P-Y CURVE TO MODEL LATERAL RESPONSE OF PILE FOUNDATIONS IN LIQUEFIED SOILS

Suresh R Dash[1], Subhamoy Bhattacharya[2], Anthony Blakeborough[3] and Masayuki Hyodo[4]

[1] *Research Scholar, Dept. of Engineering Science, University of Oxford, UK*
[2] *University Lecturer, Dept. of Civil Engineering, University of Bristol, UK*
[3] *Professor, Dept. of Engineering Science, University of Oxford, UK*
[4] *Professor, Graduate School of Science and Engineering, Yamaguchi University, Japan*

Email: suresh.dash@eng.ox.ac.uk, S.Bhattacharya@bristol.ac.uk, tony.blakeborough@eng.ox.ac.uk, hyodo@yamaguchi-u.ac.jp

ABSTRACT :

Dynamic soil-pile interaction is undoubtedly a complex phenomenon. Though this interaction is well established in competent non-liquefied soil, the same for liquefied soil is still an area of active research. One of the versatile soil-pile interaction model used in practice is the Beam on Nonlinear Winkler Foundation (BNWF) model. In the BNWF model, the soil is represented by a set of independent springs lumped at discrete locations along the pile. For earthquake loading, the lateral soil spring, which is generally referred to as a p-y spring, is one of the governing parameters determining the response of piles. Although much research has already been carried out to define the p–y curves for liquefied sand, considerable inconsistency still exist in appropriately defining its shape, salient features including magnitude and most importantly the applicability This paper critically reviews the most commonly used models for the p-y curve for liquefied soils. Some experimental test results are compared with the commonly used models and the inconsistencies in the definition of the p-y curve are highlighted. The effect of p-y curve on lateral pile response to displacement loading is discussed and a more realistic class of p-y curve is proposed.

KEYWORDS: Earthquakes, Liquefaction, Pile foundations, Lateral pile-soil interaction (LPSI), p-y curve

1. INTRODUCTION

Piles are long slender structural members often used as deep foundations to support heavily loaded structures such as; bridges, buildings, jetties, oil platforms, etc. These foundations are generally preferred where the soils at shallow depth are not strong enough to support the structural load. These are often considered as an all-safe solution due to their good performance in extreme loading conditions, for example the offshore environment. However, this is certainly not true if the structures are in a seismically active zone and in loose to medium dense saturated sandy soil. During moderate to strong earthquakes, these soils liquefy and behave like a 'solid suspension' due to the rise in pore water pressure. When the soil liquefies, it loses its strength and stiffness significantly. As a well known fact, the strength and stiffness of the soil around the pile have great influence during analysis and design of pile foundations. Though the quantification of strength and stiffness parameters is well understood for non-liquefied soils, the same for liquefied soils is still unclear and an active area of research.

The predominant seismic hazards like seismic shaking and ground failure impose mainly lateral loads on the pile foundations. Hence, the lateral response of pile foundations in liquefied soils is of major concern for earthquake engineers. In practice, designing of pile foundations is carried out in many different ways by modelling pile-soil interaction (Koo et. el., 2003). One of the widely used models is BNWF (Beam on Non-linear Winkler Foundation) model (Figure 1). BNWF model is extensively used in practice due to its simplicity, mathematical convenience and ability to incorporate most of the nonlinearities in the system. The lateral pile-soil interaction (LPSI) in BNWF model is modelled by nonlinear p-y curves, where 'p' refers to the lateral soil pressure per unit length of pile and the 'y' refers to the lateral deflection (Figure 1).

