Centrifuge Modelling of Inclined Pile Foundations under Seismic Actions

Zheng LI
LUNAM Université, IFSTTAR (L'Institut français des sciences et technologies des transports, de l'aménagement et des réseaux) & Ecole Centrale de Nantes, GeM (Research Institute of Civil Engineering and Mechanics), CNRS UMR 6183, France

Sandra ESCOFFIER
LUNAM Université, IFSTTAR (L'Institut français des sciences et technologies des transports, de l'aménagement et des réseaux), GER, F-44341 Bouguenais, France

Panagiotis KOTRONIS
LUNAM Université, Ecole Centrale de Nantes, GeM (Research Institute of Civil Engineering and Mechanics), CNRS UMR 6183, 1 rue de la Noë, BP 92101, 44321, Nantes, cedex 3, France

SUMMARY:
The behaviour of inclined pile foundations under seismic actions is not thoroughly investigated. This paper provides new experimental data on the responses of inclined pile foundations tested using the centrifuge modelling technique. The experiments are designed to investigate soil-structure interaction. Different mock-ups are subjected to several earthquakes with low intensities and the evolution of their frequency components is identified. Experimental results are analyzed in terms of response frequencies and section forces in the piles. Numerical simulation of the soil-structure systems is conducted in the linear viscoelastic domain. Comparison of the experimental and the numerical response both in the frequency and in the time domain is satisfactory. From both the experimental and numerical results preliminary conclusions can be drawn concerning the dynamic characteristics of the soil-structure systems and the effects of inclined piles.

Keywords: Inclined pile foundation; Centrifuge modelling; Seismic response; Soil-structure interaction; Numerical simulation

1. INTRODUCTION

Nowadays, with the increasing need for infrastructures and the decreasing availability of space, both structural and geotechnical engineers are challenged to design, analyze, and evaluate more expensive and strategic structural systems (e.g. high-rising buildings, offshore platforms, multi-story highways, etc.) submitted to extreme lateral loadings (e.g. earthquakes, gusty winds, terrorist attacks, etc.). Inclined piles are sometimes used to provide sufficient lateral stiffness but also in order to avoid interference between the piles and underground pipes (for example in power-plant structures). Although their behaviour under static loadings is fully investigated, their response under seismic loadings is not clear; the use of inclined piles is not recommended in different recommendations and design codes such as AFPS (1990) and Eurocode EC8 (2003). The main problems that appear during an earthquake loading are summarized hereafter:

- inclined piles induce large axial forces translated to the pile cap;
- inclined piles are subjected to "parasitic" bending stress due to soil densification and/or soil consolidation;
- the bending capacity of inclined piles is reduced due to important tensile stress;
- the spatial configuration of inclined piles (e.g. symmetric or asymmetric) has significant influence on the dynamic response of the super-structure.

Different references can be found in the literature. Poulos (2006) studied the response of a 3 × 2 pile group subjected to lateral ground movement and found out that an increase in pile batter leads to a reduction in settlement, to a significant increase in pile-cap rotation, and an increase in axial force and bending moment at the pile head. Sadek and Shahrour (2006) studied the seismic response of inclined micro piles and showed that the symmetrically inclined micro piles of a 4 × 4 group supporting a short structure develop smaller bending moments and larger axial forces compared to a similar group of vertical micro piles. Deng (2007) performed analysis for a large pile group containing inclined piles and showed that kinematic loading can have a major impact on the magnitude of the maximum axial force that developed in the batter piles. In the study, inclined piles developed 5-8 times greater axial forces than the vertical piles. The use of batter piles leads to a concentration of high axial forces that can induce major damages in the pile cap and/or pile head.

However, other studies showed that inclined piles perform well during an earthquake event. A proper design can be beneficial both for the super-structure and the piles themselves (G. Gazetas and G. Mylonakis, 1998). Recent research (Guin 1997) indicated that the response of a typical bridge type structure submitted to a representative seismic excitation may improve when supported by inclined piles. I. Lam and G. Martin, 1986 showed that both cap displacements and pile bending moments may be reduced dramatically in liquefied soil due to the stiffening effect of the inclined piles Field investigations also proved the beneficial performance of inclined piles during earthquake events. M. Pender 1993 presented some case studies during the Maya Warf in Kobe earthquake (1995), and the Landing Road Bridge in New Zealand during the Edgecumbe earthquake. Until today, the beneficial or detrimental role of batter piles to the seismic response of the superstructure or the foundation itself has not yet been thoroughly clarified.

