AN APPROACH TO STUDY DYNAMIC STABILITY OF PILE-SUPPORTED STRUCTURES IN LIQUEFIABLE SOILS

S.Bhattacharya[1)], N.A.Alexander[2)] S.Adhikari[3)]

1) Lecturer in Dynamics, Earthquake Engineering Research Centre, University of Bristol (U.K)
2) Senior Lecturer in Structural Engineering, Earthquake Engineering Research Centre, University of Bristol (U.K)
3) Professor, Department of Aerospace Engineering, Swansea University (U.K)
S.Bhattacharya@bristol.ac.uk, Nick.Alexander@bristol.ac.uk, S.Adhikari@swansea.ac.uk

Abstract: Collapse and/or severe damage of pile-supported structures are still observed after strong earthquakes despite the fact that a large factor of safety against axial capacity and bending due to lateral loads is employed in their design. A recent investigation has conclusively shown that a pile may become laterally unsupported in the liquefiable zone during strong shaking and due to the axial load acting on it. This led to another failure mechanism: Buckling instability. Also, during seismic liquefaction, the axial load on the pile in the liquefied zone increases due to the loss of shaft resistance. Due to this extra axial load, the stiffness of the pile-soil system reduces and so does the first natural vibration frequencies. This paper aims to show that at the point of instability the fundamental vibration mode and buckling mode shapes are identical. A method is shown to analyse dynamic stability of piles in liquefiable soils.

1. INTRODUCTION

Collapse and/or severe damage of pile-supported structures are still observed after strong earthquakes despite the fact that a large factor of safety against axial capacity and bending due to lateral loads is employed in their design. All current design codes employ a large margin of safety against the hinge formation (using partial factors on load and material stress), yet occurrences of pile failure due to liquefaction are abundant. Bhattacharya (2003) has shown that in some cases, the overall factor against plastic hinging at collapse has been of the order of 4 to 8. This is strong evidence that there are perhaps other mechanisms governing these failures, which the code does not consider. A critical review of the current theories of pile failure can be found in Bhattacharya and Madabhushi (2008).

In Japan, following some earthquakes, piles were excavated or extracted from the subsoil, borehole cameras were used to take photographs, and pile integrity tests were carried out. These studies hinted the location of the cracks and damage patterns for the piles. The main observations in terms of hinge formation or cracks in the pile are:

1. Cracks were observed near the bottom and top boundaries between liquefied and non-liquefied layers. Often cracks were observed at the pile head.
2. Plastic hinges also formed not only at the boundaries of the liquefiable and non-liquefiable layers but also at various depths.
3. Horizontal displacements of the piles were measured in some cases of pile failure. In some cases, the displacement of the piles was found to agree with the horizontal displacement of the ground.
4. There are few cases where plastic hinges formed at the middle of the liquefied layer. It has been reported, Yoshida et al (2005) that this damage was observed when the structure was very close to the quay wall which moved towards the sea significantly.

The above observation indicates that the stresses in the pile during the earthquakes exceeded the yield stress of the material of the pile. As a result, design of pile foundation in seismically liquefiable areas still remains a constant source of attention to the earthquake geotechnical engineering community.

2. THEORIES OF PILE FAILURE

2.1 Bending mechanism due to kinematic loads on the pile and inertia of the superstructure

The current understanding of pile failure as hypothesised by some codes of practice is as follows: In an earthquake if loose sands are saturated, they lose strength as excess pore water pressure is generated and the soil tends to liquefy. This means that if the soil is on a slope it will flow down-slope which is often termed as lateral spreading. Lateral spreading is a term used to represent the permanent lateral ground displacement after an earthquake. Up to now it has been assumed that the failure of these buildings were caused by the lateral pressure of the flow of the liquefied sand and any non-liquefied stabilised crust resting on the top of the liquefied soil. This mechanism is therefore based on flexure of the pile causing a bending failure. This mechanism forms the basis for the design guidelines developed by JRA (1996, 2002). Many researchers recently suggested that in the presence of non-liquefied crust, the lateral loading created by liquefied ground can be ignored as the lateral passive earth pressures created in the non-liquefied crust dominate the lateral loading generated on the pile foundation.

