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Effect of Cap Stiffness and Soil Dilatancy in Pile Groups Subjected To Lateral Loads

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Abstract-This paper explores the effect of cap stiffness and soil dilation on the structural response of pile groups subjected to lateral increasing monotonic loads. For such scope, a finite element-based model for studying pile configurations of 3x3 to 3x7 in homogeneous sand deposits is proposed. The model employs solid elements with twenty nodes to represent the soil and pile domains, while zero-thickness interface elements are used to model the contact state at the soil-pile interface. The adequacy of the model is firstly validated with experimental data from literature and then the associated parametric studies involving the two aforementioned aspects are performed. For the studied examples, the obtained results indicate that the reduction of the cap stiffness substantially increases the lateral displacement of the system. This increment under ultimate loads can be as high as three times the lateral displacement of the original system with a rigid cap. Finally, soil dilation did not significantly affect the pile group response as the soil mass seems to be restricted by the surrounding piles.

***Keywords*: pile group, homogeneous soil, static load, finite element.**

1. Introduction

Foundations are essential to transfer loads from the superstructure to the soil in which they rest. To build safe structures, careful analyses should be performed by taking into account the soil-structure interaction effect. Specially, deep foundations are commonly employed in practice because they permit to attain larger soil depths. The piles which are typically joined by a rigid or flexible cap are in charge of transferring the vertical and lateral loads from the superstructure to the soil, being this mechanism possible due to the soil-pile interaction effect. Furthermore, the adequacy of the pile material and the selection of its slenderness depend upon this complex mechanism that develops with the surrounding soil.

Various standards still permit to employ quase-static procedures using lateral loads to represent seismic and/or wind actions, together with a simplified numerical model based on nonlinear springs and frame elements to represent the soil and piles, respectively. Here, the *p-y* curves are defined to model the soil-pile interaction alongside provisions for the group reduction factor concept, so that the well-known shadowing effect can be included in the analysis. However, with the advance of more sophisticated numerical tools based on the Finite Element Method (FEM) more complex analyses can be performed, which may lead to safer and more economical designs as an accurate prediction of the deformational behavior of the foundation can be established.

Works related to pile groups subjected to lateral loads are customary found in the literature. For instance, in Zhang *et al*. [1] the numerical prediction of a single pile and 3x3 to 3x7 pile groups using a numerical model based on nonlinear springs was carried out to validate the model with the experimental data presented in McVay *et al*. [2]. Later, in McVay *et al*. [3] a series of lateral load tests were performed on 3x3 and 4x4 pile groups embedded in loose and medium-dense sand deposits. The study explores the effect of caps located at variable elevations in relation to the ground surface. A simplified model based on nonlinear springs using *p-y* curves proved to reasonably match the experimental data at all cases. It is shown that the lowering of the cap favours a more resistant system.

Yang and Jeremic [4] studied the numerical response of 3x3 and 3x4 pile groups founded in loose and medium sands using a three-dimensional Finite Element (FE) model. Various numerical static pushover tests are carried out to investigate the soil-pile interaction effect on the final pile bending moment and soil reaction stress distributions with depth. The same authors [5] expanded their previous model later to include the layering effect in the problem. Therein, the mechanical response of single piles embedded in elasto-plastic soils with alternate layers of clay and sand were studied. In Karthigeyan *et al*. [6], a FE model was proposed to investigate the effect of the vertical load on the lateral response of piles under static loading, regarding both homogeneous clay and sandy soils. It is found that the effect of vertical load has a significant impact on the response of sandy soils. Yao *et al*. [7] presented a 3D nonlinear FE model to study the mechanical performance of super long piles with slenderness ratio greater than 50 subjected to both lateral and axial loads in layered soils. In Jin *et al*. [8] a three-dimensional finite element analysis of a real-scale-group-pile foundation system subjected to horizontal cyclic loading was conducted. The material nonlinear behavior of the pile is included in the analysis, while the response of the saturated soil is described using an effective stress approach. Other useful works can be found in [9-13].