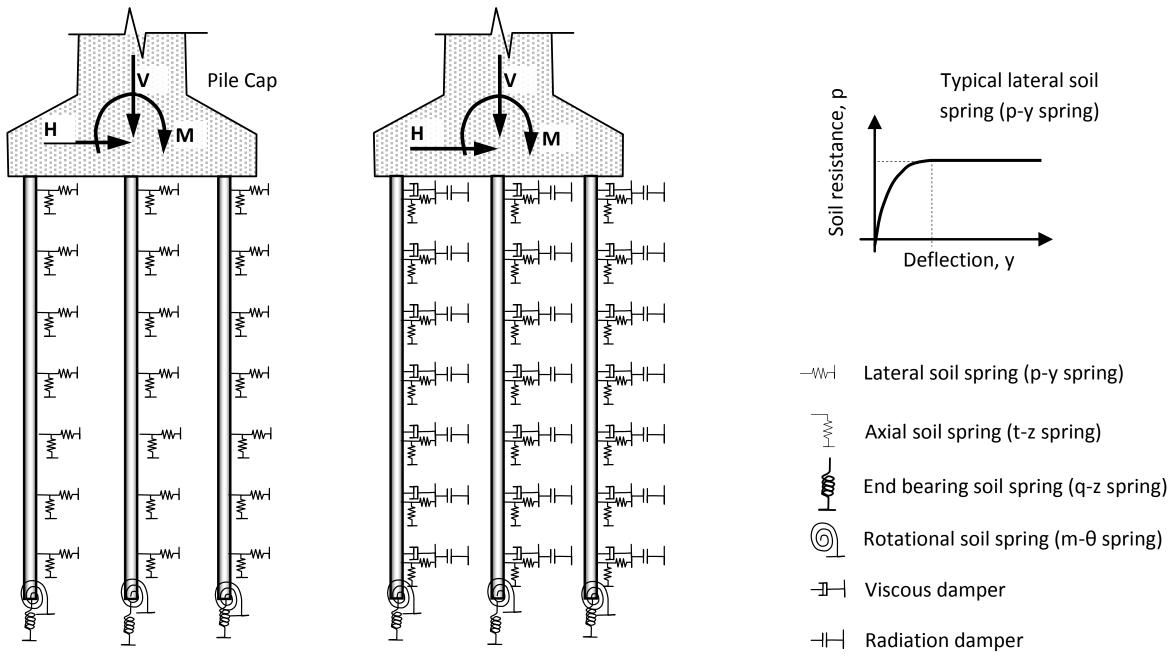


Figure 1 a) Pseudo-static, and b) dynamic BNWF model of Pile-Soil Interaction.

2. CURRENT PRACTICE OF MODELLING P-Y CURVES IN LIQUEFIED SOIL

The p-y curve in a BNWF model for normal soil condition (i.e., non-liquefied soil at this context) is well understood and used in the practice with confidence from last 30 years (API 2000, JRA 2002). However, these soil parameters alter for the liquefied soil. The soil changes its state from solid to fluid during liquefaction and the interaction between pile and liquefied soil hence, becomes much complicated. Traditionally, engineers uses simple solutions to tackle this complex situation and suggests the use of reduction factor in the soil resistance “p”. This reduction factor depends on various field conditions such as, degree of liquefaction, depth of liquefied soil versus depth of non-liquefied crust, etc. Four most commonly used p-y curve model in liquefied soil is summarized in Table 1. The reliability of these methods for representing lateral resistance of liquefied soil may simply be limited by the fact that they are approximations of a rather complex phenomenon that is poorly understood.

3. OBSERVATIONS FROM VARIOUS EXPERIMENTAL STUDIES

Looking at a different prospective to a spectrum of experimental test results such as, the laboratory tests by Yasuda et al. (1999), the centrifuge tests by Wilson et al. (2000), the 1g shaking test by Takahashi et al. (2002) and the full scale field test by Rollins et al., (2005) suggests a radically different shape and magnitude of the p-y curve in liquefied soil than that is currently been used in practice (Table 2).

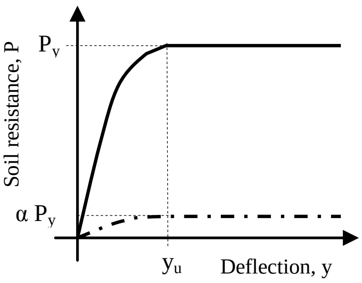
In practice, the p-y curve is characterized by the index properties of the soil and the pile dimension. The index properties of the soil are basically a representation of its stress-strain behaviour. The shape of the p-y curve is geometrically similar to the stress strain curve of the soil material. Typically the stress-strain curve of non-liquefied soil looks like a convex curve with initial stiff slope. However, in liquefied soil the initial stiffness is significantly low up to a certain deflection value, beyond which the soil attains very high stiffness (Yasuda et al, 1998, 1999). The shape of the p-y curve in liquefied soil is nearly concave, in contrast with the convex shape for non-liquefied soil. Hence, during the process of liquefaction the p-y curve changes from a convex shape to a concave shape whose changing sequence and pattern is still an area of current research.