The movement of the superstructure i.e. inertia force can also induce bending moments in the pile. However, Ishihara (1997) notes that during an earthquake soil liquefaction starts at approximately the instant of peak acceleration. He argues that since the seismic motion has already crossed its peak, subsequent shaking will be less intense, so that the lateral force applied by the superstructure will not be significant. Therefore, the effects of inertia of the superstructure on the pile stresses are considered separately and are not combined with the lateral spreading effects.

2.2 Buckling mechanism arising due to unsupported length of the pile in liquefiable zone

A recent investigation by Bhattacharya et al (2004), Bhattacharya et al (2005) conclusively showed that a pile becomes laterally unsupported in the liquefiable zone during strong shaking which led to another failure mechanism. The soil around the pile liquefies and it loses much of its stiffness and strength, so the piles now act as unsupported long slender columns, and simply buckles under the action of the vertical superstructure (building) loads. The stress in the pile section will initially be within the elastic range, and the buckling length will be the entire length in the liquefied soil. Lateral loading, due to slope movement, inertia or out-of-line straightness, will increase lateral deflections, which in turn can cause plastic hinge to form, reducing the buckling load, and promoting more rapid collapse. Therefore, this hypothesis is based on a buckling mechanism. This has later been verified by other researchers; see for example Lin et al (2005), Kimura and Tokimatsu (2007), Shanker et al (2007).

Though buckling mechanism can classify pile failures (good performance or otherwise), the location of hinge formation/ cracks in the piles cannot be explained by buckling instability theory. Criticisms of buckling mechanism can be found in Yoshida et al (2005). This led to the search of any other mechanism of failure.

3.0 DYNAMIC STABILITY OF PILED FOUNDATIONS (UNIFIED BUCKLING MECHANISM AND RESONANCE)

Structurally, buckling of a slender column can be viewed as a complete loss of lateral stiffness to resist deformation. It is commonly known as an instability phenomenon. During liquefaction, if a pile buckles - it can be concluded that the lateral stiffness of the pile is lost. From a dynamics point of view as the applied axial load approaches the buckling load it can also be observed that the fundamental natural frequency of the system drops to zero, Thompson and Hunt (1984). Essentially, at the point where the natural frequency drops to zero, the inertial actions on the system no longer contribute. Thus, the system's dynamical equations of motion degenerate into a statics stability problem.

During seismic liquefaction, the axial load on the pile in the liquefied zone increases due to the loss of shaft resistance. Due to this extra axial load, the stiffness of the pile-soil system reduces and so does the vibration frequencies. At the point of instability the fundamental vibration mode and buckling mode shapes are identical. Thus, as the soil transforms from solid to a fluid-like material i.e. from partial-liquefaction stage to full-liquefaction stage, the modal frequencies and shapes of the pile change.

Considering the first natural frequency of the pile-soil-superstructure system, it is suggested that the "other mechanism" may probably be the two effects arising from the removal of the lateral support the soil offers to the pile while in liquefied state. They are

(a) Increase in axial load in the pile in the potentially unsupported zone due to loss of shaft resistance;

(b) Dynamics of pile-supported structure due to frequency dependent force arising from the shaking of the bedrock and the surrounding soil than can cause dynamic amplification of pile head displacements leading to resonance type failure.

