Influence of *p-y* curves on Buckling Capacity of Pile Foundation

Praveen Huded M., Suresh R. Dash

***Abstract*—**Pile foundations are one of the most preferred deep foundation systems for high rise or heavily loaded structures. In many instances, the failure of the pile founded structures in liquefiable soils had been observed even in many recent earthquakes. Failure of pile foundation have occurred because of buckling, as the pile behaves as an unsupported slender structural element once the surrounding soil liquefies. However, the buckling capacity depends on the depth of soil liquefied and its residual strength. Hence it is essential to check the pile against the possible buckling failure. Beam on non-linear Winkler Foundation is one of the efficient methods to model the pile-soil behavior in liquefiable soil. The pile-soil interaction is modelled through p-y springs, there are different p-y curves available for modeling liquefiable soil. In the present work, the influence of two such p-y curves on the buckling capacity of pile foundation is studied considering the initial geometric and non-linear behavior of pile foundation. The proposed method is validated against experimental results. A significant difference in the buckling capacity is observed for the two p-y curves used in the analysis. A parametric study is conducted to understand the influence of pile flexural rigidity, different initial geometric imperfections, and different soil relative densities on the buckling capacity of pile foundation.

***Keywords*—**Pile foundation, liquefaction, buckling load, non-linear p-y curve.

I. Introduction

PILE foundations are still one of the most practiced deep foundation systems for bridges, high rise buildings and wharves when the soil supporting the foundation is weak. Pile foundations have prevented the complete collapse of structures during the past earthquakes. Many bridges and building structures have shown partial or complete collapse during the past earthquakes as a result of liquefaction [1]-[3]. The behavior of pile foundation in liquefiable soil is a transient problem involving interaction of inertial force, generation of excessive pore pressure, and reduction of soil strength [4]. Pile foundation in liquefiable soil may fail due to bending, buckling, excessive settlement or any combination of these [5]. Berrill and Yasuda [4] proposed the possibility of buckling mechanism for the pile failure, once the soil liquefies pile will act as a slender column element, lateral deflection due to soil displacement and axial load affect the lateral stiffness of the pile. Centrifuge experimental studies have demonstrated the possibility of buckling failure (due to axial load) of pile foundation in liquefiable soil [6], [7]. Nadeem et al. [8] studied the buckling behavior of end bearing piles using finite element method without considering the liquefaction effect (including initial geometric imperfections). Knappett and Madabhushi [7] used Rik's post buckling analysis method for buckling analysis of pile groups but did not include the geometric imperfections. Few studies on the critical buckling load of pile foundation in liquefiable soil are based on Euler's buckling theory without considering the non-linear behavior of pile material and residual shear strength of the liquefied soil [6], [9]. However residual shear strength of the liquefied soil and the non-linearity of the pile material influence the buckling capacity of the pile [10], [11]. Zhang et al. [11] investigated the buckling behavior of pile foundation in liquefiable soil considering the residual shear strength of the liquefied soil and non-linearity of the pile material. The soil-pile interaction is modelled using liquefied p-y curves proposed by Dash et al. [12].

In the present study, buckling capacity of pile foundation in a liquefiable deposit is evaluated using Beam on Non-linear Winkler Foundation (BNWF) method. Soil-pile interaction is modelled using *p-y* curve proposed by API [13] and a liquefied *p-y* curve given by Dash et al., [12] to study the influence of the types of *p-y* curves. A centrifuge experiment conducted by Bhattacharya [6] is modelled numerically using both the *p-y* curves and a parametric study is conducted to study the influence of various parameters and results are presented.

II. BNWF Method for Buckling Load Calculation

The soil-pile system is modelled in OpenSees® platform (Open System for Earthquake Engineering Simulation [14]). Critical buckling load for pile foundation in liquefiable soil is evaluated using BNWF method (Fig. 1), following section describes the methodology adopted.

