CONCEPT OF A NEW PILE CAP TO MITIGATE LIQUEFACTION PROBLEMS

A. K. Dey¹, Kaushik Deb²

ABSTRACT

Soil liquefaction is one of the primary concerns during construction of pile foundations on saturated sand in seismically active areas. A lot of experiments have been done to study the behaviour of pile foundation buried in liquefiable sand subjected to dynamic loads. However these studies are concentrated mostly on liquefaction induced lateral movement of soil and pile-soil interaction. In the present study, the vertical settlement of pile groups founded in liquefiable soil subjected to dynamic loading has been observed by conducting model tests on 2x2 pile groups in the laboratory and a new geometrical modification of the pile cap has been used to mitigate the problem of vertical settlement. The model study was conducted in a steel tank of length 1m, breadth 1m and height 1m. Locally available poorly graded sand was used as the foundation soil. The sand was compacted at a relative density of 40% up to a height of 0.8 m from the bottom of the tank. Controlled volume method of compaction was used to maintain the relative density. The soil was fully saturated up to its total depth. The model piles in the 2x2 pile groups had a diameter of 20 mm and a length of 260 mm with a centre to centre spacing of 60 mm. A total of three geometrical shapes were considered in the study. All the pile caps had the same square plan area of 100mm x 100mm. The first pile cap was designed as a conventional pile cap with a depth of 20 mm. But the other two pile caps had a triangular and semicircular cross section respectively, with a depth of 50 mm. All the piles were designed as fixed piles and were embedded into the pile cap by 15 mm. The pile caps were designed to have such geometrical configuration so that they have a larger surface area and volume so that during liquefaction, when the foundation is surrounded by liquefied soil, the upward force acting on the pile cap is more and consequently, the vertical subsidence of the foundation is less. Strain gauges were attached at the junction between the pile caps and the piles to determine the bending moment variation and LVDTs were attached to the top of the pile caps to measure the vertical settlement. Accelerometers were attached to the pile caps and the tank to determine the dynamic parameters. One dimensional variable speed shake table was used to impart steady state vibrations to the steel tank. Three tests were conducted for 10 cycles on each of the model pile groups at three different frequencies of 1Hz, 1.5 Hz and 2 Hz. The results from the tests were analysed. Comparative study of the different pile groups was made and definite conclusions

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were drawn regarding the suitability of the new designs in relation to mitigation of vertical settlement in piles founded on liquefiable soil subjected to dynamic loading. Mathematical model involving simple equations was used to find out the vertical settlement of saturated sand under dynamic load for comparison with the analysed test data. Also to check the suitability of the model piles in field, static tests were performed on the pile groups and the load settlement characteristics of the pile groups were obtained. The same were obtained for the pile groups with rectangular and triangular cross-section of pile cap under plain strain condition using PLAXIS 2D software and the results were compared. The pile group with semi-circular cross-section of pile cap could not be analysed with PLAXIS 2D due to modeling limitations. It was found that the pile groups of different cross-sections of the pile cap had almost the same load bearing capacity and the pile group with semi-circular cross-section of pile cap underwent least vertical settlement under dynamic loading.

Keywords: Liquefaction, Pile cap, Pile foundation, Plaxis 2D, Vertical settlement, Shake table, Static test.
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ABSTRACT: This paper presents the results of static and dynamic tests on 2x2 model pile groups with three different cross-sections of pile cap, viz. rectangular, triangular and semi-circular founded on liquefiable sand, the objective being to find a suitable design of pile cap to mitigate vertical settlements in liquefiable soil induced due to dynamic loads. The model tests were conducted on a fully saturated sand bed of relative density 40% in a steel tank of size 1m x 1m x 1m. The model piles were instrumented with strain gauges and LVDTs. One dimensional shake table and hydraulic jack were respectively used for the dynamic and static loading. Mathematical model involving simple equations was used to compare the dynamic test results and PLAXIS 2D was used to compare the static test results. It was found that the pile group with semi-circular cross-section of pile cap underwent the minimum vertical settlement under dynamic loading.

