INVESTIGATION OF BENDING – BUCKLING INTERACTION OF PILES IN LIQUEFIABLE SOILS

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ABSTRACT :

Post-earthquake reconnaissance of the 2001 Bhuj (India) earthquake and the 1995 Kobe (Japan) earthquake reveals that the pile supported structures still collapse after major earthquakes despite a large factor of safety against bending due to lateral loads being employed in their design. The authors believe that the cause of this phenomenon is due to omission of the effect of axial loads in code based design of piles which assumes the failure of piles in liquefied soils to be due to lateral spreading of the soil resulting in a flexural failure. A number of experimental investigations at University of Cambridge reveal that when the soil liquefies the reduction in the supporting lateral stiffness of the soil can cause a buckling failure of the pile which can be initiated by lateral spreading or inertia or any imperfection. Bending and buckling requires different approach in design. Moreover, both the mechanisms (bending and buckling) interact and ultimately the member in question fails by exceeding the bending moment. The bending-buckling interaction mechanism is thereby being hypothesized as the primary reason leading to the collapse of many pile-supported structures in earthquakes. An analytical investigation and a numerical study using SAP and OpenSees are being carried out in this manuscript to support this hypothesis and also to highlight the importance of this interaction.

KEYWORDS: Pile, Earthquake, Pile-soil interaction, Bending-buckling interaction, Liquefaction

1. INTRODUCTION

Collapse of several pile foundations in liquefiable soils are still observed after most strong earthquakes. Many of these failures are associated with the formation of plastic hinges in the pile, i.e. structural damage to the piles. Currently, structural design of piles in liquefiable soils is carried out assuming the piles as beams that resist bending. This bending is due to lateral loads from inertia of the superstructure and/or lateral spreading of the ground. The well understood mechanisms of pile failure in liquefiable are: (a) excessive settlement; (b) shear, or (c) bending. These mechanisms are the basis of the design guidelines proposed in many codes of practice. Of these mechanisms, bending due to lateral spreading of the ground has been regarded as the root cause of many pile failures; see for example Hamada (1992a and 1992b), Ishihara (1997), Tokimatsu et al. (1998), Goh and O’Rourke (1999), Abdoun and Dobry (2002) and Finn and Fujita (2002).

Recent research (Bhattacharya et al. (2004), Knappett and Madabhushi (2005), Kimura and Tokimatsu (2005), Shanker et al. (2007) etc.) suggests another possible mechanism of pile failure in liquefiable soils: buckling instability of piles. Essentially, during liquefaction, the pile suffers a significant loss of lateral support in the liquefied zone, and if the axial load is very high (i.e. near to the critical buckling load)- the combination may lead to pile instability and the structure may deform excessively.

Though, the bending and buckling mechanism of piles in liquefied soils have been studied in detail separately, little has been reported on their interaction. The aim of this paper is to investigate the importance of bending buckling interaction for piles in liquefiable soils.

2. CONCEPT OF BENDING-BUCKLING INTERACTION

Bending and buckling requires different approaches in design as bending is strength-based design and buckling is stiffness-based. Pure bending is a stable mechanism as long as the pile is elastic. This implies as the load is withdrawn the pile returns to its original position. On the other hand, buckling represents a sudden instability of the pile and initiates when the axial load in pile reaches a certain value. In reality, during an earthquake in liquefiable soils, piles are subjected to both axial and lateral loads and hence act like beam-column members. The presence of the axial load amplifies the lateral deflection caused by lateral load. The ensuing large deflection of the beam may then induce plasticity in the beam resulting in a premature failure. This is basically the P-delta effect of the axial load. Stability analysis of pure elastic columns (Timoshenko and Gere 1961) shows that the lateral deflection (Δ_0) due to lateral loads is amplified in the presence of axial load (P) producing a final deflection of pile (Δ) as given by Equation 1. Here P is the applied axial load in the member and P_{cr} is the Euler Critical load defined later on.

