Behaviour of Single Piles in Liquefied Soils during Earthquake

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Abstract

Analysis of pile foundations for earthquake loads, requires the consideration of inertial loads due to soil-pile-super structure interaction and also need the evaluation of kinematic interactions due to the movement of the surrounding soil and the pile. Also such soil-pile interaction analysis, must consider the stiffness degradation due to earthquake loading in liquefying soils. In the present study, pile-soil interaction analysis is attempted by considering stiffness degradation effects for a range of earthquakes with different amplitudes (Maximum horizontal acceleration, (MHA)), frequency contents, and different durations. The pile response is observed for both rigid piles and flexible piles under earthquake loading. Effects of both kinematic and inertial interactions are considered by using seismic deformation method. Results of ground response analysis obtained from separate study were used for soil-pile interaction analysis. Pile response for kinematic interactions is validated with the available solutions in the literature. Parametric studies have been carried out to understand the effect of depth of embedment, depth of liquefying layer etc. and their results are presented. It is observed that the effect of depth of liquefying layer has significant influence on the pile bending response. Also it is observed that the peak bending moment occurs at the interface of liquefying and non-liquefying layer.

Introduction

Pile foundations supporting super structure shall be designed for lateral loads arising due to wind loads, seismic loads, wave forces etc. in addition to the vertical loads. During earthquake loading the pile will be subjected to lateral loading due to kinematic and inertial interactions. For piles in liquefying soils the presence of liquefying layer makes the pile vulnerable to buckling due to significant stiffness degradation. Also this involves the consideration of bending-buckling interaction phenomenon [Dash et al. 2010]. For soil-pile response analysis several approaches have been proposed including sophisticated mathematical or numerical analyses and simplified methods. The finite difference technique is most commonly adopted due to its simplicity. However, finite element techniques have more flexibility to perform seismic analysis of soil pile interaction in both frequency and time domain. In the present study a pseudo static analysis is carried out using seismic deformation method and the behavior of liquefying soils under earthquake loads is studied and their results are presented.
Design Approaches

The current design methods are based on pile design against bending failure due to lateral loads such as inertia load and loads due to lateral spreading. The most commonly used methods [Bhattacharya 2007, Liyanapathirana and Poulos 2005] are discussed below:

i) Force based method or limit equilibrium method: In this method, lateral pressure acting on the pile is estimated and response of the pile is evaluated. Pile yielding and allowable deflection are checked in this method.

ii) Displacement based method or p-y method or seismic deformation method: In this method, free field ground displacement is evaluated and the displacement profile is applied on the pile and pile response is evaluated.

For the seismic analysis of piles in liquefying soils Liyanapathirana and Poulos 2005, proposed pseudo-static method which is simple and practicable and yet gives reasonably accurate results. The method mainly involves two solution stages. In the first stage a free field ground response analysis is carried out to obtain maximum ground displacement along the depth of the pile and ground surface acceleration. In the second stage a pseudo-static analysis is carried out for the pile, subjected to the maximum ground displacements along the depth, and the pseudo-static loading at the pile head is computed by multiplying the cap mass with the maximum ground acceleration. In the present study the above approach is adopted and the ground response analysis results obtained in stage I, using different earthquake loadings are used to obtain the pile response in the second stage.

Limit Equilibrium Method

Dobry et al. 2003, presented a simplified limit equilibrium method for computing maximum bending moment in a pile and is given as:

$$M_{max} = (0.5A_pH_p + A_cH_c)p_l$$  (1)

where $A_p$ = Area of pile exposed to lateral liquefied soil pressure and $H_p$ = length of pile exposed to lateral liquefied soil pressure,

and $A_c$ = Area of pile cap exposed to lateral liquefied soil pressure and $H_c$ = Height of force $F_c$ above the bottom of liquefied sand layer; $F_c$ = Lateral equivalent force on the pile cap and $p_l$ is the limiting liquefied soil pressure.