The study of pile groups under lateral loads is an active topic, which deserves special attention. Hence, the objective of this paper is to explore two important aspects related to the influence of the cap stiffness and soil dilation on the numerical response of 3x3 and 3x7 pile groups by means of pushover analyses, regarding the pile-cap-soil interaction and using a 3D FE model. For such scope, a particular numerical tool was coded and validated with experimental data.

2. The numerical approach

A computational model to predict the numerical response of pile groups under lateral is developed in this study. The response of piles subjected to horizontal loading is dictated by a linear elastic behavior in the case of aluminium piles. The linear elastic behavior is defined by means of the shear modulus G_c and Poisson's ratio ν_c. In relation to the soil material, a linear elastic perfectly plastic model using the Drucker-Prager criterion and nonassociated flow rule is considered for modeling the behavior of sandy soils, where the shear modulus G_s, Poisson's ratio ν, internal friction angle ϕ, cohesion c and dilation angle ψ are needed. This simple model has been preferred over others because of its simplicity and acceptable behaviour for representing soil paths under monotonic loads. For the soil-pile interface, a shear-strength criterion of Mohr-Coulomb type is employed. This criterion reads as follows.

$$\tau_L = \mu\sigma_n \tag{2}$$

where τ_L is the limiting shear stress, σ_n is the normal stress acting at the integration point and μ is the friction coefficient at the soil pile interface. Sliding occurs when the shear stress $\tau_S = k_s.u_i$ exceeds the limiting stress τ_L, where k_s is the tangential stiffness (penalty) and u_i is the tangential relative displacement along the *i*-th direction according to the local system depicted in Fig. 1(a). The separation between the pile and the soil develops when tensile stresses are detected at the soil-pile interface. The relationship between the normal stress $\sigma_n = k_n.u_n$ and the relative displacement in the normal direction u_n is also shown in Fig. 1(a) by means of a normal stiffness k_n. The joint may have also a specific tensile strength σ_M, and once it is reached the normal stress drops drastically to zero. Since the local coordinate system is orthogonal at the interface element, it is assumed that the displacement in each direction only yields stress in that direction.

The finite element mesh discretization adopted for soil and pile is based on the use of 20 node brick finite elements, while the soil-pile interface is modeled by using 16 node quadrilateral contact elements of zero-thickness, which joint one face of the pile element to the corresponding face of the soil element as shown in Fig. 1(b). The lateral and bottom surfaces of the finite element mesh are fully fixed and the lateral surfaces are located at an adequate distance from the foundation to avoid boundary effects. A linear geometric behavior is deemed at all analyses.

The initial stress state in the soil deposit is computed under the k_o condition, thereafter the horizontal load at the cap is applied incrementally and the corresponding lateral displacement is recorded at all load levels. More details can be found in [14].