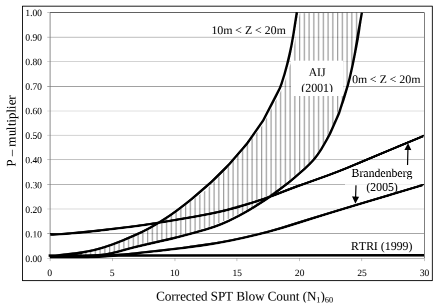
Table 1: p-y curve models used in practice for liquefied soils

<table>
<tr><th>Sl.</th><th>Method</th></tr>
<tr><td>1.</td><td>

p-multiplier (α)

This method assumes a strength degradation factor according to the SPT blow count $(N_1)_{60}$. This is the most common method being used in practice to model p-y spring for liquefied soil in a BNWF model.

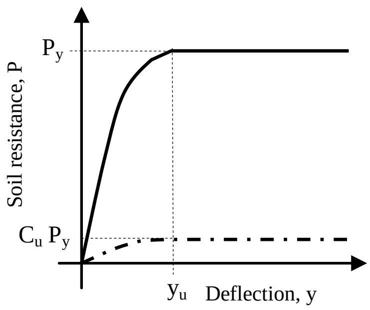


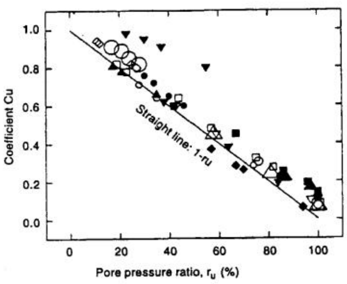


</td></tr>
<tr><td>2.</td><td>

C_u-factor

Instead of a p-multiplier for a fully liquefied soil, this method assumes a strength degradation factor according to the degree of liquefaction. Form centrifuge model tests, Liu and Dobry (1995) have derived the C_u as a linearly varying function with excess pore pressure ratio (i.e., $C_u = 1\text{-}r_u$).

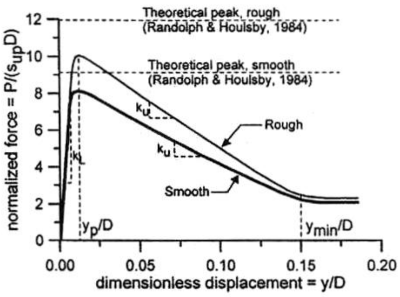


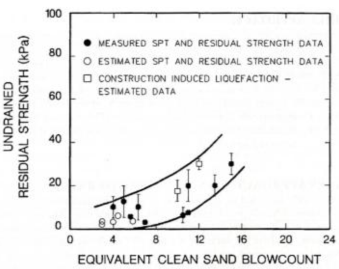


</td></tr>
<tr><td>3.</td><td>

Residual strength method

Many researchers suggested that the sandy soil at full liquefaction behaves like soft clay. Goh and O'Rourke (1999) proposed a p-y curve with the shape similar to the same for soft clay for cyclic loading, as proposed by Matlock (1970). However, the maximum lateral strength is replaced with the undrained residual shear strength of the liquefied sand.





</td></tr>
<tr><td>4.</td><td>

Zero strength method

The residual shear strength of liquefied soil depends on many parameters and also complicated to evaluate. Ideally, as the effective stress becomes zero at full liquefaction, the shear strength of soil should be zero considering it as liquid. However, studies by Castro (1969) showed that even after liquefaction, many sands do retain a significant amount of shear resistance. In contrast, Eurocode 8 (part 5, Sec. 5.4.2 of EC8, 1998) advises to ignore the side resistance of soil layers that are susceptible to liquefaction or to substantial strength degradation while analyzing pile foundations. Many researchers, hence ignores the lateral strength of soil while doing pile analysis and design in liquefiable soils. See for example Bhattacharya et al (2005).

