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# The Dynamic Behaviour of Pile Foundations in Seismically Liquefiable Soils: Failure Mechanisms, Analysis, Re-Qualification

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## Abstract

This chapter presents a concise overview of the mechanics of failure, analysis and requalification procedures of pile foundations in liquefiable soils during earthquakes. The aim is to build a strong conceptual and technical interpretation in order to gain insight into the mechanisms governing the failure of structures in liquefaction and specify effective requalification techniques. In this regard, several most common failure mechanisms of piles during seismic liquefaction such as bending (flexural), buckling instability and dynamic failure of the pile are introduced. Furthermore, the dynamic response commentary is provided by critically reviewing experimental investigations carried out using a shaking table and centrifuge modelling procedures. The emphasis is placed on delineating the concept of seismic design loads and important aspects of the dynamic behaviour of piles in liquefiable soils. In this context, using Winkler foundation approach with the proposed  $p$ - $y$  curves and finite-element analyses in conjunction with numerical analysis methods, are outlined. Moreover, the feasibility of successful remediation techniques for earthquake resistance is briefly reviewed in light of the pile behaviour and failure. Finally, practical recommendations for achieving enhanced resistance of the seismic response of pile foundation in liquefiable soil are provided.

**Keywords:** liquefaction, dynamic behaviour, pile, failure mechanisms, requalification

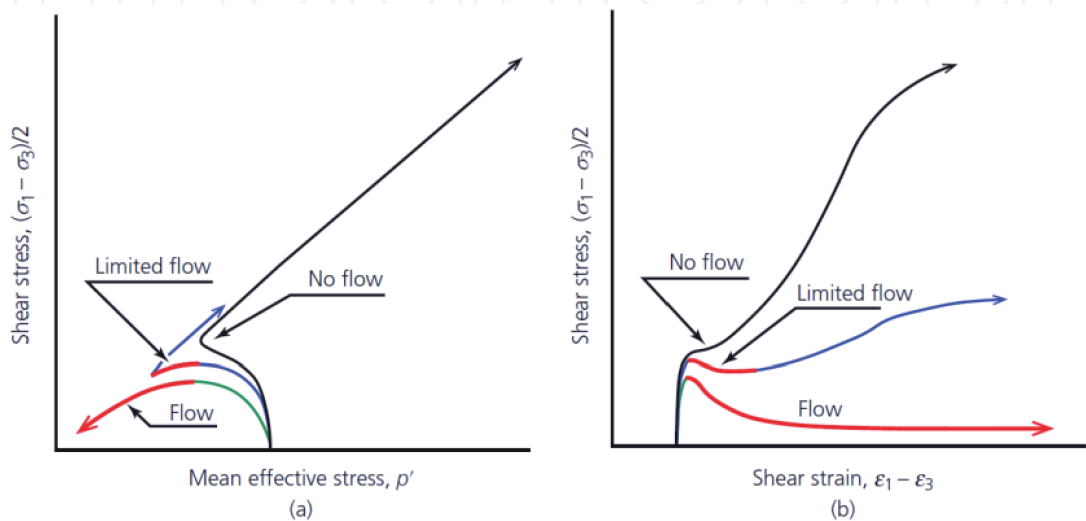
## 1. Introduction (Characterisation of liquefaction behaviour)

The liquefaction of loose, saturated sands, particularly cohesionless soils is caused by earthquake shaking or cyclic (monotonically increasing) undrained loading. The early work in liquefaction soil in the laboratory apparently emerged from the experience of the Fukui earthquake in 1948 in Japan [1]. It was regarded as a milestone from researchers since its devastating failures were prevalent following the major earthquakes in Niigata, Japan and Alaska, USA, in 1964 [2–4].

Soil liquefaction has been responsible for extremely damaged structures and foundation piles of bridges and buildings and has resulted in severe loss of strength and stiffness of saturated cohesionless soil. Liquefaction was first introduced by Hazen [5] that he used to describe the 1918 collapse of Calaveras Dam in California [6]. Typically, liquefaction occurs when a deposit of loose saturated sand layers are subject to shaking during a seismic event, the progressive build-up of excess pore water pressure and the stiffness of the liquefied soil drops to a value of near-zero. The reduction in strength and stiffness of liquefied soil often leads to permanent deformation in sloping grounds, commonly termed as lateral spreading (a few centimetres or metres) or flow failure (hundreds of meters). Flow failures have been observed in a number of hydraulically filled earth dams, constructed tailings dams and in coastal and/or offshore areas [6]. This hydraulic problem was observed as secondary and progressive liquefaction surrounding the majority of slides as a result of the generation of excess pore pressure and the upward flow of water almost immediately prior to the initiation of the slide flow [6]. This also may lead to formation of sand boils, which have been illustrated by Ishihara [7] in terms of the relative thicknesses of liquefiable and overlying non-liquefiable layers in case history data from the 1976 Tangshan and 1983 Nihonkai earthquakes. In this respect, Huang and Yu [8] also classified the liquefaction related damage to soils and foundations during earthquakes in the first part of the twenty-first century in: ground subsidence, lateral spread, and damage induced by buoyancy (uplift).

Laboratory studies carried out to investigate the liquefaction susceptibility and conventionally evaluate the undrained behaviour of sandy soils under monotonic shearing. This tendency is generally expressed in terms of void ratio ( $e$ ) and relative density ( $D_r$ ). In this state, sand is flowing under constant shear stress at constant effective minor principal stress and at constant volume [6, 9–11]. Poulos [11] included the requirement of constant velocity, the “steady-state of deformation,” and the relationship between the steady-state effective stress and the void ratio i.e. the “steady-state” line. The response for very loose sand shows fully contractive behaviour is reached at large strains, as delineated in **Figure 1**. This behaviour reported as “spontaneous liquefaction” and also known as flow. In the case of sand with slightly higher density, the strain softening is followed by the strain hardening and the sand recovers its strength and restores stability. This type of behaviour was first called “limited liquefaction” by Castro [9] and known as limited flow.

In medium-dense and dense sands exhibiting dilative behaviour, ever-increasing shear stress is needed to induce shear strain and eventually obtain the steady state



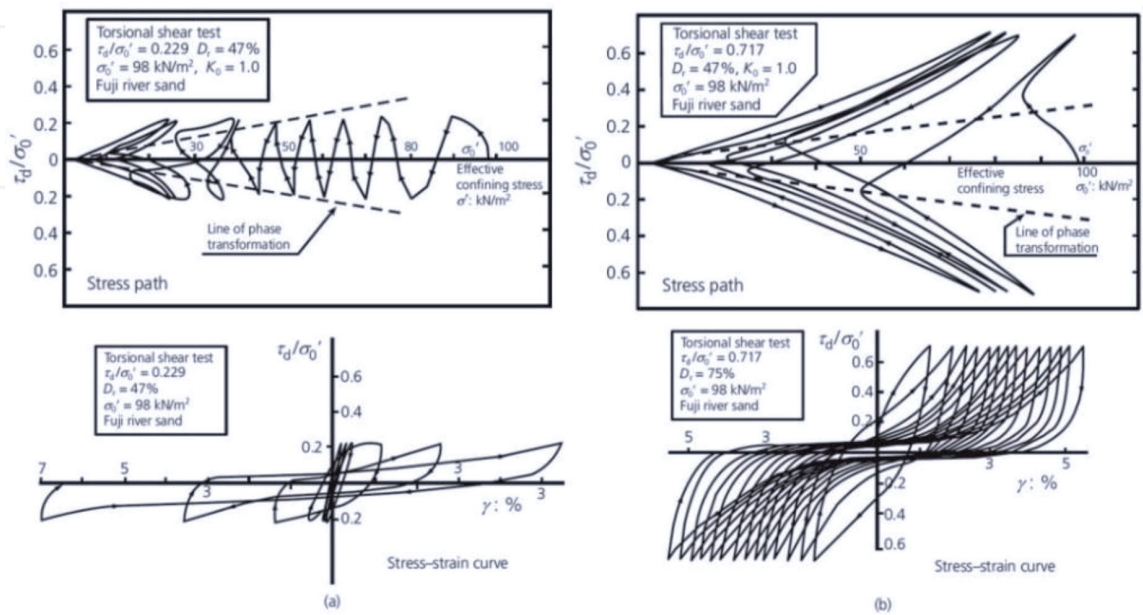
**Figure 1.** Monotonic behaviour of different sands: (a) effective stress path; (b) shear stress–shear strain relation [12].

of deformation (no flow) [12]. Kramer et al. [6] indicated that at relative densities greater than those corresponding to the steady state line (slightly greater than 44%), the soil exhibit dilative behaviour, with no potential for liquefaction.

**Figure 2a** and **b** illustrate a typical cyclic loading test on loose sand (2a) and dense sand (2b) using a torsional shear apparatus. The upper graphs describe the time history of cyclic shear stress ratio applied to constant-amplitude cyclic loading, while the lower graphs show the development of shear strain and the generation of excess pore pressure with time. In the early stages of loading the effective stress path moves to the left and the shear strain is negligible. However, as the loading progresses, the pore pressure builds up until the stress path begins to cross the phase transformation line identified by Ishihara [7] and eventually reaches a value equal to the initial confining pressure, which is called cyclic mobility. It can be seen that in both sand the effective confining stress decreases, but the shear strain increases in a slower manner for dense sand.