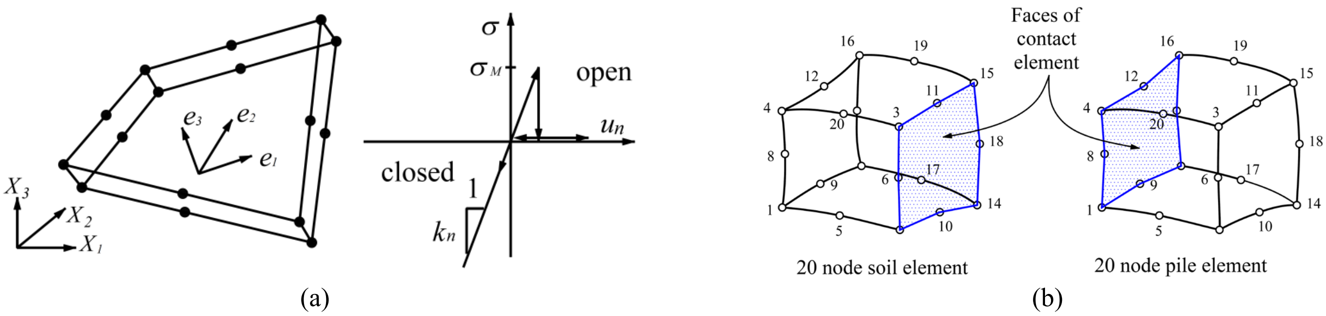


Fig. 1: Interface, hexahedral and contact finite elements

2.1. Pile groups tested by McVay et al. (1998)

To validate the adequacy of the current numerical model, pile groups of 3x3 to 3x7 configurations tested by McVay and co-workers [2] using a centrifugal equipment at 45g and subjected to lateral increasing loads are studied in this section. The piles are embedded in a homogeneous soil deposit and rigidly connected to a cap. Both the cap and the piles are made of aluminium with an elasticity modulus of 69 GPa and Poisson's ratio of 0.33. In the experiment, the soil is inserted into a rectangular container with model dimensions of 0.254m wide, 0.457m long and 0.305m high, which are equivalent to prototype dimensions of 11.4m, 20.6m and 13.7 m respectively. The container borders were positioned at an adequate distance from the foundation to minimize boundary effects. For instance, the layout of the pile group of 4x4 at 3D diameter spacing, where D = 0.429m is the prototype pile side, is illustrated in Fig. 2(a).

The soil corresponds to a dense sand with internal friction and dilation angles of 37.1° and 0°, respectively, with the elasticity modulus varying with depth according to $E = E_o(p/p_a)^{0.5}$, where E_o is the elasticity modulus at the atmospheric pressure $p_a = 101$kPa and $p = \sigma_{ii}/3$ is the initial mean pressure of the soil. The value of E_o is taken as 17.4MPa as suggested by Yang and Jeremic [4]. The soil Poisson's ratio is 0.35 and its specific weight is 14.5 kN/m^3. The applied target lateral loads are 2000 kN, 2970 kN, 3400 kN, 3800 kN and 4400 kN for pile groups of 3x3, 3x4, 3x5, 3x6 and 3x7, respectively. The cap dimensions vary according to the size of the current group. As already said, the soil behaviour is based on the Drucker-Prager constitutive model, while an elastic behaviour is adopted for the aluminium piles. Zero-length contact elements are positioned at the soil-pile interface in order to model interaction effects at this zone, e.g., slipping and opening. A typical finite element mesh used for the pile group of 3x3 is depicted in Fig. 2(b). Due to symmetry considerations only half of the mesh is deemed. Mesh refinement is adopted at zones where high gradient of stresses are expected to occur. Here, various horizontal layers are defined to capture property variation with soil depth.

As the normal stiffness of the interface controls the lateral capacity of the group, its value was obtained by calibration of the numerical model with the corresponding experimental data. The procedure is as follows, an initial stiffness value was adopted based on the recommendation presented in [6] and then this value was varied up to approximately fit the available experimental load-displacement curve at all cases. The results shown in Fig. 3(a) indicate the use of $k_n = 400G_s$ and $k_s = 0.7k_n$, where G_s is the soil shear modulus, in average better fits the experimental data at all cases.

