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Kinematic Bending of Pile Foundations on Layered Liquefiable Soils

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ABSTRACT

Pile foundations are used invariably as a safer foundation system in many problematic soil profiles, which includes potentially liquefiable soils. An important parameter used for the structural design of pile foundations is the maximum bending moment induced, which is a function of the stiffness contrast between the soil layers. Dynamic loads impose high curvatures on pile foundations inducing large bending moments at the depth of stiffness contrast. The kinematic effect depends on many parameters such as soil layer stiffness, slenderness ratio of the pile, flexibility of the pile and the location of the stiffness contrast. Many recent studies have shown that the initial soil stiffness during liquefaction becomes nearly zero. Hence, the boundaries between the liquefied and non- liquefied layers make a zone of high stiffness contrast in the soil. This study is focused on the performance of pile foundations in liquefiable soils subjected to earthquake loading. Design parameters such as the maximum bending moment induced due to the stiffness contrast at variable depths is studied in detail using Finite Element Analysis.

Introduction

The kinematic and inertial modes of failure in deep foundations form a dichotomy in understanding the bending modes of failure of piles during earthquakes. These kinematic effects have been especially profound in cases where there exist a stiffness contrast between soil strata.

Various authors (Mylonakis (2000), Nikolau et al. (2001), Tokimatsu et al. (2005), Sica et al. (2011), Di Laora et al. (2012)) have parametrically studied the kinematic bending moments induced in pile foundations at the location of stiffness contrast and in many countries, design codes are well in practice. Some buildings codes (*e.g.* Eurocode 8 (1998), NEHRP (2000)) enforce that "piles should be designed to withstand strains induced from propagation of the seismic waves through the soil". Even though various codes prescribe studying the kinematic effects in non-homogenous soil deposits (layered soils), many aspects of this mode of failure are still uncertain, including the effect of possible layering of soil (even in an initially homogenous soil deposit) that happens during the progression of liquefaction.

When a homogenous liquefiable soil deposit is excited by seismic waves, possible liquefaction may occur and in its progression through the deposit, the depth of the liquefied layer will transiently vary, due to erratic nature of the shear stresses produced during earthquakes. This may cause even an initially homogenous soil deposit to manifest itself as a layered profile during

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the passage of seismic waves. This transient stiffness contrast maybe high (as liquefied soil offers very little stiffness compared to the non-liquefiable layer) and the interface may develop large bending moments. As this is not evident during design, comprehensive studies need to be done in order to understand this complex soil pile structure interaction problem in liquefiable soils under seismic loading. The scope of this study is focused on carrying out time history analysis of the system to study kinematic bending moments induced in pile foundations for various depths of liquefaction.

Model Description

The numerical model used in the present study is developed in three stages as follows.

1) Selection of Spectrally matched time histories (at least 4 cases.),
2) Site response study to find the input ground motion, and
3) A beam on non-linear Winkler foundation (BNWF) model to carryout analysis.

Selection of time histories

In the absence of histrorical groundmotion records in the region, the generation of artificial ground motion has become a common practice for performing time history analysis. Various methods exist to generate artificial ground motion histories from existing earthquake records, from which the widely used methods include: (a) scaling the time history with a uniform factor (Shome and Cornell (1998)) or (b) scaling the ground motion response spectrum to match a particular design spectrum for the given period of the structure (Kunnath et al. (2006)) or a range of periods (Kalkan and Kunanth (2006), Watson and Abrahamson (2006)). More recently, to incorporate the inelastic behavior for design of structures near faultlines, a MPS (Modal Pushover Based Scaling) technique has been suggested by Kalkan and Chopra (2010). Also, the selection of the number of records to be used is a major source of concern for designers. UBC (1997) and IBC (2000) state that the maximum response should be considered if 3 records are taken, and the average should be considered if 7 are taken. The ISO 19901-2 (2004) proposes that a minimum of 4 time histories that comply with the site conditions and faulting style of the location can be utilized while carrying out analysis.