The liquefaction potential of the soil is generally estimated by comparing the anticipated earthquake loading and its inherent liquefaction resistance. This comparison is most commonly base on cyclic shear stress amplitude usually normalised by initial vertical effective stress and known as a cyclic stress ratio (CSR) for loading and a cyclic resistance ratio (CRR) for resistance. The potential for liquefaction is expressed in terms of a factor of safety against liquefaction,  $FL = CRR/CSR$ . If the  $FL > 1.0$  soil profile can be safe against liquefaction. Standard penetration test (SPT) and cone penetration test (CPT) are the two empirical methods that use to obtain the cyclic resistance to liquefaction. The susceptibility of soil deposits to liquefaction is determined by a combination of various factors such as soil properties, geological conditions and ground motion characteristics. The soil's CRR is also affected by the duration of shaking (which is correlated to the earthquake magnitude scaling factor, MSF) and effective overburden stress (which is expressed through a  $K\sigma$  factor). The evaluating of the soil liquefaction potential based on the SPT and CPT values are well explained in [12].

The seismic response of a soil profile is strongly influenced by the effective stress of an earthquake ground motions. The nature of ground motions at sites containing potentially liquefiable soils can affect the potential damage to pile foundations. The

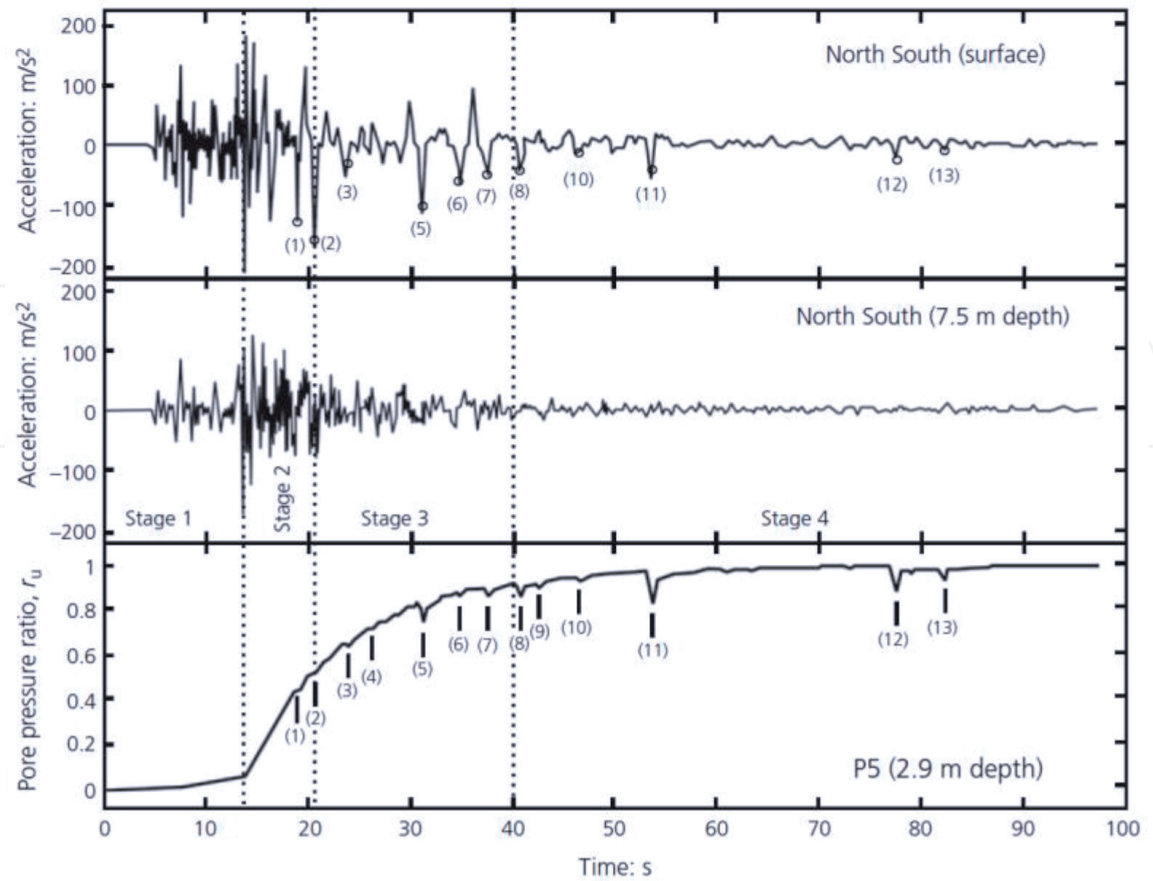


**Figure 2.**  
Stress path behaviour and stress-strain curve of (2a) loose sand and (2b) dense sand from the cyclic torsion shear test (Ishihara, [7]) ( $D_r$ : Relative density of the soil,  $K_0$ : Coefficient of earth pressure,  $\gamma$ : Shear strain,  $\tau_d / \sigma_0$ : Shear stress ratio).



relevant characteristics of ground motions are the frequency content, amplitude, and duration, and they can provide insight into the effects of ground motion duration on liquefaction hazards. The initial characteristic site period for a simple layer (ground of thickness  $H$ ) with constant initial shear wave velocity,  $V_{so}$ , is given by  $T_{so} = 4 H / V_{so}$ . The velocity is related to frequency  $f$  and wavelength  $\lambda$  by  $v = f \lambda$ . The relation between  $v_s$  and the SPT  $N$  value indicates the soil type [12]. The effects of liquefaction and generation positive of pore pressure leads to decrease in the effective stresses and the shear modulus of the soil. As a result, there is a reduction in soil stiffness which, in turn, increases prevalence of low frequency motions. The accelerations recorded of the Wildlife liquefaction array in the 1987 Superstition Hills earthquake (NS component) at the ground surface and at 7.5 m depth and the associated excess pore water pressure measured at 2.9 m depth are shown in **Figure 3**.

In each case, a clear and gradual stiffness degradation associated with an increase in the pore water pressure can be observed. Acceleration time history of the surface record showing clear visual evidence of the high-frequency portion of the motion from the beginning of the record (about 18 s); which is consistent with a series of isolated high-frequency pulses of acceleration (see numbers) [12]. Kramer et al. [14] reported that these pulses have amplitudes smaller than those of the pulses that occur prior to liquefaction, but in some cases, the peak ground acceleration of the entire motion is produced by a strong dilation pulse occurring near, or even after, the initiation of liquefaction. Hall et al. [15] examined the transient vibration characteristics of two  $2 \times 2$  pile-group models based on the wavelet. They found that liquefaction causes a decrease in structural frequency, whose reduction depends on the rate of excess pore pressure build-up, whereby high rates (“fast liquefaction”) lead to greater reduction, i.e. up to 51%.



**Figure 3.** Acceleration time histories time histories and associated pore water pressure of north-south components at wildlife site during the 1987 Superstition Hills earthquake (Zeghal and Elgamel, [13]) (numbers show high-frequency pulses of acceleration).

Ozener et al. [16] used of Stockwell spectrograms and indicated that as a result of the changes in stiffness; the response of a liquefied soil is often markedly different before and after triggering of liquefaction.

The liquefaction potential index (LPI) of Iwasaki et al. [17] is an integral number of factors of safety (FS) values weighted by depths of soil layers, which has increasingly been used for assessing the severity of liquefaction hazards [17, 18]. This is expressed by Eq. (1):

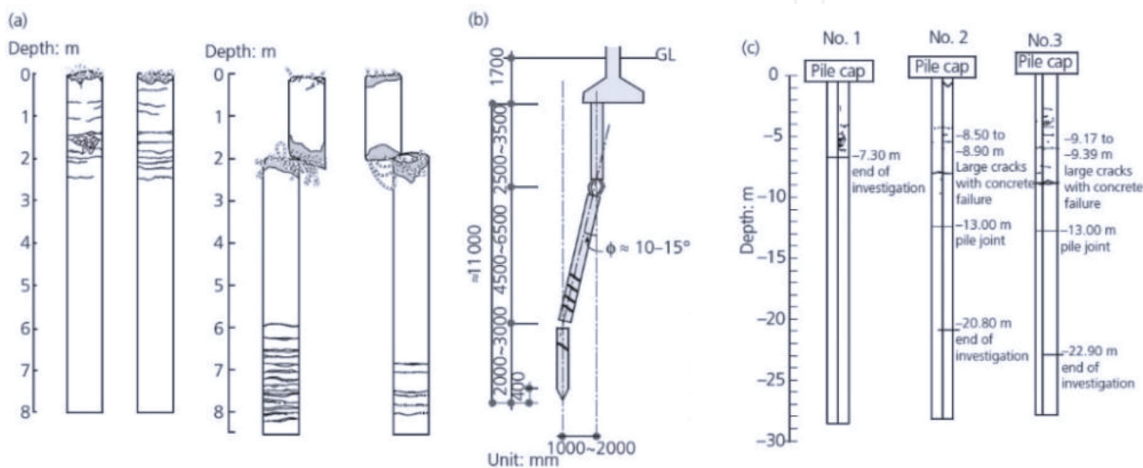
$$LPI = \int_0^{20} F \cdot w(z) dz \tag{1}$$

Where  $z$  is the depth (0–20 m) and  $w(z)$  = weighting function ( $10-0.5z$ ) and  $F$  is a function of the liquefaction resistance. Sites with  $LPI > 15$  have increasing susceptibility to liquefaction and potential for severe damage. The risk of liquefaction tends to decrease with depth while if  $LPI < 5$ , the effects are minor due to increasing effective stress.