</td></tr>
</table>

The shape of stress-strain curve of the liquefied sand as obtained from the tri-axial test by Yasuda et al (1999), is similar to the results of the centrifuge test carried out by Wilson et al, (2000) and the field test by Rollins et al (2005). Though the actual p-y curve for liquefied soil is different as compared to what is followed in the practice, not much of research has yet been done to fully characterize it. Ashour and Norris (2003) proposed the strain wedge model to assess the p-y curve for piles in saturated sands as liquefaction develops by using the stress-strain model of the pile and soil. However, there still lies a considerable amount of inconsistency in the appropriate definition of its shape, magnitude, failure envelope and most importantly its applicability (Figure 2). It must be mentioned that the correctness of the analytical solutions using a BNWF model depends on the appropriate p-y curve being used.

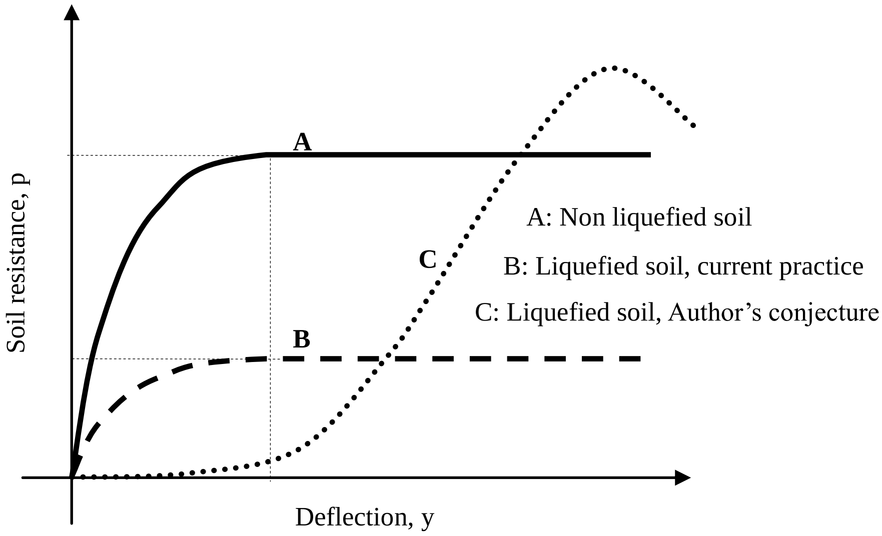


Figure 2 p-y curve for liquefied soil

4. EFFECT OF STRAIN RATE ON STRESS-STRAIN BEHAVIOUR OF LIQUEFIED SOIL

One of the important factors affecting the p-y curve, as observed in some model tests (see Table 2), is the rate of displacement loading. According to authors' knowledge, research is very limited on the study of the rate effect on the stress-strain behaviour of liquefied sand. Studying stress-strain behaviour provides a good understanding of the basic index properties which can directly be related to the p-y behaviour of pile-soil interaction. This section presets the results obtained from a study conducted in an un-drained triaxial test setup. The Toyoura sand, having mean diameter of (D_{50}) 0.2mm and maximum and minimum void ratios of (e_{max}) 0.977 and (e_{min}) 0.597, respectively, was used in this test to investigate the effect of strain rate in liquefied sand at a constant confining stress of 50 kPa. The soil sample is first subjected to a cyclic loading causing the soil to liquefy followed by a monotonic loading at different strain rates. The schedule of tests is presented in table 3.