A. Modeling of Liquefied Soil (p-y Spring)

Centrifuge experiments have credibly shown that end bearing pile (both single and pile group) fails in buckling when the soil surrounding the pile liquefies [6], [7]. Any non-liquefiable soil or partially liquefied soil would give restraint to the pile deformation and would increase the buckling capacity of the pile. In the present study only the soil which is completely liquefied is considered. Hence the p-y curve for the liquefied soil is modelled.

In BNWF model the soil pile interaction is modelled using non-linear springs called *p-y* springs, these are discrete *p-y* springs for lateral loading (Fig. 1 (b)). The p-y springs are modelled according to the procedure given by API [13] with a

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degradation factor of 0.1 (stiffness reduced to 10% of the initial values to account for liquefaction) and as proposed by Dash et al. [12] for the liquefied soil as shown in Fig. 2.

The *p-y* curve proposed by Dash et al. [12] for the liquefied soil is included in the present study. In p-y curves, *p* refers to the pressure per unit length by the lateral soil, and *y* refers to the relative pile-soil displacement [15]. In most of literature the *p-y* spring for the liquefied soil is modelled based on the recommendations of API [13]. However, *p-y* curve recommended by Dash et al., [12] retains the essential features of the liquefiable soil such as zero strength and stiffness at the initial lower strains and at large strains liquefied soil exhibit strain hardening behavior which are essentially not incorporated in the *p-y* curve of API code [12]. Dash et al. [12] proposed the methodology based on the typical bore log data which could be conveniently used by the designers. The p-y curve can easily be constructed from (1) as given by [12]:

$$p=\omega\frac{p_1}{y_1}y+A(1-\omega)\left[\frac{p_u+p_1}{2}+\frac{p_u-p_1}{2}\tanh\left(\frac{2\pi}{3(y_u-y_1)}\left(y-\frac{y_u+y_1}{2}\right)\right)\right] \quad (1)$$

where p_u and y_u are ultimate lateral resistance and lateral displacement respectively, p_1 and y_1 are initial lateral resistance and lateral displacement respectively, weighted function ω is given by equation 2. Constants in (1) and (2) can be evaluated based on the bore log data [12].

$$\omega=\frac{1}{2}\left(1-\tanh\left[\frac{6\pi}{y_u}\left(y-\frac{4y_1+y_u}{6}\right)\right]\right) \quad (2)$$

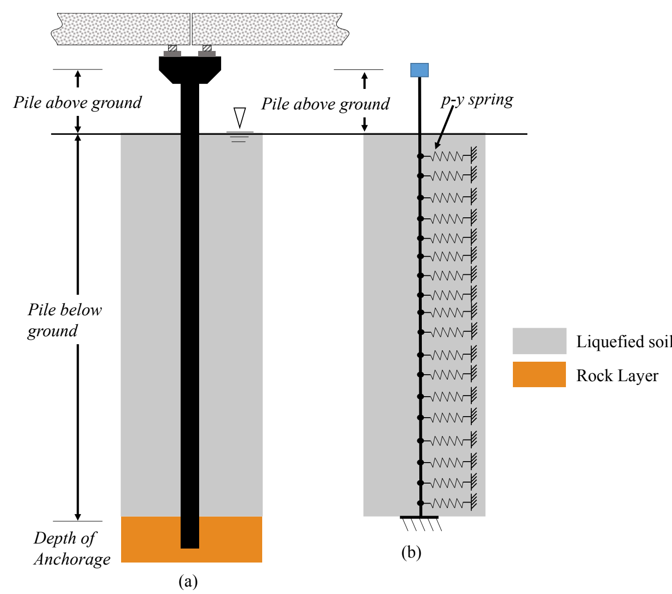
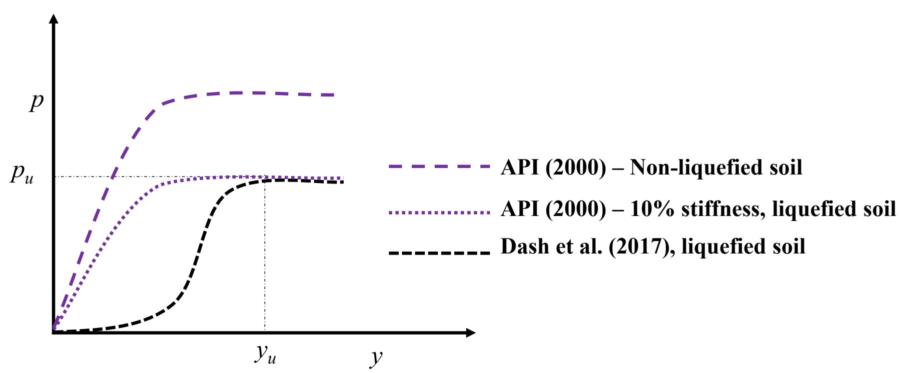