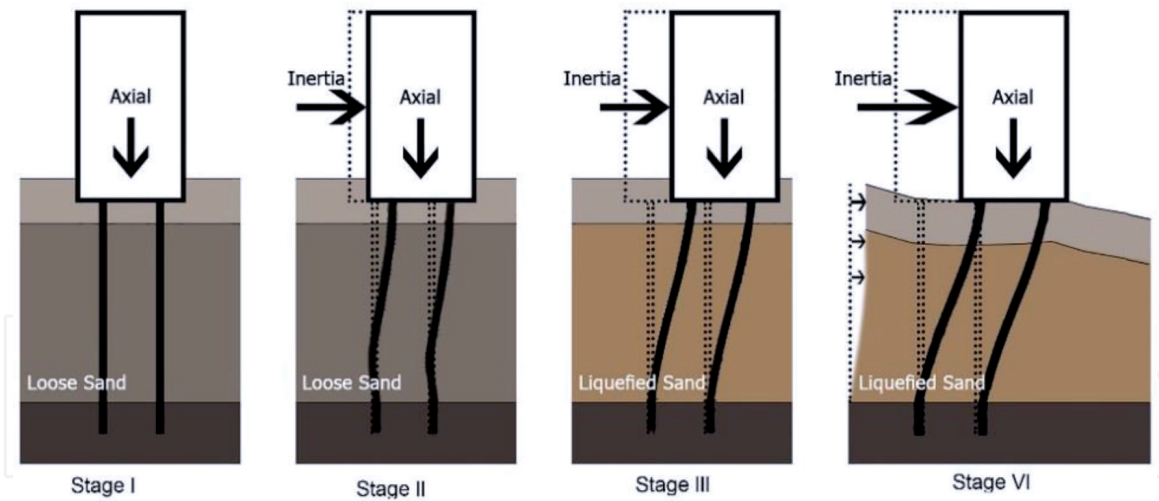
## 2. Pile failure

Piles are a particular type of deep foundation generally constructed to support heavily loaded structures to transfer the loads from superstructures to the deeper layers of soil, relying on both skin friction and tip resistance [19]. Piles are also used in seismic-prone zones comprising loose to medium-dense sandy soil or soft clay. However, during earthquake shaking when the soil around the pile loses much of its stiffness and strength due to liquefaction, the pile will act like a long laterally unsupported column and could buckle under the high axial load from the superstructure, affecting the foundations. Collapse and damage of pile-supported structures due to liquefaction have been observed after many major earthquakes [2–4]. The observations from many historic cases indicated that the failure of foundations occurred at unexpected locations (see **Figure 4**).

During earthquakes, the response of pile-supported structures to liquefiable soils would depend of the stiffness of the pile foundation type, the response of the soil surrounding the pile, and the soil-pile interaction effects. The analysis of this response requires accurate characterisation of the interaction effects include the inertial loading exerted by the superstructure and the kinematic loading induced by the soil surrounding the pile. **Figure 5** illustrates four critical stages of loading on the piles during a seismic liquefaction-induced event.



**Figure 4.**  
(a), (b) Buildings in Niigata city and (c) Building in Kobe city [12].



**Figure 5.**  
Different stages of loading and failure mechanism of pile during earthquake [20].

Before, or just at the onset of the earthquake, the axial load on the piles can be estimated based on static equilibrium. Upon commencement of the seismic vibration, and before the excess pore water pressure build-up, this axial compressive load may increase/decrease further due to the inertial effect of the superstructure (due to oscillation of superstructure) and the kinematic effects of the soil flow past the foundation (due to ground movement). This change in loading can be transient (during the vibration, due to the dynamic effects of the soil mass) and residual (after the vibration, due to soil flow, often known as “lateral spreading” [21]). However, at this stage, with pore water pressure built up (at full liquefaction, the excess pore water pressures reach the overburden vertical effective stress), the soil loses its strength and stiffness, and the pile acts as an unsupported slender columns over the liquefied depth. Most of the efforts have been made to greatly improve understanding of pile failure mechanism due to liquefaction [20–24]. Two plausible mechanisms of pile failure: bending (due to inertia of the superstructure and/or kinematic loads due to lateral soil pressure) and buckling (due to axial load), have been studied in detail separately. However, dynamic failure (bending–buckling interaction) of a pile foundation may also occur in a seismically liquefiable soil deposits and lead to failure of the structure.

## 2.1 Bending failure

The bending failure mechanism due to liquefaction-induced lateral spreading of soil is often demonstrated as one of the predominant causes of pile foundation failures during earthquakes [21–24]. The bending failure mechanism can occur when soil liquefies and loses much of its stiffness, causing the piles to act as unsupported slender columns. Piles can exhibit bending failure as a result of one or both of two mechanisms. First, during seismic shaking the lateral flow of soils at a particular depth induces additional forces on piles and simultaneously the bending moment is generated in the pile due to the summation of inertia and kinematic loads. Second, at the end of the shaking, the bending moment is expected only due to kinematic flow as a result of the full dissipation of pore pressure [25]. The bending behaviour of a pile depends on the bending strength (e.g. yielding of the pile materials) and the flexural stiffness (changes in geometry of the moment-resisting pile section) [26]. Most of the current design methods, such as JRA [27], NEHRP [28], IS:1893 [29], and Eurocode 8 [30] focus on bending strength of the pile to avoid bending failure due to lateral loads (combination of inertia and/or lateral spreading). When pile-supported structures are embedded deep enough



to move with the non-liquefiable layer, the displacement at the pile cap is equal to the ground displacement, and this condition yields maximum bending moments different from the free-end condition. Hwang et al. [31] evaluated the influence of liquefied ground flow on the pile behaviour. It was found that by increasing the slope angle of the liquefying ground, the shear force, the bending moment, and the lateral displacement of the pile increased. For the pile head condition, the bending moment also increased with depth. However, for the fixed pile head condition, the maximum bending moment of free head was about 1.5 times greater than that under the fixed pile head condition. The head supports for numerical analysis will be explained in Section 3.3.2.

## 2.2 Buckling failure

The second mechanism is the buckling instability under the interaction of axial and lateral loads, and piles acting as beam-columns under both axial and lateral loading [32–35]. Bhattacharya [35] argued this failure mechanism and suggested piles become laterally unsupported in the liquefiable zone during strong shaking which the axial load applied on pile and the soil around the pile liquefies loses of its stiffness and strength. Next, the piles act as unsupported long slender columns, and soils cannot support the corresponding action. Buckling failure depends on the geometrical properties of the member (i.e. slenderness ratio). The buckling mechanism is in the length of in touch with liquefied soil. The lateral loads for structural elements, due to slope movement increase lateral spread displacement demands, which in can cause plastic hinge to form and reducing the buckling load.

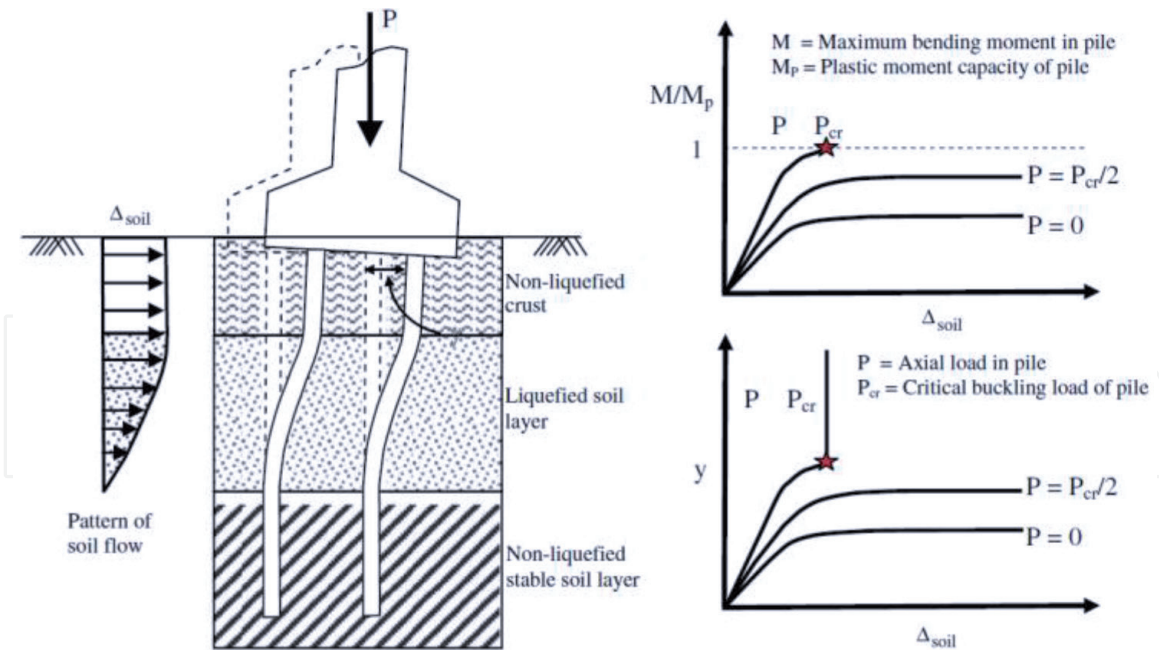
Extensive research has been carried out on the buckling instability of pile in liquefied soils. One early method for the stability of beams on elastic foundations proposed by Hetenyi [36] may be the base of the buckling analysis of pile foundations. The lateral loads, due to inertia or lateral spreading, could increase the lateral deflection of pile and thus reduce the buckling load [35]. On the other hand, there will always be confining pressure around the pile even if the soil has fully liquefied, and it could provide some lateral support to the pile and increase the buckling load [37]. As observed by Bhattacharya et al. [24], Knappett and Madabhushi [32], and Zhang et al. [38], buckling failure of the end-bearing pile normally occurs when the soil is fully liquefied. And pile buckling in partially liquefied soil would require a higher buckling load than that in the fully liquefied soil. In other words, when predicting the critical buckling load of pile in liquefiable soils, only the soil that has fully been liquefied needs to be considered. Zhang et al. [38] found that the critical buckling load of piles in liquefied soils increases with the increase of soil relative density and flexural rigidity of the pile and decreases with the increase of initial geometric imperfections of the pile and pier height. Shanker et al. [39] proposed an analytic method to predict the critical buckling load of pile under partial to full loss of lateral support.