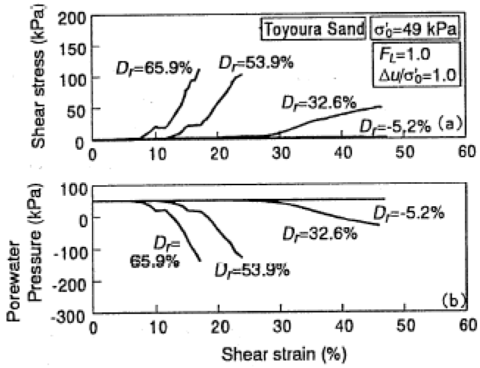
The test results (Figure 3) were very promising in terms of the pattern of the stress-strain behaviour as compared with the other experimental studies in liquefied sand. The shear strain in the liquefied soil increased with very low or negligible shear stress up to ~ 5-10% strain, after which the resistance increased rapidly. This observation is very similar to the same observed by Yasuda et al. (1998, 1999). Another significant observation is that the stiffness of the liquefied soil in its hardening stage is not affected by different amount of strain rate. This suggests that the p-y curve can be modelled as a pseudo-static curve and the effect of strain rate (i.e., dynamic effect), which becomes significant during dynamic analysis, can be related to the damper properties in the BNWF model.

Table 2: Observations from experimental studies on liquefied sand

<table>
<tr><th>Sl.</th><th>Experimental study and its key findings</th></tr>
<tr><td>1.</td><td>

Laboratory test

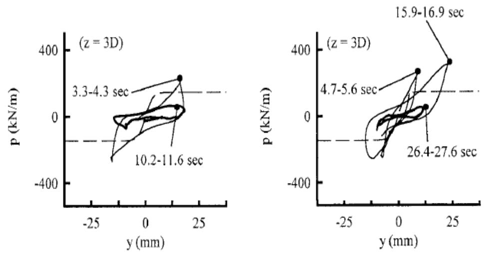
The stress-strain relationship of sand after liquefaction is studied by many laboratory tests by Yasuda et. al., (1999). Here, Toyoura sand is used as the sand sample, which is subjected to a monotonic loading after subjected to cyclic loading causing the soil to liquefy. In contrast with the present practice, the stress-stain behaviour obtained is quite different in shape. The stiffness and initial take off strain of the stress-strain curve is strongly dependent on its relative density. The similar kind of behaviour has also been observed in torsional shear tests by Yasuda et al (1998) for several varieties of sands.



</td></tr>
<tr><td>2.</td><td>

Model test in centrifuge

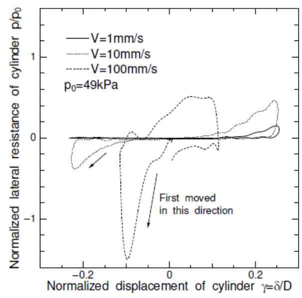
A series of centrifuge tests were carried out by Wilson et al. (2000) to study the dynamic response of pile foundations in liquefying sand during seismic loading. The p-y curve obtained from the experimental study shows some interesting observations with respect to the shape and magnitude of the p-y curve currently being used in practice. Even at full liquefaction state, there still remained significant resistance in soil against lateral displacement of pile. The shape of the p-y curve is similar to the stress-strain curve as obtained in the laboratory tests.



</td></tr>
<tr><td>3.</td><td>

1g model test

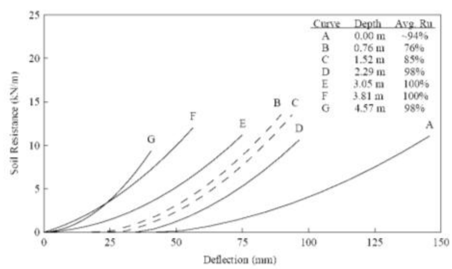
Takahashi et al. (2002) have carried out a set of simple 1g tests in a watertight sand box by pulling an instrumented pipe in it to study the rate effect on lateral resistance of liquefied soil. The results were very promising to see a great dependency of the lateral resistance of pile with respect to the rate of lateral displacement



</td></tr>
<tr><td>4.</td><td>

Full scale field test

Rollins et al. (2005) carried out full scale testing of a single pile and group piles subjected to blast induced liquefaction to study the pile-soil-pile interaction effect in liquefied soil. The estimated p-y curves from the test results have also shown similar kind of concave pattern at full liquefaction. Instead of the fact that at full liquefaction the effective stress is zero and does not vary with depth, still the dependency of depth over the p-y curve stiffness was observed.