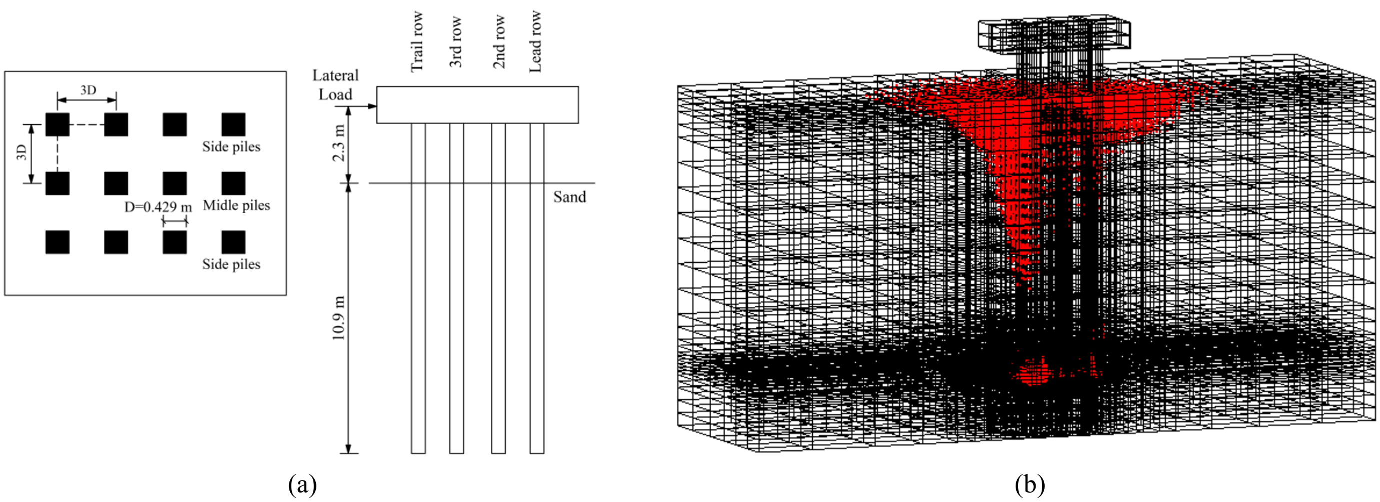


Fig. 2: Sketch of 4x4 group and typical finite element mesh

The evolution of bending moments along the pile length for the last load step and for the side piles of the 3x7 pile group are depicted in Fig. 3(b). As it may be observed, the side piles within the lead row are the most demanded ones with absolute maximum bending moments occurring at the pile-cap interface, being consistent with the fixed pile condition generated by the cap. Otherwise, the maximum bending moment at the buried part of the pile occurs approximately at a soil depth of 3m (7D). It is also noted that the experimental bending moment close to the pile-cap interface is well predicted by the numerical model. Furthermore, the bending moment diagrams for the middle and trailing piles are almost similar.

The spreading of the plastic points in a typical analysis such as the 3x3 configuration is displayed in Fig. 2(b) for the load step. It can be noticed that the passive and active plastic wedges develop in front and behind the aluminium foundation, respectively, as the lateral load is applied from the left to the right. This visualization is relevant as it corroborates the assumptions commonly employed in some analytical procedures based on the Theory of Limit Analysis to determine the load capacity of pile groups under lateral loading.