INTRODUCTION
Pile foundations buried in liquefiable soil have always been a cause of concern for the foundation designer and geotechnical experts. Loose cohesionless sands and silts below the water table develop high pore water pressures or liquefy during strong earthquake shaking. This leads to significant loss in strength and stiffness of the soil. Depending on the occurrence of liquefaction, the pile foundation may undergo substantial shaking, while the soil is in a fully liquefied state and soil stiffness is at a minimum. Due to the loss in strength of soil due to liquefaction, pile foundations undergo damages like lateral spreading, vertical sinking etc. which ultimately leads to settlement of the superstructure. The traditional principle of mitigation of liquefaction-induced damages was to prevent the 100% development of excess pore water pressure. However, with recent advances in performance based design concept in geotechnical engineering, it is believed to be more cost-effective to allow the onset of liquefaction, but prevent the consequent damages. Many numerical and experimental studies have been conducted to find out the response of pile foundations buried in liquefiable soil under dynamic loading but most of them are concentrated on the lateral response of the pile foundation. A simplified solution can be used to find the permanent settlement in saturated sandy soil subjected to dynamic loading by using the concept of relative compression of sand [1]. A numerical model taking into account the reduction of soil stiffness and strength due to pore pressure generation and subsequent liquefaction has been presented to obtain the pile response [2] in addition to the material nonlinearity. A large scale shake table test has also been performed on a model pile to determine the lateral deflection, moment and shear force along the pile length and the post liquefaction soil strength [3]. Hatsukazu et.al. [4] carried out shaking table tests in near-to-full scale models of pile foundation in water saturated sands and found out that pile behaviour is influenced by inertial forces initially and later it is influenced by soil motion when acted upon by dynamic load. They further studied the effect of ground water table on liquefaction of soil. Shake table test on 3x3 pile group behind a sheet-pile quay wall shows that frequency and amplitude of input motion had significant effect, while the effect of pile head fixity has negligible effect on the distribution of maximum lateral force on the group of piles [5]. Ramin Motamed et. al. [6] presented the results of large-scale test on liquefaction-
induced lateral spreading in a model consisting of gravity quay wall and pile group and it was found that lateral spreading decreased as the distance from the quay wall increased landward. The group interaction effect on small scale pile groups with different spacing at two different sites was studied and found out that dynamic loads caused a permanent non-linear response in the pile groups [7]. The response of two single piles, a relatively flexible and a relatively stiff pile was investigated to determine the stiffness characteristics of liquefied soil [8]. The back-calculated stiffness of the liquefied soil was found to be in the range between 1/30 and 1/80 of the initial stiffness. Installation of sheet piles and anchor piles were suggested to be the best remedial measures for liquefaction hazards [9].

In the present study, a new technique which involves changing the shape of the pile cap has been adopted to mitigate the vertical settlement of pile groups due to earthquake generated dynamic loading. To study the feasibility of the new structures in terms of load carrying capacity, load tests have also been performed on them. Comparative studies of both the static and dynamic tests have been made to find out the best suited remedial measure for the liquefaction hazard.

**DYNAMIC AND STATIC TESTS ON PILE GROUPS**

**Modelling Considerations**
Typical prototype conventional pile diameter varies from 0.3 to 6 m. In this study model conventional pile of 20 mm diameter and length of 260 mm was used which gives rise to a similitude ratio between 0.003 and 0.067 with l/d ratio of 13. Fig. 1 shows the general outline of model pile groups.

**Test Setup**
The test setup consisted of a steel tank of dimensions 1m x 1m x 1m. Locally available poorly graded sand was poured up to a height of 800 mm from the bottom of the tank compacted at a relative density of 40 % and used as the sand bed. The sand was fully saturated by water sedimentation process. Table 1 shows the properties of sand used in the tests for making the sand bed. The piles were constructed as pre-cast piles and placed in position on the sand bed and later the caps were constructed.

**Table 1 Properties of sand used in the tests**

<table>
<thead>
<tr>
<th>Soil property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>2.65</td>
</tr>
<tr>
<td>Uniformity coefficient</td>
<td>1.64</td>
</tr>
<tr>
<td>Grain size D&lt;sub&gt;50&lt;/sub&gt; (mm)</td>
<td>0.26</td>
</tr>
<tr>
<td>Maximum void ratio</td>
<td>0.88</td>
</tr>
<tr>
<td>Minimum void ratio</td>
<td>0.55</td>
</tr>
</tbody>
</table>

For the dynamic tests, the pile groups were instrumented with LVDTs and strain gauges to measure the vertical sinking of the pile group and the bending moment generated in the pile foundation due to dynamic loads. The LVDT was attached to the top of the cap and the strain gauges were attached at the junction between the cap and the pile. Accelerometers were attached to the pile groups to record the dynamic parameters. Dynamic excitation was imparted to the pile group by means of one dimensional shake table at three different frequencies of 1 Hz, 1.5 Hz and 2 Hz for ten cycles.

For the static tests on the pile groups, the same sand bed was used. The setup consisted of
hydraulic jack, proving ring and dial gauges to apply load, measure load and settlement respectively.

Test Results
The strain gauges attached to the piles gave values of strain on the piles due to dynamic loading which were then converted into bending moment by necessary calculations. The LVDTs mounted on the pile groups gave the value of vertical settlement undergone by the pile groups due to dynamic loading. Fig. 2 to Fig. 4 show the variation of bending moment (N-m) with time and Figs. 5 - 7 show the variation of vertical settlement (mm) with time for the three pile groups at three different frequencies respectively.

![Graph 1](image1.png)

**Fig. 2** Bending moment vs time for three cross-sections of pile cap at a frequency of 1 Hz.

![Graph 2](image2.png)

**Fig. 3** Bending moment vs time for three cross-sections of pile cap at frequency of 1.5 Hz.

![Graph 3](image3.png)

**Fig. 4** Bending moment vs time for three cross-sections of pile cap at frequency of 2 Hz.
Fig. 5 Vertical settlement vs time for three cross-sections of pile cap at frequency of 1 Hz

Fig. 6 Vertical settlement vs time for three cross-sections of pile cap at frequency of 1.5 Hz.

