) / (b+x^d), \text{ with:} \quad [23]$$

$$a = 3092.5 \quad b = 177.2 \quad c = 3394 \quad d = 2.381$$

b. DF = 300 N

$$y = (ab+cx^d) / (b+x^d), \text{ cu:} \quad [24]$$

$$a = 3088.7 \quad b = 227.9 \quad c = 3364 \quad d = 2.44$$

c. DF = 450 N

$$y = (ab+cx^d) / (b+x^d), \text{ cu:} \quad [25]$$

$$a = 3077.49 \quad b = 53.96 \quad c = 3274 \quad d = 1.55$$

d. DF = 600 N

$$y = (ab+cx^d) / (b+x^d), \text{ cu:} \quad [26]$$

$$a = 3077 \quad b = 301.4 \quad c = 3233 \quad d = 2.18$$

The obtained results are presented in the following table:

DF (N) (valoare model)	M _{static} (mm)	M ₁₅ (mm)	M ₁₀₀ (mm)	M ₁₀₀₀ (mm)	M ₁₀₀₀₀ (mm)	M ₁₀₀₀₀₀ (mm)	M ₁₀₀₀₀₀₀ (mm)	Creștere procentuală (%)
150	3074	3341	3393.1	3393.8	3393.9	3394	3394	10
300	3074	3312	3363	3363.7	3363.9	3364	3364	9.4
450	3074	3188	3265	3273	3273.5	3274	3274	6.5
600	3074	3162	3230	3232	3232.6	3233	3233	5.2

Table 9 – Results synthesis – maximum bending moment evolution in pile at various cycles

Based on data in table here above it can be observed that the maximum bending moment in the pile, for all 4 amplitudes, is stabilising at cycle 100000. Between cycle 100000 and 1000000 the bending moment is constant. The average increase of the maximum bending moment after cycle 100000 compared to the monotonic loading, for all 4 amplitudes, is of about 8%.

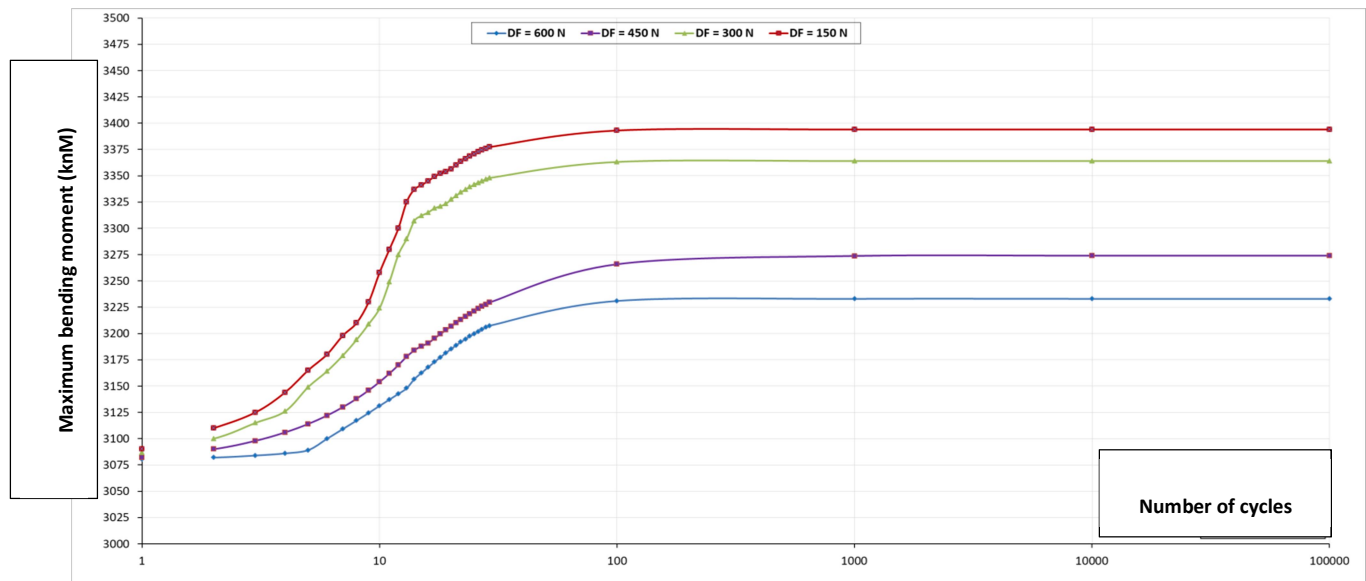


Figure 23 – Maximum bending moment evolution with 100000

The figure here above at logarithmic scale shows the evolution of maximum bending moment along the cycles and emphasizes its stabilisation starting with cycle 100000, regardless the amplitude. It can also be noted that cycles have a maximum importance for the first 100 cycles and this decreases after cycle 1000.

The function and graphs describing the evolution of the maximum bending moment after 100 000 cycles for the 4 considered amplitudes and a maximum lateral force on the model of 600 N (or 960 N prototype-value) and for a sand with the following characteristics ($E = 30 \text{ MPa}$ and $\Phi = 30^\circ$) is the following:

