

On the mechanics of failure of pile-supported structures in liquefiable deposits during earthquakes

Subhamoy Bhattacharya^{1,*}, Suresh R. Dash² and Sondipon Adhikari³

¹Department of Civil Engineering, University of Bristol (Previously University of Oxford), Queens Building, BS8 1TR, UK

²Department of Engineering, University of Oxford, UK

³Department of Aerospace Engineering, University of Wales (Swansea), UK

Piles are long, slender members inserted deep into the ground to support heavily loaded structures such as bridges, buildings, jetties or oil platforms, where the ground is not strong enough to support the structure on its own. It is not an overstatement to state that most small to medium span river bridges and most G + 4 buildings are supported on piles. In seismic-prone zones, in areas of loose to medium dense sand, where the groundwater table is near the ground surface, piles are also used to support structures such as buildings and bridges.

Under moderate to strong shaking, loose to medium dense, saturated, sandy soil liquefies and behaves like a 'solid suspension' due to the rise in pore water pressure. In other words, the sand behaves like 'quick sand' and cannot bear any load. These soils are termed as 'liquefiable deposits' and the phenomenon is termed as 'liquefaction'. Collapse and/or severe damage to pile-supported structures is still observed in liquefiable soils after most major earthquakes. Therefore, this still remains a great concern to the earthquake engineering community. This article explains the mechanics behind the failure of these structures.

Keywords: Buckling, earthquake, liquefaction, pile, pile design.

PILES are a particular type of foundation inserted deep into the ground. Essentially, they are long, slender members that transfer the load of the superstructure to greater depths (Figure 1). These deep foundations are generally chosen at a construction site where the soils at shallow depths are weak and have low bearing capacity. The piles transfer the load of the superstructure through two ways: (a) Shear generated along the surface of the pile due to soil–pile friction; (b) Point resistance due to the bearing of the pile at its bottom. It is not an overstatement to say that piles are used in most of the heavily loaded structures such as multi-storied buildings, bridges, flyovers or oil platforms, etc.

In seismic-prone zones having loose to medium dense sandy soil, structures are often founded on piles because

the sand is not strong enough to support the load of the structure through conventional shallow footing. If these sands are saturated (due to shallow water table), they lose their strength and stiffness during earthquake shaking. Essentially, the soil behaves like a thick fluid quite similar to 'quick sand'. The soil which was solid before the earthquake transforms into a fluid-like material during shaking. This phenomenon is termed as 'liquefaction' and has been reported to be one of the main causes of destruction to the built environment; for example, the 1964 Niigata earthquake, the 1964 Alaska earthquake, the 1995 Kobe earthquake or the 2001 Bhuj earthquake. The soils that liquefy are termed as 'liquefiable soils'.

Pile-supported structures still collapse and/or get severely damaged in liquefiable soils after strong earthquakes; for example, the 1995 Kobe earthquake, the 1999 Koceli earthquake, the 2001 Bhuj earthquake and the 2004 Sumatra earthquake. This is despite the fact that a large factor of safety is apparently employed in their design. Figure 2 shows few case histories of failure of pile-supported structures in liquefiable soil during earthquakes. The photographs show that the superstructure (part of the structure above the ground) is intact/undamaged and it tilts or rotates as a whole, rendering it useless following an earthquake. This suggests that the foundations may have

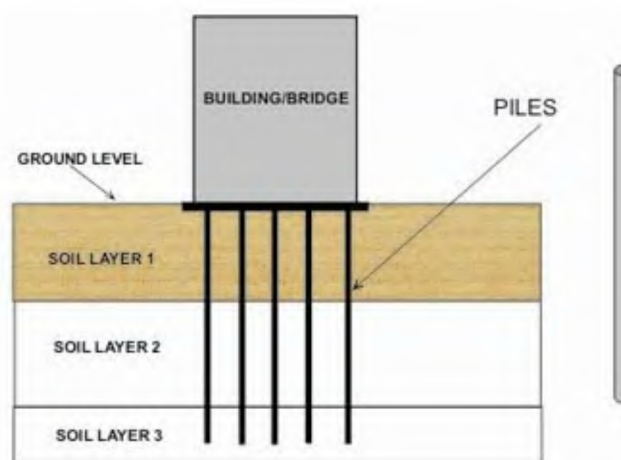


Figure 1. Schematic diagram of a pile-supported structure.