In this paper, both experimental work and numerical simulations were carried out to study the performance of inclined pile foundations under earthquake actions. Experiments were done on the centrifuge of IFSTTAR (Institut français des sciences et technologies des transports, de l'aménagement et des réseaux) using two configurations: a symmetric vertical pile group and symmetric inclined 15° pile group. Numerical simulations were performed with a 3D ABAQUS finite element model considering the characteristics of the soil and the superstructure.

2. CENTRIFUGE MODELLING OF THE SEISMIC RESPONSE OF INCLINED PILES

In geotechnical engineering, it is often very difficult or even impossible to do in-situ test on full-scale models. The centrifuge technique is a good alternative based on the idea that the same stress state is adopted on the prototype and the small-scale mode. Dynamic centrifuge experiments have been performed at 40g gravity level, and the results are discussed hereafter.

2.1. Dynamic Centrifuge Tests of Pile Foundations

Dynamic centrifuge tests were performed on a short building on pile foundations. Two pile configurations were adopted: vertical and inclined. The connection between the building and the pile cap was assumed perfect. The building was designed to have a frequency of 2 Hz in the prototype scale considering a fixed base condition. In this article, results are compared both in the frequency and in the time domain. The bending moments along the piles are also studied. A sketch of both experimental configurations is shown in Fig.2.1.

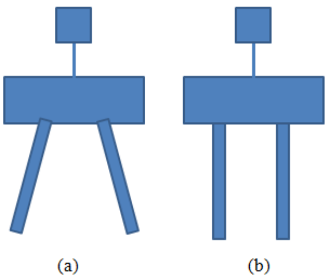

Figure 2.1 Short building on different pile foundation configurations: (a) inclined pile group; (b) vertical pile group;

2.1.1. Soil Properties

Fontainebleau NE34 sand was used in the experiment. The parameters of the soil were controlled by the pluviation technique (Air Pluviation at 1g gravity level). Several parameters, see in **Table 2.1.**, were determined to achieve an 80% relative density of the soil.

Table 2.1. Parameters of pluviation

Soil	H (cm)	HV (Hz)	Gap (mm)	Number A/R	I_d (%)	γ_d (kN/m^3)
Fontainebleau NE34	70	22	3	4	80%	16.155

The sand used in the experiment has a minimum and maximum void ratio of 0.545 and 0.866 respectively. The evolution of the sand Young's modulus Vs. the depth is calculated according to the equations proposed by Estelle Delfosse-Ribay (2004):

$$G_{max} = A \cdot \frac{(B-e)^2}{(1+e)} \cdot \sigma_3'^{C} \tag{2.1}$$

$$E_{max} = 2 \cdot G_{max} \cdot (1+\nu) \tag{2.2}$$

Where: G_{max} is the maximum shear modulus of the NE34 sand; A, B and C are constants 200, 2.17 and 0.47 for the NE34 sand respectively; e is the void ratio 0.716 respect to the relative density in the experiment; $\sigma_3'^{C}$ is the confining pressure; E_{max} is the Young's modulus of the NE34 sand; ν, 0.25, is the Poisson ratio.

In prototype scale, the profile of the evolution of the Maximum Shear Modulus of the sand with respect to the soil depth is shown in Fig. 2.2.

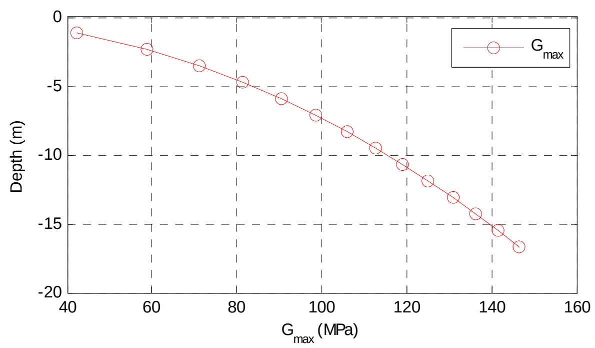

Figure 2.2 Evolution of the Maximum shear modulus of the sand

Eqn 2.2 is used to calculate the profile of the Young modulus used in the finite element model.

2.1.2. Input seismic signal

The Martinique Jara real seismic signal was selected. Its time representation and normalized frequency contents are shown in Fig.2.3 (prototype scale). Its energy is concentrated in the frequency range between 1.8Hz ~ 4Hz. The peak ground accelerations for Martique Jara earthquake is around 0.1g. The Arias intensity is calculated using Eqn. 2.3.

$$AI = \frac{\pi}{2g}\int_0^{tr} a^2(t)dt \quad (2.3)$$

Where: $a(t)$ is the acceleration time history, t^r is the total duration of the accelerogram and g is the gravity acceleration. The Arias intensities of the small earthquakes are found 0.2 m/s.