## 2.3 Dynamic failure (bending–buckling)

A collapse of pile-supported structures in liquefiable deposits may occur under the combined action of lateral load and axial load. Bhattacharya et al. [37] included the dynamics failure on the combined axial and lateral loads on a pile foundation. In this mechanism, piles are subjected to both axial and lateral loads during seismic shaking and piles act like beam-column members (**Figure 6**).

As a result of this combination (axial- and lateral-loading) on piles during a seismic liquefaction-induced event, the influence of the axial load,  $P$ , in piles causes a loss of lateral stiffness ( $y$  is the lateral displacement) until the axial load





**Figure 6.**  
Schematic of the effect of bending–buckling interaction on the response of pile foundation [40].

approaches the critical value ( $P=P_{cr}$ ). The loss of lateral stiffness in association with the axial load (i.e., pile deflection),  $\Delta$ , is dominated by the excessive moment caused by the  $P$ - $\Delta$  effect (see Section 3.3.1). Subsequently, the large deflection of the beam may then induce plasticity in the beam resulting in an early failure. The same failure point of the pile is also possible when bending moment reaches  $M_p$  and pile continues to deflect without any additional loading.

Dash et al. [40] investigated the importance of bending–buckling interaction in seismic design of piles in liquefiable soils using numerical techniques. They concluded that if a pile is designed for bending and buckling criteria separately and safe for these individual design criteria, it may fail due to their combined effect. Recently, this is also suggested by Zhang et al. [41] to consider the buckling mechanism together with the effect of lateral load. It is hence important for the designers to consider a possible boundary for safe design to avoid failure of the pile.

### 3. Dynamic behaviour of pile

A variety of design procedures have been adopted by design guidelines and codes for assessing the behaviour of piles in liquefiable ground. Dynamic loading during earthquake be superimposed onto the working loads of the piles. Predicting seismic response of pile foundations in liquefied soil layers is much more complex due to uncertainties in the mechanisms involved in soil–pile–superstructure interaction (different dynamic loads, the stiffness and shear strength of the surrounding soil and pore water pressure generation). In practice, different design procedures have been used for the seismic design of pile-supported structures. The Japanese Highway Code of Practice (JRA) [27], for example, advises the practicing engineers to consider both of the loading conditions mentioned above. However, it suggests a separate bending failure check for the effects of kinematic and inertial forces. Similarly, BS EN ISO 2008 [30] advises pile design against bending due to inertial and kinematic forces arising from the deformation of the surrounding soil. In the event of liquefaction, Eurocode 8 also suggests that “the side resistance of soil layers

that are susceptible to liquefaction or to substantial strength degradation shall be ignored". The NEHRP [28], on the other hand, focuses on the bending strength of the piles by treating them as laterally loaded beams and assuming that the lateral load due to inertia and soil movement causes bending failure.

Since the mid-1960s, significant research has been conducted to understand the dynamic behaviour of pile foundations in liquefiable soils using various experimental techniques as well as various numerical modelling methods. These investigations can be divided into three categories: field observations (case histories), laboratory tests (dynamic centrifuge experiments, shaking table tests and full-scale field tests), and numerical modelling (Winkler analyses with linear-elastic or hysteretic soil behaviour, finite-element analyses). These will be discussed in detail in the following subsections.

### 3.1 Field observations (case histories)

This section provides a brief review of case histories. This can help to appropriately understand the phenomena involved and to identify important aspects of pile-soil-interaction behaviour. These case histories are primarily from the past 50 to 60 years, which describes the observed of some of the damaged piled foundations from the literature (see **Table 1**).

Iwasaki [51] reported the results of investigations on seismic damages to highway bridges during major eight earthquakes in Japan (occurred in 1923 to 1983). Their observation described that many of reinforced concrete buildings, highway bridges and other structures sustained considerable damage due to liquefaction of sandy soils (e.g., Showa Bridge 1964, Yuriage Bridge 1978, Shizunai Bridge 1982, Gomyoko Bridge 1983). The Showa bridge collapse has been a case history of interest in many publications and it was as an iconic example of the detrimental effects of liquefaction-induced lateral spreading on the ground. Hamada [2] argued that a more plausible explanation could be offered based on the ground displacements suffered due to liquefaction induced lateral spreading. In this respect, the JRA code [27] tried to formalise this research and presented methods of estimating the loading due to lateral spreading ground on pile foundations. This problem was revisited by Yoshida et al. [52] and they collated a number of eye-witness accounts to establish the timing of the bridge collapse as well as the lateral spreading of the river banks. It was suggested that lateral spreading of the surrounding ground started after the bridge had collapsed. Madabhushi and Bhattacharya [21] reanalysed the bridge and showed that lateral spreading hypothesis could not explain the failure of the bridge. A similar explanation was reported by Kerciku et al. [53]. As a final remark, Bhattacharya et al. [12, 54] and Mohanty et al. [55] suggested that the Showa Bridge could have collapsed because of bending, buckling, and combined action of bending of pile foundations.

The Niigata Family Court House building was a four-storey building constructed on concrete pile foundations. Hamada [2] suggested that one pile suffered relatively modest damage, as it did not penetrate into the deeper, non-liquefied ground. Madabhushi et al. [56] concluded that the laterally spreading ground around the piles caused the observed distress in these piles.

Further example on the probability of identifying collapse mechanisms is the Kandla Port one of the largest ports in India, located in the western state of Gujarat. Following the Bhuj earthquake of 2001, there was some damage to the port facilities [49]. Dash et al. [57] used conventional analysis of a single pile or a pile group to predict collapse. They concluded that the foundation mats over the non-liquefied crust shared a considerable amount of load of the superstructure and resisted the complete collapse of the building.

| Case history                          | Earthquake event | Magnitude (M <sub>L</sub> ) | Pile type                     | Pile diameter (m) | Pile length (m) | Pile length in liquefiable soil (m) | Pile performance   | References               |
|---------------------------------------|------------------|-----------------------------|-------------------------------|-------------------|-----------------|-------------------------------------|--|--------------------------|
| 10 storey-Hokuriku building           | 1964 Niigata     | 7.5                         | Reinforced concrete           | 0.4               | 12              | 5                                   | Cracking on piles  | Hamada [2]               |
| Showa bridge                          | 1964 Niigata     | 7.5                         | Steel tube                    | 0.6               | 25              | 19                                  | Collapse due to buckling of pile foundations   | Hamada [2]               |
| Landing Road bridge                   | 1987 Edgecumbe   | 6.3                         | Square Reinforced concrete    | 0.4               | 9               | 4                                   | Minor cracking at pile heads   | Berrill et al. [42]      |
| 10-story Hotel east of State Highway  | 1989 Loma        | 7.0                         | Prestressed concrete piles    | 0.36              | 12              | 5                                   | Piles performed well.  | Adib et al. [43]         |
| Buildings on Port Island              | 1995 Kobe        | 6.9                         | Reinforced concrete           | 0.45              | 9<br>9          | 4<br>4                              | Severe damage at liquefiable/non-liquefiable interface   | Fujii et al. [44]        |
| Building near the Higashi Kobe bridge | 1995 Kobe        | 6.9                         | Prestressed concrete pile     | 0.45              | 10              | 6                                   | Failures at about 1-m below the reclaimed fill, many cracks between 3 and 6 m below ground surface | Fujii et al. [44]        |
| 14 storey building in American park   | 1995 Kobe        | 6.9                         | Reinforced concrete           | 2.5               | 33              | 12.2                                | Damage most common at pile heads.  | Tokimatsu et al. [45]    |
| Hanshin expressway                    | 1995 Kobe        | 6.9                         | Reinforced concrete           | 1.5               | 41              | 15                                  | Heavy damage including collapse  | Ishihara [46]            |
| NHK and NFCH buildings                | 1964 Niigata     | 7.5                         | Reinforced concrete           | 0.35              | 16              | 8                                   | Severe damage at interfaces between liquefied and non-liquefied soils.                             | Hamada [2]               |
| Buildings on Fukaehama                | 1995 Kobe        | 6.9                         | High-strength concrete piles, | 0.35–0.6          | 20              | 16                                  | Pile failures at the interface between liquefied and non-liquefied layer.                          | Tokimatsu and Asaka [47] |