Fig. 1 Numerical model developed (a) Typical bridge foundation; (b) BNWF model (Soil-pile interaction)



P: Resistance of soil   y: Soil-pile relative displacement

Fig. 2 p-y curve for the liquefiable soil

B. Modelling of Pile

In the present analysis, an end bearing pile is modelled using nonlinear beam column elements [14]. The nodes of the pile elements are modelled in two-dimension with three degrees of freedom (two translational and one rotational). The pile material non-linearity is captured by integrating stress-strain response of pile material through fiber section approach, the pile is meshed in vertical direction. Corotational transformation is adopted to capture the geometric non-linearity of the modelled pile [11], which can be constructively be used for evaluating the buckling load [16].

C. Initial Geometric Imperfections of the Pile

Initial imperfections are induced in piles during the manufacturing, transportation, or during construction (during driving or boring). It is very impractical to achieve zero radius of curvature in the installation of pile foundation [8]. The initial curvature induced in the pile profile makes it more vulnerable towards the buckling mode of failure. Also, lateral load from the super structure, soil slope movement, and initial geometric imperfections decrease the buckling capacity of pile foundation. End bearing and friction pile response is governed by the degree of pile curvature and slenderness ratio [17], [18]. However initial bent pile surrounded by the soil will help in increasing the bending capacity till the yielding of the surrounding soil. Once the soil yields, stresses in the pile are localized at a point along the pile length of the pile. Dunlope et al. [19] based on the experimental work recommended the bent pile shape approximated to a quarter sine profile. The initial imperfections were modelled through pile nodes coordinates based on (3) for quarter sine profiles, where 'a' is an induced initial imperfections and 'L' is length of the pile (Fig. 3).

$$x = a \sin\left(\frac{\pi y}{2L}\right) \tag{3}$$

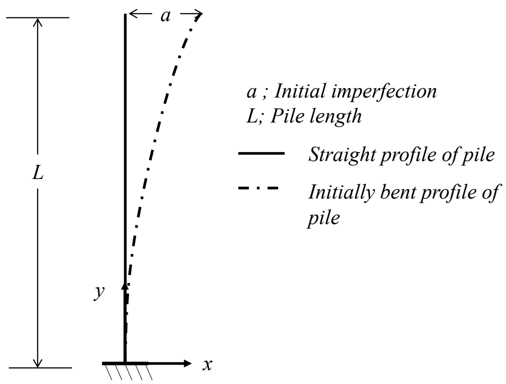


Fig. 3 Quarter sine profile of the pile foundation

D. Estimation of Critical Buckling Load

The soil-pile interaction is modelled through *p-y* springs. The pile is meshed in vertical direction at 0.25 m and the *p-y* springs are also spaced at 0.25 m [20]. The properties of the *p-y* springs are derived based on the soil properties such as soil relative density, unit weight of soil and dimensions of pile, based on the recommendations of Dash et al., [12] and American Petroleum Institute (API) [13]. Based on the procedure explained in previous sections a BNWF model is developed incorporating geometric imperfection of pile and pile-soil material non-linearity. Non-linear buckling analysis of pile foundation is carried out by applying displacement controlled axial compression which resulted in Load deflection curve as shown in Fig. 4. The peak of the load-deflection curve is considered as the critical buckling load of the pile foundation [8], [11].