Seismic Deformation Method or p-y Method

The most commonly used model for predicting the non-linear behavior of soil is using p-y curves (API, 2003). The basic differential equation of laterally loaded pile is as given below:

$$EI \frac{d^2y}{dz^2} + Ey = F$$  (2)

Where,

$y$ = lateral deflection of pile
$z$ = distance along the pile from the top
$EI$ = flexural rigidity of pile
$E_s$ = soil modulus
$F$ = applied force per unit length of the pile

In earthquake engineering the equation (2) is modified as given below [AIJ, 2001]:

$$EI \frac{d^2y}{dz^2} + k_sD(y-y_g)$$  (3)

$y_g$ = ground displacement; $D$ = diameter of pile; $k_s$ = subgrade modulus.

$p$-$y$ curves are usually employed for obtaining the pile response (Matlock 1970, Reese et al. 1974)
Using the above equation, pile response due to kinematic interactions may be evaluated. Also by adding inertia loads on right side of equation (3) and using principle of superposition the combined pile response due to kinematic and inertial loading may be obtained.

In case of liquefying soils, the subgrade modulus is degraded and the degradation of $k_{hn}$ with increasing displacement is expressed as [Tokimatsu et al. 1998, Tokimatsu 1999]:

$$k_h = k_{hn} S_f$$  \hspace{1cm} (4)

where $S_f$ is the scaling factor for the liquefied soil.

Variation of horizontal subgrade modulus, $k_{hn}$ (for non-liquefied soils) with depth in the soil deposits is correlated with the Standard Penetration Test (SPT) N values. The modulus of subgrade reaction for non-liquefied soils $k_{hn}$ proposed by AIJ (2001), JRA (1996) is given as:

$$k_{hn} = 80 E_o B_o^{-0.75}$$  \hspace{1cm} (5)

$$E_o = 0.7N$$  \hspace{1cm} (6)

Where $k_{hn}$ is the modulus of subgrade reaction in $MN/m^3$, and $E_o$ is the modulus of deformation in $MN/m^2$, $N$ is the SPT value, and $B_o$ is the diameter of the pile in $cm$.

As soil liquefies, the stiffness of soil degrades. It can be found from the case studies that the modulus of subgrade reaction for the laterally spreading soils can be reduced by a scaling factor, termed as stiffness degradation parameter, $S_f$ varying from 0.001 to 0.01 (Ishihara and Cubrinovski, 1998) as compared to normal soil condition where there is no liquefaction. The degree of stiffness degradation in the laterally flowing deposits is related to the displacement of the pile relative to the surrounding soil.

**Validation of Pile response for Laterally spreading grounds**

Based on the Equation (3) a computer program is developed for single piles in liquefied soil by using MATLAB (2004). The pile top (node ‘1’, Fig. 2) is assumed to be free headed and pile tip (node ‘n+1’) as floating tip in the present study.

Fig. 1: Pile passing through liquefied layer

Fig. 2: Soil-pile analysis considering ground deformations using finite difference technique

By applying central difference method at point ‘i’ (Fig. 2):

$$EI \frac{\dot{y}_{i+1} + \dot{y}_{i} - 2\dot{y}_i}{(h)^2} + k_i (y_i - y_{i+1}) = 0$$  \hspace{1cm} (7)

$n=$ number of elements along the pile. $h=$ segment length $L/n$ and $k_i$ is modulus of subgrade reaction.
Equation 7 may be used for points ‘2’ to ‘n’ to give n-1 equations. The remaining unknowns may be obtained by using appropriate boundary conditions and equilibrium equations.

In the present study kinematic and inertial loads are imposed separately and the combined response is algebraically added. The kinematic interaction response is obtained by considering the ground deformations $y_i$ alone. In the second stage inertial loads (H) alone are applied at the pile top as equivalent static loads and the pile bending response is obtained. In order to estimate the peak bending moment due to combined inertial and kinematic loads, Tokimastu et al. (2005), Tokimastu and Suzuki (2005) suggested that when $T_b$ (natural period of structure) > $T_g$ (ground natural period), the peak pile bending moment in the pile can be estimated by the SRSS (square root of sum of squares) of the individual moments due to the inertial and kinematic loads. When $T_b$ < $T_g$, the peak bending moment in the pile can be computed by the algebraic addition of inertia and kinematic components. In the present study the bending moments in the pile considering kinematic and inertial interactions are algebraically added at various nodes along the pile length.