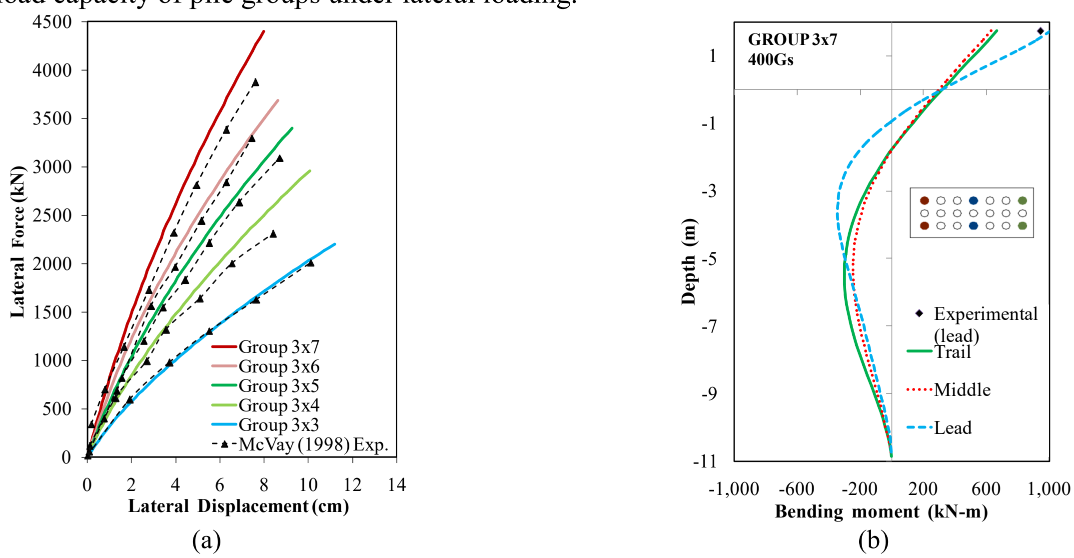


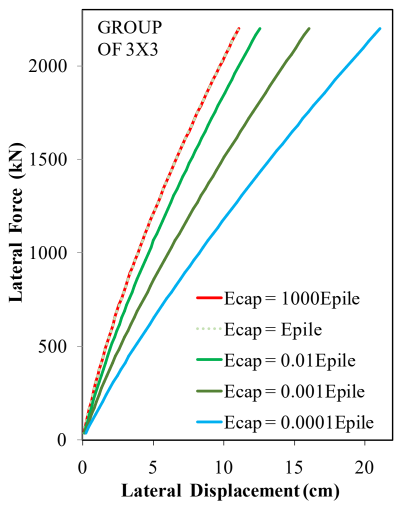
Fig. 3: Load-displacement curves and pile bending moment

3. Parametric studies

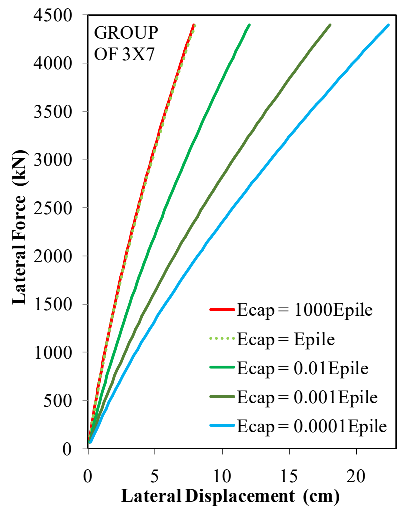
In this section, the importance of the cap stiffness and soil dilation on the numerical response for the pile groups of 3x3 and 3x7 previously studied in the last section are explored. It is considered that these cases define the bounds of other intermediate pile groups.

3.1. Effect of cap stiffness

In this section the cap stiffness was varied from a rigid behavior (E_{cap}=1000E_{pile}) to a flexible one (E_{cap}=0.0001E_{pile}), where E_{cap} and E_{pile} are the Young modulus of the cap and pile, respectively. The outcomes expressed in terms of load-displacement curves for the pile groups of 3x3 and 3x7 are depicted in Fig. 4(a) and Fig. 4(b), respectively. As it may be observed, when the cap stiffness is decreased the lateral displacement at the cap augments substantially in relation to the associated stiffer system. This increment can be as high as two and three times for the pile groups of 3x3 and 3x7, respectively. Also, the numerical response of the cases E_{cap}=1000E_{pile} and E_{cap}=E_{pile} overlap each other in practical terms.



(a)



(b)

Fig. 4 : Load-displacement curves

The variation of the bending moment with soil depth along the axis of the side trail, middle and lead piles for pile groups of 3x3 and 3x7 are illustrated in Fig. 5. It can be noticed again that the response of the cases E_{cap}=1000E_{pile} and E_{cap}=E_{pile} are practically the same. Also, it is interesting to note that when E_{cap}=0.0001E_{pile} (or E_{cap}=E_{pile}/10000) the bending moment at the pile top is almost zero, indicating thus the pile group behaves as if the cap does not exist. Also, the maximum bending occurs in the buried part of the piles at a depth between 3m and 5m. Otherwise, as the cap stiffness increases the maximum bending in the pile occurs at its top close to the pile-cap interface primarily for the lead piles.

