Dynamic response and load distribution of pile groups in layered liquefiable ground

Xing Liu i), Rui Wang ii) and Jianmin Zhang iii)

i) Ph.D Student, State Key Laboratory of Hydrosience & Engineering, Tsinghua University, Beijing 100084, China.
ii) Lecturer, State Key Laboratory of Hydrosience & Engineering, Tsinghua University, Beijing 100084, China.
iii) Professor, State Key Laboratory of Hydrosience & Engineering, Tsinghua University, Beijing 100084, China.

ABSTRACT

Three dimensional finite element simulation of 3×5 pile groups in layered liquefiable ground during earthquake was carried out in this paper to investigate the dynamic load distribution within the pile groups. A unified plasticity model for large post-liquefaction shear deformation of sand was used for the appropriate simulation of soil liquefaction. Load distribution over the corner, edge and central piles was studied. The influence of pile spacing on the load distribution was also studied. Numerical results showed the corner piles, the edge piles and the central piles were subjected to varying dynamic loads. The pile spacing had significant impacts on the load distribution of pile groups.

Keywords: pile group, liquefaction, FEM, load distribution

1 INTRODUCTION

Pile groups are widely used in liquefiable ground to enhance the bearing capacity. In general, pile groups are considered to be capable of resisting earthquakes. However, since Hamada et al [1] found the fact that pile groups were damaged during the Niigata earthquake, many researchers began to research the dynamic response of pile groups in liquefied foundations during earthquakes. According to earthquake investigations, damage to pile groups in liquefied ground mainly occurred in two directions, lateral and vertical. Lateral damage to pile groups in liquefied ground was mainly caused by pile-soil kinematic interactions, pile-structure inertial interactions, dynamic interactions coupled by kinematic and inertial interactions in horizontal ground during pre- and post-liquefaction, and lateral spreading in inclined ground post-earthquake [2-4]. Vertical damage to pile groups in liquefied ground was mainly caused by settlement of pile groups post-earthquake [5-6]. Although physical modelling is an experimental method of good repeatability and high reliability, it is costly and complicated. As the finite element method (FEM) is economical and practical to research the dynamic response of pile groups, numerical simulations is more and more popular to simulate dynamic time-history analysis. Wu and Finn [7, 8] raised the dynamic linear and nonlinear FEM to study the dynamic response of pile group in the time-domain and frequency-domain. Yang et al [9] used the OpenSees finite element framework to simulate the response of pile groups. The complicated mechanical behaviour of liquefiable sands should be finely and reasonably described in good numerical simulations. Based on many results of undrained triaxial tests and torsion shear tests, Jian-min Zhang, Gang Wang [10-12] formulated a plasticity model for large post-liquefaction shear deformation of sand considering physical mechanisms. Rui Wang et al [13, 14] developed a unified plasticity model for large post-liquefaction shear deformation of sand, which was able to achieve unified description of the behaviour of sand at different states under monotonic and cyclic loading during both pre- and post-liquefaction regimes. This constitutive has been implemented in OpenSees platform [15], named CycliqCPSP.

This paper employed the unified constitutive model, CycliqCPSP, to research the dynamic response of pile group in liquefiable ground subjected to earthquake loads. Attention was paid to the distributions of moments and shear forces in different piles.

2 PILE-SOIL-STRUCTURE MODELS AND PARAMETERS

2.1 Geometric model

The geometric model with three layers shown in fig. 1, an 11 meters thick non-liquefied layer at the top, a 12 meters thick liquefied layer at the middle and a 12 meters non-liquefied layer at the bottom. The pile cap was 19×11×2 m. The surface of the pile cap was at ground level. The 5×3 pile groups contains 5 piles in x direction, 3 piles in y direction. Piles were square, and side length was 1 meter. The length of piles was 24 meters. The pile space was 4 times diameter (4D). Pile group was connected with the pile cap rigidly. A
structure was built above the pile cap, 15 meters long, 7 meters wide and 30 meters tall. The mesh of the computation model is shown in Fig. 2. The model consisted of 5670 nodes and 4520 elements.

![Mesh of the computation model.](image)

**Fig. 2.** Mesh of the computation model.

### 2.2 Materials model

The unified plasticity model, CycliqCPSP material model, was employed to simulate the liquefied layer at the OpenSees platform. Material parameters are shown in Table 1. Non-liquefied layers, pile groups, pile cap and the structure were simulated by elastic model. Material parameters were shown as Table 2. Pile and structure elements were brick elements. Soil elements were solid-fluid fully coupled u-p elements.