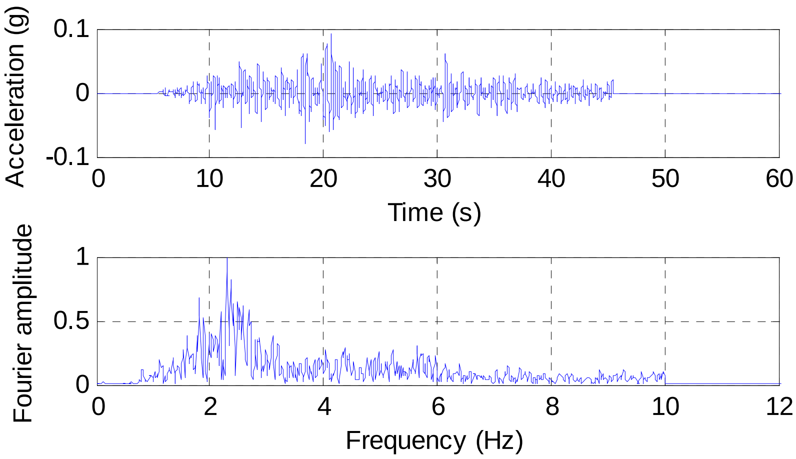

Figure 2.3 Input signal: Martinique Jara earthquake (-1dB). Accelerogram and Fourier spectrum (prototype scale)

2.1.3. Arrangement of sensors

During the dynamic centrifuge tests, the responses of the soil and the soil-pile system were captured by series of accelerometers. Bending moments and axial forces were measured using strain gauges positioned along the piles. Fig.2.4 shows the arrangement of the accelerometers for the inclined pile group (similar positions are used for the vertical group): the red triangles represent their positions, and the peaks of the triangles define the positive direction. Accelerometers are divided in two groups related to the measured response e.g. the vertical and horizontal group. For the vertical group, the two vertical sensors on the pile cap named CH12 and Ch13 were used to measure its rotation. In order to capture the response of the soil column, the accelerometers were aligned along a vertical line. Three additional laser sensors, marked by blue arrows, were used to measure the displacements of the pile cap and the building.