*For correspondence. (e-mail: subhamoy.bhattacharya@eng.ox.ac.uk)

been damaged. It is also clear that not only in developing countries (such as India) but also in developed countries (such as Japan) the same kinds of failure are being observed. It can be argued that in the developed countries higher degree of quality control in design and construction is maintained. This is strong evidence that the correct failure mechanism/mechanisms governing the failure have not been properly taken care of while designing them.

As earthquakes are rapid events and as much of the damage to piles occurs beneath the ground, it is hard to ascertain the detailed pattern of failure unless deep excavations are carried out. Twenty years after the 1964 Niigata earthquake and also following the 1995 Kobe earthquake, investigation has been carried out by excavating and extracting the pile from subsoil and using borehole cameras to take photographs^{1,2}. The detailed field investigation provided important information about the location of cracks and damage patterns for the piles. Figure 3 shows the result of one such excavation where extensive damage has been observed in the piles. The piles were completely damaged with the reinforcements exposed. Design of piled foundations in liquefiable soils therefore still remains a major concern to the earthquake geotechnical engineering community.

This article, therefore, has two aims:

- (1) To describe and summarize the plausible failure mechanisms of pile-supported structures in liquefiable soils that has been identified in recent research. It must be mentioned that seismic pile design is a

constantly evolving subject. Earthquakes in the past have shown the shortcomings of current design methodologies and construction practices, at the cost of structural failures and loss of lives. Post-earthquake investigations have led to improvements in engineering analysis, design and construction practices. It is therefore necessary to summarize the recent findings and it is expected that this article will serve the purpose.

- (2) To compare the state-of-the-art understanding and state-of-the-art practice (codes of practice) of seismic pile design in liquefiable soils.

Different stages of loading in the pile during earthquake

During earthquakes, soil layers overlying the bedrock are subjected to seismic excitation consisting of numerous incident waves, namely shear (*S*) waves, dilatational or pressure (*P*) waves, and surface (Rayleigh and Love) waves, which result in ground motion. As the seismic waves arrive in the soil surrounding the pile, the soil layers tend to deform. This seismically deforming soil tries to move the piles and the embedded pile-cap with it. Subsequently, depending upon the rigidity of the superstructure and the pile-cap, the superstructure may also move with the foundation. The pile may thus experience two distinct phases of initial soil–structure interaction.

(1) Before the superstructure starts oscillating, the piles may be forced to follow the soil motion, depending on the flexural rigidity (*EI*) of the pile. Here the soil and pile may take part in kinematic interplay and the motion of the pile may differ substantially from the free field motion. This may induce bending moments in the pile (Figure 4, Stage-II).

(2) As the superstructure begins to oscillate, inertial forces are generated. These forces are transferred as lateral forces and overturning moments to the pile through the



Figure 2. *a*, Kandla Port tower after the 2001 Bhuj earthquake. The building is located near a water front. *b*, Collapse of a building in level ground following the 1995 Kobe earthquake (photo courtesy: K. Tokimatsu). *c*, Failure of the Showa Bridge during the 1964 Niigata earthquake (NISEE¹⁷).



Figure 3. Excavated piles of a building twenty years after the 1964 Niigata earthquake. The superstructure is three-storey RC building. The piles are precast RC piles 10 m long and 300 mm in diameter. (Photo courtesy: T. Tazon.)

