SEISMIC DESIGN OF PILE FOUNDATIONS: KINEMATIC INTERACTION IN LAYERED SOILS

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ABSTRACT

Ground deformation due to the passage of seismic waves during an earthquake imposes loadings to embedded piles referred to as kinematic loads. This is different from the inertial loads that the superstructure applies at the pile head. In practice, pile foundations are usually designed for inertial loads from the superstructure, while determining the influence of kinematic loads requires additional analysis and may not be necessary for all cases. This study aims to investigate the significance of kinematic interaction in the seismic design of piles in a low- to moderate-seismicity region like Hong Kong and identify the conditions under which kinematic loads need to be considered in the design. The case of soil profiles containing consecutive layers with sharply different stiffness is considered, and the pile capacity ratio limit is defined to derive the criteria. A parametric study has been conducted by varying the key parameters to cover the common design cases. The results clearly indicate that in stiff ground the kinematic moment arising from the earthquake design ground motion in Hong Kong is insignificant and can be ignored in the design. However, in the softer ground and in the presence of high stiffness contrast at a soil layer interface, the kinematic loads can be considerable and should be checked.

Keywords: Pile foundations; Soil structure interaction; Kinematic loads; Seismic design; Layered soils.

1. INTRODUCTION

Pile foundations are usually employed to transmit the foundation loads through the soil strata of low stiffness/bearing capacity to deeper soil or rock strata having higher stiffness/bearing capacity. During earthquakes, piles are subjected to two main sources of loading as shown in Figure 1. Inertial loads are applied at the pile head as a result of superstructure’s vibration producing base shear, moment and torsional excitation, while kinematic loads are induced along the pile due to soil seismic deformation and can exist even in the absence of superstructure. In practice, pile foundations are usually designed for inertial loads from the superstructure, while determining the influence of kinematic loads requires additional analysis and may not be necessary for all cases. This study investigates the significance of kinematic interaction in the seismic design of piles in a low- to moderate-seismicity region like Hong Kong and identify the conditions under which kinematic loads need to be considered in the design. Evidence from case histories as well as previous analytical and experimental studies (Mizuno 1987, Sica et al. 2011) revealed that the maximum values of kinematic bending moments in piles are concentrated either at the interfaces of soil layers with sharply different stiffness or at the pile head.

Two critical cases can be distinguished for studying the significance of kinematic loads on pile foundations as follows:

Case 1) homogenous soils;
Case 2) soil profiles containing consecutive layers with sharply different stiffness.

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This paper presents the results of the second part of the study – Case 2) soil profiles containing consecutive layers with sharply different stiffness.

![Diagram of kinematic load on piles due to soil displacement](image)

Figure 1. Schematic of kinematic load on piles due to soil displacement

2. METHODOLOGY

The methods used for the analysis of piles due to the movement of surrounding soil can vary in complexity and computational effort and can be categorised into three groups:

(i) simplified analytical methods (Margason and Halloway 1977, Dobry and O’Rourke 1983);
(ii) two-stage subgrade reaction methods (Pappin et al. 1998, Mylonakis 2001, Nikolaou et al. 2001);
(iii) methods based on 2D or 3D numerical simulations of the soil-foundation-structure system in a continuous medium through time-history analysis (Di Laora et al. 2012, Pappin 2002, Hokmabadi et al. 2014).

Both time-domain and frequency-domain analyses have been frequently used in the literature. A brief review of the available methods for the calculation of kinematic loads on piles at the interface of two soil layers is provided below.

2.1 Dobry and O’Rourke (1983)

Dobry and O’Rourke (1983) developed a simplified method for computing the kinematic bending moment at the interface of two soil layers (see Figure 2). This approach is based on the following main assumptions:

1. the soil in each layer is homogenous, isotropic, and linearly elastic;
2. the soil layers are sufficiently thick so that boundary effects do not influence the response at the interface;
3. the pile is long, vertical, and linearly elastic;
4. perfect contact exists between pile and soil;
5. the soil is subjected to uniform static stress field.
Based on these assumptions, and modelling the pile as a beam on Winkler foundation, Dobry and O’Rourke (1983) derived an explicit solution for the pile bending moment ($M_k$) at the interface of two soil layers as shown in Equations 1 and 2.

$$M_k = 1.86 \left( \frac{E_p I_p}{G_1} \right)^{3/4} \left( \frac{G_1}{G_2} \right)^{1/4} \gamma_1 F$$  \hspace{1cm} (1)

$$F = \frac{1-c^{-4}}{(1+c)(c^{-4}+1+c^{-2})}, \ c = \left( \frac{G_1}{G_2} \right)^{1/4}$$  \hspace{1cm} (2)

Where $E_p$ and $I_p$ are pile Young’s modulus and cross-sectional moment of inertia respectively, $G_1$ and $G_2$ are soil shear modulus in soil layer 1 and 2 respectively, and $\gamma_1$ is soil shear strain in layer 1.