3.2. Effect of dilation angle

The effect of dilation angle on the numerical response is explored in this section. As it was stated before, a dilation angle of 0º was adopted in the computations from section 2.1. Here, the additional dilation angles were taken as 6º, 12º, 24º and 36º. In Fig. 6(a) and Fig. 6(b) are depicted the load-displacement curves for the 3x3 and 3x7 pile groups. As it may be observed, the effect of the dilation angle is rather limited at the global level. This may be explained as most of the soil mass among the piles is physically limited to dilate because of the restriction imposed by the surrounding piles. In Fig. 7 is sketched the bending moment variation along the pile axis for the 3x3 and 3x7 pile groups and for the trail, middle and lead piles. It can be noticed here that soil dilation has also a minor effect, yielding slightly larger values when the dilation angle is smaller.

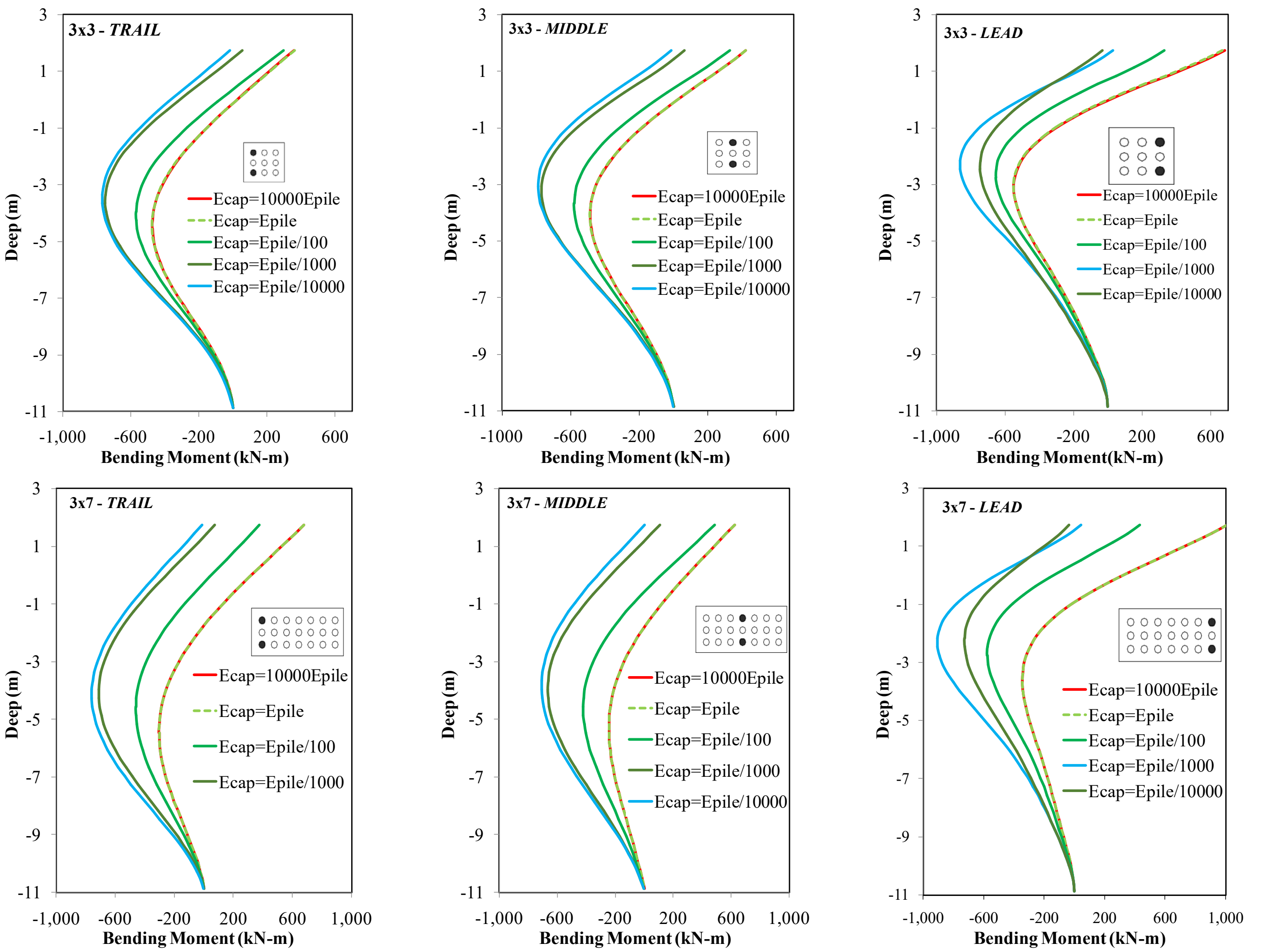
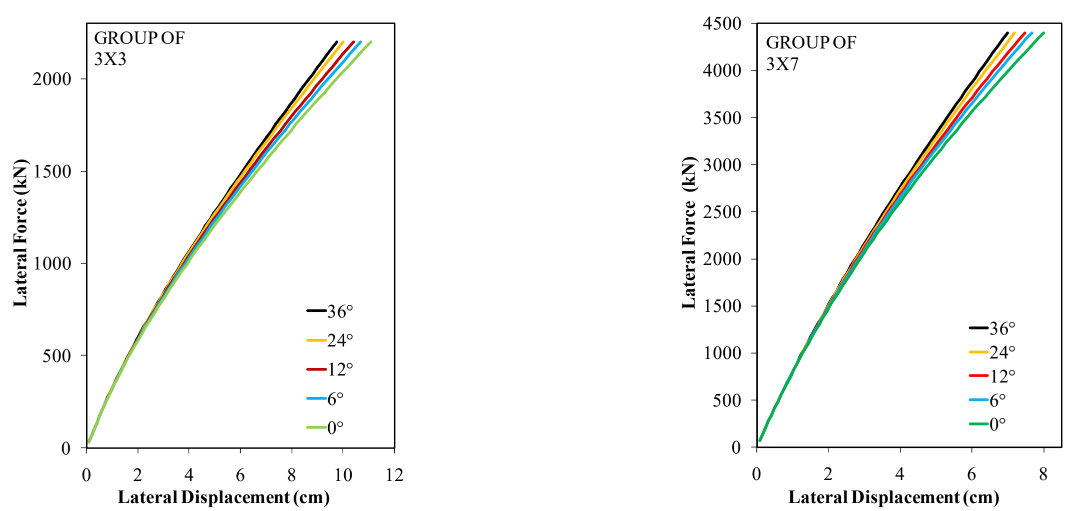


Fig. 5 : Bending moment versus depth for 3x3 and 3x7 pile groups



(a)

(b)

