Chapter 2: Literature review

2.1 Introduction

Over the last four decades since the 1964 Niigata and Alaska earthquakes, considerable research has been carried out to understand the failure mechanisms of pile foundations during earthquake-induced liquefaction. This research generated information through physical modelling, numerical modelling and the study of field case records, thereby improving the understanding of pile behaviour.

This chapter reviews the literature relevant to the aims and scope of this research work as mentioned in section 1.8. The review includes liquefaction, pile foundations and the current understanding of pile failure in areas of seismic liquefaction. Emphasis is given to the hypotheses in the current codes of practice. A brief review of design methods in the major codes of practice is presented. A brief historical development of pile design in the Japanese Code of Practice (JRA) is also outlined. The chapter ends with a summary and critical review of the current understanding of pile behaviour during seismic liquefaction.

2.2 Liquefaction

Liquefaction is one aspect of the behaviour of sandy soils that has engrained fear in geotechnical engineers for more than two generations due to its destructive effects, for example dam failures, slope failures etc. This behaviour of sandy soils has received great public attention after the 1964 Niigata earthquake where all kinds of modern infrastructure were destroyed due to this behaviour of soil (Ishihara, 1993). However, the study of liquefaction phenomena dates back as early as 1920 and remains a subject of debate and controversy in the entire soil mechanics community.

Castro and Poulos (1977) referred to Hazen (1920) as the first engineer to use the word “liquefies” in 1920 to refer the failure of the Calaveras dam in California. Ishihara (1993) in the Rankine lecture refers to Terzaghi and Peck (1948) coining the term “spontaneous liquefaction” to describe the phenomenon of sudden change of loose deposits of sand into flows much like those of viscous fluid, triggered by a slight disturbance.

This section of the literature review discusses the current theoretical framework of behaviour of soil that liquefies, citing typical triaxial test data. The necessary conditions for liquefaction and its manifestations have also been discussed in relation to the apparent meaning of the word in science or in common language.

2.2.1 Theoretical framework for behaviour of sandy soil

The Critical State concept developed at Cambridge by Roscoe, Schofield and Wroth (1958) provides a strong framework to describe the behaviour of soils. Three parameters, namely p' (mean confining stress), q (deviator stress) and v (specific volume) can define the state of a soil sample. The Critical State concept states that soil, if sufficiently distorted, will come into a well-defined critical state where it shears without any change in stress or volume (Schofield and Wroth, 1968). This state can be depicted as a line, in p'-q-v space, known as Critical State line.

The behaviour of granular soils under cyclic loading has been an active field of research since the 1964 Niigata earthquake. Element tests have been done using triaxial, simple shear, torsional cylinder apparatus and resonant column tests. A summary of some published work can be seen in Wood (1980). Among these works the studies carried out by Ishihara et al (1975) and by Luong and Sidaner (1981) are commonly cited and there is similarity between the two.

Luong and Sidaner (1981) studied the cyclic behaviour of drained and undrained samples of cohesionless soils and introduced the ‘Characteristic state line’ 'CL' similar to the definition of Critical State line in a q-p' plot. Ishihara et al (1975) defines a phase transformation line, which is similar to the characteristic state line. This line lies below the failure line of the soil (FL) and divides the cyclic behaviour of soil into two domains namely the subcharacteristic domain and the surcharacteristic domain in a q-p' plot as shown in Figure 2.1.

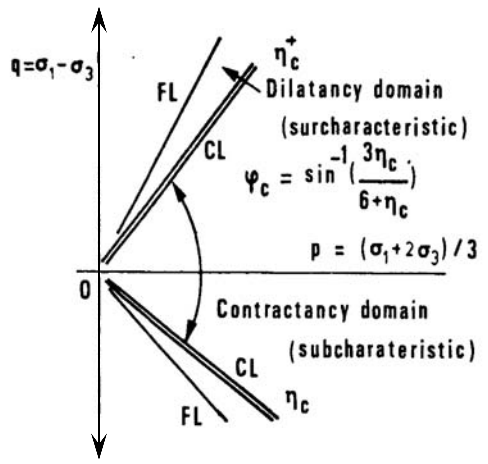


Figure 2.1: Characteristic state for granular material, Luong and Sidaner (1981).

In the subcharacteristic domain, granular aggregates tend to become more closely packed under drained conditions. Under undrained conditions, the pore pressures increase, causing the effective stress to decrease. On the other hand, in the surcharacteristic domain the aggregate dilates to looser packing under drained conditions. Under undrained conditions this will lead to the creation of suction pressures leading to an increase in effective stress.

During an earthquake, a soil element starting in the subcharacteristic domain will exhibit generation of positive pore pressures throughout the loading and the stress path will progress towards the origin until it reaches the characteristic state line, as seen in Figure 2.2. Once the line is reached on one side of the p' axis, pore pressure and strain development will accelerate and the stress path runs up and down like a butterfly wing passing through or near the origin. If the applied deviator is sufficient to hit both the characteristic state line on either side, a cycling of pore pressure at double the frequency of loading will be seen.

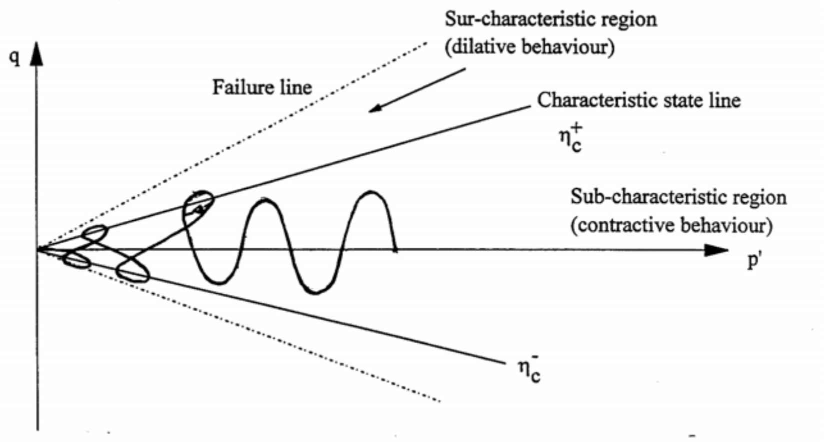


Figure 2.2: Schematic representation of the liquefaction stress path for cyclic loading, Peiris(1998)