</td></tr>
</table>

Table 3 Test conditions

	γ_d (g/cm^3)	E	Dr (%)	$\sigma_d/2\sigma'_c$	S.R. (%/min)
Test.1	1.463	0.805	49.8	0.143	0.1
Test.2	1.462	0.805	49.6	0.147	0.5
Test.3	1.455	0.814	47.0	0.145	1.0
Test.4	1.456	0.813	47.3	0.146	5.0
Test.5	1.459	0.809	48.5	0.151	10.0
Note: σ'_c: Confining stress, γ_d: Dry Density, Dr: Relative Density, $\sigma_d/2\sigma'_c$: Cyclic stress ratio, S.R.: Strain ratio					

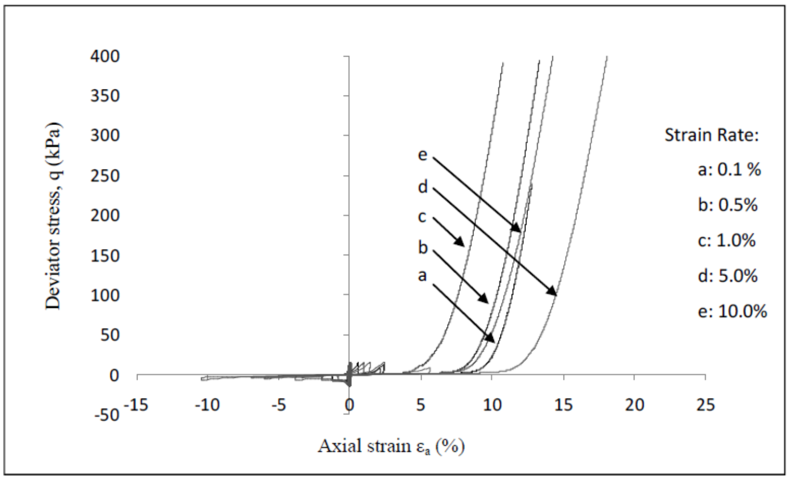


Figure 3 Stress-strain behaviour of liquefied soil obtained from tri-axial cyclic shear test of saturated sandy soil in undrained condition.

5. CHARACTERIZATION OF P-Y CURVE FOR LIQUEFIED SOIL

In line with the above discussion, the pseudo static p-y curve for liquefied soil can be hypothesized as an S-curve as illustrated in Figure 4a. The pore water pressure ratio (r_u) in soil at any particular time may be used to characterize the p-y curve while the sandy soil transit from non-liquefied state to fully liquefied state. For a liquefied soil, the p-y curve can be simplified and represented as shown in Figure 4b. The parameters of interest for this are; a) the initial takeoff ground displacement (y_t), b) stiffness of liquefied soil (K_l), and c) Maximum load that the soil can take (p_u) or d) the maximum mobilizing displacement (y_u).

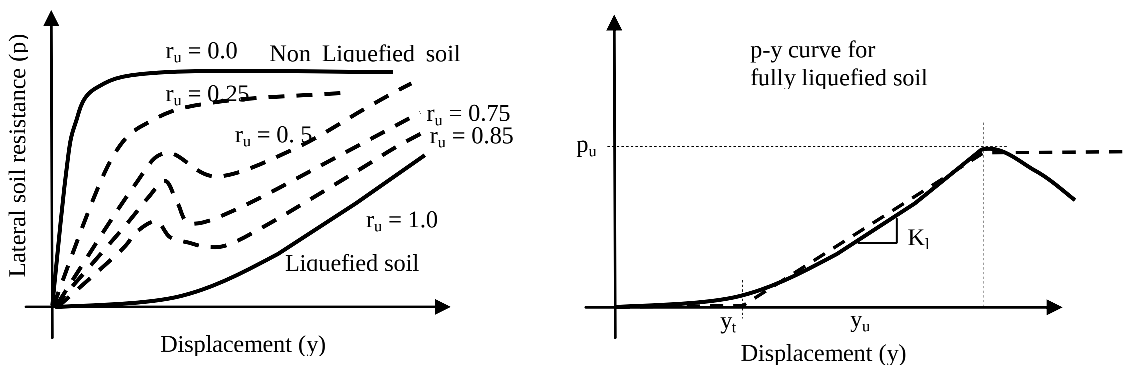


Figure 4 (a) p-y curve for saturated sandy soil during the process of liquefaction. (b) Simplified p-y curve model for liquefied soil. (Authors' conjecture)