$$y = a - be^{-cx^d}, \text{ cu:} \quad [27]$$

$$a = 3420 \quad b = 49900 \quad c = 25.6 \quad d = -0.239$$

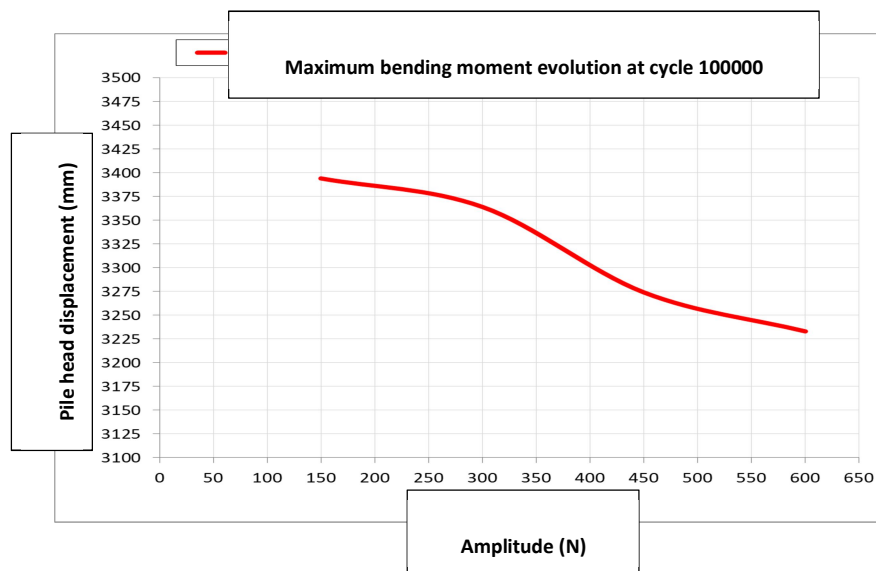


Figura 24 – *Bending moment evolution at 100000th cycle of stabilisation*

Partial conclusions

By developing a 3D numerical model using CESAR-LCPC software for studying the influence of cyclic parameters (amplitude and number of cycles) on the behaviour of a single pile submitted to cyclic lateral loading, for a sandy soil with an internal friction angle of 30° and a deformation modulus of 30 MPa the following aspects have been revealed:

- * Regardless the amplitude, the cycles have as effect the accumulation of displacements on pile head and the increase of bending moment.
- * When the amplitude increases, the rate of displacement accumulation on pile head increases also and stabilizes later in comparison with a lower amplitude.
- * The lateral displacement of the pile versus the depth is, as well, larger for higher amplitudes.

- * The maximum bending moment is recorded for a low amplitude, the most plausible explanation being that sand is densifying for more intense loading (denser sand, smaller displacements, higher moments)
- * Taking into account the fact that the foundation of an offshore structure is submitted during its lifetime to millions of cycles coming from wind or waves it was necessary to perform a study over a higher number of cycles regarding the evolution of maximum displacement and maximum bending moment.
- * Both the maximum displacement on pile head and the maximum bending moment were stabilised after 100 000 cycles. After the 100 000th cycle and up to 1 000 000 cycles there are no more differences.
- * For the maximum displacement on pile head, for the 4 considered amplitudes (150, 300, 450 and 600 N) there was an increase of 35.3%, 39%, 39.4% and 40%, respectively after the cycle 100 000, compared to monotonic loading.
- * For the maximum bending moment, for the 4 considered amplitudes, this has increased after cycle 100 000 with 10%, 9.4%, 6.5% and 5.2%, respectively, compared to the end of monotonic stage.
- * For a real structure is inappropriate to consider that along 100 000 cycles there is a constant amplitude of 150, 300, 450 or 600 N. It is obvious that this varies so that, in theory, one can consider an average of the above mentioned values.
- * **Therefore, for design, when the soil is a sand having the above mentioned geotechnical parameters (medium dense), whatever the amplitude it can be considered that the maximum displacement on pile head is increasing after 100 000 cycles with 38%, while the maximum moment increases with 8%.**
- * These percentages are confirmed by a centrifuge tests on the same type of sand carried out by Rakotonindriana (*Rakotonindriana, 2009 - test C1231ic*) for 75000 cycles and two random amplitudes, The author has observed that even after 75000 cycles the displacement is not stabilised, while the increase of the bending moment after 75000 cycles was of 9%.
- * In conclusion, for a very high number of cycles the exact values of the amplitude doesn't play an important role, having eventually the same consequences for the considered sand.
- * It is mandatory also to study the influence of geotechnical parameters which can have much more influence for single pile foundation submitted to lateral cyclic loads.

Section III

Parametric studies regarding the influence of geotechnical parameters (relative density, internal friction angle and deformation modulus) on the behaviour of a single pile in sand

Introduction

The soil mass plays a fundamental role in the analysis of a pile foundation submitted to lateral cyclic loads. Logically, better the soil, smaller the displacements. But this aspect should be analysed in detail, especially if considering the accumulation of displacement on pile head. A special attention has to be paid to the displacement accumulation rate and to determine the number of cycles for obtaining a stabilisation of this accumulation, for various types of soils, thus for various geotechnical parameters. The main goal of this part is to study the influence of geotechnical parameters on the behaviour of a single pile foundation submitted to laterally cyclic loads, for sandy soil. The analysis will be carried out using 3D numerical modelling and CESAR-LCPC software.

The study has been carried out for two cases:

1. The combined influence of geotechnical parameters, going from less good soil to a very good one:

^a medium dense to loose sand ($\gamma = 15.1 \text{ kN/m}^3$; $\Phi = 30^\circ$; $E = 30 \text{ MPa}$);

^b medium dense sand ($\gamma = 16 \text{ kN/m}^3$; $\Phi = 35^\circ$; $E = 35 \text{ MPa}$);

^c dense sand ($\gamma = 16.5 \text{ kN/m}^3$; $\Phi = 40^\circ$; $E = 40 \text{ MPa}$).

2. The individual influence of the geotechnical parameters of the sand for a less good soil (type a here below)

^a influence of unit weight γ ;

^b influence of deformation modulus E ;

^c influence of internal friction angle Φ .

Both studies have been carried out for the same model-pile described and used in the previous chapters, for a maximum lateral force $F = 600 \text{ N}$ (or 960 kN prototype value) applied on pile head and an amplitude $DF = 600 \text{ N}$ (therefore the most unfavourable from the point of view of displacement accumulation, as it has been shown previously). The number of cycles was 15.

CAPITOLUL 1 – Numerical results for combined influence

The following results have been obtained after using CESAR-LCPC software:

1.1. Evolution of displacements on pile head

Analysing the figure below one can note that the couples of value considered for the geotechnical parameters have a major role on the maximum displacements on pile head and also on their evolution along the cycles. It is interesting that the slope of the maximum displacement versus cycles graph is less steep for the good sand, which can be explained by a faster stabilisation.

It can therefore be affirmed that the better the soil, the faster the stabilisation of the displacements on pile head. This is valid for a small number of cycles as considered in this analysis ($n = 15$), being required a further confirmation for a very high number of cycles.