2.2.2 Typical data of liquefiable soils under monotonic and cyclic loading in triaxial apparatus

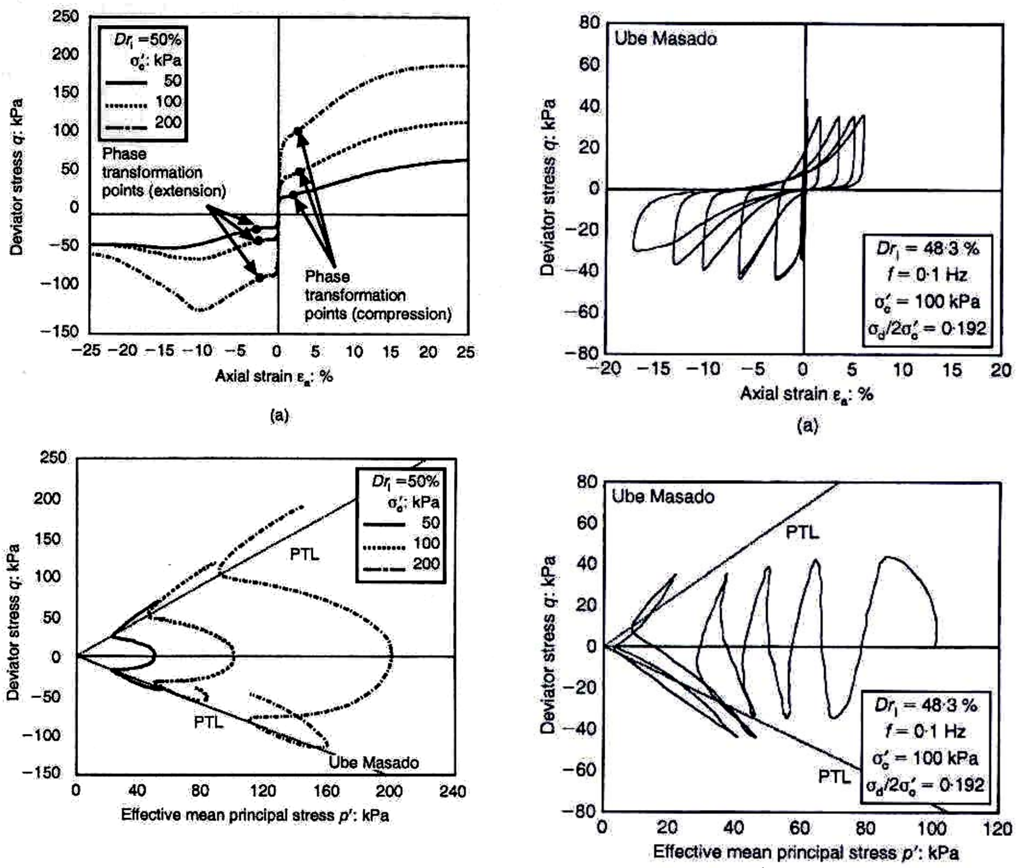
The observed behaviour of two types of soil in a triaxial apparatus (Hyodo et al 1998) will be discussed in this section.

The soils are Ube Masado and Shirasu, which have been extensively used as fill material for reclaiming land in Japan. Ube Masado is crushable highly angular decomposed granite soil used to reclaim land from the sea during the formation of Rokko and Port Island in Japan (Hyodo et al 1998). Structures resting on these soil deposits were badly damaged during the 1995 Kobe earthquake as the soil fully liquefied. Shirasu is a volcanic soil and is also used as fill material, Umehara et al (1975).

Details of the test set up and properties of the soil can be seen in Hyodo et al (1998). Relevant results are only discussed here.

2.2.2.1 Comparison between "undrained monotonic" and "undrained cyclic behaviour" of loose (approx 50%) Ube Masado soil

Figure 2.3 (a) shows the undrained behaviour of loose soil subjected to monotonic shear. From the effective stress path, it can be observed that there is an initial contraction followed by a point of phase transformation at approximately 3% axial strain. After this point axial compressive strain rapidly increased to 20% where a critical (steady) state is achieved.



(a) (b)

Figure 2.3: Behaviour of "Ube Masado" soil of 50 % relative density after Hyodo et al (1998); (a): under "undrained monotonic" loading; (b): under "undrained cyclic" loading.

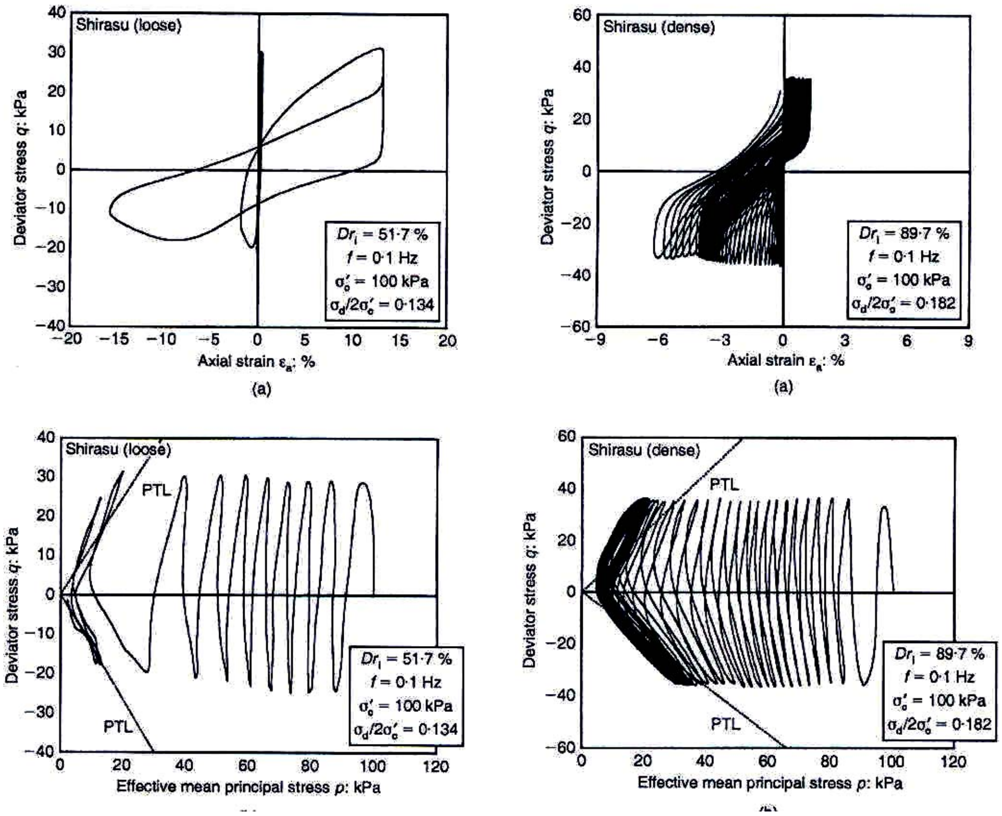
Figure 2.3(b) shows the undrained behaviour of the same soil at approximately same initial relative density but under cyclic shear. It must be observed that strain amplitude accelerated rapidly to amplitudes in excess of 10% particularly on the extension side of the cycle as the pore pressure rise reduced p' to zero. It must also be mentioned that this occurred under cycles of constant amplitude of deviatoric stress. Earthquake shaking amplitudes are seen to attenuate strongly as they propagate upwards through liquefying soils. In this sense, most cyclic triaxial tests create unreliably severe events.