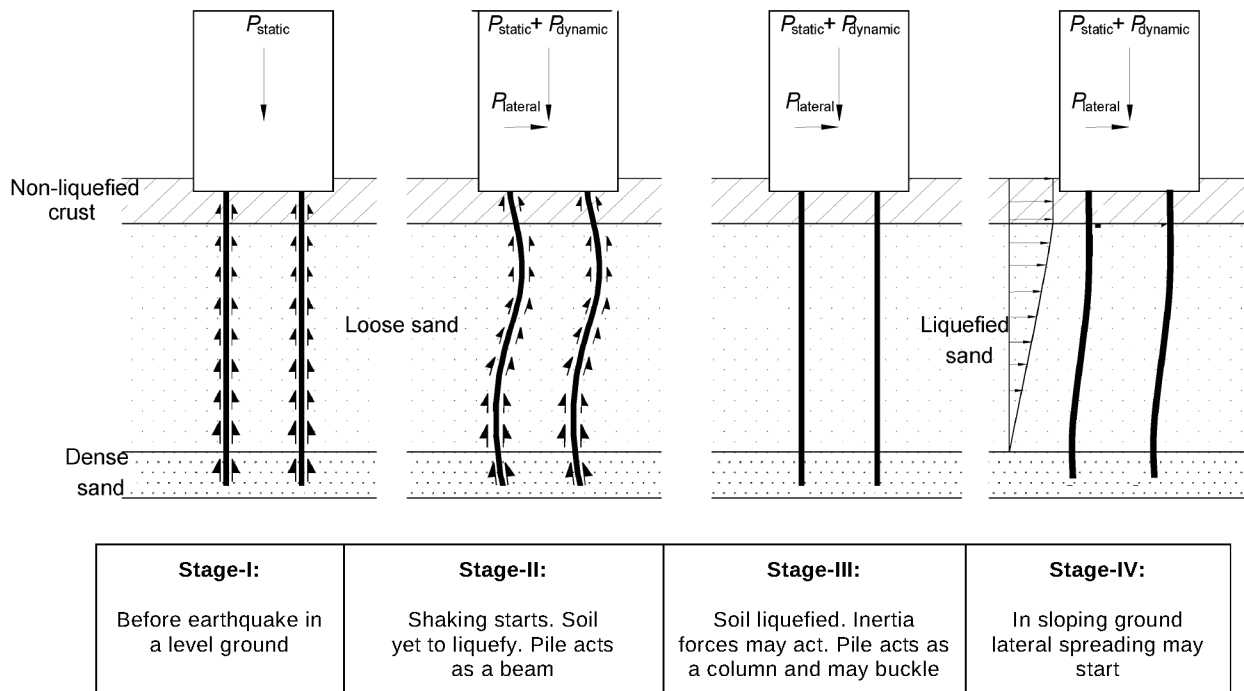


Figure 4. Loads and collapse mechanisms on a piled foundation.

pile-cap. The pile-cap then transfers the moments as varying axial loads and bending moments in the piles. Thus the piles may experience additional axial and lateral loads, which cause additional bending moments in the pile.

The above two effects occur with only a small time lag. If the section of the pile is inadequate, bending failure may occur in the pile. The behaviour of the pile at this stage may be approximately described as a beam on an elastic foundation, where the soil provides sufficient lateral restraint. The available confining pressure around the pile is not expected to decrease substantially in these initial phases. The response to changes in axial load in the pile would not be severe either, as shaft resistance continues to act. However, the pile should be strong enough to take the additional axial load induced by inertia of the superstructure mass.

In loose, saturated, sandy soil, as the shaking continues pore pressure builds up and the soil begins to liquefy. With the onset of liquefaction, an end-bearing pile passing through liquefiable soil will experience distinct changes in its stress state. The following two distinct states may be used to describe the state of the soil–structure interaction during the earthquake.

(1) The pile will start to lose its shaft resistance in the liquefied layer and shed axial loads downwards to mobilize additional base resistance. If the base capacity is exceeded, settlement failure will occur (Figure 4, Stage-III). As the unsupported length of piles increases due to liquefaction, the piles may fail due to buckling under high axial load.

(2) The liquefied soil will begin to lose its stiffness so that the pile acts as an unsupported column (Figure 4, Stage-IV). Piles that have a high slenderness ratio will then be prone to axial instability and buckling failure will occur in the pile, enhanced by the actions of lateral disturbing forces and also by the deterioration of bending stiffness due to the onset of plastic yielding. Dynamic centrifuge tests, study of case histories and analytical work carried out^{3–5} have conclusively shown the above failure mechanism. This particular mechanism is currently missing in all codes of practice.

In sloping ground, even if the pile survives the above load conditions (i.e. safely carry the axial load and the lateral inertial loads under fully liquefied condition), it may experience additional drag load due to the lateral spreading of soil. Under these conditions, the pile may behave as a beam column (column with lateral loads).