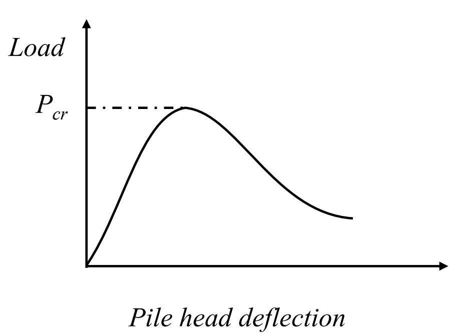


Fig. 4 Load deflection curve

III. Validation of Centrifuge Tests

The proposed numerical modelling is validated against the centrifuge experiments conducted by Bhattacharya [6], where a series of centrifuge experiments were carried out at various ratios of P/P_E (P; is applied load and P_E is Euler buckling load). Experiments named SB-04-Pile_ID-07, SB-04-Pile_ID-08, and SB-06-Pile_ID-10 are modelled in the present study. All the experiments consisted of single soil layer of depth 9m of saturated silica sand. Table I presents the soil properties used in the present validation work. The pile used in the centrifuge experiment is made of aluminum, Table II lists the properties of the pile. The pile is fixed at the base of the model container. In centrifuge experiment pile is in both gravity acceleration of 1g and centrifugal acceleration (50g) which are perpendicular to each other; hence pile may be accompanied a geometric imperfection in the form of quarter sine wave profile [11]. As the data of this geometric imperfection are not explicitly measured during the experimental work, an imperfection amplitude of 0.5% times the length of pile (L) is used. BNWF model is developed to determine the critical buckling load. The soil pile interaction is modelled using *p-y* curves based on API [13] and *p-y* curve proposed for the liquefied soil by Dash et al. [12]. The load deflection plots for the experiment SB-04-Pile_ID-07 is shown in Fig. 5. It is evident from Fig. 5 that *p-y* curves based on the API [13] with a reduced stiffness do not predict the failure however the experimentally the pile has failed. The analysis with the new *p-y* model proposed by Dash et al. [12] predicts the failure due to buckling. This is due to the fact that API model has higher initial stiffness at the small strain levels as shown in Fig. 2. Zhang et al. [11] have also drawn the similar conclusion.

The results of the experimental work and the present numerical work are given in Table III. The results of the proposed BNWF method (using the p-y curve proposed by Dash et al., [12]) are in comparison to the centrifuge results. Hence in the subsequent parametric study the model proposed by Dash et al. [12] is adopted in critical buckling analysis of pile foundation in liquefiable soil deposit.

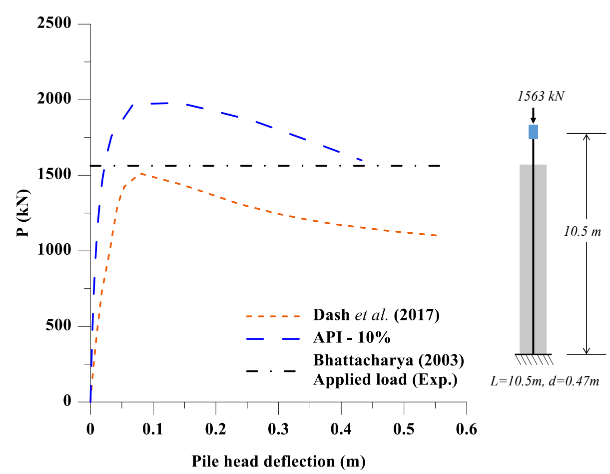


Fig. 5 Load deflection curve for experiment SB-04-Pile_ID-7

TABLE I
SOIL PROPERTIES USED IN THE VALIDATION WORK

Parameter	SB-04	SB-06	Reference
Relative Density of sand D_r (%)	43	40	Bhattacharya [6]
Specific gravity of sand G_s	2.65	2.65	Bhattacharya [6]
e_{min}	0.613	0.613	Bhattacharya [6]
e_{max}	1.014	1.014	Bhattacharya [6]
Saturated unit weight ρ (ton/m^3)	1.896	1.890	Bhattacharya [6]
$(N_1)_{60cs}$	8.5	7.4	$(N_1)_{60cs} = 46D_r^2$
C_D	41	41	Dash et al. [12]
Residual shear strength of liquefied soil (Lower bound) S_r (kPa)	2.0	1.7	Cubrinovski and Bradley [21]
Scaling factor for *p-y* curves (smooth soil-pile interface) N_s	9.2	9.2	Dash et al. [12]