2.2.2.2 Comparison between "undrained cyclic behaviour" of loose (approx 50%) and dense Shirasu soil (90%)

Figure 2.4 compares the behaviour of Shirasu soil in two different states: loose (initial relative density 50%) and dense (initial relative density 90%) subjected to the same cyclic loading. The cyclic loading applied was at a frequency of 0.1 Hz with a sinusoidal motion, Hyodo et al (1998). Two different kinds of cyclic behaviour can be distinguished:

For the loose soil the effective mean principle stress cycles through zero and the axial strains rapidly accelerate to failure (15% in this case), which is often referred to as liquefaction.

On the other hand, for the dense sand the stress path cycles through or close to zero p' condition but the axial strain increases at a steady rate to large values, but not to failure. This phenomenon is known as "cyclic mobility" and the term was proposed by Casagrande (1969) to explain progressive softening. Castro and Poulos (1977) relate this to internal re-distribution of void ratio in the laboratory specimen (top and bottom of the sample). They expressed doubts whether this behaviour would occur to the same degree in-situ during earthquakes.



(a) (b)

Figure 2.4: Behaviour of Shirasu sand under cyclic loading, after Hyodo et al (1998); (a): Loose (50% relative density); (b): Dense (90% relative density).

2.2.3 Definition of liquefaction

The word "liquefaction" means transformation to liquid, as in melting, and in this case it refers to a change from "soil being in a solid state" being converted to a liquid state maintaining its original density. The defining characteristic of a liquid is that it does not offer resistance to flow or shearing. Thus going by literal meaning, liquefied soil would require no significant shear stress to produce large shear strains.

Casagrande (1936) introduced a term called "Critical Void Ratio" and postulated a hypothesis that sands above this void ratio would be susceptible to liquefaction under undrained conditions and the sands below would be safe against such type of failure. In his 1970 lecture "On Liquefaction phenomenon" to the British Geotechnical Society, he concluded through the analysis of Fort Peck Dam failure that his postulate may be incorrect and formulated a new hypothesis based on "flow structure". He explained the phenomenon of liquefaction as a change from "static structure" to "flow structure" using the analogy of boulder flowing through a hydraulic pipeline which only in one position can be moved by water without wedging in the pipe. He supposed liquefaction to be a chain reaction.

100% pore pressure build up in a soil is commonly known as liquefaction, Seed (1979). However, it is clear from sec 2.1.2 that this definition cannot distinguish the behaviour of loose and dense sand under cyclic loading. Ishihara (1993, 1996) added a clause of development of 5% double amplitude (DA) axial strain in addition to the 100% pore pressure rise to define liquefaction, presumably for soils in triaxial testing.

Ishihara (1993) also defines liquefaction as "*A state of particle suspension resulting from release of contacts between particles of sand constituting a deposit. Therefore, the type of soil most susceptible to liquefaction is one in which the resistance to deformation is mobilised by friction between particles under the influence of confining pressure*".

Castro and Poulos (1977) explain the difference between liquefaction and cyclic mobility through the use of state diagram as shown in Figure 2.5. The axes are void ratio (e) and effective minor principal stress and the steady state line is similar to Critical State line (Schofield and Wroth, 1968) where soil can shear at constant void ratio and at constant shear stress. He stated that liquefaction only occurs in specimens that are highly contractive i.e. loose.