Predominant loads acting on piled foundations during earthquakes

Based on the above description, the predominant loads acting on a pile can be summarized as follows:

1. Axial load (P) that acts at all times. P_{static} (Figure 4, Stage I) represents the axial load on piles in normal condition in the absence of environmental loading. This can be estimated based on statical equilibrium. This axial compressive load may increase/decrease further by $P_{dynamic}$ due to inertial effect of the super-

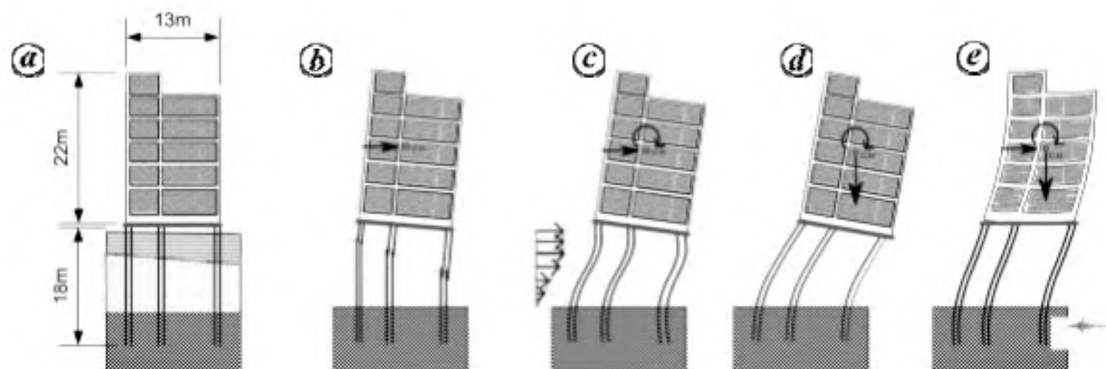


Figure 5. Different failure mechanisms: **a**, Kandla Port building in Figure 2 *a* with pile foundations; **b**, Shear failure mechanism; **c**, Bending failure mechanism; **d**, Buckling mechanism; and **e**, Dynamic amplification mechanism.

structure and kinematic effects of the soil flow past the foundation.

2. Inertial lateral loads due to oscillation of the superstructure (P_{lateral}).
3. Kinematic loads due to ground movement. This load may be of two types: transient (during shaking, due to the dynamic effects of the soil mass) and residual (after the shaking ceases due to soil flow, often known as 'lateral spreading').

Fundamental failure mechanisms of pile foundation

This section describes the fundamental mechanisms that may cause yielding/failure of the pile. The failure of the Kandla Port tower building (Figure 2 *a*) has been used as an example to discuss the various mechanisms. The building is located in laterally spreading ground in the city of Kachchh (Arabian Sea). The lateral dimension of the building is 13 m and it is 22 m high, supported on 18 m long piles. The feasible failure mechanisms are given below.

Shear failure

Shear failure of pile may occur due to lateral loads such as inertia or kinematic loads or a combination of the above. Figure 5 *b* shows this mechanism of pile failure due to inertia load. This is particularly damaging to hollow, circular, concrete piles (non-ductile) with low shear capacity.

Bending failure

Bending failure of piles may occur due to the lateral loads either due to inertia or due to kinematic loads or a combination of the two. This would depend on the type of earthquake motion, the time of onset of liquefaction and regaining of strength of the soil after liquefaction. Bending in the pile due to lateral spreading of ground is often re-

garded as the root cause of many bridge failures⁶⁻¹². Figure 5 *c* explains the hypothesis of this failure mechanism. Japanese Bridge Code of Practice JRA (1996, 2002) has codified this mechanism. They advise practising engineers to design the pile considering passive earth pressure for non-liquefied crust and 30% of the total overburden pressure for the zone of liquefied soil.

Buckling instability

Buckling failure in slender piles may occur due to the effect of axial load acting on the pile and loss of the surrounding confining pressure offered by the soil owing to liquefaction^{3-5,13}. Lateral loading due to slope movement, inertia or out-of-line straightness in the pile will increase lateral deflections, which in turn can increase the chances of instability failure even at lower axial loads. This may cause plastic hinges in the piles leading towards collapse of the structure.

Dynamic failure

All the above failures can occur due to static loads. During the earthquake, the dynamic soil-pile interaction becomes much complicated and has significant effect on the pile response. The dynamic properties of soil and pile and their interaction properties change during the earthquake. This change can lead to amplification of structural response and eventually to the failure of the structure (Figure 5 *e*). The following effects have been identified.