6. EFFECT OF P-Y CURVE IN SOIL-PILE INTERACTION

The main parameters of a load-displacement (p-y curve) relationship are stiffness and strength. The stiffness of p-y curve is the resistance of soil to unit pile deformation. During transient vibration, the stiffness of soil plays an important role. When the differential soil-pile movement is small (i.e., the soil is not pushed to its full capacity), the resistance on pile depends on the initial stiffness of the soil and the value of deflection (Figure 5a). In contrast, the strength of soil is an important parameter while dealing with high amplitude soil-pile interaction. When the differential soil-pile movement is large, the resistance offered by soil over pile is governed by the ultimate strength of the soil (Figure 5a).

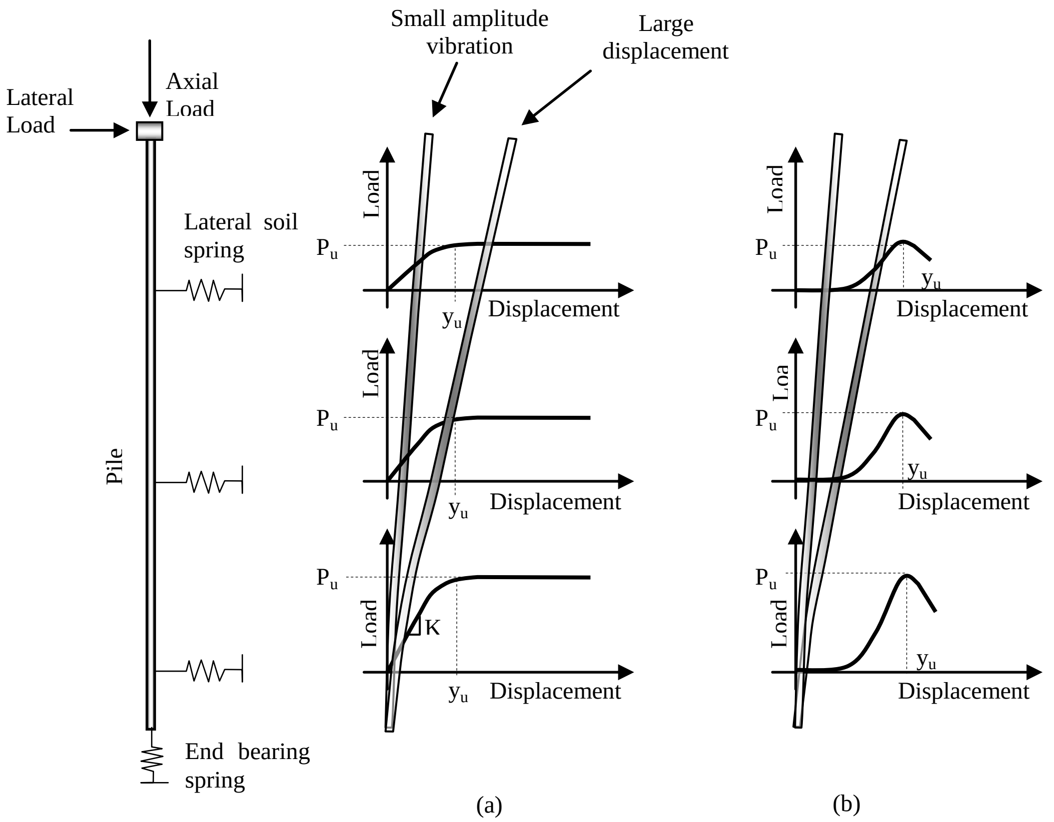


Figure 5 Soil-pile interaction, (a) for small amplitude soil-pile movement, (b) large amplitude soil-pile movement.