The computer program developed is initially validated by using available solutions in literature (Meera and Basudhar 2008) for kinematic interactions. The validation on flexural behavior of pile is performed for free headed pile with floating tip at the base. The notations used are: non-liquefied depth factor $r (= L_1/L)$, liquefied depth factor $s (= L_2/L)$, embedded depth factor $t (= L_3/L)$, pile flexibility factor $R (= E_p I_p/E_s L^4)$, ratio of young’s modulus of the pile to the soil modulus $K (= E_p/E_s)$, the pile length to pile diameter ratio $L/D$ (slenderness ratio), soil modulus to soil strength ratio $Q (= E_s/s_u)$, vertical load factor $V (= 4P/\pi D^2 E_s)$, horizontal load factor $H (= H/s_u D^2)$, moment factor $M (= M/s_u D^3)$, ratio of distance of location of pile from the waterfront to the affected distance of lateral spreading, i.e., location factor $L_i (= \alpha L)$, scale factor for liquefied soil ($S_f$), and gradient of surface topography ($s_l$).

The soil pile interaction is performed with the above input parameters and the results of the present study are given in terms of non-dimensional coefficients. The non-dimensional deflection coefficient, is $Y^* = y/D$, where $y$ is pile deflection, $D$ is diameter of pile and non-dimensional bending moment coefficient is, $M^* = M/s_u D^3$, where $M$ is the bending moment developed at the pile soil interface, $s_u$ is the shear strength of soil. The various input parameters considered for obtaining the effect of lateral spreading are: $L/D=25$; $r = 0.20$; $s = 0.60$; $K = 500$; $R = 1.0 \times 10^{-4}$; $Q = 200$; $S_f = 0.01$; $L_i = 0$; $V = 0$; $H = 0$; $M = 0$.

Table 1: Variations of non-dimensional deflection coefficient $Y^*$ and non-dimensional moment coefficient $M^*$ with slope – Present study vs. those obtained by Meera and Basudhar (2008).

<table>
<thead>
<tr>
<th>Slope (S%)</th>
<th>Present study</th>
<th>Meera and Basudhar (2008)</th>
<th>% diff.(absolute)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$Y^*$</td>
<td>$M^*$</td>
<td>$Y^*$</td>
</tr>
<tr>
<td>10</td>
<td>2.214</td>
<td>217.11</td>
<td>2.232</td>
</tr>
<tr>
<td>20</td>
<td>2.783</td>
<td>272.90</td>
<td>2.805</td>
</tr>
<tr>
<td>30</td>
<td>3.181</td>
<td>311.98</td>
<td>3.207</td>
</tr>
<tr>
<td>40</td>
<td>3.498</td>
<td>343.05</td>
<td>3.526</td>
</tr>
</tbody>
</table>
are compared with results obtained by Meera and Basudhar 2008, and are given in Table 1. Clearly it can be seen that the results are in good agreement with that of available solutions in the literature. Also the variation of pile deflections and bending moments are plotted in Fig. 3A and Fig. 3B respectively by considering various ground surface topographies (slopes). It is observed that the ground surface slope has significant influence on the pile response and it is found that the amplification factor is about 1.60 for non-dimensional deflection and bending moment coefficients when the ground surface slope is increased from 10% to 40% and hence pile design must consider the slope effects in laterally spreading grounds. Also it can be seen that the maximum bending moments are observed at the interface of liquefying soil layer to non-liquefied layer.

**Behavior of Piles under Seismic Loads**

Having validated the pile response to kinematic loading in liquefying soils, the pile behavior is studied for both non-liquefying and liquefying conditions for selected ground motions. The boundary conditions are considered as free headed top and floating tip base. In the present study four typical strong motion earthquake events corresponding to Bhuj (2001) earthquake, Loma Prieta (1989) earthquake, Loma Gilroy (1989) motion and Kobe (1995) earthquake are considered for soil pile interaction analysis. These earthquakes represent wide range of amplitudes [maximum horizontal accelerations (MHA) of 0.106g for Bhuj (2001), 0.278g for Loma Prieta (1989), 0.442g for Loma Gilroy (1989) and 0.834g for Kobe (1989) motion respectively], frequency contents and durations. Time history of Kobe (1995), Loma Prieta (1989) and Loma Gilroy (1989) are available in DEEPSOIL v5.0 library [Hashash et al. 2008].