| Case history                      | Earthquake event | Magnitude ( $M_L$ ) | Pile type                  | Pile diameter (m) | Pile length (m) | Pile length in liquefiable soil (m) | Pile performance  | References             |
|-----------------------------------|------------------|---------------------|----------------------------|-------------------|-----------------|-------------------------------------|---|------------------------|
| Building on Higashinada-ku        | 1995 Kobe        | 6.9                 | Prestressed concrete piles | 0.4               | 12              | 5                                   | Cracks on piles at near the pile head, in the middle and the bottom of the liquefied layer. | Tokimatsu et al. [48]  |
| Harbour Master's building, Kandla | 2001 Bhuj        | 7.7                 | Reinforced concrete        | 0.4               | 25              | 15                                  | Piles performed poor.   | Madabhushi et al. [49] |
| Miaoziping Bridge                 | 2008 Wenchuan    | 7.9                 | Reinforced concrete        | —                 | 100             | 45                                  | One approach bridge span collapsed.   | Kawashima et al., [50] |

**Table 1.**  
*Summary of case histories on pile foundation performance in past earthquakes (adapted from Bhattacharya et al. [33]).*



Bhattacharya et al. [33] collated 15 case histories of pile earthquake performance and classified them according to their Euler buckling load when the soil was fully liquefied. In most of the cases where the axial load in the pile was 50% or more of the buckling loads, the foundation suffered significant damage.

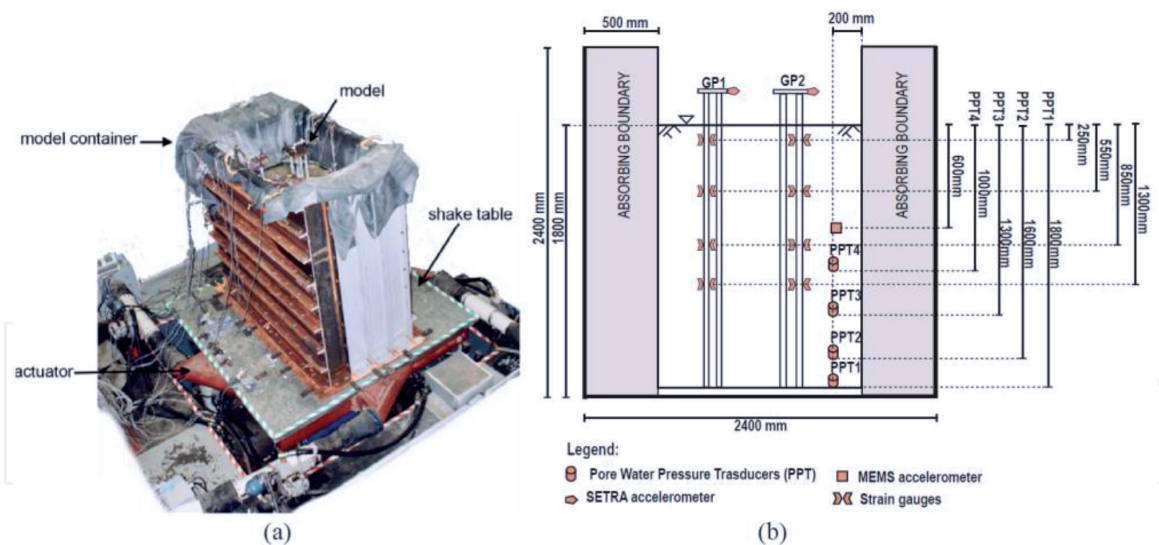
Based on these observations, the failure of pile foundations occurred in both laterally spreading ground and in level ground where no lateral spreading would be anticipated. The cracks observed were near the bottom and at interfaces between liquefied and non-liquefied layers and often at the pile head. Additionally, severe damage had also formed at the boundaries of the liquefiable and non-liquefiable layers and at various depths between. The plastic hinge formation occurred at the boundaries of the liquefiable and non-liquefiable layers and at various depths.

### **3.2 Laboratory testing**

Laboratory studies are also critical to elucidate the failure mechanisms and the behaviour of soil–pile interaction in liquefiable soils and its relevant fundamental parameters such as relative density, confining pressure, shear strength, frequency content and amplitude, damping ratio, pile bending moment, and deformed shape of the soil profile. Therefore, while the aim of the work presented in this section is to review and provide well-interpreted field of the dynamic response of piles by different physical model tests that can be used to evaluate analytical procedures and design methods. Many studies have investigated the seismic response of pile, soil and superstructure using shake-table experiments [58–63], dynamic centrifuge experiments [19, 64, 65], and full-scale field tests that utilise blast-induced liquefaction [66, 67]. The requirements for a model container for carrying out seismic soil-structure interactions (SSI) at 1-g (shaking table) and N-g (geotechnical centrifuge at N times earth's gravity) are well introduced in [68].

#### *3.2.1 Shaking table tests*

Iwasaki et al., [69] used a shaking table to estimate the liquefaction potential by using fundamental properties of the soil. Meymand [70] carried out a set of soil-pile interaction tests using the large shaking table operated by U.C. Berkeley. It was reported that damping for the single piles computed from 10 to 20%. Yao et al., [71] highlighted that the transient state prior to soil liquefaction was important in the design of piles due to dynamic earth pressure showed peak response in this state. Tokimatsu et al. [72] investigated pile under the combination of inertial and kinematic forces and reported that the pile foundation response depends on the time period of the ground as well as the superstructure. Cubrinovski et al. [73] discussed the behaviour of pile foundations under lateral spreading. Chau et al. [74] suggested that seismic pounding between the laterally compressed soil and the pile near the pile cap level can be one of the probable causes of pile damages. Motamed & Towhata [75] carried out a series of 1 g small-scale shake table model tests on a  $3 \times 3$  pile group located behind a sheet-pile quay wall. It reported that fixed-end mitigating sheet pile can reduce the bending moment of pile. This is depended on the pile position within the group [76]. Gao et al. [61] studied the dynamic interactive behaviour of soil–pile foundations in liquefying ground under different shaking frequency and amplitude. They reported that the frequency of motion does not have a significant effect on the pile and soil response; however, these responses depend on the shaking amplitude. Besides, Lombardi & Bhattacharya [34] concluded that natural frequency of pile foundation decreases due to liquefaction; also they found that the damping ratio may increase due to liquefaction in excess of 20% (**Figure 7**).



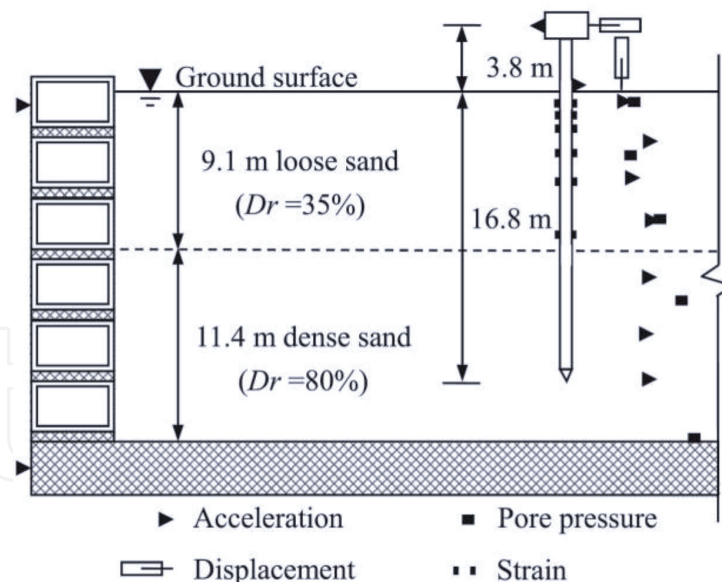
**Figure 7.**  
 (a) The shaking table test and (b) elevation view of test [34].

A similar observation was reported by Tang and Ling [62] and Tang et al. [63], which conducted a shaking table experiment to investigate the dynamic behaviour of a reinforced-concrete (RC) elevated cap pile foundation during (and prior to) soil liquefaction. These works indicated decreasing the frequency and increasing the amplitude of earthquake excitation. Next to this, Chen et al. [77] suggested that the seismic response of the soil and structure depends on input motion with richer low frequency components. On the other hand, Su et al. [58] document thicker pile having higher displacement. Likewise, the work performed by Liu et al. [78] was demonstrated that pile group bending moment was able to increase dramatically as the diameter increases. Four large laminar-box shaking table experiments used by Ebeido et al. [59] to examine pile response due to the mechanism of liquefaction-induced lateral spreading. They concluded that the highest pile lateral loads occurred at the initial stages of lateral deformation. Zhang et al. [41] reported that a pile collapsed due to buckling instability, which happened after the soil fully liquefied.