In this study the pile is considered as free headed and is designed to remain elastic for a Maximum Considered Earthquake (2% probability of exceedance in 50 years, IBC 2000) in a potentially liquefiable soil deposit. Four time histories are chosen to represent the ground motion and are matched with the IS 1893(2002), design response spectrum in medium - hard soil for a seismic zone - V. The time histories are chosen in such a manner that after spectral matching, their average response spectrum lies above the design spectrum for the range of 0.2 T-1.5 T before liquefaction, where T is the fundamental frequency of the pile soil system. The finally selected four time histories are: (a) Bhuj Earthquake (2001), (b) Kocaeli Earthquake (1999), (c) Kobe Earthquake (1995) and (d) Loma Prieta Earthquake (1989). Figure 1 illustrates how the spectral matched time history has been converted to bed rock level and the site response study is done to estimate the time history at various depths applied to the fixed end of soil springs. The 4 time histories are matched at the rock outcrop location at A. The outcrop acceleration time history is scaled by a factor of 0.5 to obtain the bedrock time history (Meija et.al, 2006).

Rayleigh damping is considered 5% for the site response analysis and a global damping of 5% is considered for the time history analysis. Since the bedrock for the site is assumed to be the same, the bedrock motion is then convoluted through the deposit using a site response analysis program *Cyclic*1D (Elgamal et al. (2006)) and time histories at different depths obtained. The Soil Structure Interaction is then modelled through the application of the time histories at the far end of the Winkler springs as defined by API code (2010).

Far field behavior (Site Response Analysis)

The far field behavior is modeled through a site response analysis software called *Cyclic* 1D, which uses an advanced constitutive model for the liquefiable soil developed by Parra et al. (1996) and Yang (2000). A 1D time history analysis is done through direct integration to simulate the kinematic response of the free headed pile. The thickness of the liquefiable layer is parametrically varied and the model is run against the 4 ground motion accelerograms to obtain the maximum bending moment due to each ground motion. This process would not represent the actual field condition, as this layering (due to liquefaction) happens transiently with the ground motion and not in discrete steps. Hence in this study, an assumption is made that the soil remains liquefied throughout the entire ground motion duration for each configuration.

Near field behavior (Soil Structure Interaction)

In present study, the soil-pile system is modeled as a beam on nonlinear Winkler foundation (BNWF) where soil elements are modeled as discrete non-linear springs as per API (2010). The BNWF model combines the far field behavior and near field behavior of the soil through a series of springs. The post liquefaction behavior of the soil is obtained by the use of a *p*-multiplier on strength and stiffness, keeping the nature of the curve same as the non-liquefied case. It is also noted that various authors have also studied the post liquefaction behavior of soils through full scale models (Rollins et al. (2005)) and laboratory testing. Dash (2010) noted that the liquefied soil offers very less or zero initial strength and stiffness. Hence the degradation of the strength and stiffness of the API *p-y* curves through multipliers (Brandenberg (2005), JRA 2002) may sometimes prove unconservative in modelling the behavior of liquefied soils. Various other calibrated BNWF models exist in practice, for which the frequency dependent springs and dashpots were calibrated against theoretical models provided by Novak et al. (1978) or numerical solutions (Kavvadas and Gazetas (1993)).

Numerical Modeling

The pile is modeled as a linear elastic concrete member with a characteristic compressive strength of 25 MPa and is designed to remain elastic for the maximum considered earthquake in the region. The pile head is considered to be free head. In this paper the aim was to investigate the bending response for possible variation in the depth of liquefaction. The pile tip is modelled as a roller support to allow translation and rotation at the pile tip. The settlement has not been considered in this model. Soil-pile Interaction is modeled through nonlinear springs at regular intervals according to the API (2010) code. Since only the lateral pile response is studied, soil springs are provided only in the horizontal direction (*i.e.* the direction of earthquake motion). A modal analysis is performed to study the mass participation factors of various modes and

damping of 5 % is defined at the first and N^{th} frequency, where N was the frequency where the cumulative mass participation factor for all modes before N was 0.85 (Chopra (2007)). To study possible effects of geometric nonlinearity (P - Δ Effects), the allowable axial load is back calculated for a 40 cm diameter pile resting on the given soil conditions through end bearing and skin friction springs in the longitudinal direction of the pile. The axial load of 1073.7kN is applied to the top of the pile while performing the time history analysis. For kinematic study, the top mass is ignored, however for studying the inertia and kinematic effect, the top mass equivalent to the top axial load is provided at the pile head. The analysis is performed through Hilbert-Hughes-Taylor direct time integration method in SAP2000 (2014). Figure 2 shows the BNWF numerical model used in this study representing the considered field condition.