Fig. 6 : Load-displacement curves

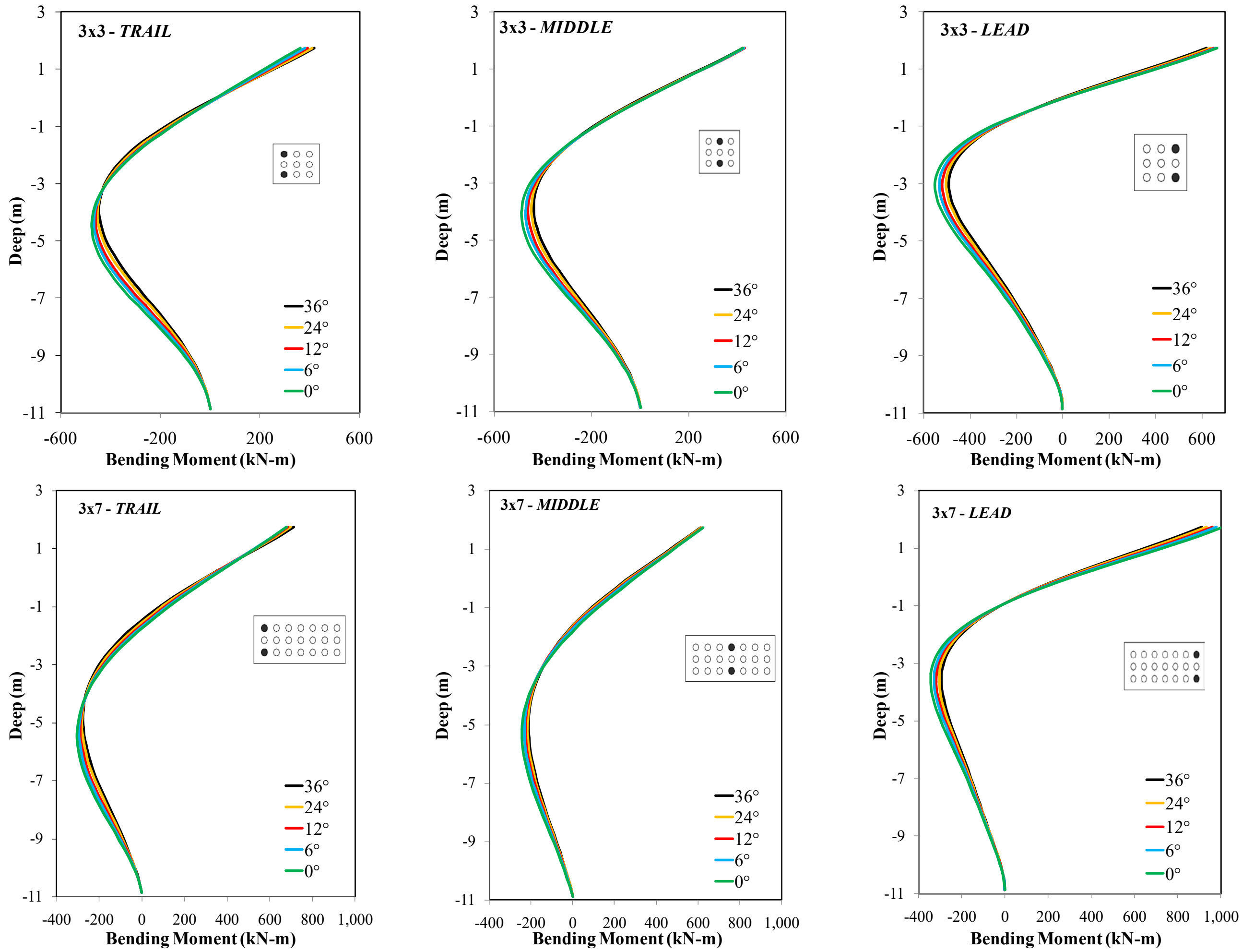


Fig. 7: Bending moment variation for 3x3 and 3x7 pile groups

4. Conclusion

In this paper a numerical model was proposed to model pile groups under lateral loads. Particularly, pile group configurations of 3x3 to 3x7 were studied. Two main aspects were explored in the parametric studies, the effect of the cap stiffness and soil dilation on the numerical response. It was found that as the cap stiffness is progressively reduced, the lateral displacement at the pile cap centre significantly increases under ultimate loads up to approximately two times the value of the corresponding rigid system for the 3x3 pile group, and almost three times for the 3x7 pile group. These increment ratios reflect the difference between a pile group with a rigid cap and that without a cap (or fully flexible cap). Otherwise, the effect of soil dilation expressed by means of various dilation angles was found to be negligible at both the global and local level. This is because the presence of surrounding piles limits the soil expansion.

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