### 3.2.2 Centrifuge tests

Similar to the shaking table test, centrifuge test enables to address liquefaction, lateral spreading, and their effect on pile foundations by the simulation of gravity-induced stress conditions in scale models of soil structures at  $N$  times earth's gravity. Conceptually, this technique consists of the linear dimensions in the model soil by a factor  $1/N$  and the confining stress is identical by a factor of unity. Thus, scale models law to simulate the behaviour of full-scale earth structures by reduced scale and provide data applicable to full-scale problems [68].

Several experimental studies have investigated of piles subject to liquefaction by using centrifuge method testing. McVay et al. [79] analysed the behaviour of the laterally loaded on pile group in sand with different pile group models. They reported that by changing the size of the group, there was no change in the group's lateral resistance, but only was a function of row position. Wilson [80] and Wilson et al. [81] performed dynamic centrifuge tests on the soil-pile interaction (**Figure 8**), which was directly obtained from the observed  $p$ - $y$  response through back analysis of a single pile. This analyse presented the first dynamic characterisation of  $p$ - $y$  behaviour of pile foundations in liquefying sand.



**Figure 8.**

*Layout of the centrifuge test setup by Wilson [80].*

Abdoun et al. [82] estimated the peak subgrade reaction values in liquefiable sand from centrifuge tests and concluded that the largest free head pile bending moment occurred at the boundary between the liquefied and non-liquefied strata. Similarly, a large displacement due to the liquefaction of the backfill soil was observed between the rubble mound and the bearing stratum, which produced a large bending moment at the top of the pile [83]. Brandenburg et al. [84] conducted various aspects of bending failure mechanism. In contrast, Bhattacharya et al. [85] proposed buckling instability failure mechanism as a new theory of pile failure mechanism verified by dynamic centrifuge tests. Another experiment concluded that an increase in excess pore pressure around and beneath end-bearing piles might induce the instability failure caused by liquefaction [86]. A similar observation was reported to clarify the buckling instability failure mechanism by Knappett and Madabhushi [32]. Recently, Garala et al. [19] conducted unexplored aspects of kinematic pile bending.

### 3.3 Numerical modelling

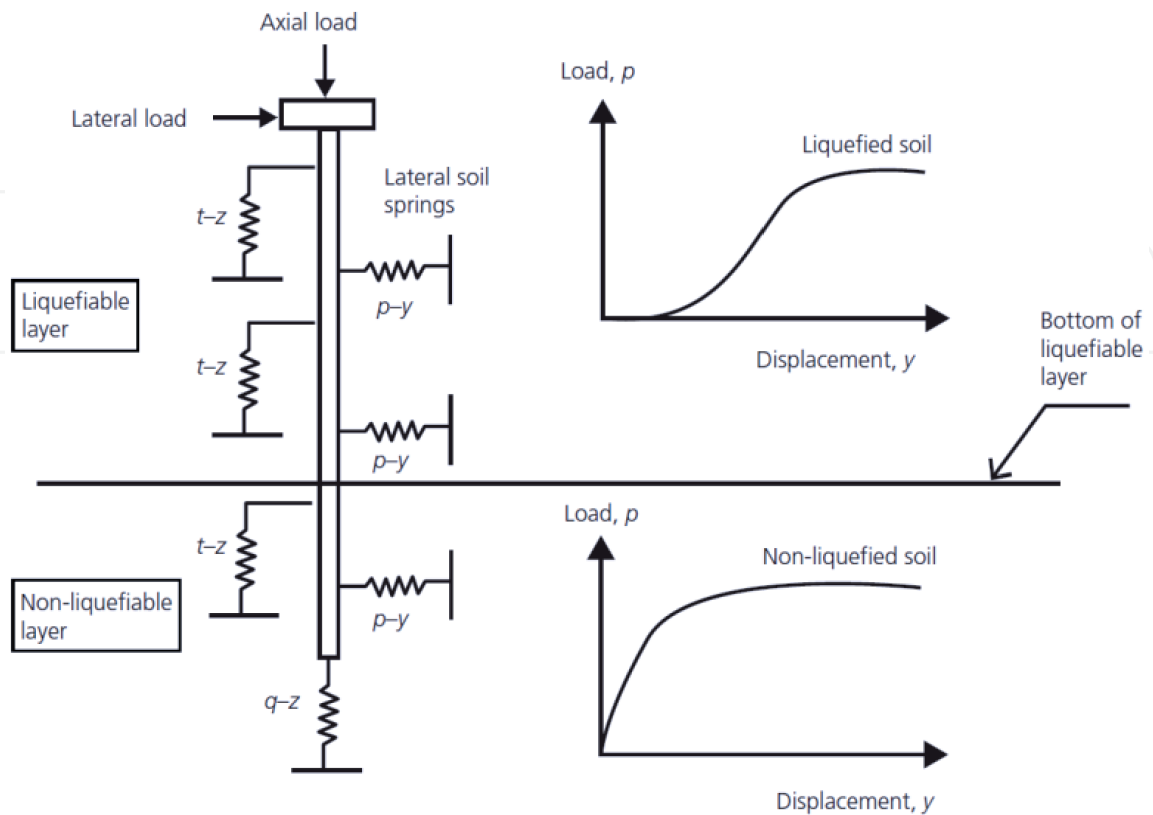
The numerical simulation tools have been prominent for analysing liquefaction problems in the light of potential disadvantages of physical models used in experimental simulation. This section presents different numerical platforms used in modelling of pile foundations under dynamic loading and their capabilities and limitations. Review of the recent relevant works delineates the important aspects of the seismic analysis of piles in liquefiable soils.

Numerical modelling can be divided into three categories: Beam on nonlinear Winkler Foundation (BNWF) approach with the proposed p-y curves, two-dimensional numerical modelling and the full three-dimensionality of model. Due to computationally complex and time-consuming of two- and three-dimensional numerical modelling, most of the researchers prefer to use the pseudo-static analysis based on Winkler method for the seismic analysis of pile foundations. Winkler models are approximately capable of predicting maximum lateral displacement and maximum bending moment of pile foundations in liquefied soils. However, it is not able to simulate the prototype model accurately because it is difficult to estimate the accurate values for the springs and dashpots coefficients, which considerably change over time, especially during strong shaking.

3.3.1 *p-y* curves

The beam-on-nonlinear-Winkler-foundation (BNWF) method (also known as *p-y* method or Winkler method) is widely used in the modelling of soil–pile–structure interaction due to its simplicity in modelling and computational efficiency. This method is based on the hypothesis that the reaction exerted by the soil at a given depth on the pile shaft is proportional to the relative pile–soil lateral deflection. In the BNWF method, soil–pile interactions are modelled by a set of nonlinear soil springs, whereas the horizontal responses are analysed using *p-y* spring, for simulating shaft friction controlling vertical loading characteristics by *t-z* spring and end-bearing at the bottom of the piles responses are represented via the *q-z* spring (**Figure 9**). Each spring can be defined by means of a non-linear relationship between the soil reaction (per unit length of the pile) *p* and the corresponding relative soil–pile displacement.

This method is based on the beam on elastic foundation approach of Hetényi [36] and Winkler [88]. The *p-y* curves have been used to model the reaction of the foundation with consideration of inertial effects and seismic soil–pile interaction. Guidelines for the *p-y* curves as prescribed by current codes of practice are based on the works of Matlock [89] for soft clay, Reese [90] for cohesionless soils and Cox et al. [91] and O’Neill and Murchison [92] for sand, published by API [93] and DNV [94]. A different approach based on the assumption that the liquefied soil behaves like soft clay applied to account for the effects of liquefaction on the *p-y* curves, which is known as “residual strength” method [95]. However, applications of these curves were developed from a number of field tests with relatively few inherent limitations. Therefore, numerous works have been carried out to evaluate *p-y* curves for laterally loaded piles in liquefiable soils, such as Dobry et al. [96], Yasuda et al., [97], Sivathayalan and Vaid [98] and Rollins et al. [99]. Subsequently, the *p-y* method was extended to liquefiable soils by applying a “*p* multiplier” [96, 100], which is a reduction factor ( $m_p$ ). Combining the force- and displacement-based methods,



**Figure 9.**  
*Method of pile analysis using *p-y* curves, Dash et al. [87].*



Cubrinovski et al. [100] proposed to use limit pressures for non-liquefied crust layers and linear springs with a “stiffness degradation factor” (known as the  $p$ -multiplier) for liquefied layers during liquefaction-induced lateral spreading. Several analyses of the full-scale tests [81, 101–103] conducted and observed the actual shape of post-cyclic stress–strain response of liquefied soils. They suggested an S-curve shape of the “ $p$ – $y$ ” curve for liquefied soil. Similarly, the post-liquefaction behaviour of sands observed in element tests by [104, 105]. Lombardi et al. [106] and Dash et al. [107] adopted a new set of  $p$ – $y$  curves that can be obtained by modifying the conventional  $p$ – $y$  curves (for non-liquefied soils) in such a way that replicates the strain hardening behaviour with practically-zero stiffness at low strain.