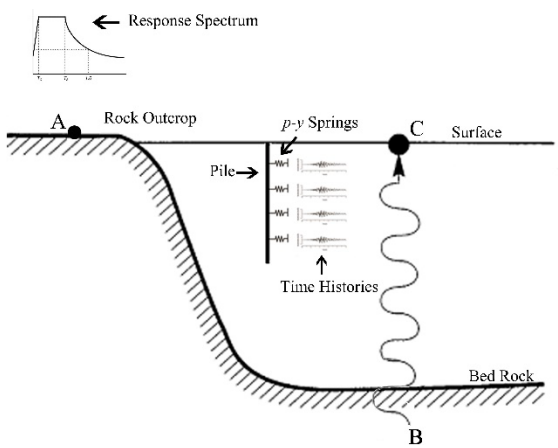

Figure 1: Spectrally matched time history estimated at Rock Outcrop (A), TH Converted to bed rock motion (B), Site Response study is done to find TH at various depths (B to C).

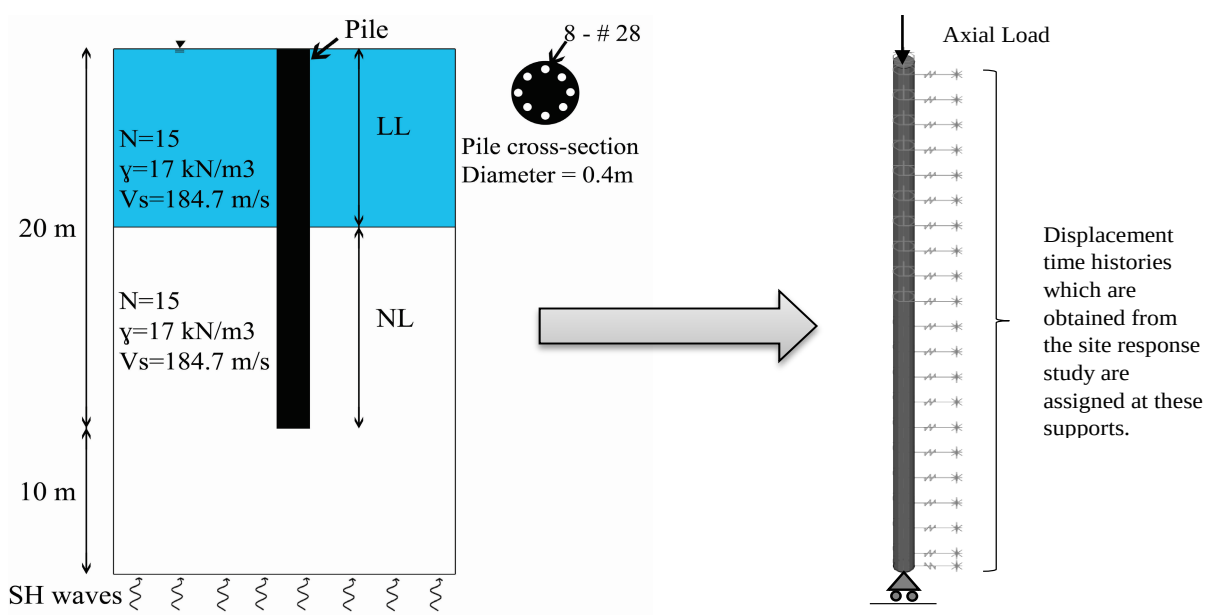

Figure 2: (a) The pile and soil condition (LL: Liquefied Layer, NL: Non liquefied layer), (b) The BNWF model implemented in SAP 2000.