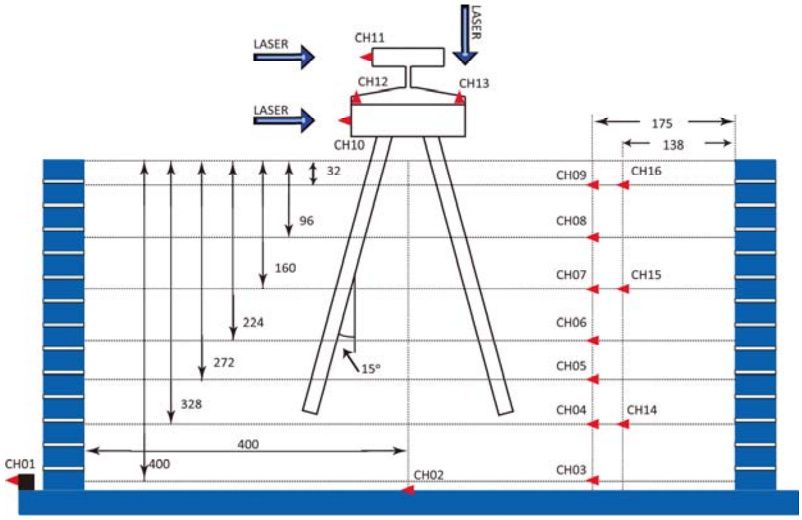

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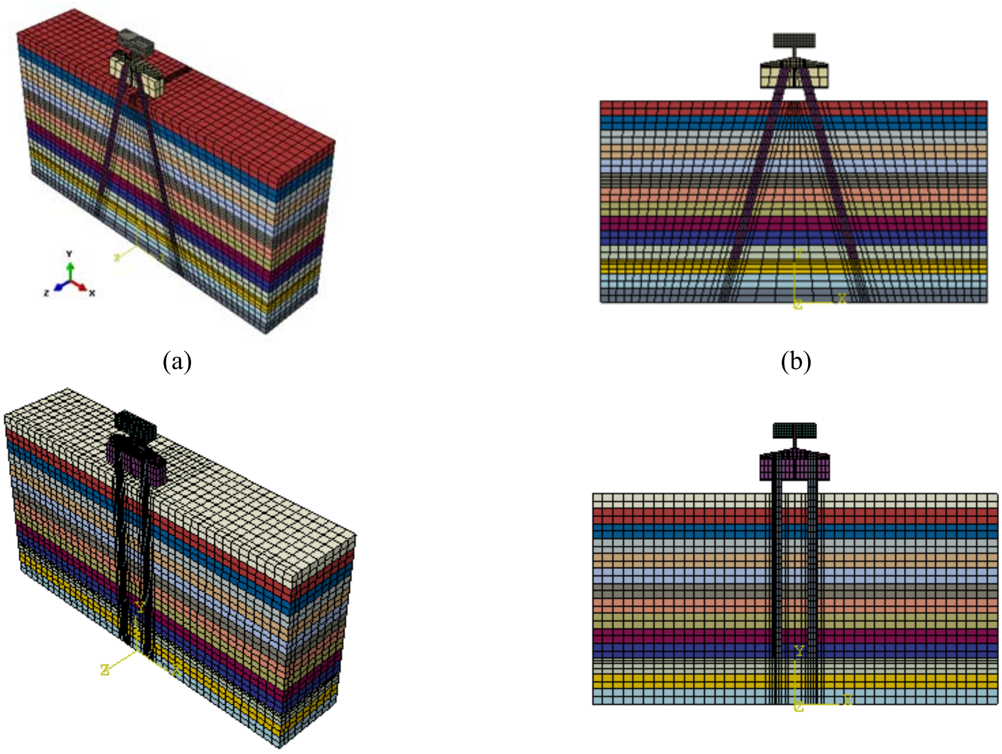
Figure 2.4 Arrangement of accelerometers and laser sensors for the inclined pile group

3. NUMERICAL MODELLING OF THE CENTRIFUGE EXPERIMENTS

The objective of the numerical modelling of the centrifuge test is to have a better understanding of the mechanism of the soil-pile-interaction and the response of the whole system. A 3D finite element ABAQUS model was adopted following the geometry and the material properties presented in section 2. Only half of the system was modelled considering the inclined and the vertical pile foundation groups (Fig. 3.1).



(a) (b)

(c) (d)

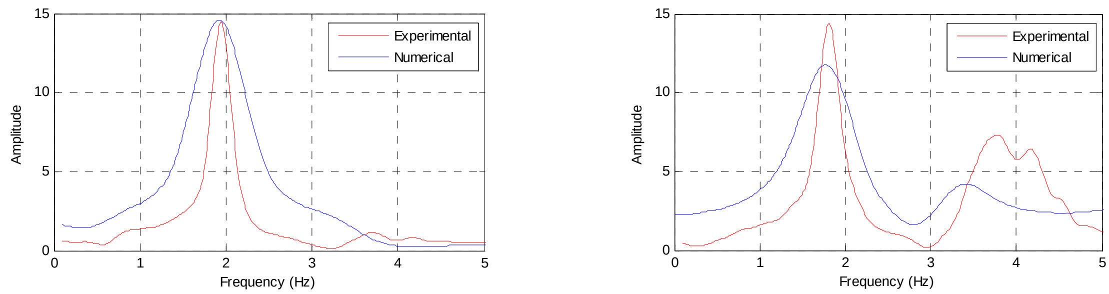
Figure 3.1 3D finite element meshes for the inclined and the vertical pile groups:
(a) iso-view (b) front view

The soil, the piles and the pile cap were modelled using 3D solid elements. The soil was divided into 14 layers according to the structure of the ESB box (Equivalent shear beam box) used in the centrifuge test. The Young's Moduli of the different layers follow section 2. As it was suggested by Zienkiewicz et al. (1989), the lateral boundaries of the mesh were constrained is such a way that the displacements of the left-hand-side boundary equal those of the right-hand-side. A 5% Rayleigh damping was used in the modeling. The behavior of the materials was considered linear elastic. The contact elements are used to model the interface between pile and soil.

4. RESULTS AND DISCUSSION

4.1. Experimental Vs. numerical results in the frequency-domain

In earthquake engineering design, during an elastic response spectrum analysis for example, the eigenfrequencies of a building are normally calculated considering fixed base conditions. However, due to soil-structure interaction, the frequencies decrease. In our experiment, the short building was designed to have a 2Hz frequency assuming fixed base conditions. With the soil-structure interaction, the frequency decreased both for the inclined (1.9Hz) and the vertical configuration (1.8Hz) as shown in Fig.4.1 where the experimental and the numerical results are presented (the precision of the transfer function is 0.01 Hz).