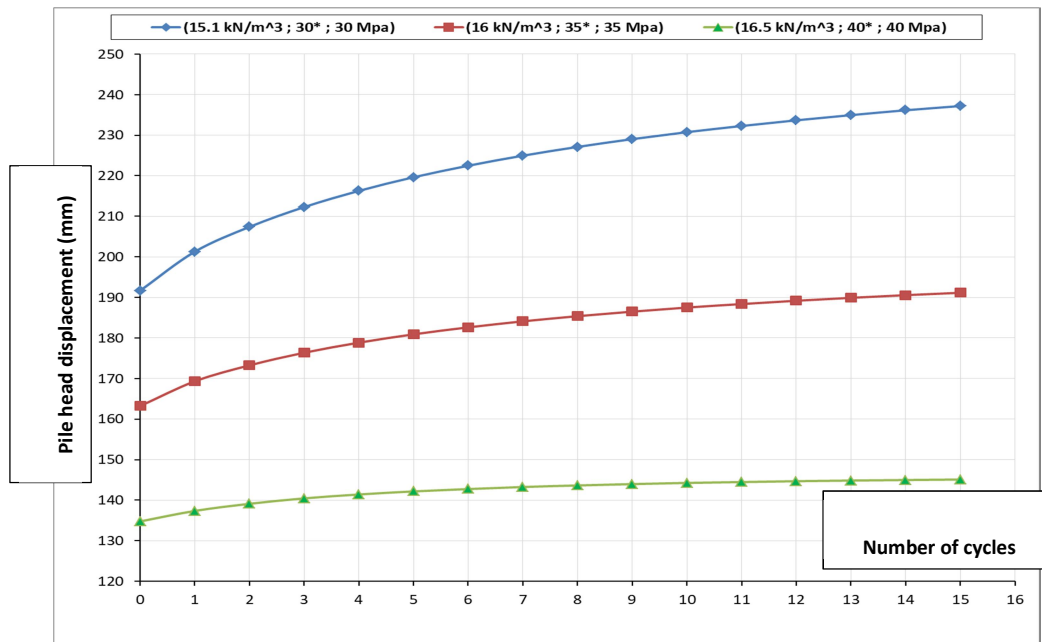


Figure 25 – Maximum pile head displacement for various types of sand

1.2. Pile deformation versus depth

The figure below shows the deformation of the pile at the end of 15 cycles.

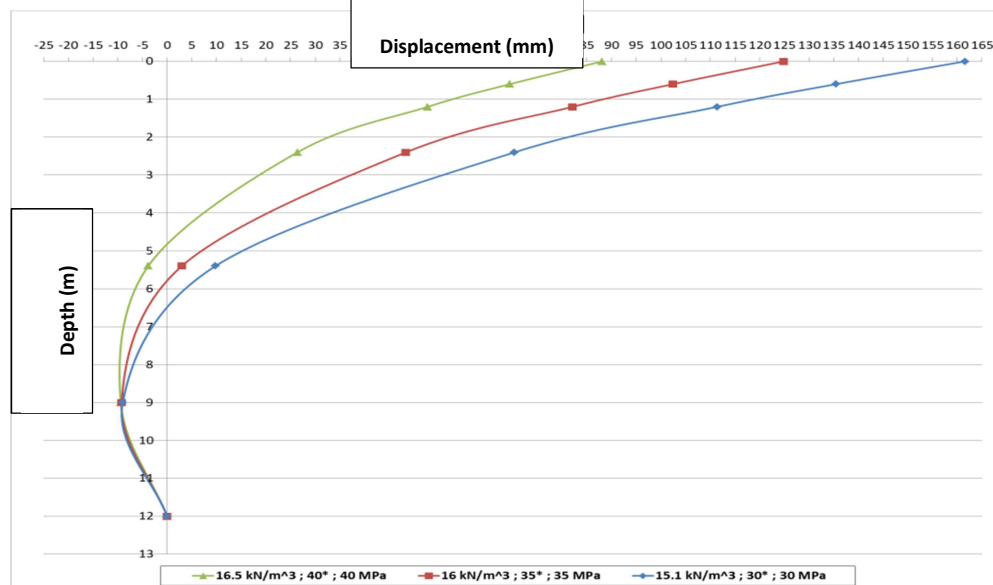


Figure 26 – Pile deformation versus depth for various types of sand

It can be easily seen that over the depth there are considerable differences between the 3 soil conditions in terms of lateral displacement of the pile. In this case also the better the soil, the smaller the displacements.

1.3. Bending moment diagram

The figure below presents the bending moment diagram at the end of 15 cycle for the various types of soil.