This method, despite its simplicity and limiting assumptions, provides a practical tool for determining kinematic moments at the interface of two soil layers (Nikolaou et al. 2001).

### 2.2 Mylonakis (2001)

Mylonakis (2001) proposed a method based on a Beam-on-Dynamic-Winkler-Foundation (BDWF) approach in which the role of the soil-pile interaction is simulated through a set of springs and dashpots continuously distributed along the pile and the wave-induced motion of the free-field soil serves as the support excitation of the pile-soil system. It is a frequency-domain approach and offers a number of improvements over the earlier methods.

Mylonakis (2001) derived a simple formulation (Equations 3 and 4) and a design chart (Figure 3) for estimating the kinematic bending moment at the interface of two soil layers under low frequency excitations (circular frequency $\omega \to 0$), where $\varepsilon_p$ is the pile bending strain, $\gamma_1$ is the shear strain in the top soil layer, $r$ is the pile radius, and $\Phi$ is the frequency factor. The frequency factor ($\Phi$), as given in Equation 5, was introduced to take into account the transient (dynamic) nature of the phenomenon. In practice, the frequency factor ($\Phi$) is generally between 1 and 1.25.

$$M_k = (E_p I_p) \left( \frac{E_p}{G_1} \right) \gamma_1 \left( \frac{\Phi}{r} \right)$$  \hspace{1cm} (3)

Figure 2. Adopted method by Dobry and O’Rourke (1983) for computing the kinematic bending moment at the interface of two soil layers.
Figure 3. Design chart for estimating the ratio between peak kinematic pile bending strain to the peak soil shear strain ($\varepsilon_p/\gamma_1$ – strain transmissibility) of a vertical solid cylindrical pile at the interface of two soil layers of different stiffness at low excitation frequencies (Mylonakis 2001)

2.3 Nikolaou et al. (2001)

Nikolaou et al. (2001) developed a method based on the BDWF approach and used frequency–time domain formulations. A parametric study was conducted and a closed-form expression given in Equation 6 proposed for estimating the kinematic bending moment of solid cylindrical piles at the interface of soil layers in steady-state conditions.

$$M_k \equiv 0.042 \, \tau_c \, d^3 \left( \frac{L}{d} \right)^{0.30} \left( \frac{\varepsilon_p}{E_i} \right)^{0.65} \left( \frac{V_2}{V_1} \right)^{0.50} \quad (6)$$

where $V_1$ and $V_2$ are the soil shear wave velocities in layer 1 and 2, respectively, $E_i$ the soil Young's modulus in layer 1, $\tau_c$ is the soil shear stress at the interface, $d$ is the pile diameter and $L$ is the pile length.

Nikolaou et al. (2001) explained that harmonic steady-state results, derived from frequency-domain analysis, can only rarely be used directly in design. This is because only a hypothetical harmonic excitation with a very large number of cycles would produce a response with amplitude equal to the steady-state value. A more transient excitation, as in earthquake shaking, would tend to produce a smaller response. Accordingly, they proposed a reduction factor ($\eta$) to be applied to the maximum steady-state pile bending strain (or moment) in the frequency domain to obtain the corresponding peak value in the time-domain (transient conditions). The reduction factor ($\eta$) depends on the effective number of cycles and resonant conditions and is generally between 0.2 and 0.5.
2.4 Sica et al. (2011)