Fig. 7 Vertical settlement vs time for three cross-sections of pile cap at frequency of 2 Hz.

From the values of vertical settlement of pile groups in foundation soil, the percentage reduction in settlement in pile groups with triangular and semi-circular cross-section of pile caps was computed as shown in Table 2. It is observed that the semi-circular cross section displaces more volume of liquefied soil and thus settlement is the least.

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>% reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triangular</td>
<td>1.48</td>
</tr>
<tr>
<td>Semi-circular</td>
<td>33.50</td>
</tr>
</tbody>
</table>

From the static load tests on the three pile groups, the load vs settlement curves were plotted as shown in Fig. 8. It is observed that for settlements equal to 10% and 20% of pile diameter, the pile group with semicircular cross section carries more load than rectangular and triangular cross sections.
Fig. 8 Load vs settlement curves for the three pile groups under static loading

MATHEMATICAL AND NUMERICAL MODELLING

A mathematical solution to the permanent settlement in sand due to liquefaction in terms of relative compression was given by Yasuhiro et al. [1]. Soil liquefaction can be uniquely related to relative compression “$R_c$” regardless of type and density of soil as,

$$R_c = \frac{\Delta \theta}{(e_i - e_{min})}$$  \hspace{1cm} (1)

Where $\Delta \theta$ is the change in void ratio, $e_i$ and $e_{min}$ are the initial and minimum void ratios. In addition, $R_c$ could also be a function of the maximum double amplitude of the shear strain $\gamma_{max}$ induced in the sand deposit during earthquakes.

$$R_n = R_0 \gamma_{max} n \hspace{1cm} (2)$$

Where $R_0$ and $n$ are constants. Post liquefaction volumetric strain $\varepsilon_{v\text{pr}}$ of sandy soil is given by the equation

$$\varepsilon_{v\text{pr}} = R_0 \frac{(e_i - e_{min})}{(1 + e_i)} \gamma_{max} n \hspace{1cm} (3)$$

The strain values obtained were then converted to settlement values by necessary calculations. Table 3 shows the settlement values obtained for the three different frequencies of dynamic loading.

<table>
<thead>
<tr>
<th>Frequency (Hz)</th>
<th>Settlement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.08</td>
</tr>
<tr>
<td>1.5</td>
<td>8.08</td>
</tr>
<tr>
<td>2</td>
<td>16.16</td>
</tr>
</tbody>
</table>

The mathematical equations do not take into consideration the effect of the foundation buried in the soil. The tests, however, were carried on liquefiable sand bed on which the model pile groups were founded. The difference in values of vertical settlements observed in actual laboratory tests and that obtained by mathematical calculations may be the result of that. Numerical analysis of the pile groups under static loading was done in PLAXIS 2D for all the pile groups except for the one with semi-circular cross-section due to modeling constraints. Fig. 9 and Fig. 10 show the load vs settlement behaviour for the pile groups with rectangular and triangular pile cap cross-sections.

Fig. 9 Load vs settlement curves for pile group with rectangular pile cap cross-section
Fig. 10 Load vs settlement curves for pile group with triangular pile cap cross-section

From the curves, the ultimate load carrying capacity for the pile groups was determined as is shown in Table 4.

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Ultimate load(kg)</th>
<th>Ultimate load(kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Experimental</td>
<td>PLAXIS 2D</td>
</tr>
<tr>
<td>Rectangular</td>
<td>44</td>
<td>39</td>
</tr>
<tr>
<td>Triangular</td>
<td>34</td>
<td>41</td>
</tr>
<tr>
<td>Semi-circular</td>
<td>48</td>
<td>-</td>
</tr>
</tbody>
</table>

CONCLUSIONS
In this study, a new design of pile cap has been adopted to mitigate the hazard of vertical settlement in pile foundations due to dynamic loading and the following conclusions have been drawn:

1. In case of static loading, all the three pile groups with different pile cap cross-sections, viz. rectangular, triangular and semi-circular behave similarly with a slight change in ultimate load carrying capacity for each of the three pile caps.

2. The bending moment in the piles increase with increasing frequency of loading with the maximum bending moment occurring in the pile group with semi-circular cross-section of pile cap.

3. The settlement in the vertical direction is the least in case of the pile group with semi-circular cross-section of pile cap for all the three frequencies of loading. The vertical settlement was found to decrease by 33.50% in case of pile cap with semi-circular cross-section. In case of triangular and rectangular cross-sectional pile caps, settlement values are comparable with marginal improvement in case of triangular cross-sectional pile cap.

However, it is obvious that this particular field of research is still in a very early stage and needs more appropriate and detailed analytical and numerical studies in future.

REFERENCES