Results

Modal analysis

By varying the depth of liquefaction, the model is analyzed for its first 12 modes of vibration. The fundamental frequency of the system is observed for two soil conditions, a) considering no soil spring for the liquefied soil zone, and b) considering a reduced strength soil spring (8% API *p-y* spring value). The no soil spring condition is in fact the representation of negligible initial stiffness of the *p-y* curve of liquefied soil, as the modal analysis is linear. The variation of fundamental frequency with respect to increasing depth of liquefaction is plotted in Figure 3. As expected, the frequency constantly decreases with increase in depth of liquefaction, making the soil-pile system flexible. However, the frequency for the model with reduced soil spring has shown an initial decrease up to a depth of 6m and the change is frequency is very minimal for larger depths. This could be due to the high initial stiffness of the liquefied soil layer, even after the strength is reduced by 92% that of the non-liquefied soil layer, at deeper depths. This may overestimate the spectral acceleration but underestimate the lateral drift at pile head, for the flexible system.

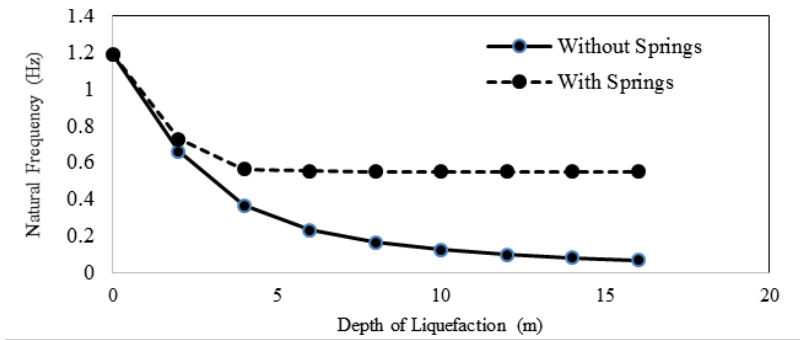

Figure 3: Variation of Natural Frequency with depth of liquefaction.

Time history analysis

Kinematic effect

The response of pile in terms of bending strain is investigated for four spectrally matched time histories without considering the superstructure mass. Figure 4 shows the envelope of bending strain with depth for the 4 time earthquake motions for different depths of liquefaction. For each pile soil configuration, as shown in the figure 4, the plot of the maximum strain envelop with depth is displayed adjacent to it. The maximum strain is always observed at the boundary between the liquefied and non-liquefied soil strata. The maximum bending strain for non-liquefied case was 0.0037% which increased to a value of 0.16% for liquefaction depth of 14m. For further increase in depth of liquefaction, there was no significant change observed. However, the location of maximum strain always remained at the boundary of stiffness contrast. Figure 5 plots the bending strain for four time histories with respect to various depths of liquefaction. It is observed from the figure that the strain induced by Kocaeli earthquake is higher for all depths of liquefaction. This might be due to the fact that the response spectrum obtained from the matched Kocaeli earthquake time history was above all other time histories, when compared with IS-1893.