Sica et al. (2011) studied the kinematic loads on piles by the means of an extensive parametric study based on a BDWF model, subjected to vertically propagating seismic S-waves (accelerogram recorded), in the form of a transient motion imposed on rock outcrop. They reviewed the frequency-domain analysis versus time-domain analysis and discussed the relationship among static bending moment, resonant bending moment, and transient bending moment, calculated from different methods. Three approaches are suggested by Sica et al. (2011) including one purely static approach, which does not require dynamic analysis, and two dynamic approaches, which are more demanding from a computational viewpoint. In case of availability of free-field site response analysis results, the following equation (Equation 7) is recommended, which is an alternative form of the Mylonakis (2001) method:

\[ M_k = \frac{2E_pl_p}{d} \left( \frac{E_p}{\gamma_1} \right) (\gamma_1)_{dyn} \Phi_2 \tag{7} \]

where strain transmissibility \((\epsilon_p/\gamma_1)\) is the same as Equation 4, \((\gamma_1)_{dyn}\) is the soil shear strain derived from free-field site response analysis, and frequency factor \((\Phi_2)\) is equal to 1.20 to 1.25. The advantage of this approach is that the dynamic effects due to frequency content of the input motion are incorporated into parameter \((\gamma_1)_{dyn}\), whereas \((\Phi_2)\) is a constant.

2.4 Di Laora et al. (2012)

Di Laora et al. (2012) investigated the behaviour of a kinematically stressed pile in layered soil under the passage of vertically propagating seismic shear waves by means of three-dimensional finite-element analyses. They propose the following equations (Equations 8 and 9) to be used for practical estimates of peak transient kinematic bending moments:

\[ M_k = \frac{2E_pl_p}{d} \left( \frac{E_p}{\gamma_1} \right) \left[ \frac{\gamma_1}{\omega_0} \right] \gamma_1 \Phi_{1.5} \tag{8} \]

\[ \epsilon_p = \chi \gamma_1 \left[ -\frac{1}{2} \left( \frac{h_1}{d} \right)^{-1} + \left( \frac{E_p}{E_1} \right)^{-0.25} (c - 1)^{0.5} \right] \left( \frac{a_2}{a_1} \right)^{1/4} \tag{9} \]

where, \(\chi\) is a regression coefficient found to be close to unity (about 0.93), and the frequency factor, \(\Phi_{1.5}\), generally attains values below unity (a conservative selection would be 1). The other parameters have been defined previously.

3. SEISMICITY OF HONG KONG

The contour of peak ground acceleration (PGA) at bedrock for a return period of 475 years from the latest seismic hazard analysis for the Hong Kong region is shown in Figure 4 (Pappin et al. 2015). A PGA value of 1.18 m/s² (0.12g) covers all the land area in Hong Kong and is adopted in the analysis.
As part of the same study, a soil shear stress reduction factor ($r_d$) was also obtained from the mean plus one standard deviation of peak shear stress ($\tau_{\text{max}}$) of soil with depth calculated in the site-specific seismic response analyses for 41 ground profiles covering various parts of Hong Kong (Arup 2004, Arup 2012). The shear stress reduction factor with site effect ($S$ factor) adopted in this study is shown in Figure 5. It should be noted that the site-specific response analyses showed that the $\tau_{\text{max}}$ values with depth showed no significant dependence on the soil profile ground type.

In this study, the ground condition is classified based on the ground types in Eurocode 8 as shown in Table 1.
Table 1. Ground type definition

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>Description of Stratigraphic Profile</th>
<th>$v_{s,30}$ * (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>Rock or other rock-like geological formation, including at most 5m of weaker material at the surface.</td>
<td>&gt; 800</td>
</tr>
<tr>
<td>$B$</td>
<td>Deposits of very dense sand, gravel, or very stiff clay.</td>
<td>360 - 800</td>
</tr>
<tr>
<td>$C$</td>
<td>Deep deposits of dense or medium dense sand, gravel or stiff clay.</td>
<td>180 - 360</td>
</tr>
<tr>
<td>$D$</td>
<td>Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.</td>
<td>&lt; 180</td>
</tr>
<tr>
<td>$S_1$</td>
<td>Deposits of very loose cohesionless soil (with or without some soft cohesive layers), or of predominantly very soft cohesive soil.</td>
<td>&lt; 100 indicative</td>
</tr>
<tr>
<td>$S_2$</td>
<td>Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types $A-D$ or $S_1$</td>
<td>Not applicable</td>
</tr>
</tbody>
</table>

* $v_{s,30}$ is the geometric average shear wave velocity in upper 30m.