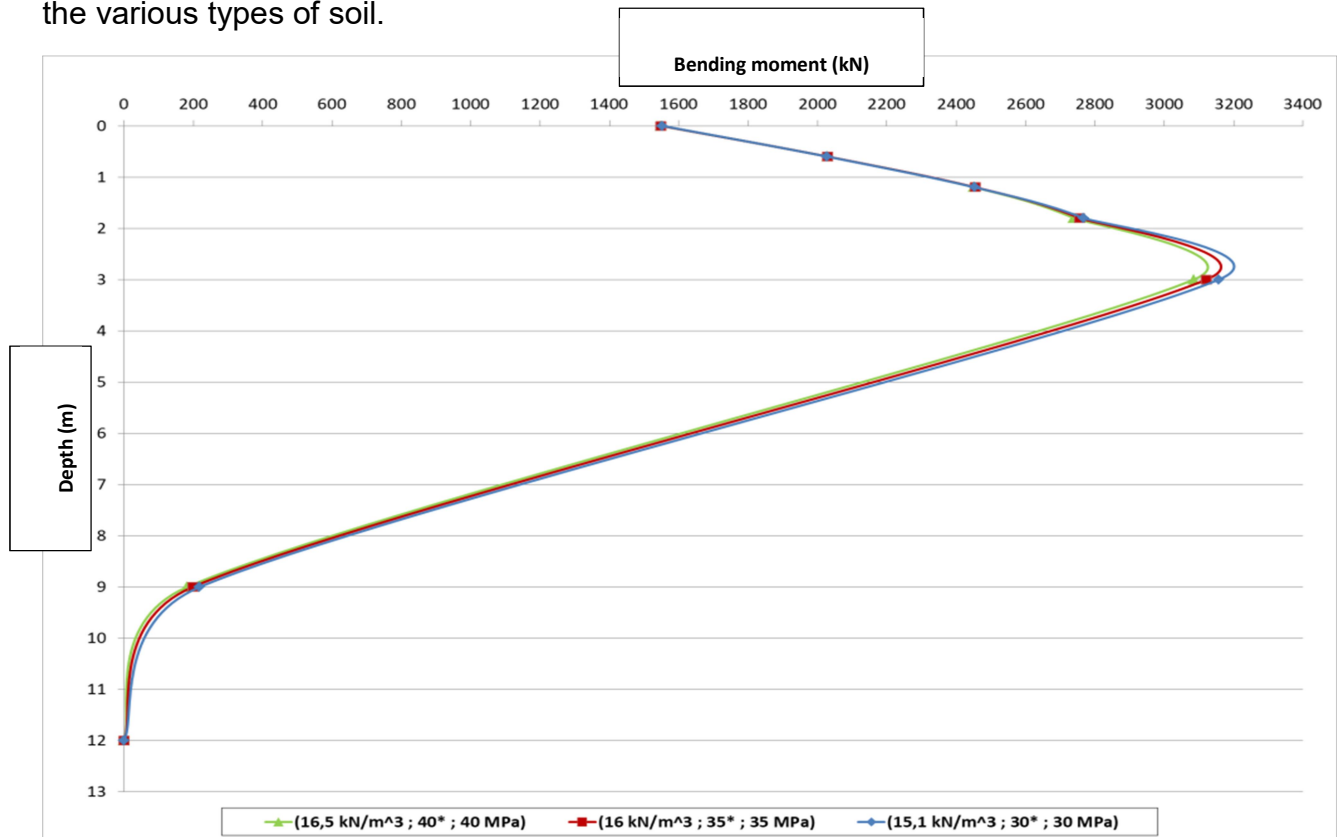


Figure 27 – **Bending moment in pile for various types of sand**

The bending moment diagrams are practically the same for all 3 cases, with differences of maximum 3 %.

1.4. Evolution of bending moment along the cycles

The increase of the bending moment at the end of 15 cycles compared to the end of monotonic stage is, for all 3 cases, of 2.7 % (for the considered amplitude of DF=600 N).

In the next graph is presented the evolution of maximum bending moment along the cycles for the 3 considered cases.

It can be noted for all 3 cases that the evolution is the same.

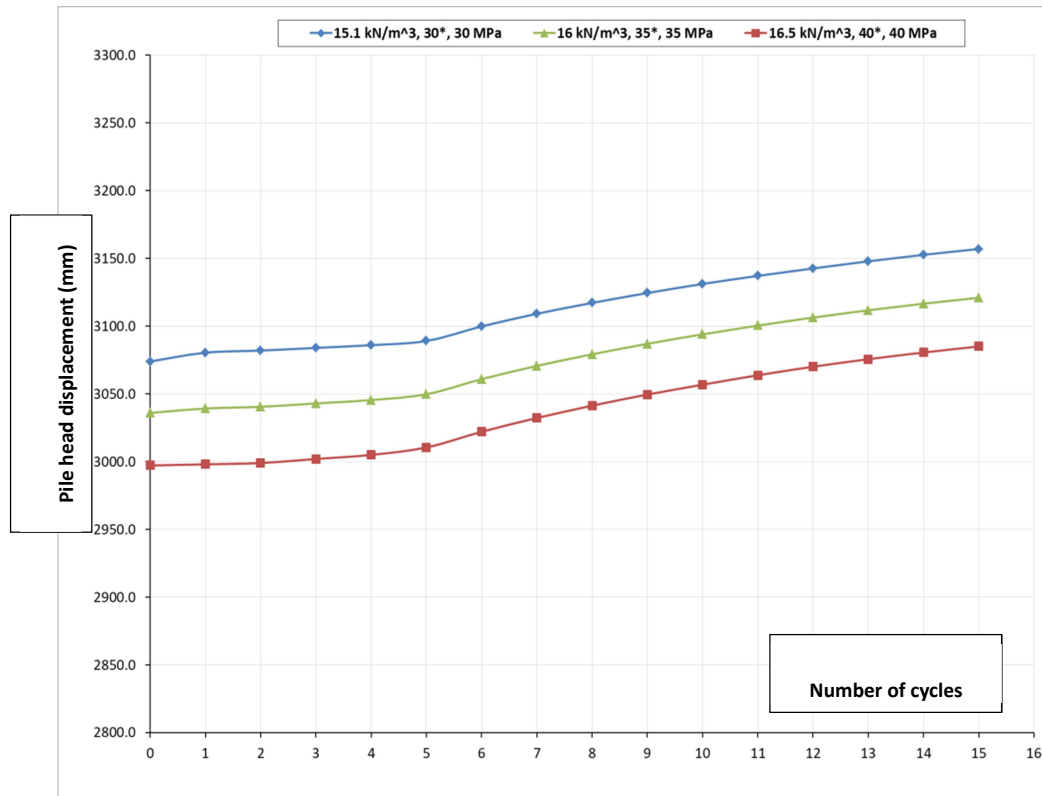


Figure 28 – Maximum bending moment evolution for various types of sand

CHAPTER 2 – Numerical results for individual influence

In this chapter, the individual influence of geotechnical parameters (unit weight, internal friction angle and deformation modulus) is presented, regarding the behaviour of a laterally cyclic submitted foundation pile, in medium to loose sand.

The following results have been obtained with the CESAR-LCPC software.

1.1. Unit weight influence

For a medium to loose sand, characterised by an internal friction angle of $\Phi = 30^\circ$ and a deformation modulus of $E = 30$ MPa, the unit weight has been varied as follows: $\gamma = 15.1$ kN/m³, $\gamma = 15.5$ kN/m³, $\gamma = 16$ kN/m³ and $\gamma = 17$ kN/m³.