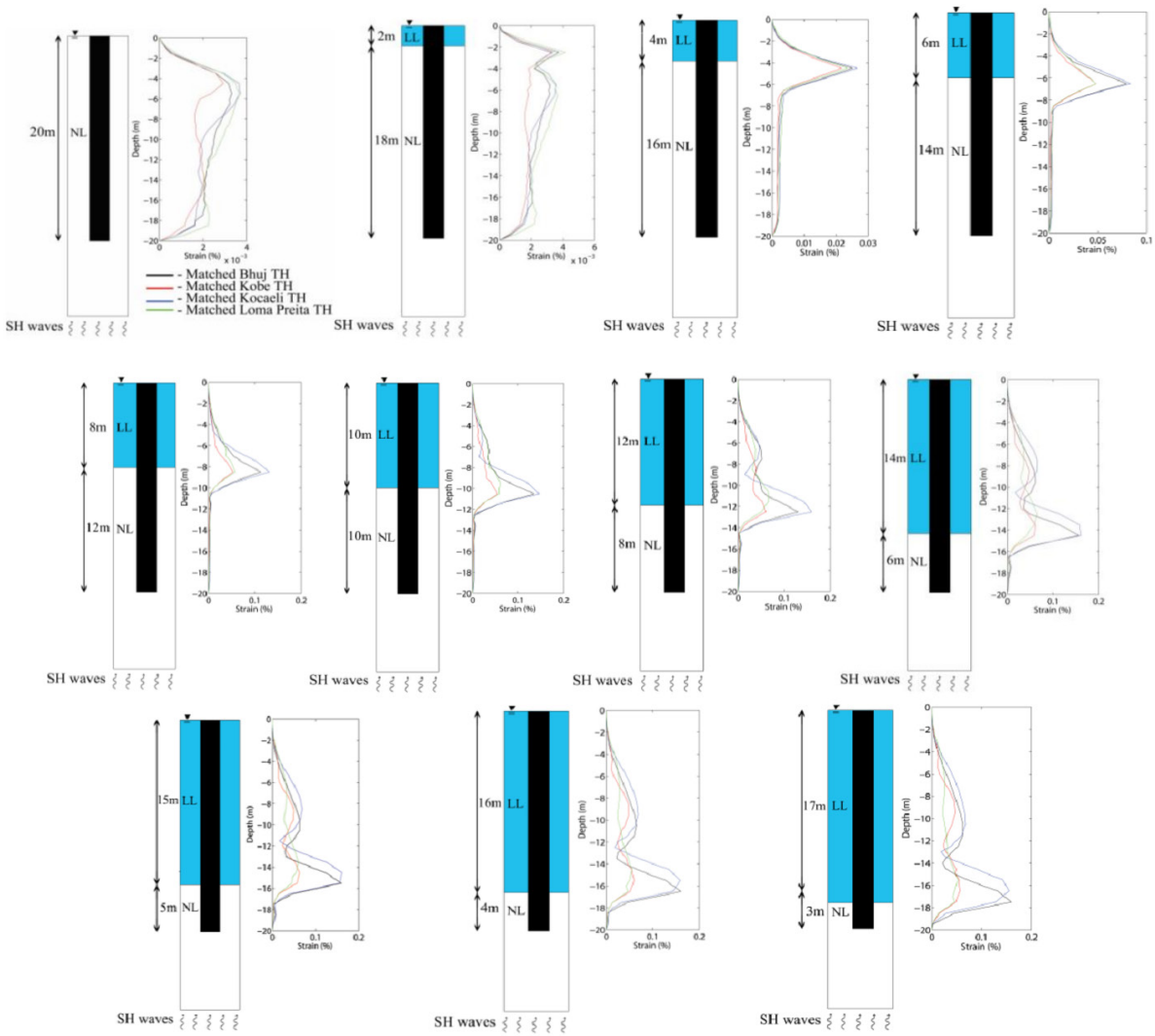

Figure 4: Envelop of maximum bending strain in the pile for different depths of liquefaction.

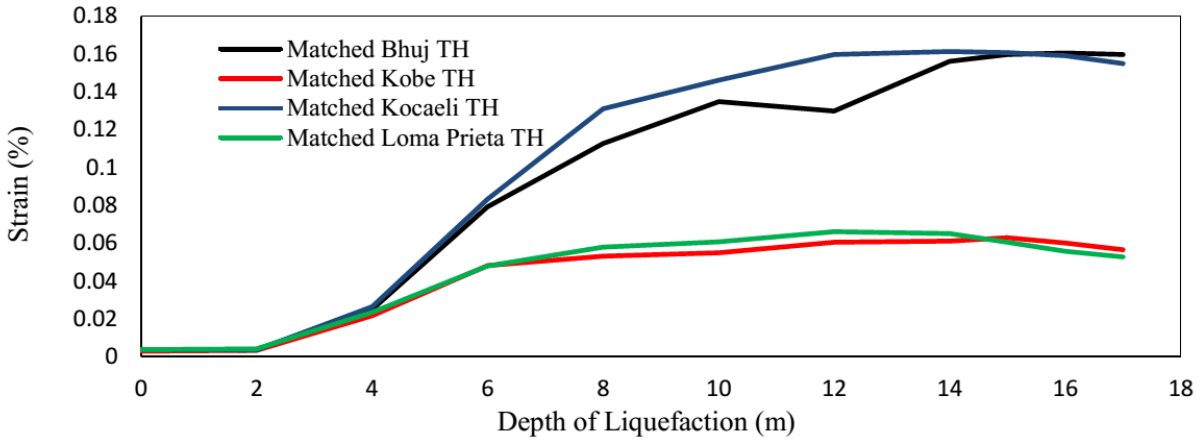

Figure 5: Variation of bending strain with depth of liquefaction.