4. ANALYSIS

4.1 Use of Bending Strain as Fundamental Measure of Pile Kinematic Bending

It is convenient to use a deformation-related quantity such as bending strain to quantify kinematic pile bending. The maximum bending strain at the outer fibre of the pile cross section, $\varepsilon_p$, is related to bending moment, $M$, as shown in Equation (10):

$$\varepsilon_p = \frac{M}{E_p I_p} \frac{r}{r}$$

where $r$ is the distance from the neutral axis to the farthest fibre in the cross section. The bending strain has been widely used in the literature for studying the kinematic loads on piles (Mylonakis 2001, Sica et al. 2011, Di Laora and Mandolini 2011, Aw et al. 2014). Using bending strain as a fundamental measure of pile bending has several advantages over the direct use of bending moment (Mylonakis 2001):

a. it is dimensionless;

b. it can be directly measured in experiment;

c. it can be used to quantify damage: ultimate (failure) bending strains do not vary significantly among common structural materials. Typically, strains in the order of 0.1% can inflict damage in conventionally designed concrete or steel beams.

Therefore, bending strain has been used in this study to quantify the effect of kinematic interaction on piles.

4.2 Combination of Inertial and Kinematic Loads

When combining kinematic and inertial loads in a two-stage equivalent static approach, one must account for the fact that individual peaks of the contributing inertial loads from the superstructure and kinematic loads from the surrounding soil displacement generally occur at different times (not likely to be in phase). Therefore, to avoid being over-conservative, the appropriate kinematic and inertial load combinations may be: (i) peak kinematic load plus some fraction of peak inertial load, and (ii) peak inertial load plus some fraction of peak kinematic load. As discussed in Boulanger et al. (2003), the appropriate fraction is a topic of ongoing research, and it depends on the relative phasing of the two
load contributions, which will be a function of the earthquake motion’s characteristics (e.g. frequency content, duration, intensity), the dynamic site response characteristics (e.g. effective site period, timing of liquefaction), the pile foundation stiffness, and the dynamic superstructure response characteristics (e.g. effective periods).

The SRSS (Square Root of the Sum of the Squares) or the following 100%:30% load combination rule have conventionally been used for non-liquefied conditions (Pappin 2002, Tokimatsu et al. 2005):

- 100% kinematic ± 30% inertial;
- 30% kinematic ± 100% inertial.

The 100%:30% load combination rule has been adopted in this study.

4.3 Pile Capacity Ratio

The pile capacity ratio is used to quantify the significance of kinematic loads on the pile foundation design. It is defined as the ratio of the imposed kinematic bending strain (Equation 10) to the damage bending strain. The damage bending strain is the bending strain corresponds to the moment capacity of the piles. Assessing the damage bending strain for typical piles revealed that for common design cases the damage bending strain is usually higher than 0.1%. As such, the damage bending strain of 0.1% is selected for calculating the pile capacity ratio, which is in line with the previous publications (i.e. Mylonakis 2001).

In this study, the impact of kinematic loads is only considered to be significant when the pile capacity ratio is more than 30%. This can be explained considering the combination of kinematic and inertial loads (Section 4.2), where limiting the pile capacity ratio induced by kinematic loads to 30% only causes less than 9% increase in the pile overall bending moment (30% × 30% = 9%) for the “30% kinematic ± 100% inertial case”. For the “100% kinematic ± 30% inertial” case, suppose the inertial load consumes 90% of the pile bending capacity, the pile overall bending moment would be 30% (from kinematic) + 90% × 30% (from inertial) = 57% and therefore would not be controlling.

5. RESULTS AND DISCUSSION

A parametric study has been conducted to assess the significance of kinematic loads in the pile foundations embedded in layered soils for Hong Kong seismicity level. To cover the common design cases, various pile diameters (d = 0.5, 1.0, 1.5, 2.0, 2.5, 3.0m), surficial layer’s stiffness (Vd = 100, 125, 180, 360 m/s), stiffness contrast between two layers (Vd/Vd = 1.5, 2.0, 2.5, 3.0, 3.5, 4.0) and interface depth (h = 7, 15, 20, 30m) are considered. The approach by Sica et al. (2011) (Equation 7) has been used for the parametric study, where the frequency factor (Φ) is conservatively taken as 1.25. Referring to Mylonakis (2001), a lower bound of 0.05 for strain transmissibility is assumed (εs/γt)min = 0.05.