#### Table 1. Parameters of CycliqCPSP material model

| $G_1$ | $k$ | $h$ | $M$ | $d_{0,1}$ | $d_{0,2}$ | $\gamma_{de}$ | $\kappa$ |
|-------|-----|-----|-----|-----------|-----------|--------------|----------|
| 125   | 0.006 | 0.8 | 1.5 | 0.05      | 1.5       | 0.05         | 45       |
| 1.8   | 0.023 | 0.7 | 0.837 | 1.1       | 8         | 1.961        |

#### Table 2. Parameters of material models

| Material        | $E$/kPa | $\nu$ | $\rho$/g/cm$^3$ |
|-----------------|---------|-------|-----------------|
| Non-liquefied   | 40000   | 0.3   | 1.8             |
| Piles           | 700000000 | 0.33  | 2.7             |
| Pile cap        | 900000000 | 0.33  | 2.7             |
| Structures      | 200000000 | 0.33  | 3.5             |

### 2.3 Seismic wave

The seismic wave was Parkfield seismic wave. The time history curve is shown as Fig. 3. The maximum acceleration was -4.835m$^2$/s at 6.75s.

![Parkfield seismic wave.](image)

**Fig. 3.** Time history of Parkfield earthquake.

### 3 RESULTS AND DISCUSSIONS

Accelerations, pore pressures and displacements of all nodes and stresses and strains of all elements were recorded. Dynamic responses of foundations and pile groups were analyzed.

As there was a liquefied layer in the foundations, the seismic wave was weakened spreading from the bottom to the surface. As shown in Fig. 4, the acceleration amplification factor in the liquefied layer decreased from the bottom to the top.

![Envelope of the acceleration amplification factor in different soil layers.](image)

**Fig. 4.** Envelope of the acceleration amplification factor in different soil layers.

As shown in Fig. 5, the excess pore pressure ratio eventually reached 1.0 within the middle layer after 8s, thus achieving liquefaction.

![Time history of the excess pore pressure ratio of different depth.](image)

**Fig. 5.** Time history of the excess pore pressure ratio of different depth.
According to stresses of pile elements, shear forces and moments of all piles can be obtained. Pile IDs within the pile group are shown in Fig. 6.

Fig. 6. Serial number of pile groups.

Fig. 7 shows maximum shear forces of each pile head. The maximum shear force of pile No. 1 was the biggest, about 6070 kN, while that of pile No. 8 was the smallest, 4560 kN, with a difference of 32%. Fig. 8 shows that maximum moments of each pile were significantly different. The maximum positive moment of the corner pile was about 1600 kNm, while that of the central pile was about 1240 kNm, with a difference of 30%. The load distribution of pile group in liquefied ground was that peripheral piles carried more than inner piles.

As shown in Fig. 9, not only the maximum moments of corner pile were different from that of the central pile, but also the locations of the maximum moments were different. The maximum moment of the corner pile occurred at the interface between the upper non-liquefied layer and the middle liquefied layer, while the maximum moment of the central pile occurred at the pile head connected with the pile cap.

The reason why the maximum moment of pile No. 1 occurred at the interface between the upper non-liquefied layer and the liquefied layer may be that the soil layers inside the pile group were hard restrained by the pile group, while the soil layers outside the pile group were not restrained by the pile group.
As shown in Fig. 10, the maximum mean shear strain difference between upper non-liquefied layer and liquefied layer inside pile group was larger than that outside pile group. Thus maximum moment of the corner pile occurred at the interface between upper non-liquefied layer and liquefied layer. So it was important to enhance the strength of pile head and interface between non-liquefied layer and liquefied layer in the design.

As shown in Fig. 8, the standard deviation of maximum moments of different piles was about 120. To determine the influence of pile spacing on the load distribution, models with pile spacing from 3D to 7D were investigated. Fig. 11 showed the standard deviation of maximum moments and the ratio of largest to smallest maximum moment. Note the distribution pattern remained the same regardless of the pile spacing, with the corner pile bearing the largest moment. The standard deviation of maximum moments peaked at a pile spacing of 4D, and then decreased with increasing pile spacing.

![Fig. 11. Influences on unevenness by elastic shear modulus constants.](image)

**4 CONCLUSIONS**

This paper employed the OpenSees finite element framework to analyze the dynamic response of pile groups in layered liquefiable ground during earthquake. The following conclusions can be drawn:

- The computation model and method employed in this paper can reproduce the dynamic response of pile groups in liquefiable ground.
- In the investigated pile groups in layered liquefiable ground, the maximum moment of the corner piles was the largest, and that of the central piles smallest. The maximum moment of the corner piles occurred at the interface between upper non-liquefied layer and middle liquefied layer, while that of the central piles was at the pile head.
- Pile spacing had a significant impact on the load distribution of pile groups.

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