Change in natural frequency of vibration of the pile-supported structure during the process of liquefaction: The frequency of a pile-supported structure will change with the stiffness degradation of the soil surrounding the pile¹⁴. Usually, the time period of vibration of a pile-supported structure is estimated based on formulas which are derived from internationally calibrated data¹⁵, IS: 1893 (2002). This time period depends on the dimension

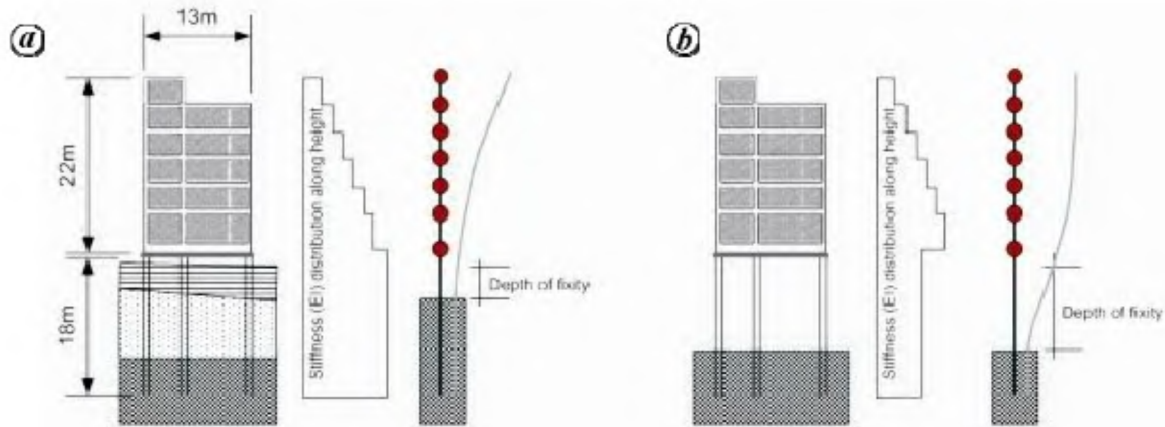


Figure 6. Structural configuration before liquefaction (a) and after liquefaction (b). (It is assumed in the example that 8 m of pile is unsupported.)

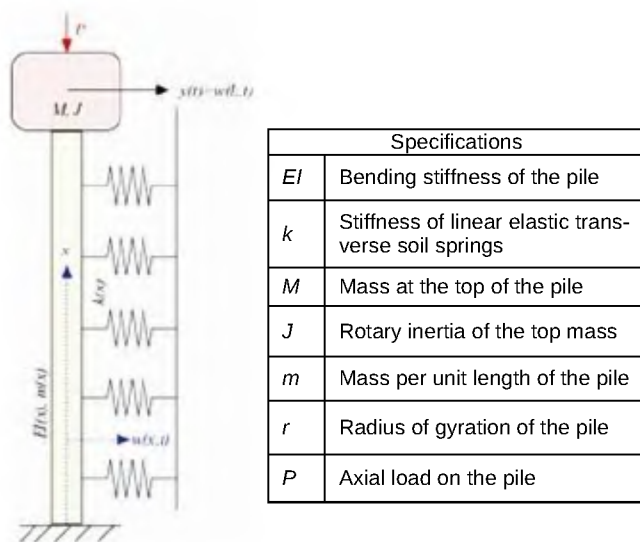


Figure 7. Combined pile-soil model using Euler-Bernoulli beam with top superstructure mass and axial force resting against a distributed elastic support.

of the superstructure without any consideration to the foundation. However, during and after liquefaction, as the pile loses its lateral confinement, it becomes an integral part of the superstructure. The frequency of the structure may alter substantially and in most cases will reduce. Reduction in fundamental frequency of the structure will increase its flexibility and the structure may suffer more lateral deformation. The bending moment in the piles may increase significantly if the altered natural frequency of the structure comes close to the driving frequency of the earthquake. Designers must therefore ensure that the frequency of the structure at full liquefaction should not come close to the driving frequency of the earthquake, to avoid resonance effect.

Change in the behaviour of structure: The structure in fully liquefied soil behaves like an inverted pendulum/

open ground-storey structure with piles resembling the long ground columns, which is not considered an ideal design for seismic vibration. This situation is also similar to the soft ground-storey phenomena. Figure 6 explains the situation. Due to loss of lateral soil stiffness in liquefiable soil, stiffness ratio between the superstructure and stiffness of the pile group becomes large. This stiffness change along with long unsupported length of piles may induce large lateral displacement at pier-cap level.