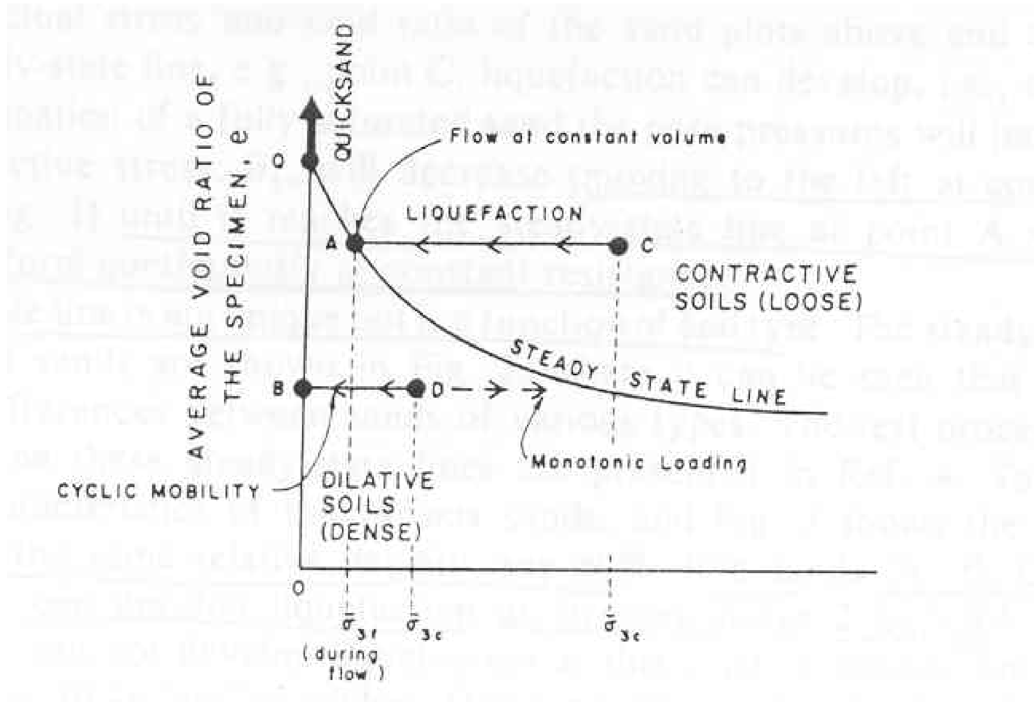


Figure 2.5: Steady State line after Castro and Poulos (1977)

Florin and Ivanov (1961) had a different idea on liquefaction. They defined liquefaction as a mechanical breakdown of structure of sand and expressed the "degree of liquefaction" as the "degree of break down" of the sand structure that can be expressed as percentage of breakdown of contacts. They mentioned two conditions necessary for liquefaction. The conditions are (a) collapse of the structure with the possibility of sand consolidation, (b) either partial or complete saturation of the sand with water.

The authors categorically mentioned that the criterion of collapse of soil structure should not be "critical void ratio" or density of sand, but "critical" values of the intensity of dynamic disturbances, stress condition of the soil or weight of the surcharge and hydraulic gradient of water flow through it.

Schofield (1981) in a St Louis Conference held an opposite view of liquefaction in comparison to Casagrande (1936, 1969) and Castro and Poulos (1977) and defined liquefaction as a class of instability (channelling, piping, boiling, fluidising) seen in soil far on the dry side (denser than critical) of critical states near zero effective stress and in the presence of a high hydraulic gradient. This is quite similar to the views of Florin and Ivanov (1961).

Muhunthan and Schofield (2000) noted that the 100% pore pressure rise is a necessary condition for liquefaction but not a sufficient condition. The formation of openings and the presence of a high hydraulic gradient, which leads to the disintegration of the continuum into clastic blocks of soil, is another important requirement, Figure 2.6. Thus to summarise, for true liquefaction to be observed with complete loss of shear strength and bearing capacity, Muhunthan and Schofield (2000), Schofield (1981) postulate

(a) A source of water i.e. high hydraulic gradient from below to the point of inspection.

(b) Shearing of the soil or mechanism of mixing of i.e. to open up the cracks or fissures.

(c) The effective stress close to zero.

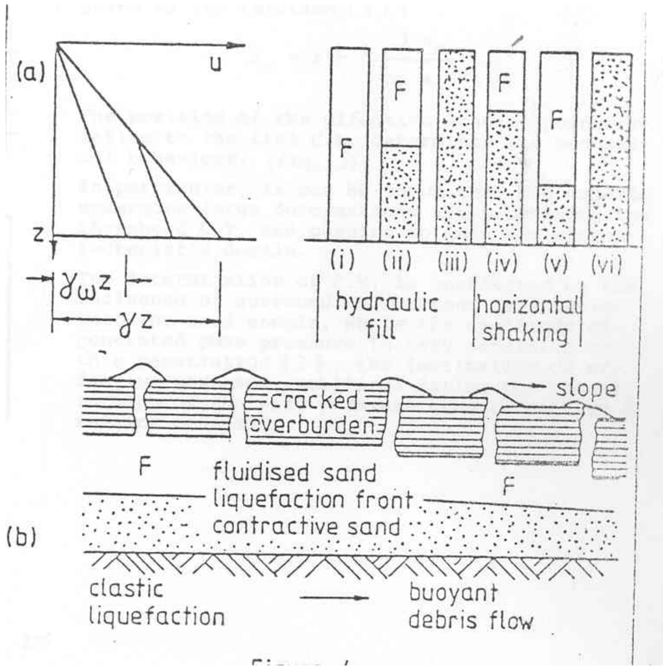


Figure 2.6: Liquefaction concept, Schofield (1981)

2.2.4 Liquefaction susceptibility

In engineering practice or in research into soil liquefaction, it is often required to evaluate whether the soil sample has a potential for liquefaction. In this regard, several attempts have been made to classify soil based on laboratory testing and field-testing.

Laboratory testing mainly aims at assessment based on the grain-size distribution of the soil, whereas field-testing includes SPT (Standard Penetration Test), CPT (Cone Penetration Test) and shear wave velocity measurements in the soil deposit.

H. Tsuchida in 1970 proposed grain size distribution boundary curves to identify soils that are not susceptible to liquefaction and published in Port and Harbour Research Institute in Japanese. These are shown in Figure 2.7, which is translated in English and is obtained from NRC (1985).

Fraction E sand, which was used in the centrifuge tests to be reported later, and the properties of which are mentioned in chapter 4, falls within the boundaries for most liquefiable soil and is marked by the red line in Figure 2.7.

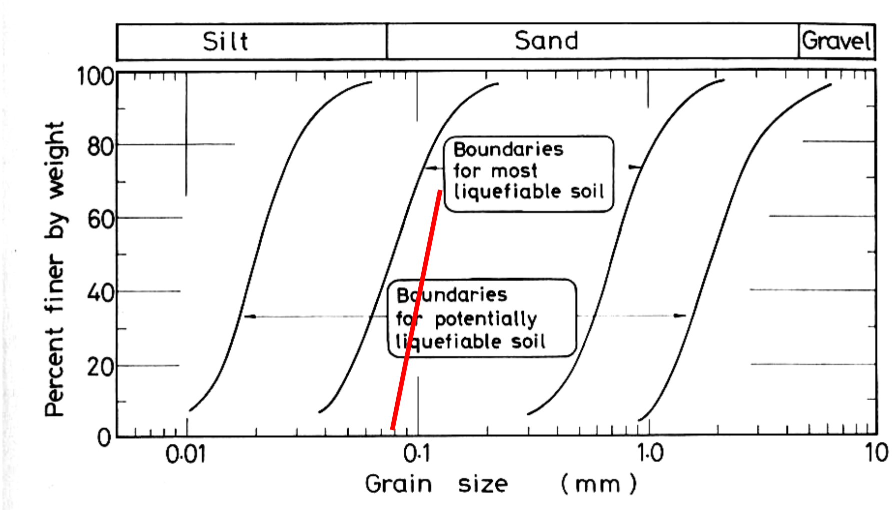


Figure 2.7: Limits in the grading curves separating liquefiable and non-liquefiable soils (after NRC, 1985). Red line denotes the grading curve of fraction E used in the centrifuge tests.