However, if the shape of the p-y curve is chosen as S-curve as shown in figure 5b, the pile response as described above for small and large amplitude vibrations will certainly change. The lack of initial stiffness and strength of the liquefied soil will increase the p-delta effect in the small amplitude vibration, and will promote buckling mode of failure of piles. However, the advantage achieved by using the later model (Figure 5b) is the higher strength and stiffness at large differential pile-soil movement, which may prevent a complete collapse of a structure.

7. CONCLUSIONS

The main conclusions that can be drawn from the above study are:

1. There still lies significant inconsistency in defining appropriate p-y curve for liquefied soil
2. The experimental results shows that the shape of p-y curve shall look like a concave curve, in

contrast with the convex shaped p-y curve that is used practice.

3. Soil deformation increases with very low resistance up to several diameter of pile. After the takeoff point the resistance increases rapidly.
4. The stiffness of liquefied soil does not depend on the rate of loading. Hence, the pseudo-static p-y curve and damper properties used in a dynamic BNWF model can be defined as uncoupled.
5. The shape, strength and stiffness parameters of the p-y curve do have great impact on the response of piles subjected to lateral seismic loading. Hence, there is a need for further research to characterize realistic p-y curves in liquefied soils.

REFERENCES

1) API (2000). (WSD), Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms-Working Stress Design." Version 21st.
2) Ashour, M. and Norris, G., (2003). "Lateral loaded pile response in liquefied soil". Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol 129, No 6, 404-414.
3) Bhattacharya, S., (2003). "Pile instability during earthquake liquefaction," PhD thesis, University of Cambridge (U.K). September 2003.
4) Brandenberg, S. J. (2005). Behavior of pile foundations in liquefied and laterally spreading ground," Phd thesis, Univ. of California at Davis, Davis, California.
5) Eurocode (1998). "Design provisions for earthquake resistance of structures- foundations, retaining structures and geotechnical aspects." En 1998-5:2004 edition.
6) Goh, S. and O'Rourke, T. D. "Limit State Model for Soil–Pile Interaction during Lateral Spread." Proc. of 7th US Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures against Soil Liquefaction, Seattle, pp. 237–260, 1999.
7) JRA (2002): Specifications for highway bridges. Part V Seismic Design, Japan Road Association, Japan, (in English).
8) Koo, K. K., Chau, K. T., Yang, X., Lam S. S. and Wong Y. L. (2003), "Soil–pile–structure interaction under SH wave excitation", Earthquake Engineering and Structural Dynamics, Vol. 32, 395–415.
9) Liu, L and Dobry, R (1995): "Effect of liquefaction on lateral response of piles by centrifuge model tests", NCEER report to FHWA. NCEER Bulletin, January 1995, Vol 9, No. 1.
10) Rollins, K. M., Gerber, T. M., Dusty L. J., and Ashford, S. A., (2005) "Lateral resistance of a full-scale pile group in liquefied sand" Journal of Geotechnical and Geoenvironmental Engineering, Vol. 131, No. 1, January 2005 , ASCE.
11) S. Bhattacharya, M.D. Bolton and S.P.G.Madabhushi (2005): "A reconsideration of the safety of the existing piled bridge foundations in liquefiable soils", Soils and Foundations (Japan) Vol. 45, No 4. (August 2005 Issue).
12) Takahashi, A., Kuwano, J., Arai, Y. and Yano, A., (2002), "Proceedings of the international conference on physical modelling in geotechnics, ICPMG-02, St Johns, Newfoundland, Canada, July 2002.
13) Wilson, D.W., Boulanger, R.W. and Kutter, B.L (2000): "Observed seismic lateral resistance of liquefying sand", Journal of Geotechnical Engineering, ASCE, Vol 126, No 10, pp 898-906.
14) Yasuda, S., Terauchi, T., Morimoto, H., Erken, A. and Yoshida, N., (1998), "Post liquefaction behaviour of several sands", 11th European Conference on Earthquake Engineering, Balkema, Rotterdam, ISBN 90 5410 982 3.
15) Yasuda, S., Yoshida, N., Kiku, H., Adachi, K. and Gose, S. (1999): "A simplified method to evaluate liquefaction-induced deformation", Earthquake Geotechnical Engineering, Balkema, Rotterdam, ISBN 90 5809 1163.