$$\frac{\Delta}{\Delta_0} = BAF = \frac{1}{\left(1 - \frac{P}{P_{cr}}\right)} \qquad \text{(Eq1)}$$

The term (Δ / Δ_0) is referred to as the *"Buckling amplification factor (BAF)"* which varies for different values of the critical axial load (P_{cr}) which in turn depends on the boundary conditions of the pile above and below the liquefiable zone. The elastic critical buckling load for a member of bending stiffness *EI* is defined by Equation 2. This is also often referred to as Euler buckling load.

$$P_{cr} = \frac{\pi^2 EI}{L_{eff}^{\ 2}} \qquad \text{(Eq2)}$$

In equation 2, L_{eff} is the effective length of member, which is a function of the unsupported length (L_0) and boundary condition of the member. Details of computing elastic critical load for piles can be found in Bhattacharya and Madabhushi (2008), Bhattacharya et al (2004). If the supporting soil surrounding the pile liquefies, the pile loses its lateral support and the unsupported length increases. Increase in L_{eff} decreases the value of critical buckling load (P_{cr}) of the member and increase the buckling amplification factor. Figure 1 shows a schematic of the effect of bending-buckling interaction to the response of a pile foundation. The pile is subjected to lateral loading due to soil flow and/or inertia and three levels of axial loading:

(a) No axial load ($P = 0$);
(b) Axial load at 50% of buckling load ($P = P_{cr}/2$); and
(c) Axial load close to buckling load ($P \approx P_{cr}$).

As the axial load (P) increases, the pile head deflection and pile bending moment also increases. When the axial load is close to the buckling load (P_{cr}), the bending moment amplification factor becomes very high, which leads the bending moment in pile to reach its plastic moment capacity, M_p. This occurs for a small value of the lateral load. The sudden rise in pile head deflection demonstrates the failure point of the pile where bending moment reaches M_p and pile continues to deflect without any additional loading. The bending buckling interaction is further verified in two ways:

(a) An analytical study for inertia force and axial load interaction, and
(b) A numerical study for lateral spreading and axial load interaction.