Different empirical correlations exist with respect to intensity of earthquake, SPT or CPT value of the soil, and cyclic stress ratio required for liquefaction. These are based on the vast amount of field performance data during past earthquakes. A recent version of this kind of correlation developed by Seed et al. (1985) based on American, Japanese and Chinese data for an earthquake of magnitude 7.5 on the Richter scale is shown in Figure 2.8. The correlation shown in Figure 2.8 is used to study the case histories of pile failure presented in chapter 3. The majority of case histories of pile foundations reported in the literature give information of the SPT N value of the soil versus depth.

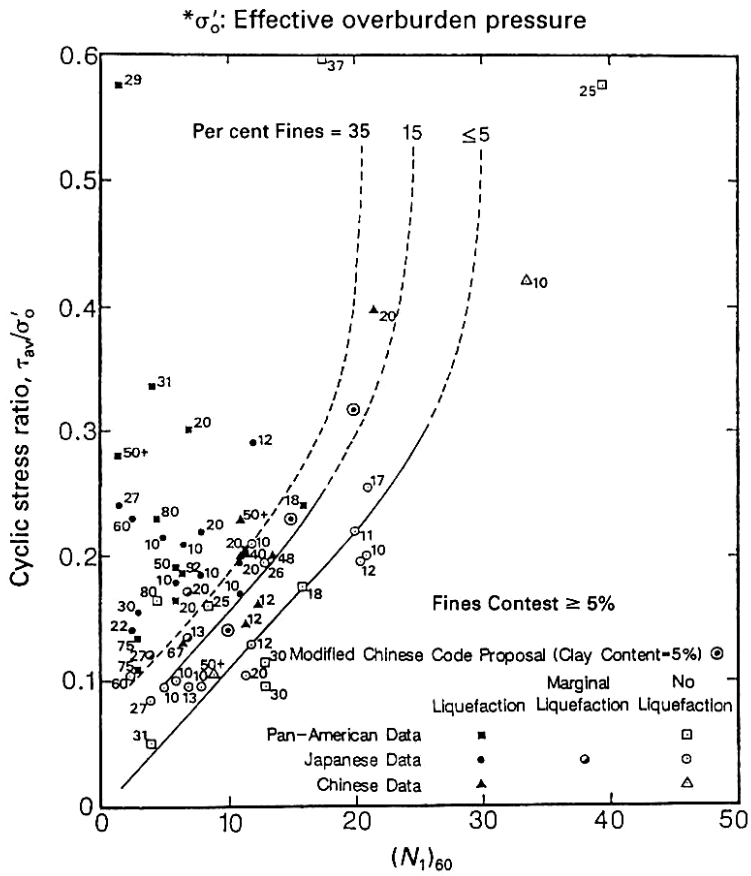


Figure 2.8: Correlations between cyclic strength and the SPT N value after Seed et al. (1985).

2.3 Pile foundations

This section of the chapter reviews some aspects of pile foundation behaviour relevant to this research. One of the main objectives of this research is to understand the failure mechanism of end-bearing piles. The failure of pile foundations can be either structural (forming a plastic hinge) or due to excessive settlement of the intact pile foundations. Thus, this section reviews the structural properties of pile and the load-settlement behaviour of end-bearing piles. This section of the chapter also reviews the current understanding of the buckling of piles.

2.3.1 Structural nature of piles

From a structural perspective, axially loaded piles are long slender columns with lateral support provided by the surrounding soil. If unsupported, these columns will fail in buckling instability and not due to crushing of the pile material. Figure 2.9 shows the length and diameter of tubular piles used in different projects around the world after Bond (1989). This figure shows that piles normally have ratios of length to diameter of 25 to 100. A parameter r_{min} (minimum radius of gyration) given by equation 2.1 is introduced to represent piles of any shape (square, tubular or circular). This parameter is used by structural engineers for studying buckling instability of long slender columns.

$$I = A.r_{\min}^{2} \quad \text{or,} \quad r_{\min} = \sqrt{\frac{I}{A}} \tag{2.1}$$

where:

I = second moment area of the pile section about the weakest axis (m^4).

A= area of the pile section (m^2).

r_{min} = minimum radius of gyration of the pile section about any axis of bending (m).

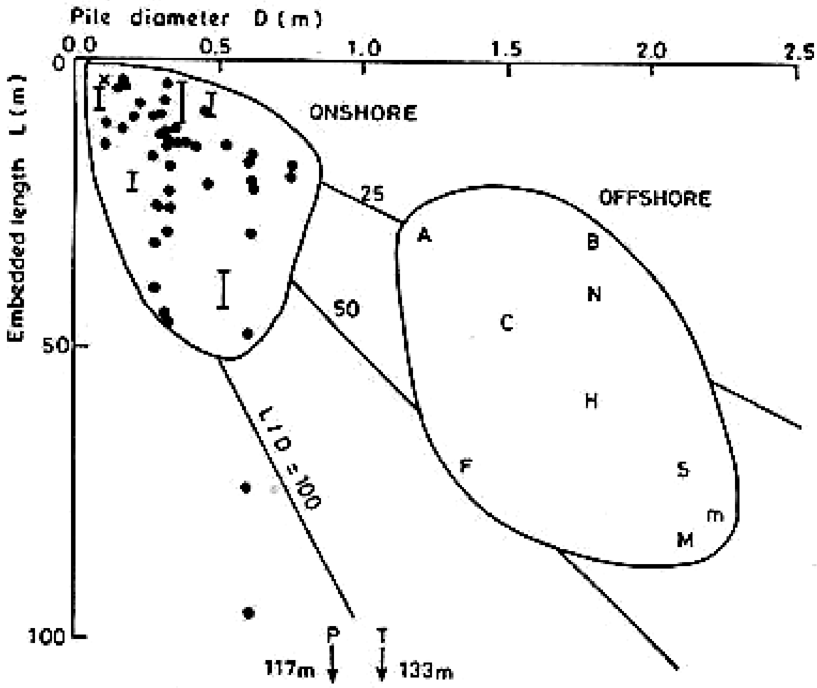


Figure 2.9: Length and diameter of tubular piles, Bond (1989).