Essentially, under service conditions (no earthquake and no-liquefaction) the first natural frequency of a structure or the fundamental time period can be estimated without considering the effect of the piles. Typically, a building will have a fundamental time period of about 0.1 times the number of storey. For a 5 storey building, the time period is 0.5sec an the first natural frequency is therefore 2Hz. However when the soil starts to liquefy, the piles become an integral part of the structure and take part in the vibration. As a result, the time period alters significantly and cannot be ignored in analysis/design. In most cases, the frequency will decrease. This paper is therefore aimed at explicitly quantifying the first natural frequency due to the effects of (1) Axial force, (2) Dynamic excitation and (3) Reduction of the lateral support due to liquefaction. An attempt has been made to develop a simplified procedure.

3.1 Winkler models for liquefied soil

BNWF [Beam on Non-Linear Winkler Foundation] or "p-y" method is commonly used in practice for analyzing piles. In "p-y" method, the soil is modelled as non-linear springs where 'p' refers to the lateral soil pressure per unit length of pile and the 'y' refers to the lateral deflection. Figure 1 shows a particular p-y model for non-liquefied soil and its corresponding liquefied condition based on an empirical method. The reduction of strength is carried out using p-multiplier and typical values of this multiplier can be found in AIJ (2001), RTRI (1999), Liu and Dobry (1995). It may be noted that the initial stiffness (i.e. the initial slope of p-y curve) degrades when the soil transforms from being solid to fluid. A detailed discussion on the shape of p-y curves for liquefied soil can be found in Dash et al (2008).

However, analysis of the full-scale tests such as Rollins et al (2005), centrifuge tests such as Bhattacharya et al (2005), laboratory tests on liquefied soil by Yasuda et al (1998), 1-g pipe pulling tests, Takahashi et al (2002) suggests that the shape of the “p-y” curve for liquefied soil should look like a S-curve. Figure 2(a) shows the change of “p-y” curves when the soil is transformed from being solid to fluid-like medium. Figure 2(b) shows the shape following Dash et al (2008).

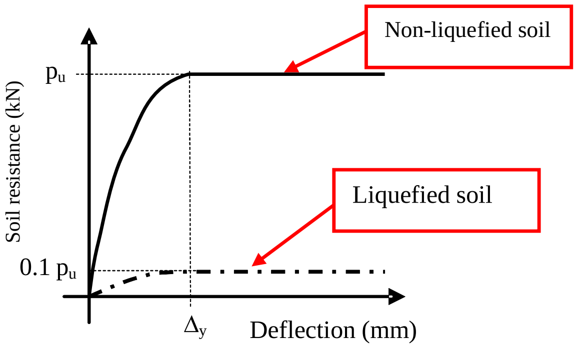

Figure 1: p-y curve for non-liquefied soil and liquefied soil using p-multiplier

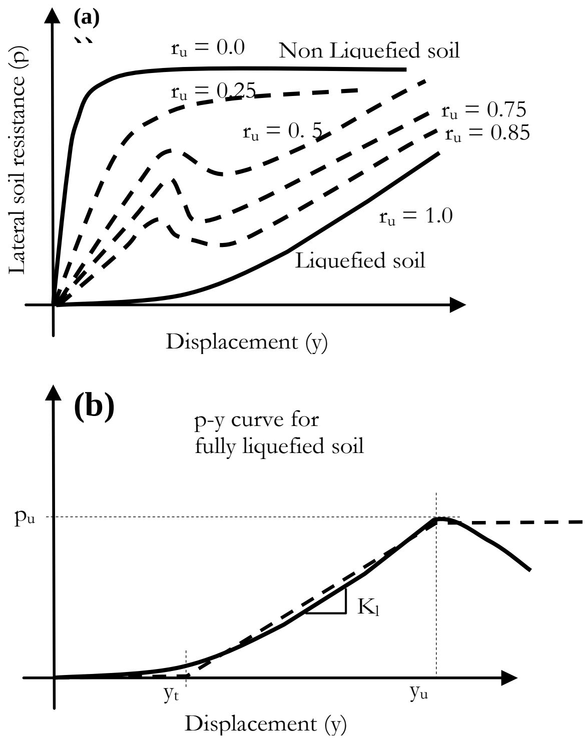

Figure 2: (a): p-y curve for saturated sandy soil during the process of liquefaction; (b) Simplified p-y curve for liquefied soil, Dash et al (2008).