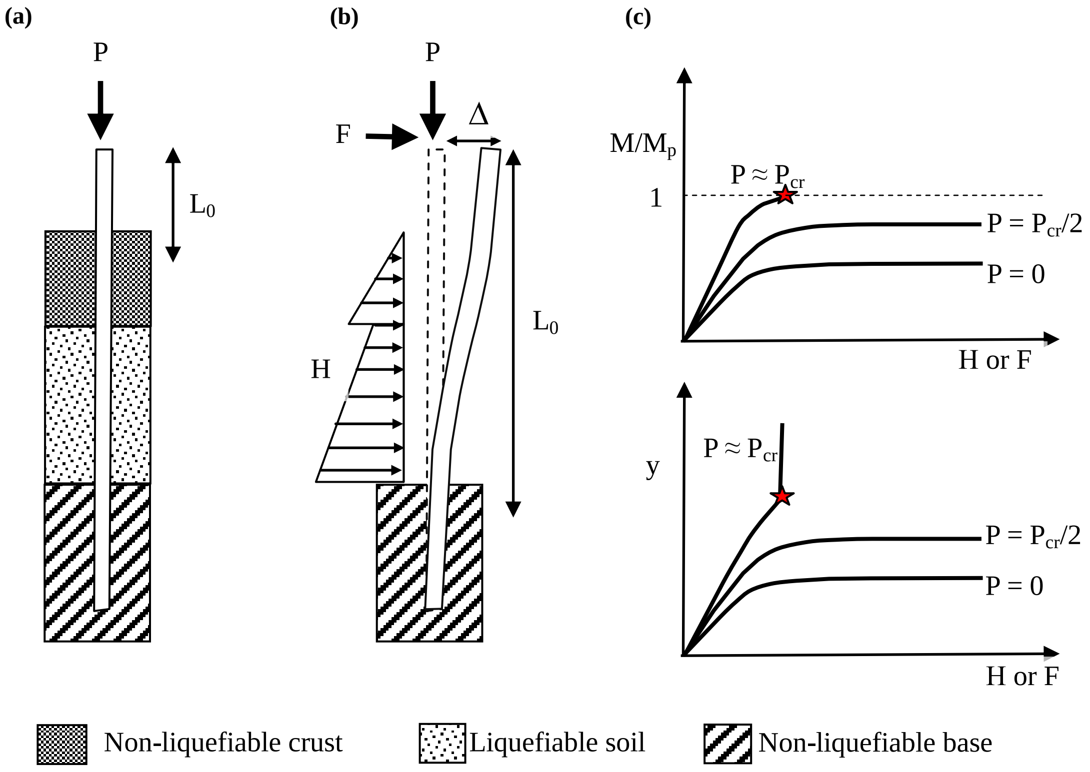


Figure 1: (a) Service condition of pile foundation; (b) Seismic condition, pile subjected to lateral spreading and/or inertia load; (c) Effect of bending-buckling interaction on the response of pile foundation. [L_0 = Unsupported length of pile]

3. ANALYTICAL INVESTIGATION OF BENDING-BUCKLING INTERACTION

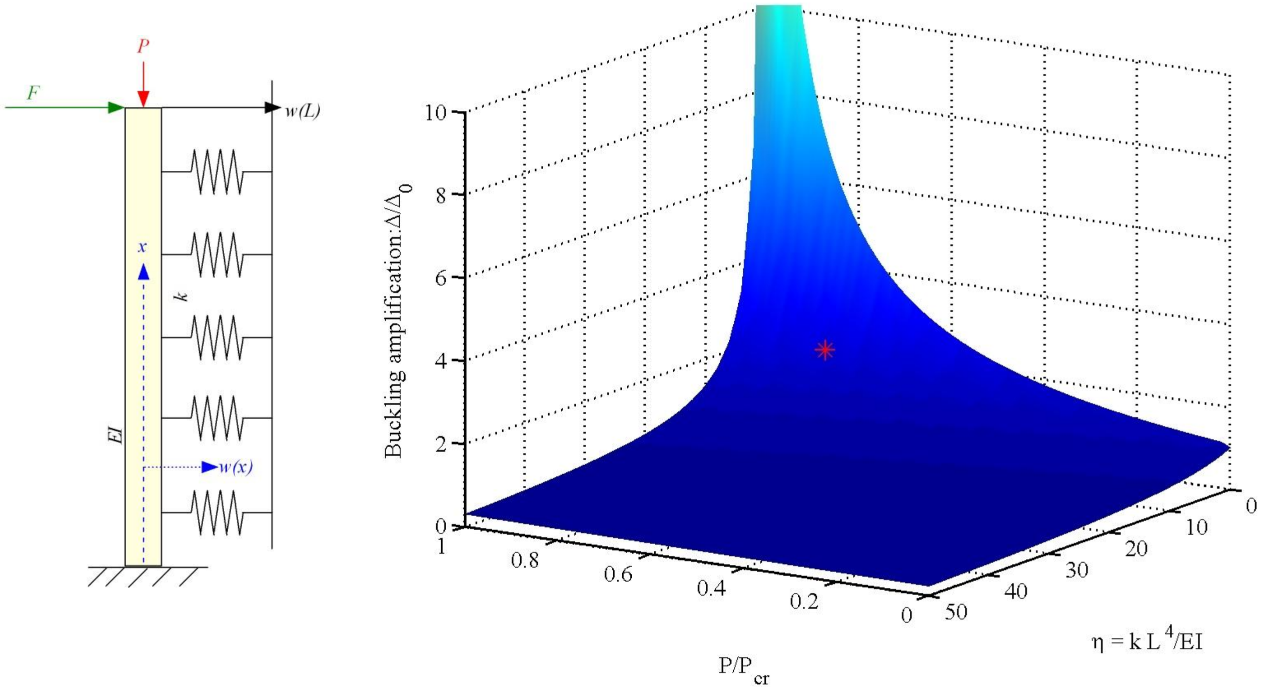
A simple beam on elastic foundation (Euler-Bernoulli beam) is considered to study the effect of axial load and soil stiffness degradation on the buckling amplification factor. Figure 2(a) shows the beam having a bending stiffness (EI) and is resting against a linear uniform elastic support of stiffness k. A constant static lateral force F is applied at the top of the beam (x = L). The well known equation of static equilibrium can be expressed as given in equation 3.