For a tubular pile r_{min} is 0.35 times the outside diameter and hence from Figure 2.9, the length (L) to r_{min} ratio of normal piles ranges from 71 to 284. This parameter is used in the study of case histories (chapter 3) and analysing the piles in the centrifuge tests (chapter 5).

2.3.2 Load settlement of end-bearing piles

For a typical pile, the ultimate load carrying capacity of a pile (Q_u) is given by the sum of the ultimate resistance of the base of the pile (Q_b) and the ultimate skin friction over the embedded shaft length of the pile (Q_s). The relative magnitude of the shaft and base capacities depends on the geometry of the pile and the soil profile.

The shaft capacity of a pile is mobilised at much smaller displacements of the pile (typically 0.5-2% of pile diameter) than the base capacity, which may require 5-10% of the pile base diameter (see Figure 2.10), after Fleming et al., 1992. The graph shows that shaft resistance carries most of the working load.

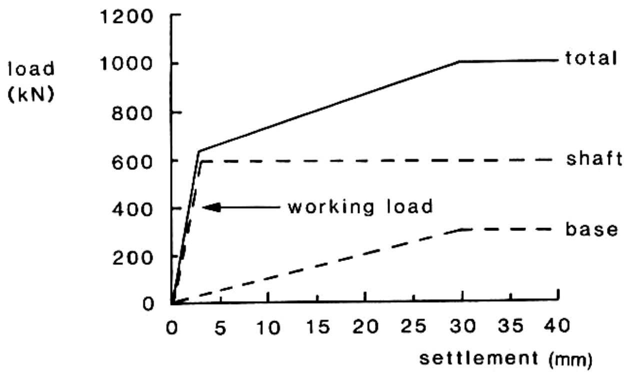


Figure 2.10: Load settlement response for a 0.6m dia. 10m long pile installed in stiff clay. The result is from an instrumented pile load test (after Fleming et al., 1992).

The load transfer behaviour of a pile can be idealised as follows. When a load is applied to the top of the pile, the same load acts along the pile stem until some resistance is encountered at the pile-soil interface. When the shaft friction can no longer resist any increment of load, it will be transferred to the base. Shaft resistance in a pile is a function of effective stress. During an earthquake a pile may lose its shaft resistance due to liquefaction and the same load may be transferred to the base resistance. This section can be illustrated by a case history of the failure of piles of Yachiyo Bridge during the 1964 Niigata earthquake. Details of the case study are shown in Appendix C. The results are quoted in Table 2.1.

Table 2.1: Estimated pile parameters of Yachiyo Bridge.

Pile diameter, length and area	300 mm dia.,10 m long and area of pile = 0.07 m^2
Ultimate bearing capacity	850 kN = 320 kN (shaft) + 530 kN (base)
Shaft resistance and settlement required for mobilisation	320 kN (226 kN from liquefiable layer and 94 kN from non-liquefiable layer); 1.5 mm (0.5% of pile dia.) to 6 mm (2% of pile dia) settlement for full mobilisation.
Base resistance and settlement required for mobilisation	530 kN; 15 mm (5% of pile dia.) to 30 mm (10% of pile dia.) settlement for full mobilisation.
Working load on the pile (allowing factor of safety of 2.5 on ultimate load).	850 kN/2.5 = 340 kN; [94% shaft and 6 % base] i.e. 320kN shaft and 20 kN base.
Stress at the junction of liquefiable and non-liquefiable layer in normal condition.	(340-226) kN/0.07 m^2=1.6 MPa
Stress at the junction of liquefiable and non-liquefiable layer just after liquefaction.	340 kN/0.07 m^2 = 4.8 MPa assuming the pile does not yield structurally.

During earthquake induced liquefaction, the pile loses its entire shaft resistance in the liquefiable layer i.e. 226 kN and the same load is transferred to the base. Thus, the base resistance becomes 246 kN and the shaft resistance is 94 kN. At these stress levels, base failure is unlikely as base capacity is 530 kN. However, the pile may need to settle down by approximately 8 mm to mobilise this base resistance.

2.3.3 Current understanding of buckling of piles

The buckling of piles is typically accounted for in design purposes by considering the following issues: (see Fleming et al.1992).

- Piles in very soft clay.
- During installation by driving, especially of an unsupported pile.
- Partially exposed piles as in jetties or offshore platforms.

Studies have shown that piles founded in soft clay can fail by buckling e.g. Golder and Skipp (1957), Bergefelt (1957), Brandtzaeg and Elvegaten (1957). In this regard Eurocode 7 (1997) suggests that:

"*Slender piles passing through water or thick deposits of very weak soil need to be checked against buckling. This check is not normally necessary when piles are completely embedded in the ground unless the characteristic undrained shear strength is less than 15kPa*".

The problem of buckling of fully embedded piles has been investigated by Granholm (1929), Davisson and Robinson (1965), and Reddy and Valsangkar (1970). The analysis shows that buckling is confined to a critical length of the pile depending on the relative stiffness of the pile and the soil. This critical length is in most cases is only a few diameters of the pile (typically 3 to 6 pile diameters).

Local buckling may be problematic for thin walled pile sections subject to pile driving stresses. Burgess (1976) gave a detailed analysis of the stability of piles driven or jacked into soil of uniform shear strength. He identifies two forms of instability; one is buckling and the other is known as "flutter". Flutter is directional instability at the advancing tip of the pile due to deviation from the proposed alignment. The results of the analysis, based on the assumption that a pile is guided until just above the ground level, showed that the buckling mode instability would rarely occur and, in any case, would always be preceded by flutter instability.