The results are plotted in Figure 6 to Figure 9 for the various values of Vd. It is evident that increase in the stiffness contrast between two layers (Vd/Vd) can substantially increase the kinematic loads, as expected. For instance, the capacity ratio (CR) at Vd/Vd = 4 can be up to four time more than the capacity ratio (CR) at Vd/Vd = 1.5. The recorded capacity ratio (CR) of large diameter piles are less than for slender piles. It should be noted that in fact the large diameter piles experience higher kinematic moments in comparison with the slender piles during earthquake due to their stiffness; however, since the failure bending capacity of the large diameter piles are much higher than the slender piles, the capacity ratio (CR) is less.

The capacity ratios (CR) of the piles embedded in stiffer ground are less significant. For instance, referring to Figure 6 and Figure 7, the maximum capacity ratio (CR) of piles embedded in ground with Vd = 360m/s (corresponding to Ground Type B) and Vd = 180m/s (corresponding to Ground Type C)
is limited to 10% and 28%, respectively, even at the presence of high stiffness contrast. The capacity ratio \( CR \) will be further suppressed for piles embedded in Ground Type A. As such, it can be concluded that in Ground Types A, B, and C, with soil shear wave velocity higher than 180 m/s, the developed capacity ratio \( CR \) is expected to be less than 30% under Hong Kong design earthquake and thus can be ignored in the design.

**Figure 6.** Capacity Ratio \( CR \) in piles with different pile diameters \( d \), interface depth \( h_1 \), and stiffness contrast at interface \( V_s/V_{s1} \) for \( V_{s1} = 360 \) m/s.

**Figure 7.** Capacity Ratio \( CR \) in piles with different pile diameters \( d \), interface depth \( h_1 \), and stiffness contrast at interface \( V_s/V_{s1} \) for \( V_{s1} = 180 \) m/s.
Figure 8. Capacity Ratio (CR) in piles with different pile diameters \((d)\), interface depth \((h_1)\), and stiffness contrast at interface \((V_{s2}/V_{s1})\) for \(V_{s1} = 125\) m/s

Figure 9. Capacity Ratio (CR) in piles with different pile diameters \((d)\), interface depth \((h_1)\), and stiffness contrast at interface \((V_{s2}/V_{s1})\) for \(V_{s1} = 100\) m/s
In Ground Type $D$ (see Figure 8 and Figure 9) with a shear wave velocity of less than 180m/s, but higher than 100m/s, the impact of kinematic loads are significant ($CR > 30\%$) only at high stiffness contrasts, and can be ignored in the design for the other cases. In the case of piles embedded in soils with a shear wave velocity of less than 100m/s corresponding to Ground Types $S_1$ or $S_2$ the kinematic moment can be significant. It should be noted that Ground Types $S_1$ or $S_2$ represents profiles consisting of very loose cohesionless soil, very soft cohesive soils, or liquefiable soils. The response of such soft ground materials under seismic excitations is expected to be highly nonlinear experiencing considerable deformations. As such, the impact of kinematic loads on piles can be significant and is recommended to be checked in the design. In such cases, the guideline proposed by Free et al. (2001) can be adopted for the estimation of kinematic loads or a more advanced analysis can be used.

6. CONCLUSIONS

The significance of kinematic interaction in the seismic design of piles in a low- to moderate-seismicity region like Hong Kong is investigated and the pile capacity ratio limit is defined to identify the conditions under which kinematic loads need to be considered in the design. The case of soil profiles containing consecutive layers with sharply different stiffness is considered.

The results of this study show that in competent soils with a soil shear wave velocity higher than 180m/s (corresponding to Ground Types $A$, $B$, and $C$) the maximum developed kinematic bending is insignificant and can be ignored in design. In Ground Type $D$, with a shear wave velocity of less than 180m/s, but higher than 100m/s, the impact of kinematic loads can be considered significant only at the high stiffness contrasts, and can be ignored in design for the other cases. In Ground Types $S_1$ and $S_2$ (represents profiles consisting of very loose cohesionless soil, very soft cohesive soils, or liquefiable soils) the kinematic moment can be significant for the earthquake level in Hong Kong and is recommended to be checked. The response of such soft ground materials under seismic excitations is expected to be highly nonlinear experiencing considerable deformation.

7. REFERENCES