**Response of laterally loaded pile in liquefying soils**

Having validated the pile response under kinematic loading, the pile response is studied for liquefying soils due to earthquake loads. The pile length is considered as 10.0m and pile radius is considered as 0.25m in the present study. Young’s modulus of the pile is assumed as $2.74 \times 10^7$ kN/m². The various input parameters considered for seismic soil pile interaction for both liquefying and non-liquefying conditions are presented in Table 2 representing SPT, modulus of deformation, degraded subgrade modulus etc., at various depths. The relative stiffness factor ($L/T$) for the pile soil system varies from approximately 2.8 to 3.4 (for liquefied case) and hence the pile behavior is expected to be rigid/semi-rigid. For non-liquefying soils it can be seen that $L/T$ varies from 7.0 to 8.5 and hence pile behavior is expected to be flexible. Results of
equivalent linear ground response analysis performed from a separate study for this soil deposit are used in the present analysis. The results obtained from ground response analysis of local soil sites are approximated as linearly varying along the depth of soil deposit. The peak ground accelerations at ground surface obtained from ground response analysis [Phanikanth et al. 2010] are 0.251 g for Bhuj (2001) motion, 0.641 g for Loma Prieta (1989) motion, 1.136 g for Loma Gilroy (1989) motion, and 0.917 g for Kobe (1995) motion respectively. It was assumed that the entire soil deposit is liquefying and hence stiffness degradation is considered as \( s_f = 0.01 \) for evaluating the pile response.

The kinematic interactions are considered based on the results obtained from the ground response analysis and the inertial interactions are considered by multiplying the cap mass and peak surface acceleration. In the present study the ultimate pile capacity is estimated as 90.0 t and the inertial loads \((H)\) are applied at the pile top obtained by multiplying cap mass with peak ground acceleration at the ground surface. Thus the inertial loads for Bhuj (2001) input motion, Loma Prieta (1989) motion, Loma Gilroy (1989) motion and Kobe

### Table 2: Input parameters for soil-pile interaction, liquefied soils

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Stratum</th>
<th>Layer thickness (m)</th>
<th>Depth below GL (m)</th>
<th>SPT value (N)</th>
<th>( E_o ) (MPa)</th>
<th>( B=D= ) pile dia. (cm)</th>
<th>( k_{soil} ) (MN/m²)</th>
<th>( S_f )</th>
<th>( k_h = k_{soil} S_f ) (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Filled up soil</td>
<td>1.5</td>
<td>1.5</td>
<td>10</td>
<td>50</td>
<td>29.78</td>
<td>0.01</td>
<td>297.83</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Yellowish loose sand</td>
<td>1.5</td>
<td>3</td>
<td>12</td>
<td>50</td>
<td>35.74</td>
<td>0.01</td>
<td>357.39</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5</td>
<td>4.5</td>
<td>13</td>
<td>50</td>
<td>38.72</td>
<td>0.01</td>
<td>387.17</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5</td>
<td>6</td>
<td>16</td>
<td>50</td>
<td>47.65</td>
<td>0.01</td>
<td>476.52</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Black clayey soil</td>
<td>2</td>
<td>8</td>
<td>20</td>
<td>50</td>
<td>59.57</td>
<td>0.01</td>
<td>595.65</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Yellowish clayey soil</td>
<td>1.8</td>
<td>9.8</td>
<td>25</td>
<td>50</td>
<td>74.46</td>
<td>0.01</td>
<td>744.56</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Greyish hard rock</td>
<td>-</td>
<td>&gt; 9.8</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
(1995) motion are estimated as 222.0 kN, 567.0 kN, 1003.0 kN and 810.0 kN respectively. The soil pile response in liquefying soils considering subgrade modulus evaluated with $s_f = 0.01 (= 1/100)$ is obtained considering both inertial and kinematic interactions together is shown in Fig. 4a with inertial response and kinematic response added algebraically. It was observed that the pile response is significantly affected in the presence of liquefiable layer and also it is observed that the amplification factor is approximately about 2.50 in peak bending moments. The pile deflections are presented for liquefying soils in Fig. 4b for kinematic and inertial interactions and in Fig. 4c for inertial interactions only. It can be seen that, the pile deflections due to inertial loads are significant and also from the load significantly and the amplification is found to be approximately 2.50. Kagawa (1992) reported that the amplification in peak bending moments for liquefying soils is as high as 6.0 compared to non-liquefying soils and many failures observed during earthquake are due to inability of pile to sustain such large bending moments. The bending moment along depth is also plotted for Loma Prieta (1989), Loma Gilroy (1989) and Kobe (1995) input motions, based on ground response results obtained and considering inertial and kinematic interaction added algebraically and are also shown in Fig. 5a.