No particular influence of this parameter has been observed. There is however an influence for $\gamma = 20$ kN/m³, but this unit weight is unjustified for a medium to loose sand.

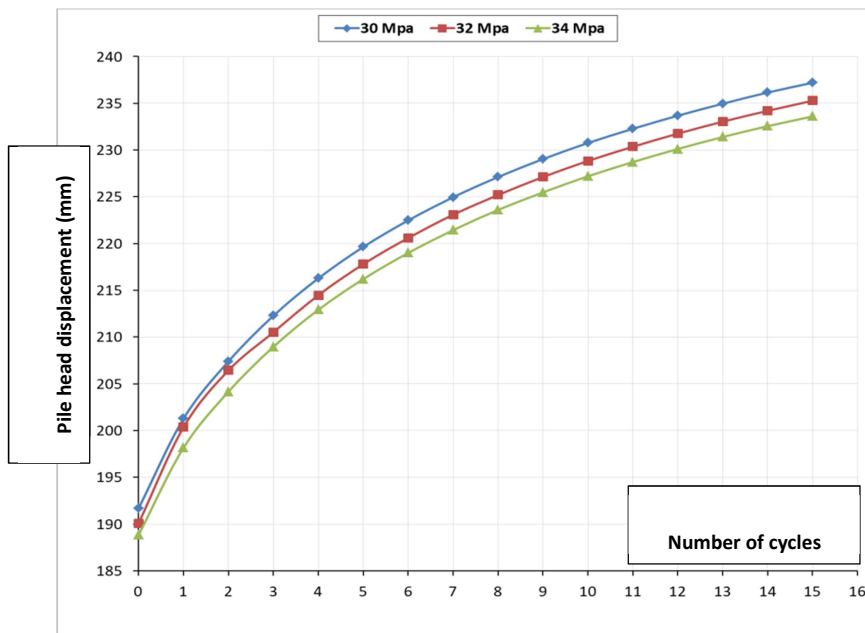
So, the unit weight does not influence the results – pile head displacements, pile deformation or bending moments.

1.2. Deformation modulus influence

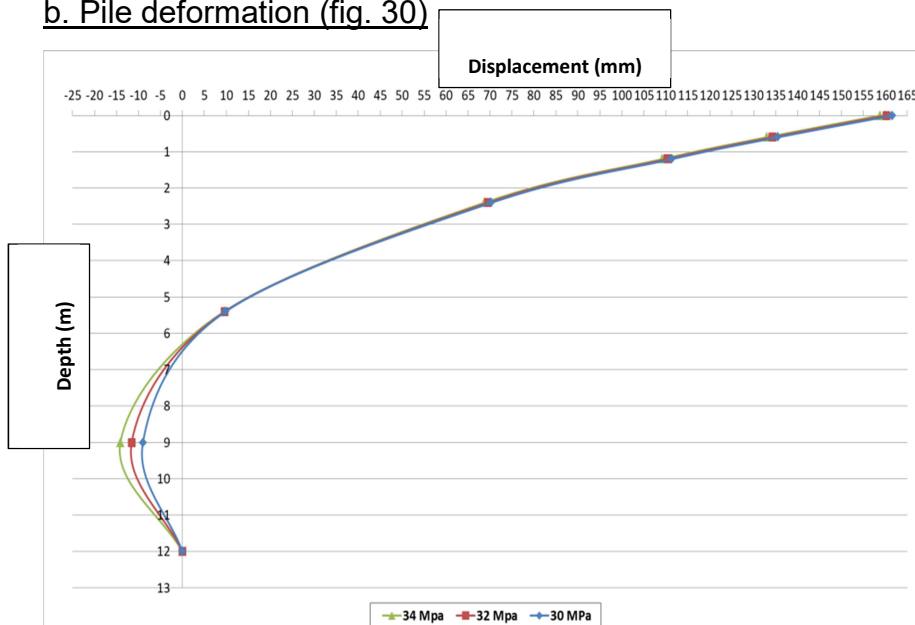
For a medium to loose sand characterised by an internal friction angle of $\Phi = 30^\circ$ and an unit weight of $\gamma = 15.1 \text{ kN/m}^3$, the deformation modulus has been varied as follows: $E = 30 \text{ MPa}$, $E = 32 \text{ MPa}$ and $E = 34 \text{ MPa}$.

The following results have been obtained.

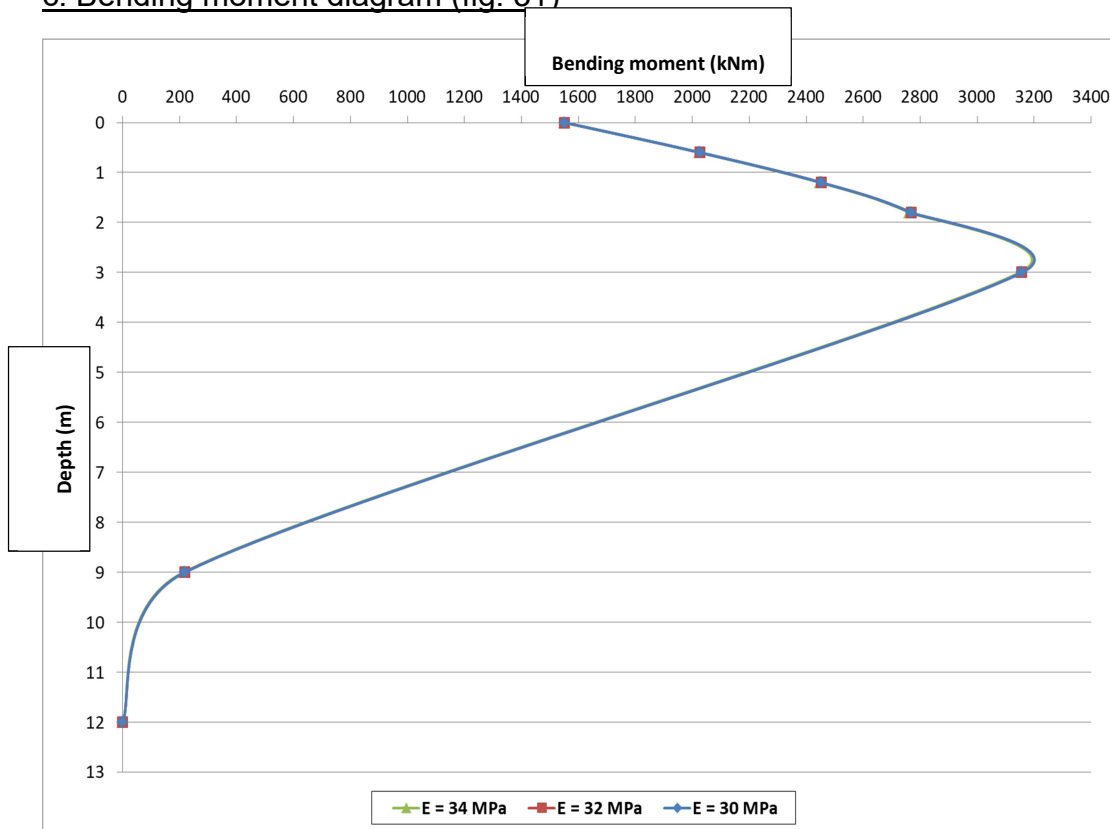
a. Maximum pile head displacements (fig. 29)