Change in soil properties: A pile-supported structure must be embedded in a competent soil layer to ensure fixity and avoid sliding. Figure 6 shows the depth of fixity of the pile before and after liquefaction. Due to liquefaction, soil stiffness reduces drastically, and the depth of fixity of pile increases. In other words, the point of fixity goes deeper, which increases the unsupported length of the pile-supported structure. Liquefied soil also acts as a damper to the vibration of the pile. The designer should consider the stiffness and damping of liquefied soil while analysing the pile foundation system.

The above forms of failure can be described as the ‘limit state of collapse’. Each of these failure mechanisms can cause a complete collapse of the foundation. However, real failure is perhaps a nonlinear combination of the above mechanisms.

It is worth noting that the pile will also lose its shaft resistance in the liquefiable region due to loss of effective stress, and thus have to settle for vertical equilibrium (Figure 4, Stage III). In order to be functional after the earthquake, settlement of the piled foundation should be within the acceptable limits for the structure. This can be termed as ‘limit state of serviceability’.

Analytical model of a cantilever pile

In this section, the results of the analysis of a cantilever pile as described in Figure 7 are presented. The soil supporting the pile is shown by discrete ‘Winkler spring

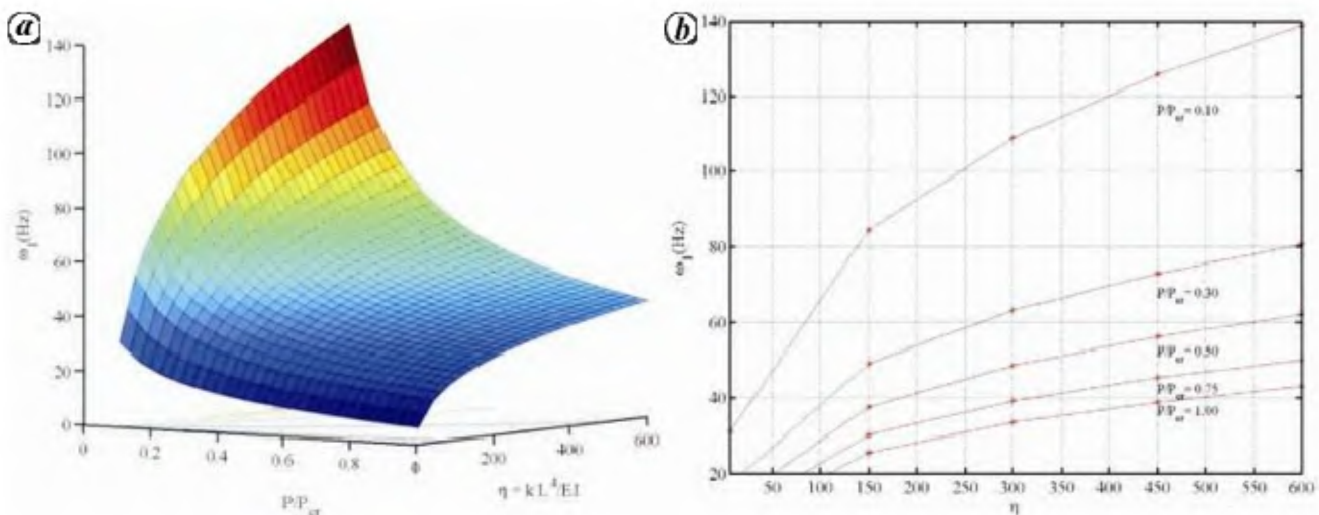


Figure 8. Variation of the first natural frequency of the cantilever pile with respect to the normalized support stiffness and axial load (a) and normalized support stiffness for different values of axial load (b).

$k(x)$ '. As the soil liquefies (Figure 4, Stage-III), its stiffness (k) will decrease. Typical estimates¹⁶ show that the value of k can reduce up to 0.1% of the original value, i.e. the stiffness of the soil while it is in liquefied condition is about 0.1% of its small-strain stiffness.