3.1 Vibration of piles in liquefied soil

A pile passes through different stages of loading during an earthquake due to the transformation of the supporting soil from being a solid to a fluid. In one stage, when the soil has not fully liquefied, the transverse bending is expected to govern the internal stresses within the pile. As the liquefaction progresses, the coupled buckling (due to the unsupported length of the pile) and frequency-dependent resonance force would govern the internal stresses in the pile. The key physical aspect that the authors aim to emphasize is that the motion of the pile (and consequently the internal stresses leading to the failure) is a coupled phenomenon. This coupling is, in general, nonlinear and it is not straightforward to exactly distinguish the contributions of the different mechanisms towards an observed failure. It is however certainly possible that one mechanism may dominate over the others at a certain point of time during the period of earthquake motion and till the dissipation of excess pore water pressure.

A coupled dynamical analysis combining (a) transverse bending, (b) dynamic buckling and (c) resonance motion can be carried out for a comprehensive understanding of the failure mechanism of piles during an earthquake. The purpose of this paper is therefore to understand the vibrational characteristics of the piled foundation at full liquefaction. This has design implications as it is necessary to predict the lateral and vertical dynamic loads in the pile at full liquefaction.

4. METHODOLOGY OF ANALYSIS FOR DYNAMIC STABILITY – ENERGY METHOD

In this section of the paper, a generalized method is adopted to analyse the dynamic stability of piled foundation in liquefiable soils.

4.1 Potential Energy

The potential Energy U of this system is composed of three terms (i) the flexural strain energy of the pile group, (ii) the loss of stiffness energy due to compressive loading of the pile group and (iii) the spring stiffness energy of the soil (c.f. Winkler) springs. The soil spring stiffnesses vary linear with depth; where k_w is the soil spring stiffness at the toe of the pile, i.e. $z = L$.

$$U = \tfrac{1}{2} E I \int_0^L y''^2 dz - \tfrac{1}{2} N \int_0^L y'^2 dz + \tfrac{1}{2} k_w \int_0^L \tfrac{z}{L} y^2 dz \qquad (1)$$

By employing a Rayleigh-Ritz spatial-temporal solution composed of a sum of a vector of orthogonal functions ϕ, equation (2), the potential energy can be re-expressed in quadratic form as (3) and (4). The non-dimensional stiffness matrices are (i) $\mathbf{K}_p$ for the pile flexural stiffness, (ii) $\mathbf{K}_s$ for the soil springs and (iii) $\mathbf{K}_b$ for the loss of stiffness due to the axial load N. $x \in \mathbb{R}^n$ is vector generalised coordinates. Two non-dimensional system parameters are introduced axial load factor ν and soil to pile stiffness ratio factor η ,see (5).

$$y = L\mathbf{x}(t)^T \phi(\xi), \qquad z = \xi L \tag{2}$$

$$U = \frac{1}{2}\frac{EI}{L}\mathbf{x}^T\left(\mathbf{K}_p + \eta\mathbf{K}_s - \nu\mathbf{K}_b\right)\mathbf{x} \tag{3}$$

$$\mathbf{K}_p = \int_0^1 \phi''\phi''^T d\xi, \quad \mathbf{K}_s = \int_0^1 \xi\phi\phi^T d\xi, \quad \mathbf{K}_b = \int_0^1 \phi'\phi'^T d\xi \tag{4}$$

$$\nu = \frac{NL^2}{EI}, \qquad \eta = \frac{k_w L^4}{EI} \tag{5}$$

Any orthogonal series that satisfies the boundary conditions can be employed; but the challenge is to find this special orthogonal series. First, let us consider employing an $n+q$ term Legendre polynomial series, where P_i is the i^{th} order Legendre polynomial. The pile displacement y can be defined (7) in term of vector of shape functions ϕ_1 and ϕ_2 can be defined as (6). Essentially the q generalised coordinates $\mathbf{x}_q$ will be constrained by the natural boundary condition leaving n unconstrained generalised coordinates $\mathbf{x}$.