### 3.3.2 Two- and three-dimensional numerical modelling

To gain further insight into the field of simulation of the soil–pile interaction, the general design process is presented here. Finn and Fujita [108] developed and recently reviewed [109] an approximate method for nonlinear, three-dimensional analysis of pile foundations using PILE-3D. Constitutive models for simulating the nonlinear behaviour of pile in liquefiable soil have been proposed and typically implemented through the finite-element using two- or three-dimensional numerical modelling. Some of the more popular computer programs used are FLAC (Itasca), DIANA (DIANA Analysis) and PLAXIS. Many researchers have been using the Open System for Earthquake Engineering Simulation (OpenSees) (Mazzoni et al., [110]) as a ground response analysis tool. However, the main challenge of numerical modelling remains the simultaneous numerical prediction of accelerations, generation and redistribution of excess pore pressures, and the resulting of deformations [111]. Yu et al. [112] suggested that the finite element method (FEM) based on solid mechanics can accurately simulate the soil behaviour (for the initial stage) and the smoothed particle hydrodynamics (SPH) method in the framework of fluid dynamics is more suitable (for the flow stage). In this regard, Finite difference numerical models were developed using ABAQUS/Explicit, SAP2000, FLAC 3D, SANISAND and PDMY02 models or FEM using DBLEAVES code. The bounding surface constitutive model simple anisotropic sand (SANISAND) developed by Dafalias and Manzari [113] and implemented in OpenSees by Ghofrani and Arduino [114], was utilised to represent the behaviour of the liquefiable soil layer. Pressure-Dependent multiyield surface constitutive model version 02 (PDMY02) developed and implemented in OpenSees by Elgamal et al. [115] and Yang et al. [116] to simulate soil behaviour. Ramirez et al. [111] compared the predictive capabilities of PDMY02 and SANISAND platforms. Furthermore, Drucker-Prager [117] and Mohr-Coulomb plasticity [118] models are also soil constitutive modelling approaches in 3D analyses of soil-foundation systems.

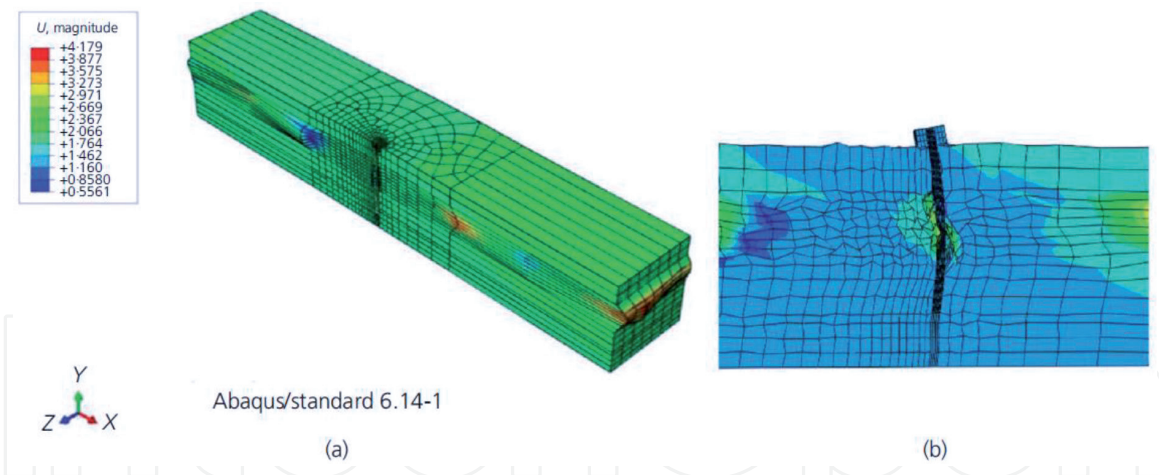
Some of the recent numerical studies on pile foundations in liquefiable soil are listed in **Table 2**.

A fully coupled formulation ( $u$ – $P$  or  $u$ – $P$ – $U$ ) has been used to analyse soil displacements and pore water pressures [136]. The  $u$ – $P$  formulation (called EightNodeBrick– $u$ – $P$  element in OpenSees framework) is the simplification of the  $u$ – $p$ – $U$  approach, which captures the movements of the soil skeleton ( $u$ ) and the change of the pore pressure ( $P$ ). More detail about description, formulation and implementation of this theory can be found in [120–122]. **Figure 10** is an example of three numerical modelling of pile in liquefiable of soil.

BNWF analyses of piles and pile groups have been exclusively employed for modelling case histories. Dash et al. [57] created 3D non-linear model of Tower of Kandla Port by using a finite element program SAP2000. A good agreement between the analytical and field observations analysis was reported. Similarly, Dash et al. [40] investigated the bending-buckling mechanism by exploring the Showa

| References                      | Analysis type     | Test type     | Soil type   | Pile configuration | Pile type           | Pile head condition |
|---------------------------------|-------------------|---------------|-------------|--------------------|---------------------|---------------------|
| Cubrinovski et al. [119]        | 3D DIANA          | Shaking table | Sand        | 3 × 3              | Stainless steel     | Fixed               |
| Cheng and Jeremic' [120]        | 3D (u-p-U) FEM    | Full scale    | Sand        | Single             | Aluminium           | Free                |
| Dash et al. [57]                | 3D (p-y) SAP2000  | Full scale    | Clay soil   | 3 × 4              | Concrete            | Fixed               |
| McGann et al. [117]             | 3D (p-y) FEM      | Full scale    | Sand        | Single             | Reinforced Concrete | Free and Fixed      |
| Rahmani and Pak [121]           | 3D (p-u) OpenSees | Centrifuge    | Sand        | Single             | Concrete            | Free and Fixed      |
| Wang et al. [122]               | 3D (p-u) OpenSees | Centrifuge    | Sand        | Single             | Concrete            | Fixed               |
| Valsamis et al. [123]           | p-y NASTRAN       | Full scale    | Sand        | Single             | Concrete            | Fixed               |
| Bhowmik et al. [124]            | 3D Abaqus         | Full scale    | Sandy clay  | Single             | steel               | Fixed               |
| Wang and Orense [125]           | 2D (p-y) OpenSees | Centrifuge    | Sand        | Single             | Steel pipe          | Fixed               |
| Finn [109]                      | PILE3-D           | Full scale    | Sand        | 4 × 4              | Concrete            | Free and Fixed      |
| Sextos et al. [126]             | 3D (p-y) SAP2000  | Full scale    | silty sands | 2 × 2, 4 × 4       | Reinforced concrete | Fixed               |
| Hamayoon et al. [127]           | 3D FEM            | Shaking table | Sand        | 3 × 3              | Aluminium pipes     | Fixed               |
| Li and Motamed [128]            | 2D (p-y) OpenSees | Shaking table | Silica sand | 3 × 3              | Steel               | Fixed               |
| Lombardi and Bhattacharya [129] | 2D (p-y) SAP2000  | Shaking table | Sand        | Single, 2 × 2      | Aluminium           | Free and Fixed      |
| Rostami et al. [118]            | 3D (p-y) Abaqus   | Shaking table | Sand        | Single             | Reinforced concrete | Free and Fixed      |
| Zhang et al. [130]              | 3D (p-u) OpenSees | Centrifuge    | Sand        | Single             | Concrete pipe pile  | Free                |
| Wang et al. [131]               | 2D (p-y) OpenSees | Centrifuge    | Sand        | 3 × 2              | Concrete            | Fixed               |
| López Jiménez et al. [132]      | 3D Flac SANISAND  | Full scale    | Sand        | 3 × 4              | Concrete            | Fixed               |
| Li et al. [133]                 | 3D Flac           | Full scale    | Sand        | 3 × 3              | XCC pile            | Fixed               |
| Zhang et al. [41]               | 2D (p-y) OpenSees | Shaking table | Sand        | Single             | Aluminium           | Free                |
| Kazemi Esfeh and Kaynia [134]   | 3D Flac SANISAND  | Centrifuge    | Ottawa sand | OWT                | Monopile            | Fixed               |
| Rajeswari and Sarkar [135]      | 3D (p-u) OpenSees | Full scale    | Nevada sand | Single             | Concrete            | Fixed               |

**Table 2.**  
*Summary of recent numerical studies on pile foundation in liquefiable soil.*



**Figure 10.**

*An example of three numerical modelling of pile in liquefiable of soil [118]. (a) Deformed shape of model of unimproved soil with 3 m thickness of liquefiable soil; (b) pile deformation.*

bridge pile failure in 1964 Niigata earthquake. In this context, McGann et al. [117] proposed a simplified procedure for the analysis of single piles subject to lateral spreading based on a parametric study. Moreover, the values of degradation factors of p-y curves in liquefiable soils computed in 3D FEM using OpenSees [137].