b. Pile deformation (fig. 30)



c. Bending moment diagram (fig. 31)



Observations:

- the deformation modulus has an influence for maximum pile head displacements (higher the modulus, lower the displacements) – however, the difference is still under 2%
- pile head displacement evolution seems to be the same for the 3 considered modulus.
- the increasing in percentage of maximum displacement from static to cyclic, for the 3 considered situations, is the same.
- the pile deformation is the same, with only difference in the passive resistance zone.
- the bending moment diagram is also almost identical for the 3 situations.

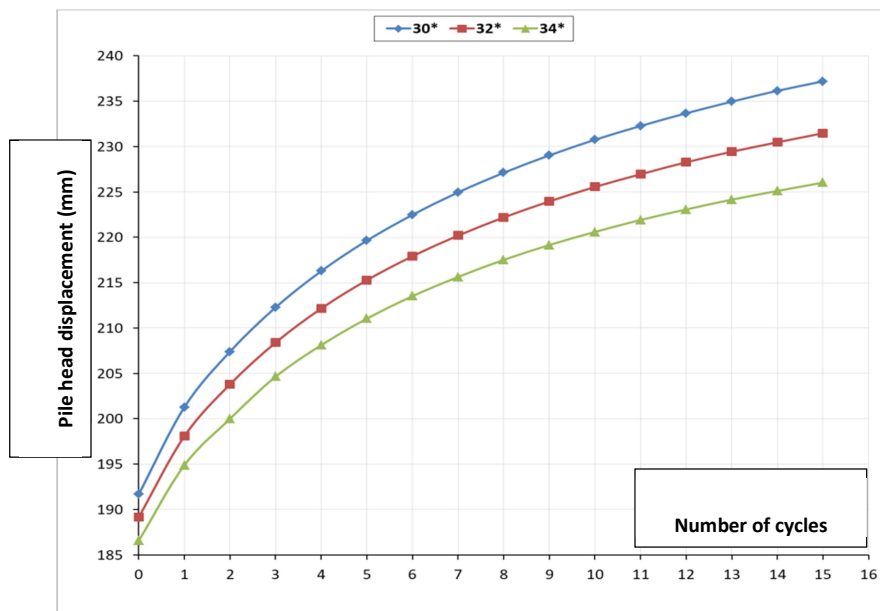
So, the deformation modulus does not a concluding role.

1.3. Internal friction angle influence

For a medium to loose sand characterised by an unit weight of $\gamma = 15.1 \text{ kN/m}^3$ and a deformation modulus of $E = 30 \text{ MPa}$, the internal friction angle has been varied as follows: $\Phi = 30^\circ$, $\Phi = 32^\circ$ and $\Phi = 34^\circ$.

The following results have been obtained.

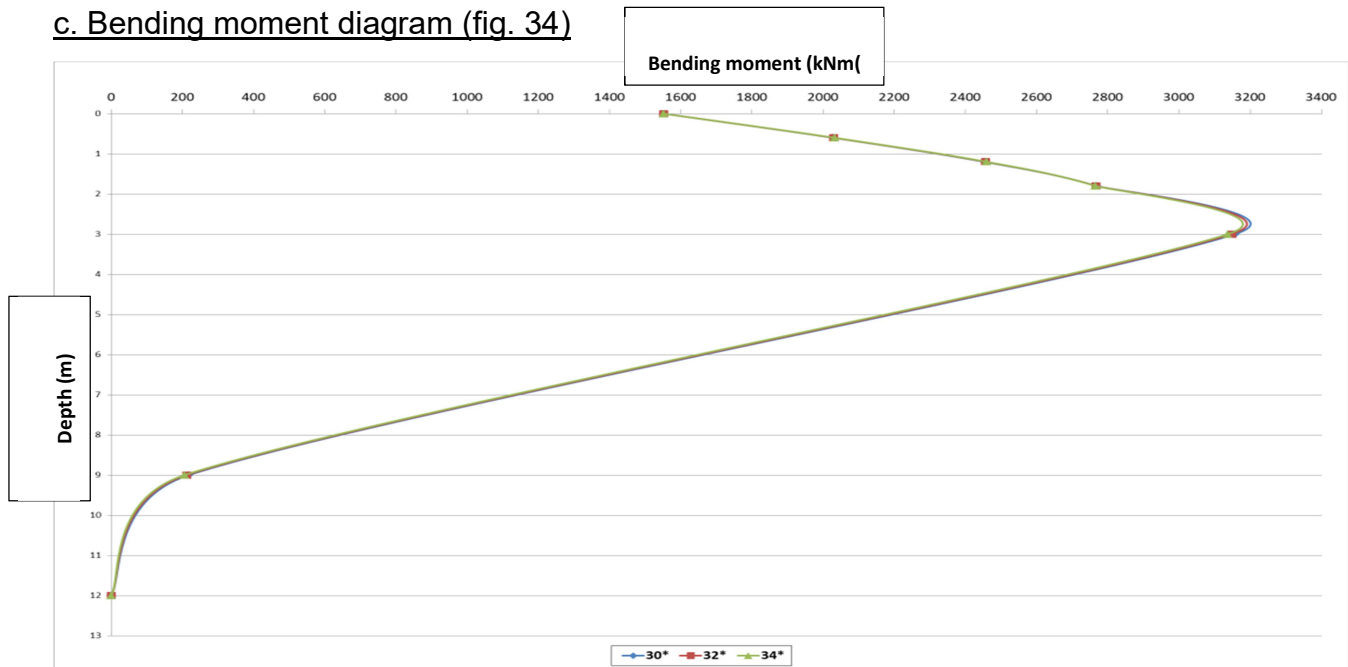
a. Maximum pile head displacements (fig. 32)



b. Pile deformation (fig. 33)



c. Bending moment diagram (fig. 34)