$$\phi_1^T = \left[P_0, P_1, \cdots P_{q-1}\right], \ \phi_2^T = \left[P_q, P_{q+1}, \cdots P_{q+n-1}\right] \tag{6}$$

$$y = L\left(\phi_1^T\mathbf{x}_q + \phi_2^T\mathbf{x}\right) = L\mathbf{x}^T\phi \tag{7}$$

The corresponding boundary condition of a pile [i.e. fixed-free (cantilever pile) etc] can be imposed.

4.1 Linear Buckling Analysis

By applying Euler-Lagrange equations to (3) we obtain an eigenvalue problem given by equation 8. From the smallest eigenvalue ν_1, the first mode and critical buckling load N_1 can be obtained, see equation 9.

$$\left(\mathbf{K}_p + \eta\mathbf{K}_s\right)\mathbf{x} = \nu\mathbf{K}_b\mathbf{x} \tag{8}$$

$$N_1 = \nu_1 \frac{EI}{L^2} \tag{9}$$

By employing the corresponding boundary conditions of the pile, appropriate orthogonal series can be obtained. This is then employed with computational algebra package to obtain matrices $\mathbf{K}_p$, $\mathbf{K}_s$ and $\mathbf{K}_b$.

4.2 Kinetic Energy

The kinetic energy of this system can be stated as equation 10; where m_b is the mass of the building, m_s is half the mass per unit depth of the soil stack, and m_p is the mass per unit length of the pile.

$$T = \frac{1}{2}m_b\dot{y}(0)^2 + \frac{1}{2}\left(m_p + m_s\right)\int_0^L \dot{y}^2 dz \tag{10}$$

This can be expressed in the quadratic form (11) where $\mathbf{M}$ is the system non-dimensional mass matrix which is expressed in terms of pile and soil to building mass ratio α.

$$T = \frac{1}{2}m_b L^2 \dot{\boldsymbol{x}}^T \mathrm{M}\dot{\boldsymbol{x}} \tag{11}$$

$$\mathbf{M} = \phi(0)\phi(0)^T + \alpha\int_0^1 \phi\phi^T d\xi \tag{12}$$

$$\alpha = \frac{\left(m_p + m_s\right)L}{m_b} \tag{13}$$

4.3 Equation of free vibration

The dimensionless Euler-Lagrange equations of free vibrations are given by (15); where time is scaled by the introduction of τ , see (14).

$$\tau = st, \ s^2 = \frac{EI}{m_b L^3} \tag{14}$$

$$\mathbf{M}\ddot{\mathbf{x}} + \left(\mathbf{K}_p + \eta\mathbf{K}_s - \nu\mathbf{K}_b\right)\mathbf{x} = \mathbf{0} \tag{15}$$

$$\left(\mathbf{K}_p + \eta\mathbf{K}_s - \nu\mathbf{K}_b\right)\mathbf{x} = \lambda\mathbf{M}\mathbf{x} \tag{16}$$

The fundamental natural frequency, of the pile/soil/structure system, ω_1 is obtained by solving the generalised eigenvalue problem (16) and determining its smallest eigenvalue λ_1.

5. AN EXAMPLE CASE STUDY

In this section an example problem shown in Figure 3 is taken. The pile is assumed to be fixed at some depth below the liquefied layer. In other words, the pile extends quite a considerable depth in non-liquefiable hard layer below the liquefiable layer. The pile head is free to translate but fixed against rotation. The boundary condition is shown by equation 17.

$$y(1) = y'(1) = y'(0) = 0 \tag{17}$$

The buckling load is calculated by solving the generalised eigenvalue problem (Equation 8) for various η, the smallest eigenvalue ν_1 is found and Figure 4 can be displayed. It may be noted that the graph tends to the Eulerian beam results as $\eta \to 0$.