Two-dimensional models have been used to study soil-structure interaction in the majority of the numerical analyses using OpenSees. Halder and Babu [138] examined the failure mechanism in piles and observed the failure mode was greatly dependent on the depth of the liquefiable soil layer. Zhang and Hutchinson [139] proposed a strategy by integrating the calculated plastic curvature at all integration points along the pile shaft. It was reported that the plastic hinge length of piles extending through liquefiable layers is about 1.4 times larger than that of non-liquefiable conditions. However, the location of plastic hinges can be affected by a variety of factors, such as material properties, pile length and thickness of the liquefied soil layer [118]. Wang and Orense [125] used a 2DBNWF finite element model implemented via Open Sees to analyse the response of raked pile foundations in liquefying ground. Bhowmik et al. [124] investigated the nonlinear behaviour of single hollow pile in layered soil subjected to varying levels of horizontal dynamic load. It found that separation of pile from the surrounding soil considerably affects the resonance frequency and amplitude of the pile foundations. Finn [109] compared different factors that can take into the behaviour of pile foundations during earthquakes in both liquefiable and non-liquefiable soils. Through a 2D nonlinear dynamic finite element (FE) modelling, Li and Motamed [128] presented a large-scale shake table test. It demonstrated that the FE model was able to reproduce the shaking table model behaviour with reasonable accuracy.

Three-dimensional analysis has become more common in the analysis of a full behaviour soil–pile–superstructure system with the greatest potential for accurately and certainly using either solid elements or beam-column elements. Zhang and Liu [140] performed a total of 90 3D finite element analyses using ABAQUS/Explicit to investigate the seismic response of different pile-raft-superstructure systems constructed on soft clay subjected to far-field ground motions. Zhang et al. [130] reported a good agreement between the numerical and the experimental data. Jiménez et al. [132] analysed the effects of this interaction, numerical models with a 3-storey reinforced concrete building using Flac 3D. Esfeh and Kaynia [134] used the software FLAC3D and the SANISAND constitutive model to conduct the non-linear dynamic analyses for Offshore Wind Turbines. It was found that SANISAND

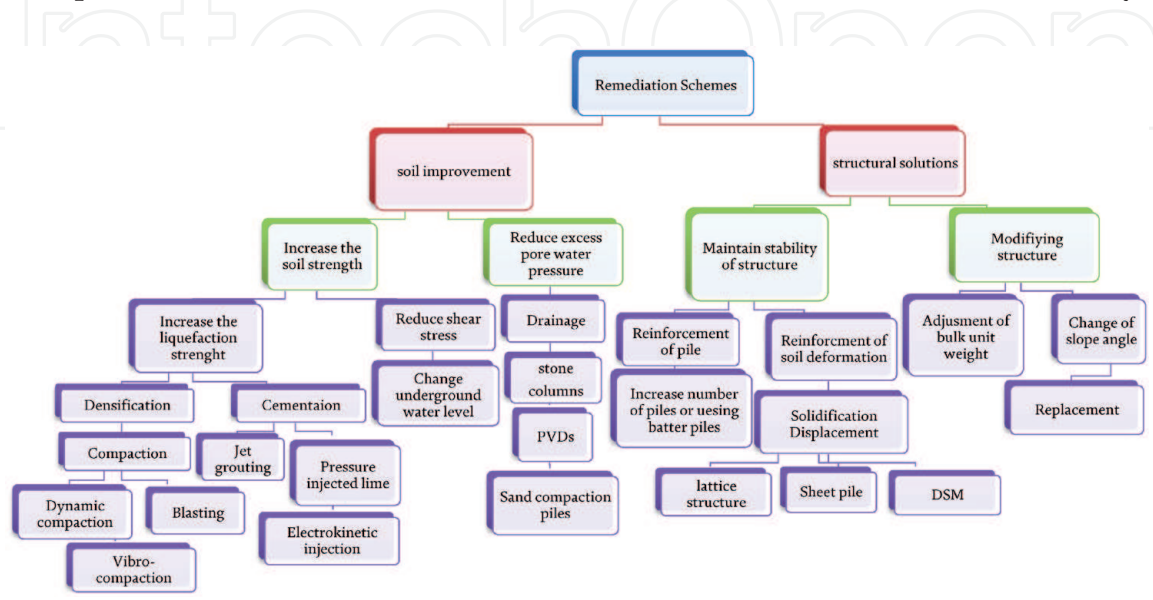
model is capable of simulating the pore pressure generation in the free-field as observed in a recent centrifuge test. Recently, Manzari et al. [141] compared 11 sets of Type-B numerical simulations with the results of a selected set of centrifuge tests conducted in the LEAP- 2017 project. They obtained a good match trends in a number of numerical simulations with their experimental results.

4. Remediation schemes

Various ground improvement techniques have been developed for remediation of piled foundations in liquefiable soils over the past few decades. New techniques are introduced either to prevent liquefaction or to minimise the resulting settlements. Piled foundations of existing buildings are often difficult to access for retrofitting and, in addition, any procedure must ensure that the superstructure is not damaged during remediation [142]. Remediation of liquefiable soils for pile foundations needs to meet the several design performances required [143]. First, the most appropriate method for remediation should be selected for a specific portion or area (e.g. ground improvement). Next, the effective of the remedial measure should be appropriately determined to eliminate liquefaction and the associated ground deformations (lateral spreading and settlement). Moreover, the economic viability of the scheme should be evaluated to reduce or avoid potential structural damage caused by liquefied soil. The most common remediation techniques for pile foundations founded in liquefiable soils are summarised in **Figure 11**.

Installation of drains (e.g., using stone columns, sand compaction piles, pre-fabricated vertical drains (PVDs)) can prevent or delay liquefaction by enhancing dissipation of excess pore pressures and preventing void redistribution and the formation of a water lens below a low permeability crust [144–147]. However, deviatoric deformation and volumetric strains due to localised drainage during shaking significantly influenced the effectiveness of drains [148].

A number of densification techniques (e.g., using deep dynamic compaction, vibro-compaction, compaction piles) have been widely studied, because these techniques are relatively simple and practical, and the resulting remediation success can be easily verified by using in-situ penetration techniques [149–152]. However, Rayamajhi et al. [153, 154] reported that the densification and drainage techniques of improvement are often ineffective while the soil-cement columns were relatively



**Figure 11.**  
*Summary of the most common remediation techniques.*



ineffective in reducing the potential for liquefaction triggering in saturated silty soils. This method also may not reduce permanent and transient tilt [148]. In this regard, Olarte et al. [155] compared drainage, densification and reinforcement with in-ground structural walls techniques. It was reported that both drainage and densification can reduce excess pore pressures and permanent foundation settlement and the performance of the reinforcement wall depended on the properties of the earthquake motion.

The soil stiffness of the liquefiable layer must be chosen carefully for a reliable analysis because it significantly affects the pile response.

Solidification methods (e.g., using deep soil mixing, jet grouting, walls, deep soil-mixed (DSM), sheet piles or lattice-shaped walls) are promising ground improvement methods which have proven to be effective in stabilising potentially liquefiable soil at several sites during earthquakes not only in controlling lateral spread but also in preventing liquefaction [20, 156–159]. This method was confirmed by a three-dimensional finite difference model using FLAC3D [157]. Moreover, the foundation ground of the 14-storey Meriken Park Hotel was improved using the deep cement mixing (DCM) method and it had good performance and survived the Kobe earthquake without damage [160]. In addition, authors [20] proposed a seismic requalification methodology of a pile-supported structure based on numerical simulations. It was recommended to use cementation or/and lattice structure techniques for reducing liquefaction hazard.

## 5. Summary

Procedures for identifying pile failure mechanisms due to liquefaction have been developed by reviewing of case histories data, experimental and numerical techniques. Examination of dynamic behaviour of pile foundations in seismically liquefiable soils led to the following practical recommendations:

- In order to consider a possible boundary for safe design and avoid failure of the pile, it is suggested to consider the buckling mechanism together with the effect of lateral load.
- Lessons from case histories reported that plastic hinge formation occurred at various locations which cannot be predicted with certainty. However, it was emphasised that, pile foundations could have collapsed because of bending, buckling, and combined action of bending of pile foundations.
- The review of results from various physical modelling techniques indicated that the dynamic behaviour of pile foundations depends not only on the site-specific characteristics (e.g., frequency of input motion, the amplitude, etc.), but also on the dynamic (modal) characteristics of the system (fixity of the pile head, the pile tip, pile position within the group, thickness of liquefying soil layer, etc.), which considerably affect piles performance in liquefied grounds.
- Overall, numerical modelling is capable of representing the most important aspects of pile failure mechanisms. The results obtained from different studies reported a good match in a number of numerical simulations compared to their experimental observations. In this respect, 3D analysis, in particular, favourably performed in the pursuit of identifying pile failure mechanisms.

- Various remediation techniques are available to mitigate the pile foundations on deposits of liquefiable sand. Nevertheless, despite being convenient for this purpose, it should be noted that considering one or more methods combined can provide economical solutions for liquefaction remediation problems. It would be rational to consider the remediation methods that have been implemented at many sites or tested/modelled by few large-scale earthquakes, which performed well. The above reasoning recommends using a combination of solidification methods.

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