Kinematic and Inertial Effect

To investigate the combined effect of kinematic bending response along with the inertia effect, the model was analyzed with top mass at pile head. The bending strain obtained from the analysis for the critical case of peak kinematic response (*i.e.* for depth of liquefaction of 14m) is studied for this combined effect. Figure 6 shows the bending strain envelops for four different earthquakes.

It is clearly seen from the figure that the effect of inertial loads is more towards the top layer of the soil, which attenuates quickly with depth. For non-liquefied soil condition, the differentiation between inertia and kinematic effect is not evident, however, for liquefied soil condition there are two peak bending strains observed. This observation is same for all four time histories. One of the peak bending strains close to the top is due to inertia effect, whereas other peak strain at the interface of stiffness contrast signifies the kinematic effect. There could be possibility of any of these two responses to become dominant. Two time histories (*i.e.* Bhuj and Kocaeli) gave larger kinematic bending strain with respect to inertial bending strain.

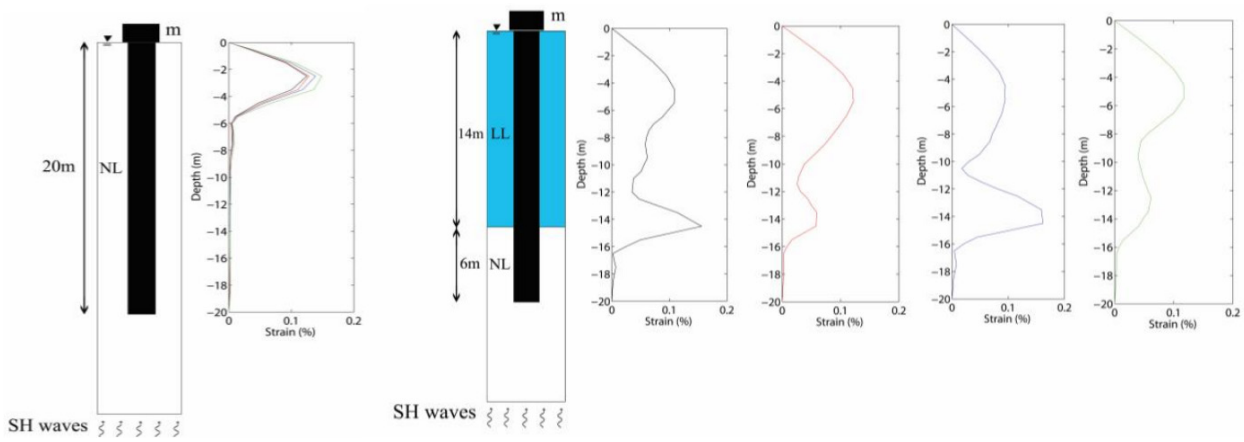

Figure 6: (a) Inertial interaction for no liquefaction case, (b) Kinematic and Inertial Interaction for Kocaeli time history with depth of liquefaction 14m

Conclusions

The performance of a pile foundation system, when subjected to earthquake loading, in liquefiable soils is presented in this paper through time history analysis. The depth of liquefaction is parametrically varied to study the kinematic bending moment for four ground motions, spectrally matched to the IS 1893 design spectrum. The study signifies that the use of multipliers on the API *p-y* curves for liquefied soils may overestimate the spectral acceleration or forces in the pile but underestimate the lateral drift at pile head, for the flexible system. Sharp rises in bending moments are observed near the depth of stiffness contrast, even for an initially homogeneous liquefiable soil deposit, which can exhibit layering arising due to the progression of liquefaction. However, this important consideration for the kinematic effect could be easily missed by designers for a homogeneous liquefiable soil deposit. It is also observed that in certain cases the maximum bending moment due to kinematic effect may be higher than that due to the inertial effect. Hence, there could be possibility of either inertial or kinematic responses governing the design in liquefiable soils, even though initially homogeneous.

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