Figure 5 displays a contour plot of λ_1 vs. the soil/pile stiffness ratio η and axial load factor ν. This is obtained for a fixed value of mass ratio α.

$$\omega_1^2 = s^2 \lambda_1 \tag{18}$$

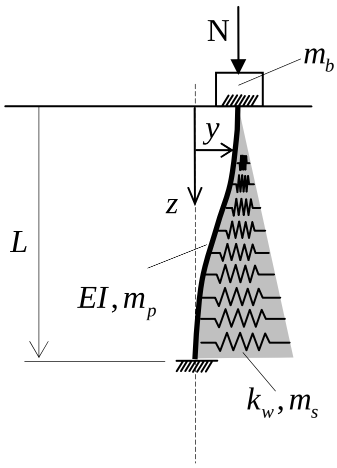

Figure 3: Example problem considered in this paper

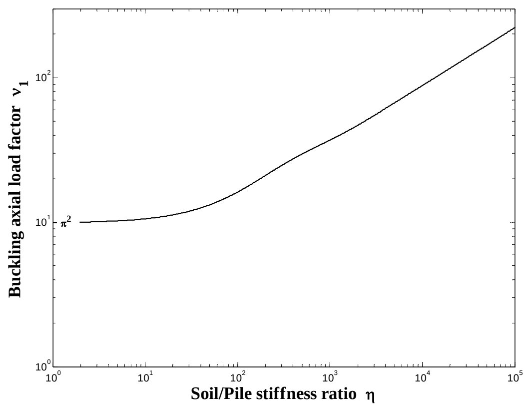

Figure 4: Buckling Load eigenvalue, ν_1 equation (8 & 9), for the boundary condition shown in Figure 3.

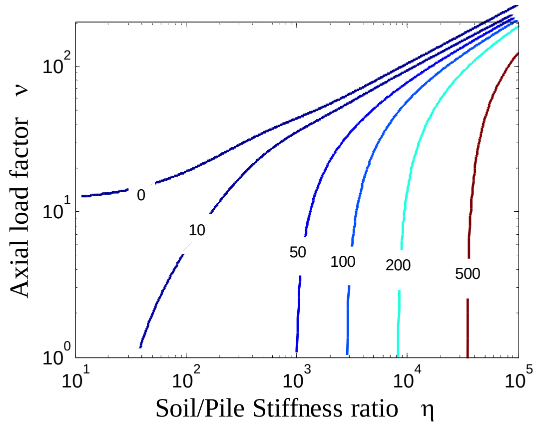

Figure 5: Natural Frequency eigenvalues, λ_1 equation (16 & 18), for the boundary condition shown in Figure 3

5. DISCUSSION

The following points may be observed:

1. As the soil/pile stiffness increases, the buckling load of the system increases. As a result, buckling of piles are never considered a problem unless the soil is extreme soft clay or liquefied soil.
2. As soil liquefies, the soil/pile stiffness decreases and the buckling load decreases (see Figure 4).
3. For low soil/ pile stiffness, the buckling load can be estimated by considering the pile as an unsupported column (see Figure 4).
4. For a particular axial load ratio of the pile, as the soil/ pile stiffness decreases, the first natural frequency of the system also decreases.
5. Zero contours in frequency plot (see Figure 5) is identical to the buckling load factors obtained in Figure 4

6. CONCLUSIONS

Dynamic stability of a pile-supported structure has been considered. During seismic liquefaction, two effects arises due to the removal of the lateral support the soil offers to the pile. They are

(a) Increase in axial load in the pile in the potentially unsupported zone due to loss of shaft resistance;

(b) Dynamics of pile-supported structure due to frequency dependent force arising from the shaking of the bedrock and the surrounding soil than can cause dynamic amplification of pile head displacements leading to resonance type failure.