The main assumptions in the analysis are:

- (i) The inertial and elastic properties of the pile are constant along the depth of the pile.
- (ii) Soil stiffness is elastic and linear, continuous and varies along the depth shown by $k(x)$. Variation of $k(x)$ with lateral displacement (y) is considered to be linear. As this investigation deals with the transition from full soil stiffness to zero soil stiffness (liquefied), this assumption will not mask the behaviour under investigation. The inertia of the soil has been ignored.
- (iii) The boundary condition at the bottom of the pile can be considered as fixed (i.e. no rotation and no displacement is allowed). This would represent a pile embedded in the non-liquefiable dense layer where strain-induced degradation is relatively negligible.
- (iv) The head mass is rigidly attached to the pile head and the axial force in the pile is constant and remains axial during vibration.
- (v) Deflections due shear force are negligible and a plane section in the pile remains plane during the bending vibration (standard assumptions in the Euler–Bernoulli beam theory).
- (vi) Flexibility of the building/structure above the ground is assumed to be uncoupled with the pile dynamics.
- (vii) The influence of other local foundations, near-field interactions and pile-group effects is neglected.
- (viii) None of the properties is changing with time. In other words, the system is time-invariant.

Figure 8 shows the results of the analysis. Figure 8a shows the variation of the first natural frequency (ω_1)

with respect to normalized support stiffness (η) and normalized axial load (P/P_{cr}), where P_{cr} is the Euler's critical buckling load. η refers to the support offered by the soil to the pile and is defined by eq. (1).

$$\eta = \frac{kL^4}{EI}, \quad (1)$$

where L is the length of the pile and definitions of the other terms are given in Figure 7.

Interpretation of the decrease in η is the decrease in soil stiffness due to the onset of liquefaction. Figure 8b shows the effect of decrease in support stiffness for a particular value of axial load. It shows that the first natural frequency of the pile decreases substantially with decrease in support stiffness.

Discussions and conclusions

Seismic pile design – theory and practice

From the above discussion, it is evident that after some initial time period, as the soil starts liquefying (Figure 4, Stage-III), the motion of a pile-supported structure will be a coupled action. This coupling will consist of: (a) Transverse static bending predominantly due to the lateral loads; (b) Dynamic buckling arising due to the dynamic vertical load of the superstructure; and (c) Resonance motion caused by the frequency-dependent force arising due to the shaking of the bedrock and the surrounding motion.

In the initial phase (Figure 4, Stage-II), when the soil has not fully liquefied, transverse static bending is expected to govern the internal stresses within the pile. As liquefaction progresses, coupled buckling and resonance would govern the internal stresses and may eventually lead to dynamic failure. The key physical aspect that the

authors aim to emphasize and no codes of practice consider is that the motion of the pile (and consequently the internal stresses leading to the failure) is a coupled action. 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 static bending, (b) dynamic buckling and (c) resonance motion is appropriate for a comprehensive understanding of the failure mechanism of piles during an earthquake.

In contrast, most codes of practice (if not all) on seismic pile design in liquefiable deposits focus on bending strength and omit considerations for the bending stiffness required to avoid buckling instability and resonance failure in the event of soil liquefaction. The current design codes need to address buckling of piles due to the loss of soil support owing to liquefaction and must also consider the dynamic response of the structure during an earthquake. A pile must also be sufficiently embedded in the non-liquefiable hard layer below the liquefiable soil to ensure fixity and avoid sliding. The frequency of pile-supported structure at full liquefaction should not be close to the diving frequency of the earthquake. Dynamic properties of structure and soil should not be neglected in the process of design. The settlement of the structure due to loss of shaft resistance of the pile in the liquefiable soils should be within acceptable limits.

Discussion on the method of analysis

We recognize that dynamic pile–soil interaction in liquefiable multi-layered soil is complex and interaction pattern changes during different phases of the earthquake. It must also be mentioned that codes of practice have to specify some simple design loads which should provide a safe working envelope for any structure of the class being considered, and in the full range of ground conditions likely to be encountered at different sites. Therefore, it is of value and interest to summarize the essential features of this type of vibration for a simple case and for simple assumptions. In particular, the first natural frequency of vibration can be estimated through this type of simplified analysis. If the natural frequency is known, then stationary random vibration analysis can be carried out, which can comment on the possibility of dynamic failure. The power spectral density of the linear system would have a peak around the natural frequency.