TABLE II
PILE PROPERTIES AT PROTOTYPE SCALE (AFTER [6])

Parameter	Pile_ID-7	Pile_ID-8	Pile_ID-10
Name of test	SB-04	SB-04	SB-06
Pile outside diameter (*m*)	0.465	0.465	0.465
Length of Pile (*m*)	11.125	11.125	11.125
Thickness (*mm*)	20	20	20
Flexural stiffness (*kN-m*^{2})	48545	48545	48545

TABLE III
VALIDATION OF CENTRIFUGE TESTS

Test Name	Axial Load applied (kN)	Computed buckling load (kN)			Centrifuge result	Numerical method
		a = 0.5%L	a = 1.0%L	a = 2.0%L		
SB-04-Pile_ID-07	1563	1512.0	1152.0	878.0	Failure	Failure
SB-04-Pile_ID-08	2225	1231.0	952.0	773.0	Failure	Failure
SB-06-Pile_ID-10	1875	1105.0	857.0	733.0	Failure	Failure

IV. PARAMETRIC STUDY

Parametric study is conducted to evaluate the influence of soil relative density, geometric imperfection of pile, and pile length above ground level on the buckling capacity of the pile foundation. For all the analysis a base line case of Pile_ID-07 is used (single soil layer of 9 m depth) and quarter sine wave profile is adopted for pile profile with amplitudes of imperfections (*a/L*) as 0.5%, 1.0%, 1.5%, 2.0%, and 2.5%. Geometric imperfection used in the present study is the possible range of imperfection which could occur during the construction of pile foundation [22], [8].

A. Effect of Relative Density of Liquefiable Soil

The residual shear strength of the liquefied soil influences the critical buckling load of the pile foundation [11]. Effect of different relative densities such as D_r = 30%, 35%, 40%, 45% and 50% are considered in this study. Table IV shows the soil properties used. The normalized buckling load (P_{cr}/P_E) with reference to Euler's elastic buckling load for a free standing cantilever pile is plotted in Fig. 6 for various initial geometric imperfection factors. For increasing initial relative density (D_r) of liquefiable soil, the buckling capacity of the pile is found to be higher, due to the higher initial stiffness and residual shear strength of soil at higher D_r. The buckling capacity of the pile foundation varies exponentially with respect to relative density of the liquefiable soil irrespective of the amount of initial geometric imperfection. However, for the geometric imperfection (*a/L*) of 0.5%, the buckling capacity is increased by 84% as D_r changes from 30% to 50%. Also, as expected, the higher is the initial geometric imperfection, the lower is the buckling capacity of the pile.

TABLE IV
SPECIFIC SOIL PROPERTIES USED FOR VARIOUS RELATIVE DENSITIES OF LIQUEFIABLE SOIL

Parameter						Reference
Relative Density of sand D_r (%)	30	35	40	45	50	-
Saturated unit weight ρ (*ton/m*^{3})	1.896	1.896	1.890	1.890	1.938	-
$(N_1)_{60cs}$	4.14	5.64	7.36	9.32	11.50	$(N_1)_{60cs} = 46D_r^2$
C_D	41	41	41	41	41	Dash et al. [22]
Residual shear strength of liquefied soil (Lower bound) S_r (*kPa*)	0.40	1.03	1.70	5.25	9.0	Dash et al. [22]
Scaling factor for *p-y* curves (smooth soil-pile interface) N_s	9.2	9.2	9.2	9.2	9.2	Dash et al. [22]