$$EI\frac{\partial^4 w(x)}{\partial x^4} + P\frac{\partial^2 w(x)}{\partial x^2} + kw(x) = 0 \qquad \text{(Eq 3),}$$

where, w(x) is the transverse deflection of the beam and x is the spatial coordinate along the length of the beam. It is assumed that the mechanical properties of the beam are constant along the length. Equation 3 is a fourth-order partial differential equation and requires four boundary conditions for its solution (zero slope and zero deflection at the bottom of the pile i.e. at x=0 and at the pile head i.e. x=L, we have moment and shear force as zero. The solution of the equation is plotted in Figure 2b. Details of the solution technique will be discussed elsewhere by Bhattacharya and Adhikari (2008). The buckling amplification factor is plotted against two non-dimensional parameters related to axial load and support stiffness as:

Non-dimensional axial load = $\dfrac{P}{P_{cr}}$, and

Non-dimensional support stiffness = $\eta = \dfrac{kL^4}{EI}$.



(a) (b)

Figure 2: (a) Pile-soil model using Euler-Bernoulli beam with axial and lateral force resting against a distributed elastic support. (b) Variation of buckling amplification factor (*BAF*) due to change in support stiffness and axial load.

The figure shows that when the axial load is high (i.e. near to Euler buckling load) and the lateral soil stiffness is low (liquefaction or soft clay), the amplification of lateral displacement is very high. Precisely, this is the loading scenario of a slender pile in liquefied soil. During the service condition, the unsupported length of pile is very less and k is large (high lateral soil stiffness), and hence, does not seem to be of major concern. However, during seismic liquefaction, considerable length of the pile (typically 10 to 15 meters in many case histories of pile failure) becomes unsupported which increases the P/P_{cr} ratio. Also, the liquefaction reduces the soil strength and stiffness significantly. Hence both of the effects of liquefaction facilitate the increase in the buckling amplification of pile. The plot clearly shows the importance of the axial load as well as soil stiffness for pile performance during earthquakes.

4. NUMERICAL VERIFICATION OF THE BENDING-BUCKLING INTERACTION

4.1 Modelling details

One of the well documented bridge piles, i.e., Showa Bridge pile, is considered for the numerical study of bending-buckling interaction. The intention however is not to figure out the actual cause of failure but to show the importance of bending-buckling interaction for piles in liquefied soil. The Showa bridge supported by pile foundations had failed during 1964 Niigata earthquake. This example is chosen as this bridge collapsed just one month after the construction, and had steel tubular piles. This ensures less uncertainty of material strength, as degradation of piles due to corrosion is not expected. The soils at bridge site were subjected to significant liquefaction, and lateral spreading were also observed near the north bank of the river. Hence, the bending buckling interaction analysis is carried out by considering simultaneously the axial load and lateral spreading forces.