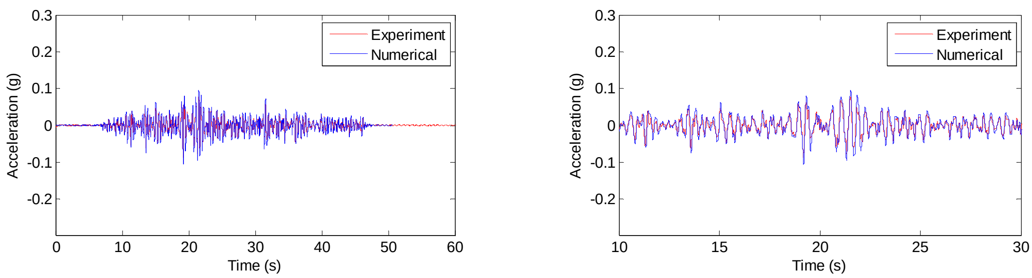
(a) (b)

Figure 4.1 Transfer function between the top building output acceleration and the base input signal: (a) inclined pile group (b) vertical pile group

4.2. Experimental Vs. numerical results in the time-domain

The experimental soil acceleration time histories at different depths are compared to the numerical results in Fig.4.1. Results show a strong linear correlation (the linear correlation factors for 16m, 6.4m and 1.28m depths are 0.98, 0.96 and 0.95 respectively), meaning that the numerical model is able to reproduce satisfactorily the behaviour of the system.

Depth 16m



Depth 6.4m

Depth 1.28m

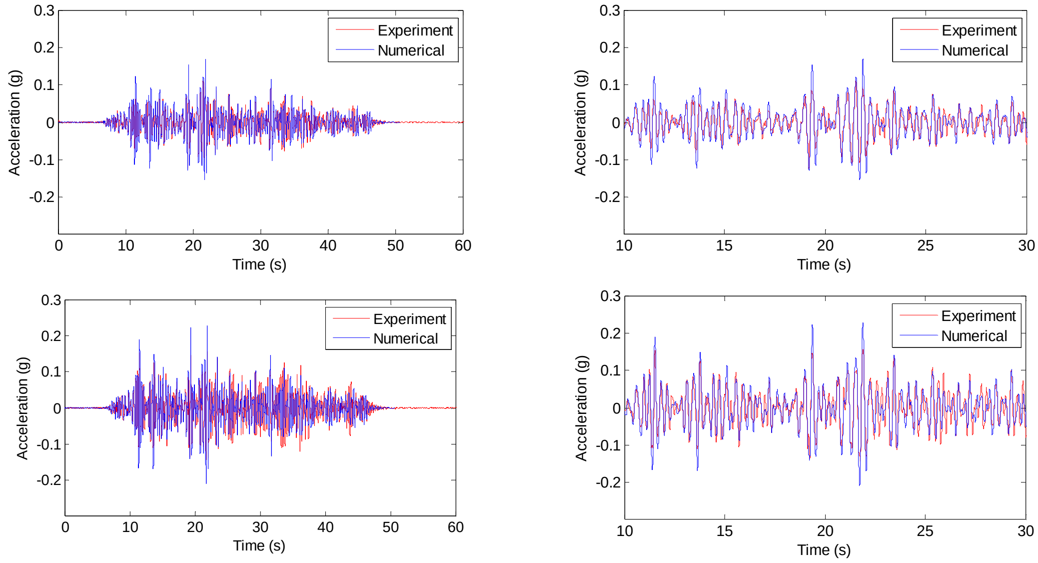
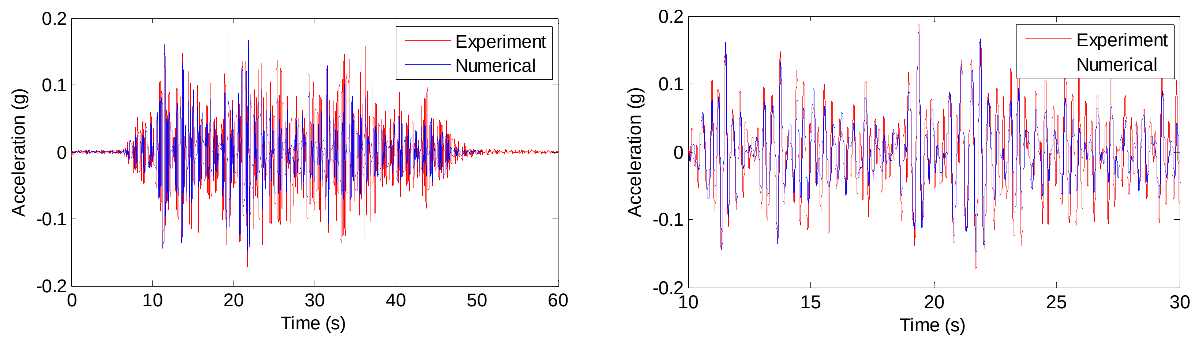