Also to understand the kinematic response of the pile with respect to the ground response under earthquake event, the pile deflections under kinematic loading are compared with ground displacements for both liquefying and non-liquefying soils. It can be seen that the pile is flexible under non-liquefied condition ($L/T$ varies from 7.0 to 8.5) where as pile tends to be rigid when soil is liquefying ($L/T$ varies from 2.8 to 3.4). The ground displacements obtained from ground response analysis results for Kobe (1995) and Loma Gilroy (1989) motion are approximated as parabolically varying to obtain the pile deflections under kinematic loading and the results are shown in Fig. 5b. It is found that when the pile is flexible the pile displacements (kinematic) nearly match with the ground displacement where as when the pile is rigid there is relative displacement between the soil and the pile.

**Comparison of responses of fully liquefying and non-liquefying soils**

The pile response is evaluated with and without stiffness degradation effects and for different ground motions. The variation of pile bending moment for kinematic and inertial interactions along depth of the pile for fully liquefying and non-liquefying soils for Bhuj (2001) motion is plotted in Fig. 5a. Clearly it can be seen that for the piles in liquefying condition the bending moments are amplified significantly and the amplification is found to be approximately 2.50.
### Conclusions

1. The ground surface slope has significant influence on the pile response and it is observed that the amplification factor is about 1.60 for non-dimensional deflection and bending moment coefficients when the ground surface slope is increased from 10% to 40% and hence pile design must consider the slope effects in laterally spreading grounds. Also Pile response under kinematic loads is validated with available solutions in the literature and the results are in good agreement with the available solutions.

### Effect of depth of liquefying layer

In general, loose saturated cohesion less soils are vulnerable to liquefaction hazards and the presence of such layers reduces the stiffness of the surrounding soil resulting in excessive bending moments ultimately leading to pile failure. In the present study the effect of depth of liquefying layer on the pile-soil response is also studied. The overall length of the pile is assumed as 10.0m. As can be seen from Fig. 1 parameter \(L_2\) denotes the depth of the liquefied layer and \(L\) is the overall length of the pile. The depth of the liquefied layer is varied in terms of \(L_2/L\) from 0.20 to 1.0 (\(L\) is considered as 0.0). The pile response is observed for various liquefied depth ratios and the results are presented in Fig. 6a showing pile deflections considering kinematic and inertial interactions. The deflection at any given depth is found to increase with the increase in depth of liquefied layer and is maximum, when the entire soil deposit is liquefying. Also the combined pile bending response are plotted in Fig. 6b. It can be seen that the maximum bending moment occurs at the interface between the liquefied layer and non-liquefied layer. Also it is observed that the bending moments are maximum when the thickness of the liquefied layer is approximately 60% of the total layer thickness. If the thickness of the liquefying layer is higher, the pile bending moments are reduced. This may possibly due to the soil failure preceding the pile failure (soil stresses exceeding the soil strength or passive pressure developed are beyond the shear strength of the soil).
2) Flexible piles tend to deform along with the ground with little relative displacements and hence in the presence of high inertial loads the total pile bending response is mainly due to inertial interactions. Rigid piles being stiff, deform with higher relative displacements between ground and pile, resulting in high passive resistance from the soil. The pile bending response due to kinematic loads is also significant.

3) The pile response in liquefied soils is significantly amplified compared to that in non-liquefying soil and the amplification in peak pile bending moment is found as high as 2.50. The effect of depth of liquefied layer has significant influence on the soil pile response. Maximum pile bending moments occurs at the interface of the liquefying and non-liquefying layer. Maximum pile bending moments are developed when the thickness of liquefying soil layer is approximately 60% of the total thickness of the soil layer.

References

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