The pile and soil properties are taken according to the pulled out pile P4 as given in Fukuoka (1966). Each pile was 25m long with outer diameter of 0.609m. The wall thickness of the top 12m of the pile was 16mm and the bottom 13m was 9mm. About 10m of the pile was in the liquefiable soil zone, which is assumed to be the source of lateral spreading force. The pile is embedded in the 6m of competent soil under the liquefied soil layer, which is assumed to provide restraint during lateral spreading (Figure 3a). This pile has been modelled in SAP (CSI 2005) as single standing column with lateral nonlinear soil springs at the bottom to simulate the restraint provided by the non-liquefied soil layer. The spring constants were obtained from the API (2000) recommendation. The nonlinear properties of the pile section are based on the stress-strain response of the pile material as shown in figure 4. The lateral spreading force is applied over the length of pile in liquefied soil zone. JRA (2001) code is one of the advanced codes of practice that suggest the lateral spreading force on pile as 30% of the total overburden pressure acting over the pile section. Figure 3b shows the pile subjected to lateral spreading force as per JRA along with axial load. The calculation of the lateral spreading as per JRA can be found in Bhattacharya (2005). He et al. (2006) also reported an experimental study which showed that the average lateral pressure from liquefied soil on a pile may vary from 20-40kPa. In the present study, a mean value of 30kPa is considered as a uniform loading to the pile subjected to lateral spreading. Details of the modeling can be found in Dash et al (2008).

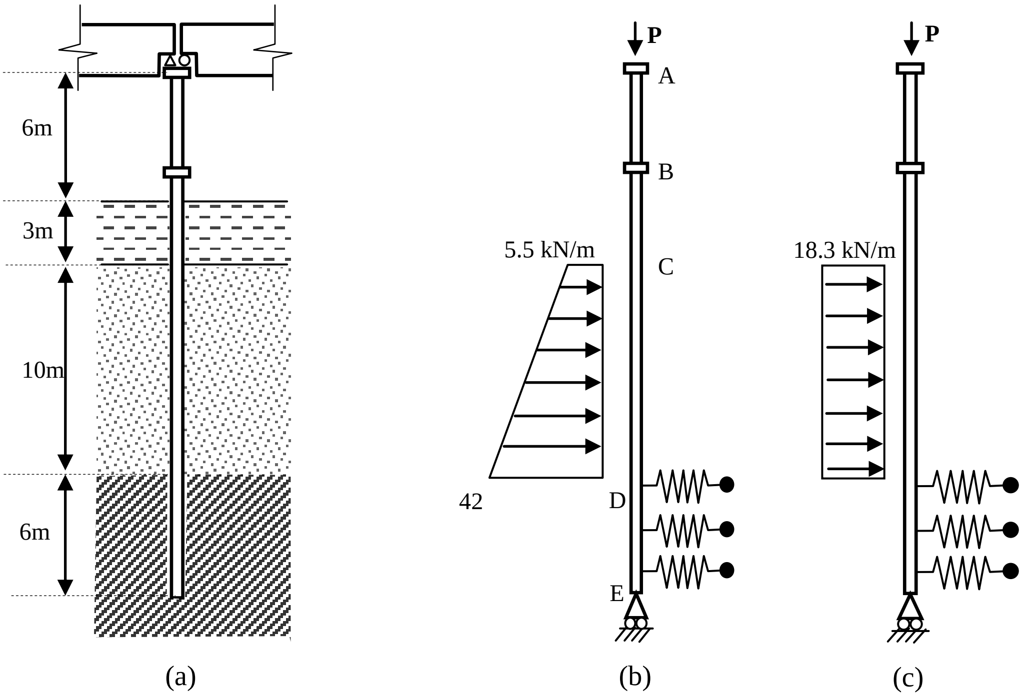


Figure 3 (a) Showa Bridge pile. (b) Numerical model with JRA loading. (c) Numerical model with He et al. (2006) uniform loading.

The present force based analysis, hence, considers two cases of lateral loading as shown in figures 3b and 3c. The analysis is carried out for a range of axial loads. The critical buckling load for the pile at full liquefaction is calculated by Bhattacharya et al (2005) to be 1030kN. Hence, the axial load (P) is varied from 0 kN to 1000 kN with an increment of 100 kN. For a particular analysis, the axial load is kept constant during the analysis, and the lateral load is applied gradually from 0% to 100%. The analysis includes the P-Delta effect.