Observations:

- the internal friction angle Φ has an influence for maximum pile head displacement (higher the angle, lower the displacements) – however, the difference is still under 5%.
- the maximum pile head displacement seems to be the same for the 3 considered angles.
- the increasing percentage of pile head displacement from static to cyclic, for the 3 considered situations is the same.
- the pile deformation is relatively the same in all 3 situations, with only visible difference in the passive resistance zone.
- the bending moment diagram is also almost the same for the 3 situations.

So, the internal friction angle individually does not have an important role.

Still, it's interesting to state that between the internal friction angle and the deformation modulus, the parameter with a more concluding influence on the results is the internal friction angle.

The deformation modulus E is controlling the slope of displacement accumulation rate on pile head, whereas the internal friction angle Φ is controlling the area of the hysteresis loops.

Partial conclusion

The geotechnical parameters of sandy soil don't play individually an important role regarding the behaviour of a pile foundation submitted to lateral cyclic action. Between the 3 considered parameters (unit weight, internal friction angle and deformation modulus), the one with most influence is the internal friction angle, with no influence being observed for the unit weight.

On the other hand, the combined influence of the 3 parameters (couples of values γ - Φ - E) is very important regarding pile head displacement and pile deformation.

In consequence, the combined influence of the 3 geotechnical parameters on the results for a high number of cycles shall be studied, with main goal being to determine if for a better sandy soil the stabilisation of displacement is happening faster by comparison with a less good soil.

CHAPTER 3 – Considering a high number of cycles. Combined influence.

It will be studied the combined influence of the 3 main geotechnical parameters on the behaviour of a pile foundation submitted to lateral cyclic loading, for a high number of cycles, through the prism of maximum pile head displacement and maximum bending moment in the pile. The chosen amplitude was $DF = 600$ N, the most disadvantageous regarding the pile head displacement accumulation rate.

3.1. Maximum pile head displacement evolution

Implementing the data in the CurveExpert software for the 15 cycles has led to the following results:

a. medium to loose sand ($\gamma = 15.1$ kN/m³, $\Phi = 30^\circ$, $E = 30$ MPa)

$$y = (ab+cx^d) / (b+x^d), \text{ cu:} \quad [28]$$

$$a = 191.68 \quad b = 6.836 \quad c = 266.52 \quad d = 0.877$$

b. medium dense sand ($\gamma = 16$ kN/m³, $\Phi = 35^\circ$, $E = 35$ MPa)

$$y = (ab+cx^d) / (b+x^d), \text{ cu:} \quad [29]$$

$$a = 163.23 \quad b = 6.01 \quad c = 205.59 \quad d = 0.906$$

c. dense sand ($\gamma = 16.5$ kN/m³, $\Phi = 40^\circ$, $E = 40$ MPa)

$$y = (ab+cx^d) / (b+x^d), \text{ cu:} \quad [30]$$

$$a = 134.7 \quad b = 3.957 \quad c = 147 \quad d = 1.075$$

The following results are obtained:

Sandy soil	y _{static} (mm)	y ₁₅ (mm)	y ₁₀₀ (mm)	y ₁₀₀₀ (mm)	y ₁₀₀₀₀ (mm)	y ₁₀₀₀₀₀ (mm)	y ₁₀₀₀₀₀₀ (mm)	Percentage increasing (%)
Medium to loose	191.68	237.43	258.39	265.17	266.12	268.4	268.4	40%
Medium dense	163.23	191.15	201.99	205.11	205.52	205.53	205.53	26%
Dense	134.77	144.83	146.66	146.97	146.98	146.98	146.98	9%

Table 10 – Results synthesis

Analysing the above table, it can be observed that the sandy soil type is influencing a lot the value and the stabilisation of pile head displacement. The better the soil, the lower the displacement and the faster the stabilisation.

More specifically, for dense sand, the displacement stabilisation happens at cycle 1000. For a medium-dense sand, the stabilisation happens at cycle 10000, whereas for the weakest sand at 100000th cycle.

Regarding the increasing percentage of the maximum pile head displacement, this increases with only 9% for the densest sand, with 26% for the medium sand and with almost 40% for the loosest sand.