However, detailed nonlinear analysis, relaxing the various assumptions outlined earlier, can be done in a routine manner using commercially available finite element software. However, such an approach can only be used in

a ‘case by case basis’ and would lack the generality necessary to be prescribed in the design codes.

1. Yoshida, N. and Hamada, M., Damage to foundation piles and deformation pattern of ground due to liquefaction-induced permanent ground deformation. In Proceedings of the 3rd Japan–US Workshop on Earthquake-resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction, 1990, pp. 147–161.
2. BTL Committee (2000): Study on liquefaction and lateral spreading in the 1995 Hyogoken–Nambu earthquake. Building Research Report No. 138, Building Research Institute, Ministry of Construction, Japan (in Japanese).
3. Bhattacharya, S., Pile instability during earthquake liquefaction. Ph D thesis, University of Cambridge, UK, September 2003.
4. Bhattacharya, S., Madabhushi, S. P. G. and Bolton, M. D., An alternative mechanism of pile failure in liquefiable deposits during earthquakes. *Geotechnique*, 2004, **54**.
5. Knappett, J. A. and Madabhushi, S. P. G., Modeling of liquefaction-induced instability in pile groups. In *Seismic Performance and Simulation of Pile Foundations in Liquefied and Laterally Spreading Ground*, Geotechnical Special Publication No. 145, ASCE, 2005.
6. Hamada, M., Large ground deformations and their effects on lifelines: 1964 Niigata earthquake. Case studies of liquefaction and lifelines performance during past earthquake. In Technical Report NCEER-92-0001, Vol. 1, Japanese Case Studies, National Centre for Earthquake Engineering Research, Buffalo, NY, 1992.
7. Hamada, M., Large ground deformations and their effects on lifelines: 1983 Nihonkai–Chubu earthquake. Case studies of liquefaction and lifelines performance during past earthquake. In Technical Report NCEER-92-0001, Vol. 1, Japanese Case Studies, National Centre for Earthquake Engineering Research, Buffalo, NY, 1992.
8. Ishihara, K., Terzaghi oration: Geotechnical aspects of the 1995 Kobe earthquake. In Proceedings of ICSMFE, Hamburg, 1997, pp. 2047–2073.
9. Tokimatsu, K., Oh-oka Hiroshi, Satake, K., Shamoto, Y. and Asaka, Y., Effects of lateral ground movements on failure patterns of piles in the 1995 Hyogoken–Nambu earthquake. In Proceedings of a Speciality Conference, Geotechnical Earthquake Engineering and Soil Dynamics III, ASCE Geotechnical Special Publication No. 75, 1998, pp. 1175–1186.
10. Goh, S. and O’Rourke, T. D., Limit state model for soil–pile interaction during lateral spread. In Proceedings of 7th US–Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures against Soil Liquefaction, Seattle, WA, 1999.
11. Abdoun, T. H. and Dobry, R., Evaluation of pile foundation response to lateral spreading. *Soil Dyn. Earthquake Eng.*, 2002, **22**, 1051–1058.
12. Finn, W. D. L. and Fujita, N., Piles in liquefiable soils: Seismic analysis and design issues. *Soil Dyn. Earthquake Eng.*, 2002, **22**, 731–742.
13. Bhattacharya, S. and Bolton, M. D., A fundamental omission in seismic pile design leading to collapse. In Proceedings of the 11th International Conference on Soil Dynamics and Earthquake Engineering, Berkeley, 7–9 January 2004, pp. 820–827.
14. Bhattacharya, S. and Adhikari, S., Dynamic behaviour of piled foundations in liquefiable soils during strong earthquakes. In *Design of Foundations in Seismic Areas: Principles and Applications* (ed. Bhattacharya), NICEE, India, 2007, Chapter 6, pp. 299–318.
15. Chopra, A. K. and Goel, R. K., Building period formulas for estimating seismic displacements. *Earthquake Spectra*, 2000, **16**, 533–536.
16. Yasuda, S., Yoshida, N., Kiku, H., Adachi, K. and Gose, S., A simplified method to evaluate liquefaction-induced deformation. In *Earthquake Geotechnical Engineering* (ed. Seco e Pinto), © Balkema, Rotterdam, ISBN 90 5809 1163, 1999.
17. NISEE, National Information Services for Earthquake Engineering, University of California, Berkeley.

Received 28 May 2007; revised accepted 23 January 2008