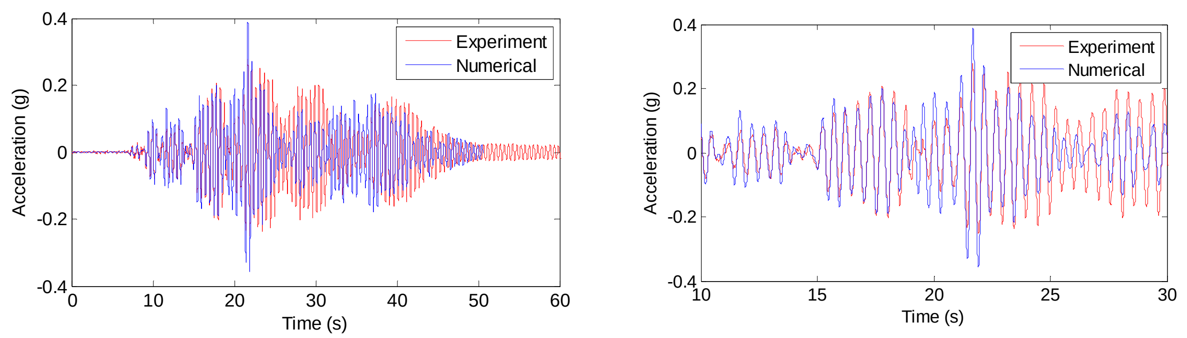
Figure 4.1 Soil acceleration time histories at different depths. Experimental Vs. Numerical results. Right sides are the detailed view of acceleration time history

The responses of the pile cap and the top of the short building are shown in Fig.4.2, Fig4.3, Fig.4.4 and Fig.4.5. Agreement is again acceptable. The peak acceleration at the pile cap for the inclined pile group is found in experiments 0.19g and 0.39g (0.18g and 0.28g in numerical simulation) for the vertical pile group. The peak acceleration at the top of the building is experimentally 0.34g and 0.46g (0.33g and 0.44g in numerical simulation) for the inclined pile group and the vertical pile group respectively. The response of the inclined pile group in terms of peak accelerations seems better as higher horizontal inertia load apply to the vertical pile group.



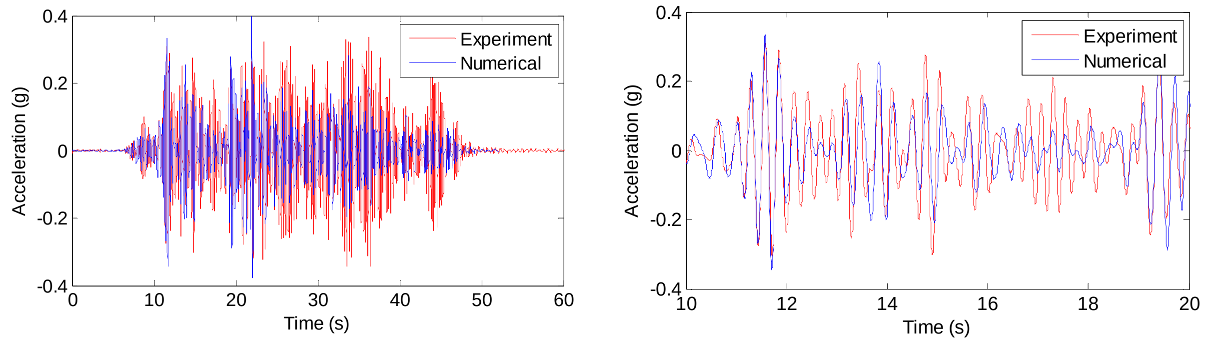
(a) (b)

Figure 4.2 Acceleration time-history at the pile cap for the inclined pile group:
(a) Whole sequence (b) Detailed view