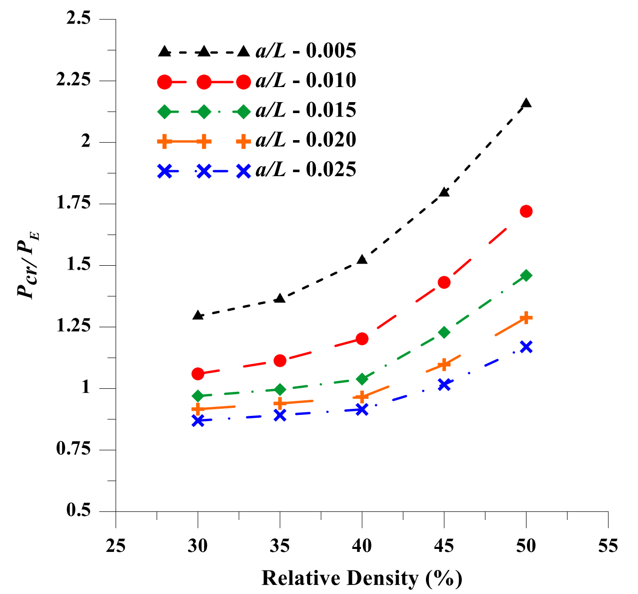


Fig. 6 Effect of relative densities of soil on buckling capacity of pile

B. Effect of Pile Length above Ground Level

The embedded depth (*L*) of the pile in soil is 9 m in the present study. To evaluate the influence of unsupported pile length above the ground (*l*) on its buckling capacity, five cases of are considered where *l* = 0.1*L*, (1 m); 0.18*L*, (2 m); 0.25*L*, (3 m); 0.30*L*, (4 m); and 0.36*L*, (5 m). Variation of normalized buckling load (P_{cr}/P_E) with respect to pile length above the ground (*l*) is plotted in Fig. 7. The figure clearly shows that with increase in unsupported pile length above the ground (*l*), the buckling capacity of the pile decreases exponentially for all types of liquefiable soil deposits due to the increase in the effective length of the pile, as observed by Zhang et al. [11]. As the pile length above the ground level increases from 1 m to 5 m, the buckling capacity decreases by 49%, 49.8%, 50%, 53%, and 56% for the soil deposits of relative density (D_r)

30%, 35%, 40%, 45%, and 50% respectively.

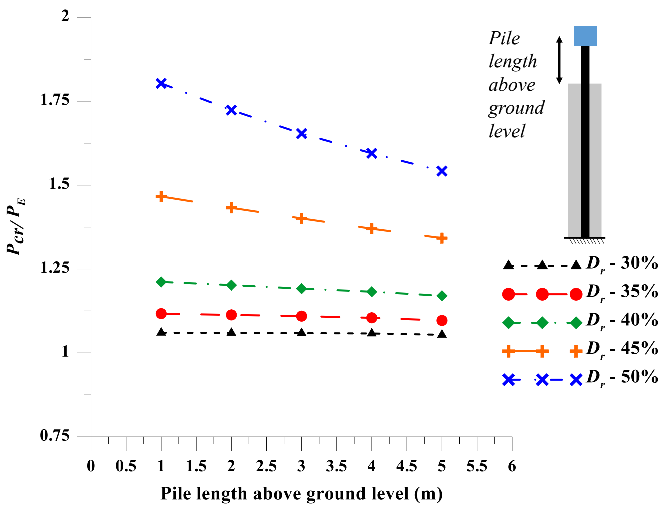


Fig. 7 Influence of pile length above ground level on the buckling capacity of the pile (*a/L*-1.0%)

V. Conclusion

The study comprehends the buckling behavior of pile foundation in liquefiable soil deposit. A BNWF model is developed to estimate critical buckling load of pile foundation. The proposed method is validated against the centrifuge experiment. Parametric study is carried out to understand the effect of relative density of liquefiable soil, pier height, and initial imperfection on the buckling capacity of the pile foundation. Major conclusions are as follows.

- For pile foundation in single layered liquefiable soil, as the relative density of liquefiable soil increases the buckling capacity of the pile foundation also increases. With increase in initial imperfection the buckling capacity decreases.
- Increase in pile height above the ground level decreases the buckling capacity of the pile foundation.
- From the study it is evident that liquefaction model proposed by Dash et al. [12] better captures the buckling behavior of the pile foundation than the conventional model proposed by the API standard.

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