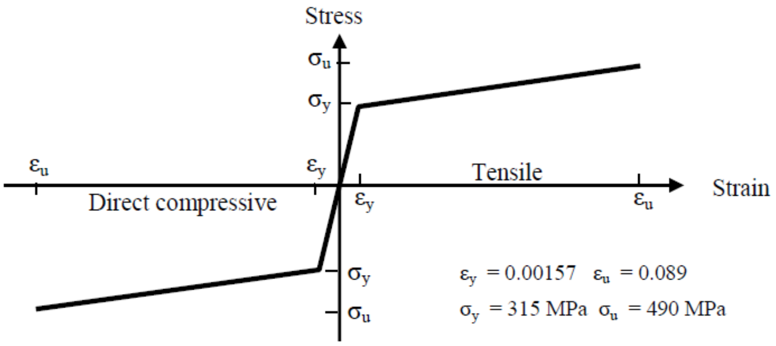


Figure 4 Stress-strain behaviour of the pile used in the present study.

4.2 Results

Figure 5 plots the normalized pile head displacement (y/D) with respect to the applied JRA type lateral load for a range of axial load. Figure 6 plots similar results for the He et al (2006) type lateral loading. As expected, as axial load increases, the failure of pile happens with lesser lateral load. Pile may be considered as failed if deflected for about 1-2D, where D is its diameter. The sudden kink in the plot represents the yielding of the pile, beyond which the deflection of pile happens in a higher rate. We can expect this result as the after yield stiffness of the stress-strain curve of the pile material is about 100 times lower than its elastic stiffness.

For present case history, Bhattacharya et al. (2005) calculated the axial load of the pile to be about 740 kN. Figure 5 hence suggests that, the pile with an axial load of 740 kN can be expected to fail at about 10% of the lateral load (JRA type). Figure 6 also shows the similar results while uniform loading as suggested by He et al (2006) is applied.

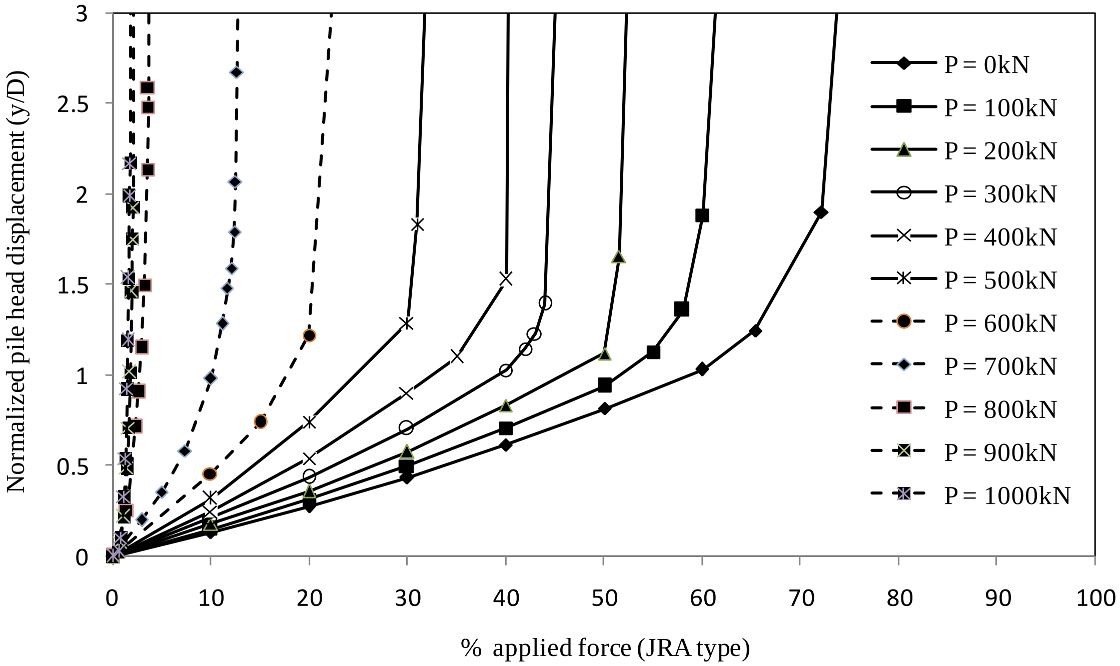


Figure 5 Normalized pile head deflection with increase in lateral loading (JRA Type) for a range of axial load.