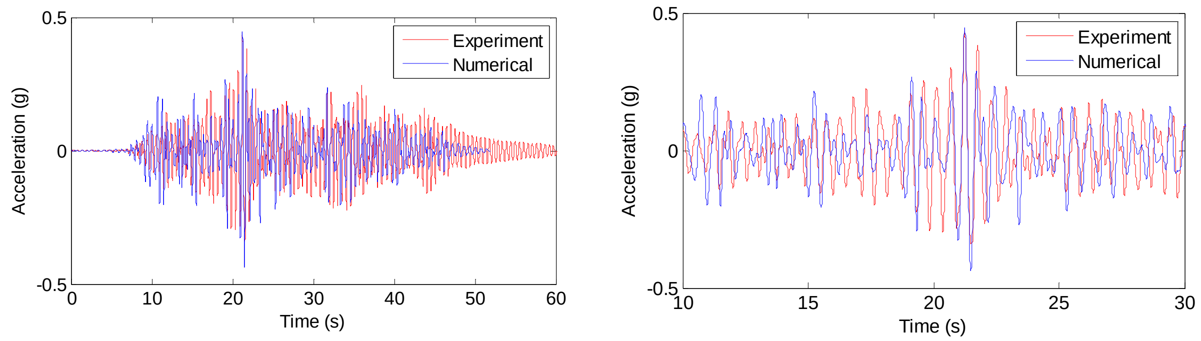
(a) (b)

Figure 4.3 Acceleration time-history at the top of the building for the inclined pile group: (a) Whole sequence (b) Detailed view



(a) (b)

Figure 4.4 Acceleration time-history at the pile cap for the vertical pile group: (a) Whole sequence (b) Detailed view



(a) (b)

Figure 4.5 Acceleration time-history at the top of the building for the vertical pile group: (a) Whole sequence (b) Detailed view

4.3. Experimental Vs. numerical results: Pile bending moments

Experimental bending moments were calculated from strain gauge measurements (in Fig. 4.6 red points indicate the location of strain gauges at the top of pile head).

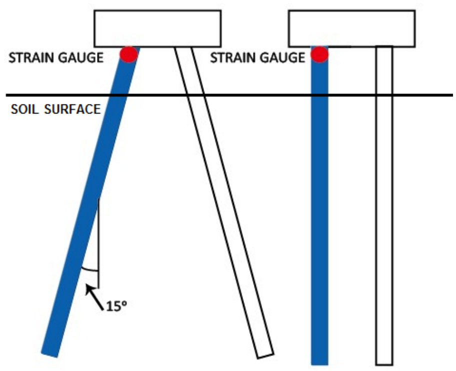


Figure 4.6 Positions of the strain gauges for the inclined and the vertical pile group

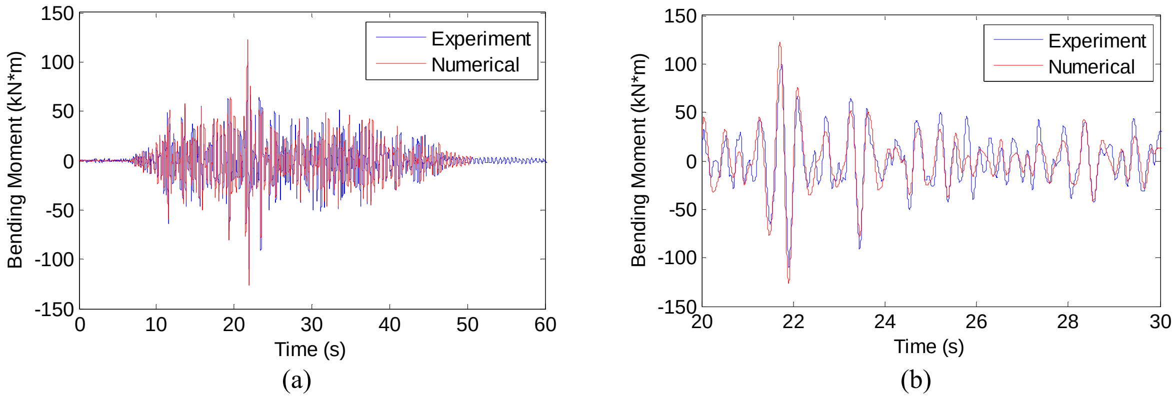


Figure 4.7 Bending moment time-history for the inclined pile group:
(a) Whole sequence (b) Detailed view