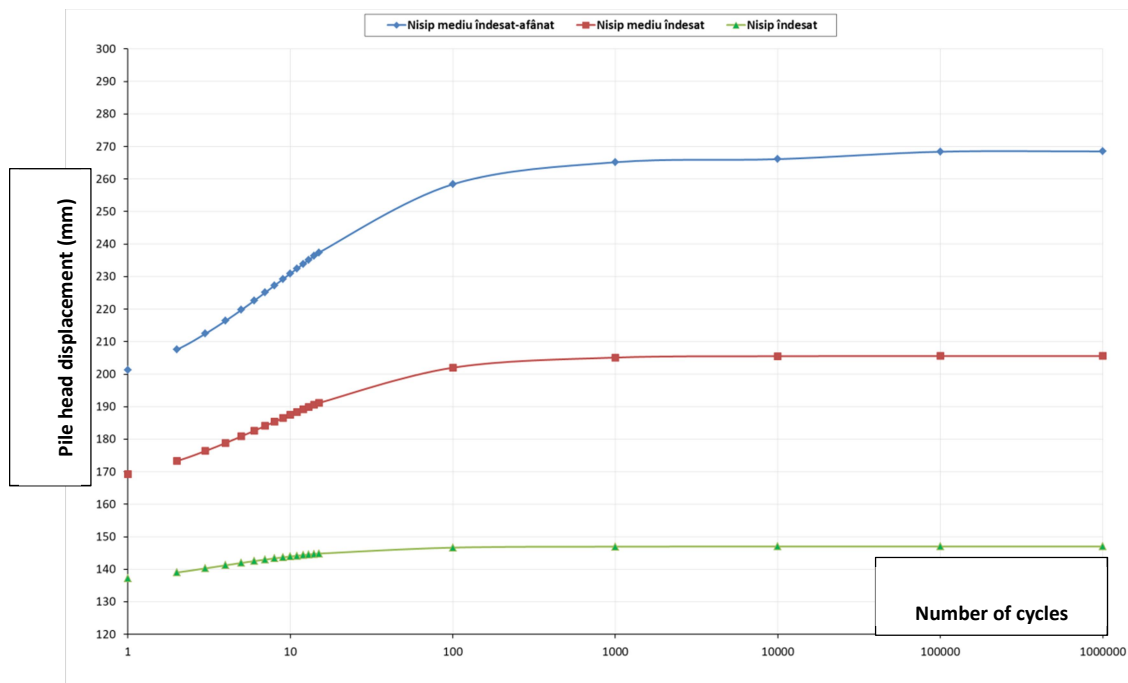


Figure 35 – Maximum pile head displacement against a high number of cycles for the 3 considered types of soil

The above graph confirms the initially hypothesis deduced for a number of 15 cycles, namely that the maximum pile head displacement is indeed lower and has a quicker stabilisation in a dense sand.

3.2. Maximum bending moment evolution in the pile

The numerical modelling results highlighted the fact that the maximum bending moment in the pile has exactly the same increasing percentage from static to stabilisation cycle, no matter the considered type of soil, namely of 5.2%, for an amplitude of 600 N.

Important observation. In parallel, there have also been performed a series of general parametric studies regarding the maximum pile head displacement evolution and maximum bending moment along the cycles, for the other amplitudes (150, 300 and 450 N), for the 3 sandy soils. The modelling showed that the results regarding the increasing percentage of maximum bending moment in the pile from static to stabilisation cycle, available for the medium to loose sand (described in detail in section II of this report) are also available for the other 2 types of sand (medium dense and dense).

SYNOPTIC TABLE

In the next table it is presented the general synthesis of the obtained results (increasing percentages from static to stabilisation cycle), regarding the maximum pile head displacement and maximum bending moment in the pile, for all 4 considered amplitudes and for the 3 analysed types of sand.

Sand	DF (N)	Max. displ. on pile head (%)	Average (%)	Max. moment (%)	Average (%)
1. Medium to loose $\gamma = 15.1 \text{ kN/m}^3$; $\Phi = 30^\circ$; $E = 30 \text{ MPa}$ (stabilizare la ciclul 100000)	150	35.3	37.7	10	7.6
	300	39		9.4	
	450	39.4		6.5	
	600	40		5.2	
2. Medium dense $\gamma = 16 \text{ kN/m}^3$; $\Phi = 35^\circ$; $E = 35 \text{ MPa}$ (stabilizare la ciclul 10000)	150	24.7	25.4	9.8	7.5
	300	25.1		9.3	
	450	25.5		6.6	
	600	26		5.2	
3. Dense $\gamma = 16.5 \text{ kN/m}^3$; $\Phi = 40^\circ$; $E = 40 \text{ MPa}$ (stabilizare la ciclul 1000)	150	8	8.5	9.6	7.4
	300	8.3		9.1	
	450	8.6		6.3	
	600	9		5.2	

Table 10 – General synthesis of the research report

The averages mentioned in the above table are justified in the context in which, in reality, along a very high number of cycles, it can't be exactly stated that the amplitude

have been resting constant, but it can vary from a small value to its maximum value. Neglecting these increasing percentages can lead to an inappropriate design of the pile foundation.

GENERAL CONCLUSIONS OF THE RESEARCH REPORT

The following general conclusions can be drawn:

- Based on some small-scale centrifuge experimental results performed by Rosquoët in 2004, officially obtained in January 2017 from IFSTTAR Nantes, a 3D numerical model has been calibrated;
- The numerical modelling has been performed in the finite element software CESAR-LCPC.
- The differences between the numerical model's results and the experimental ones are very small, under 10%, by point of view displacements and bending moments.
- On the calibrated numerical, there have been further performed a series of studies regarding the influence of certain parameters. These parameters have been grouped in 2 categories: cyclic parameters (amplitude) and geotechnical parameters (unit weight, internal friction angle and deformation modulus).
- Regarding the amplitude's influence, it has been stated that, the higher the amplitude, the higher the pile displacement and the higher the accumulation rate of displacement on pile head. The hysteresis loops are also higher for high amplitude. The bending moments are on the other side, higher for small amplitude, because in this context, the sandy soil becomes denser. For high amplitudes, the sandy soil can't become denser.
- Regarding the combined influence of the geotechnical parameters, it's obvious. The better the soil, the lower the displacement, and the pile head displacement in stabilizing faster along the cycles. The maximum bending moment after 15 cycles is practically the same, no matter the type of sand considered. The individual influence of a certain geotechnical parameter (unit weight, internal friction angle or deformation modulus) it's not that important. However, between the internal friction angle and the deformation modulus, the one with most influence is the internal friction angle.
- Taking into account that an offshore structure is submitted throughout its existence at millions cycles, the 15 previously analysed cycles served to create some extrapolation functions in order to analyse a high number of cycles.
- The results synthesis is presented in the table 10 of the present paper. So, in the design practice, an engineer must consider an increasing percentage of pile head displacement from static to stabilisation cycle with 38% for a medium to loose sand, with 25% for a medium dense sand and with 9% for a very dense sand respectively. Regarding the

maximum bending moment in the pile, this has an increasing percentage from static to stabilisation cycle with 7.5%, no matter the amplitude and the type of the consider sand.

- Neglecting these increasing percentages could led to an unconservative design of the future structure.

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