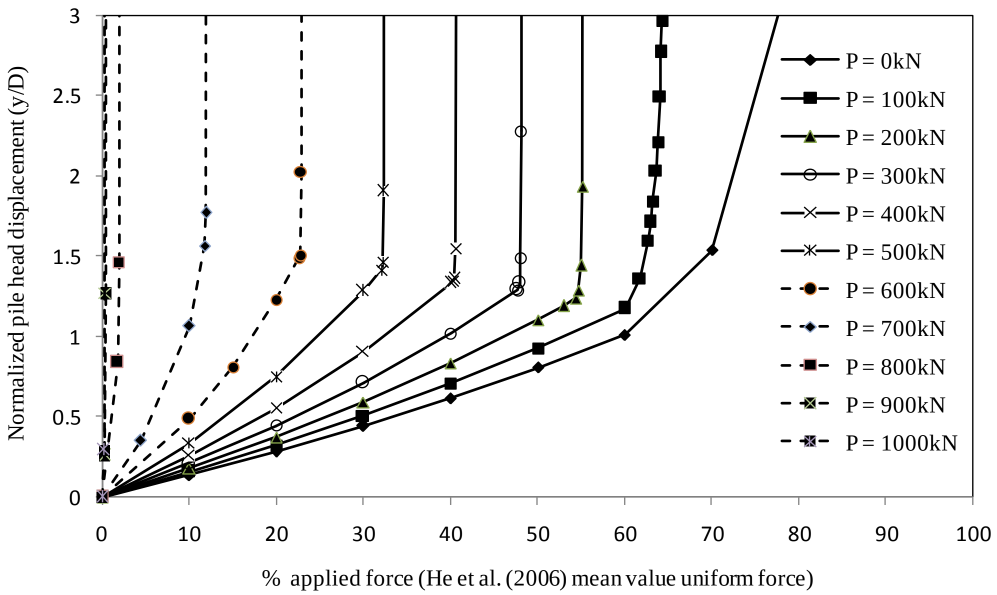


Figure 6 Normalized pile head deflection with increase in lateral loading (He et al. (2006) type) for a range of axial load.

4.3 OPENSEES Simulation

One of the limitations in SAP model described in the previous section is the detailed plasticity modelling. SAP does not allow the user to define distributed plasticity in the frame section and uses lumped nonlinear hinge properties. To avoid this limitation, the open source FE program, OpenSees, can be used to model the same frame element for bending-buckling interaction analysis. The results of this simulation shows very similar pattern of soil-pile interaction. However, due to lack of space, these results are not presented and discussed here.

5. CONCLUSION

From this study, the following conclusions are made:

(a) Buckling and bending requires different approaches in design. Bending is a strength-based design whereas buckling is a stiffness-based design. Currently, piles in liquefiable soils are designed against simple bending mechanism. Designing against bending would not necessarily satisfy the buckling requirement i.e. stiffness requirements. To avoid buckling, there is a need for a minimum diameter of pile depending on the thickness of the liquefiable layer and the boundary condition at the top and bottom of the liquefied zone.

(b) Bending and buckling mechanisms interact and therefore should not be viewed in isolation. It has been hypothesized that bending-buckling interaction of piles may be the prime cause of failure of piles in seismically liquefiable areas during strong earthquakes.

(c) A force-based method of analyzing bending-buckling interaction has been demonstrated using a structural engineering software SAP. Designers must consider the interaction of bending-buckling while designing the pile foundations in seismic zones.

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