API (1993) considers the possibility of local buckling that may occur in a freestanding pile due to pile driving stresses and recommends the following:

"*The D/t ratio of the entire length of a pile should be small enough to preclude local buckling at stresses up to the yield strength of the pile material. For steel tubular piles that are to be installed by driving where sustained hard driving is anticipated (800 blows per metre or 250 blows per foot with the largest size hammer to be used), the minimum piling wall thickness used should not be less than*

$$t = 6.35 + \frac{D}{100} \tag{2.2}$$

for t = thickness of the pile in mm , D = diameter of the pile in mm."

2.4 Theories of pile failure in areas of seismic liquefaction

Failure of piled foundations has been observed in the majority of recent strong earthquakes. The failure of end bearing piles in liquefiable areas during earthquakes is in most of the cases attributed to the effects of liquefaction-induced lateral spreading, Hamada, 1992a, Tokimatsu et al. (1996, 1998), Ishihara (1997), Finn and Thavaraj (2001). The down-slope deformation of the ground surface adjacent to the pile foundation seems to support this explanation. All these theories of pile failure treat the pile as a beam element and assume that the lateral loads due to inertia and slope movement cause bending failure in the pile.

According to the author's knowledge, "lateral spreading" was first proposed as a possible cause of pile failure during liquefaction in a report published by National Research Council (1985). This report also claims that "lateral spreading" is responsible for more damage during earthquakes than any other form of liquefaction-induced ground failure. This mechanism has been accepted as the explanation of pile failure in many earthquakes. The Japanese Code of Practice (JRA 1996) is the only code that has incorporated this understanding of pile failure as shown in Figure 1.6 in Chapter 1. In terms of pile-soil interaction the current mechanism (JRA) assumes that the soil *pushes* the pile (Figure 1.4) leading to bending failure.

This section of the chapter describes the pile failure theory based on lateral spreading as explained by Ishihara (1997) and Tokimatsu et al. (1998).

2.4.1 Ishihara's (1997) concept of pile failure

Ishihara (1997) in the Terzaghi oration of Hamburg ICSMFE summarised the seismically induced loading on the pile by introducing the concepts of 'top down effect' and 'bottom up effect'. These are described below:

- At the onset of shaking, the inertia forces of superstructure are transferred to the top of the pile and ultimately to the soil. He assumes that during the main shaking, sandy soils in a deposit have not softened significantly due to liquefaction and that the relative movement between the piles and ground are small. However, he postulates that if ground motion is sufficiently high such that the induced bending moment in the piles exceeds the limiting value, the piles may fail. Since the load comes from the inertia force of superstructure, it is referred to as 'top down effect'. He concludes that the observed failure of a pile in the upper portion after an earthquake may be attributed to this effect.

- Ishihara (1997) also reports: "*It has been known that onset of liquefaction takes place approximately at the same time as the instant when peak acceleration occurs in the course of seismic load application having an irregular time history*". Thus in sloping ground, the softened ground will start to move horizontally following the onset of liquefaction. Under this condition, lateral forces would be applied to the pile body embedded in the ground, leading to deformation of the pile in the direction of the slope. He assumes that seismic motion has already passed the peak and shaking may still be persistent with lesser intensity and therefore the inertia force transmitted from the superstructure will be insignificant. Under such a loading condition, the maximum bending moment induced by the pile may not occur near the pile head but at a lower portion at some depth and this is referred to as, 'bottom-up effect'.

2.4.2 Failure theory based on Tokimatsu et al. (1998)

Tokimatsu et al. (1998) schematically described the soil-pile structure interaction in liquefiable soil as shown in Figure 2.11. The assumptions are:

1. Prior to the development of pore water pressure, the inertia force from the superstructure may dominate. This is referred to as stage I in Figure 2.11.
2. Kinematic forces from the liquefied soil start acting with increasing pore pressure. This is referred to as stage II in Figure 2.11.
3. Towards the end of shaking, kinematic forces would dominate and have a significant effect on pile performance particularly when permanent displacements occur in laterally spreading soil.

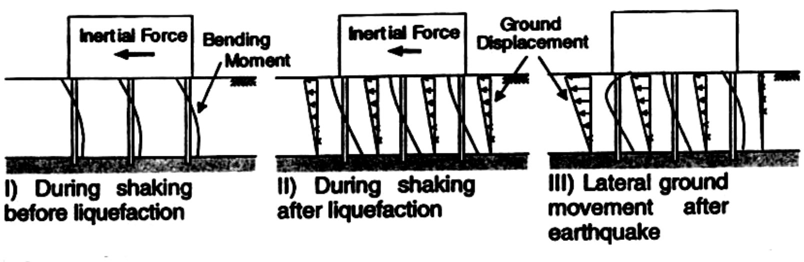


Figure 2.11: Schematic diagram showing the pile failure (after Tokimatsu et al. 1998)

2.5 Review of design methods in the major codes of practice

This section of the chapter has two objectives:

1. To review the design methods of pile foundations in areas of seismic liquefaction used in well-known codes of practice such as Japanese Highway Code of Practice (JRA 1996), NEHRP (USA code) and Eurocode 8. Emphasis is given to the postulations behind the design method.
2. To outline the history of the development of Japanese Code of Practice (JRA 1996). This code is chosen as it is the only code that has guidelines for design of pile foundations in laterally spreading soil.