Under service conditions (no earthquake and no-liquefaction) the first natural frequency of a structure or the fundamental time period can be estimated without considering the effect of the piles. However when the soil starts to liquefy, the piles become an integral part of the structure and take part in the vibration. As a result, the time period alters significantly and cannot be ignored in analysis/design. In most cases, the frequency will decrease.

This paper shows a simple method to analyse the combined problem of stability and dynamics. This paper explicitly quantified the first natural frequency due to the effects of (1) Axial force, (2) Dynamic excitation and (3) Reduction of the lateral support due to liquefaction for one boundary condition of a pile.

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References:

AIJ (1988, 2001): Architectural Institute of Japan, Recommendations for design of building foundations, (in Japanese)

Bhattacharya, S and Madabhushi, S.P.G (2008): "A critical review of methods of pile design in seismically liquefiable soils", Bulletin of Earthquake Engineering, Volume 6, No 3, pp 407-447.

Bhattacharya, S., Bolton, M.D. and Madabhushi, S.P.G. (2005): "A reconsideration of the safety of the piled bridge foundations in liquefiable soils", Soils and Foundations, Volume 45, August 2005 issue, No 4, pp 13-26.

Bhattacharya, S., Madabhushi, S.P.G., and Bolton, M.D. (2004): "An alternative mechanism of pile failure in liquefiable deposits during earthquakes", Geotechnique 54, April issue, No.3, pp 203-213.

Bhattacharya, S (2003): Pile Instability during earthquake liquefaction, PhD thesis, University of Cambridge (U.K).

Dash, S.R., Bhattacharya, S., Blakeborough, A. and Hyodo, M (2008): "P-Y curve to model lateral response of pile foundations in liquefied soils", Paper number 04-01-0089, Proceedings of the 14th World Conference on Earthquake Engineering, Oct 12-17, 2008, Beijing.

Ishihara, K (1997): Terzaghi oration: "Geotechnical aspects of the 1995 Kobe earthquake", Proceedings of ICSMFE, Hamburg, pp 2047-2073.

JRA (2002, 1996): Japanese Road Association, Specification for Highway Bridges, Part V, Seismic Design.

Kimura, Y. and Tokimatsu, K. (2007): "Buckling stress of slender pile with lateral displacement at the pile head in liquefied soils", Journal of Structural and Construction Engineering, AIJ (Architectural Institute of Japan) No 617, pp 169-175, July 2007.

Lin et al (2005): "Damage of piles caused by lateral spreading – Back study of three cases", ASCE Geotechnical Special Publication No 145 on Seismic Performance and Simulation of Pile Foundations in Liquefied and Laterally Spreading Ground, (Boulanger and Tokimatsu eds),

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.

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 Goetechnical and Geo-environmental Engineering (ASCE), Vol 131, No 1, January 2005

RTRI: Rail Technical Research Institute code – 1999 edition

Shanker, K., Basudhar, P.K. and Patra, N.R. (2007): "Buckling of piles under liquefied soil condition", Geotechnical and Geological Engineering (Springer), Volume 25, pp 303-313.

Takahashi, A., Kuwano, J., Arai, Y., and Yano, A. (2002): "Lateral resistance of buried cylinder in liquefied sand" Proceedings of the international conference on physical modelling in geotechnics, ICPMG-02, St Johns, Newfoundland, Canada, July 2002.

Thomson, J.M.T and Hunt, G.W (1984): "Elastic Instability phenomenon", Wiley, Chichester.

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

Yoshida, N, Towahata, I., Yasuda, S. and Kanatani, M. (2005): Discussion to the paper by Bhattacharya et al. (2004) on "An alternative mechanism of pile failure in liquefiable deposits during earthquakes", Geotechnique, 55, No 3, pp 259-263.