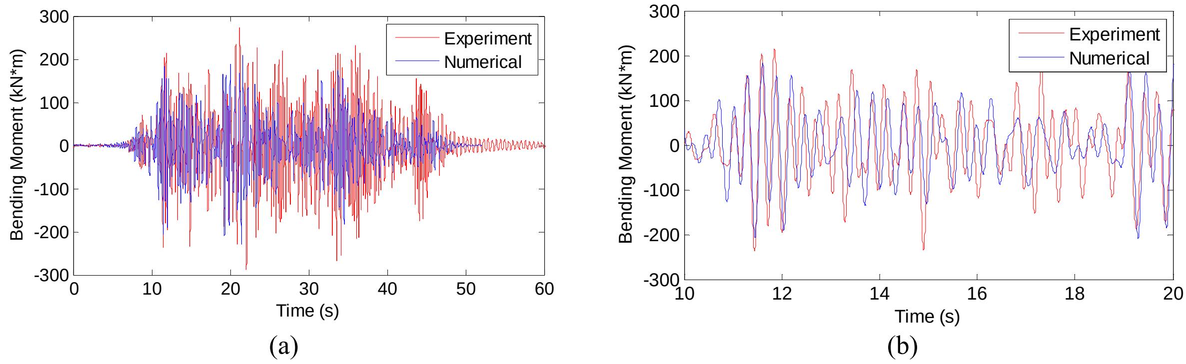


Figure 4.8 Bending moment time-history for the inclined pile group:
(a) Whole sequence (b) Detailed view

Fig.4.7 and Fig.4.8 show that the behaviour of the vertical and the inclined pile groups under small intensity earthquakes is correctly reproduced by the numerical model. As it is shown in Fig.4.9, the maximum bending envelope curve for the inclined pile group is less than for the vertical one (the maximum bending moments at the pile head are 180 kNm and 300 kNm for the inclined and the vertical pile group respectively). Significant experimental bending moments along the vertical pile group are situated not only at the pile head but also around 3m of the soil surface. The maximum bending moment for the inclined pile group is situated at the top of the pile head.

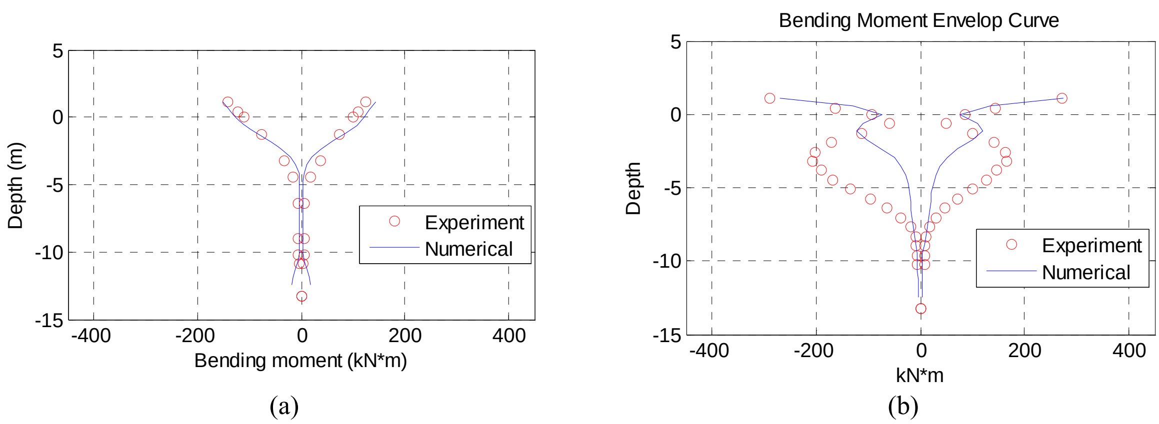


Figure 4.9 Maximum bending moment envelope curve:
(a) Inclined pile group (b) vertical pile group

5. CONCLUSIONS AND PERSPECTIVES

The behaviour of a vertical and an inclined pile foundation group under small earthquakes is studied in the paper by centrifuge modelling and numerical simulations. The numerical model assumes linear elastic material properties and free-field boundary conditions and it is able to reproduce the main characteristics of both pile foundation systems. From the results, it seems that buildings constructed on vertical pile foundations have lower frequencies and that under certain circumstances inclined pile foundations have a better behaviour than vertical pile foundations. In the near future, the response of inclined pile foundations under more severe earthquakes and the influence of different types of super-structures will be studied.

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