2.5.1 Development of Japanese Code of Practice (1972 to 1996)

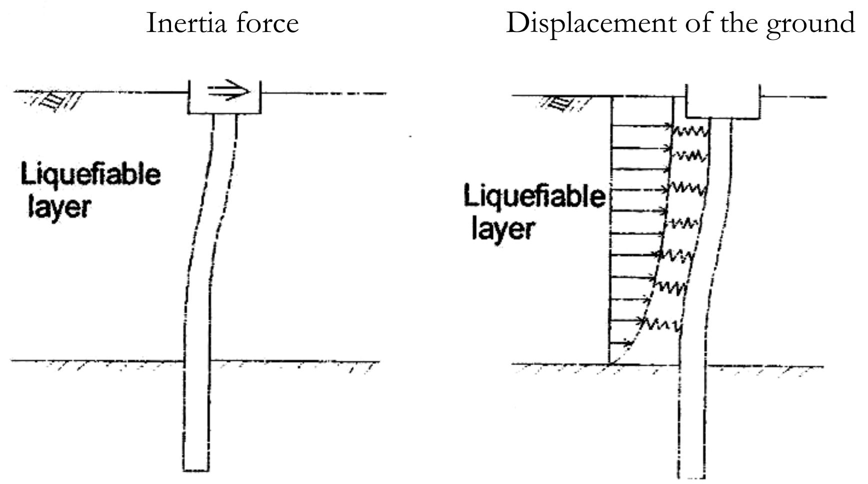
Several bridges, such as the Showa (Figures 1.2b, 1,7 and 1.8) and the Yachiyo bridges, were damaged during the 1964 Niigata earthquake due to soil liquefaction. Based on past experience and observations the "Seismic coefficient method", was introduced in the Highway Bridge Specification (JRA 1972) to take into account the effects of liquefaction, Yasuda and Berrill (2000). The code was subsequently amended in 1980 and as a result, an alternative approach known as the "Seismic deformation method" evolved. Following the damage of piled bridges in the aftermath of the 1995 Kobe earthquake, the Highway Bridge Specification was fully revised, (see Kawashima, 2000). Thus a new approach having checks on lateral spreading was introduced in this edition. This section outlines the development of the code.

Japanese Highway Bridge Specification (JRA 1972)

In the seismic coefficient method, the inertia force of the superstructure due to the earthquake shaking is applied to a pile head as shown in Figure 2.12(a). The soil spring coefficient in liquefied soil is assumed to be zero based on the assumption that liquefied soil has no resistance. Thus the pile is designed considering the maximum bending moment that is acting at the junction of liquefiable and non-liquefiable hard layer. All pile foundations were designed as such in Japan during (1972-1980).

Japanese Highway Bridge Specification (JRA 1980)

Yasuda and Berrill (2000) reported that the Japanese engineers felt that JRA (1972) led to over design owing to the fact that a large number of piles were required. This led researchers, such as Iwasaki (1981) to quantify the resistance of liquefied soil by estimating the reduction of bearing capacity of pile foundations due to liquefaction. Model plate load tests were carried out to predict the reduction of bearing capacity due to a rise in excess pore water pressure. Figure 2.13 plots the variation of excess pore water pressure ratio (L_u) with the reduction factor for bearing capacity (D_E). Shaking table tests were also carried out to find the relationship between pore water pressure (L_u) and safety factor against liquefaction F_L. By combining the two test results the relationship between F_L and D_E was found as shown in Table 2.2, which was adopted in the Highway Bridge Specification JRA (1980).



(a) Seismic Coefficient method (b) Seismic Deformation method

Figure 2.12: Two types of method for static analysis, Yasuda and Berrill(2000).

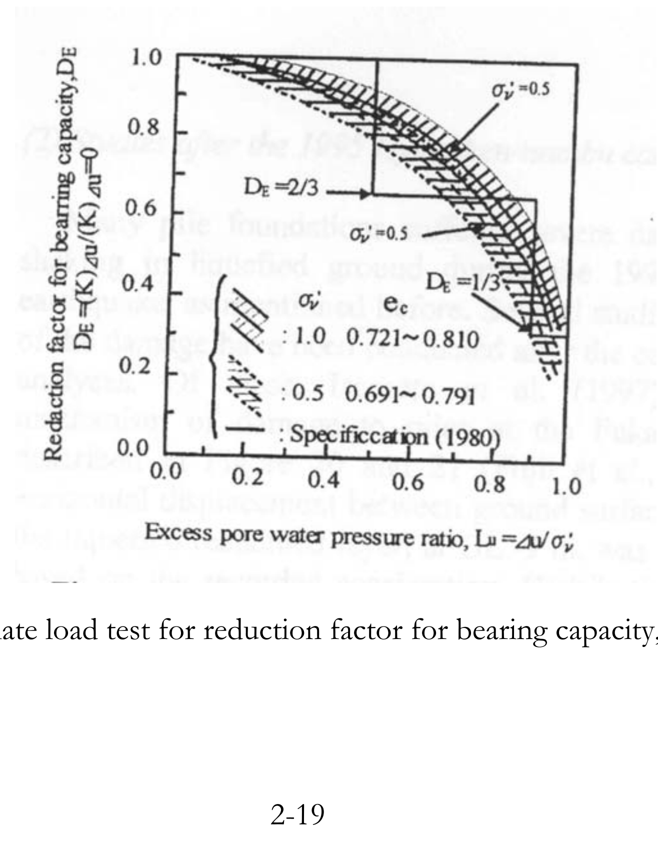


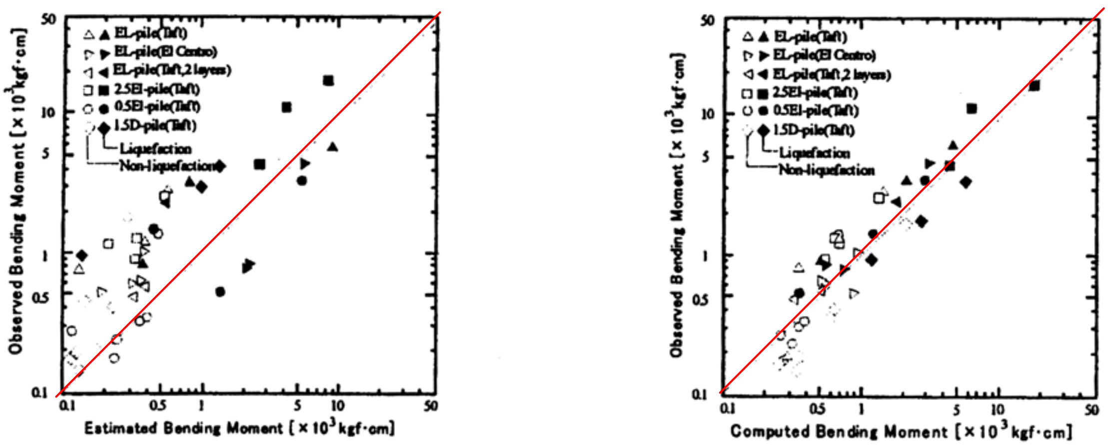
Figure 2.13: Plate load test for reduction factor for bearing capacity, Iwasaki (1981)

Table 2.2: Reduction factor introduced in JRA (1980)

F_L(Factor of safety against liquefaction)	Depths (m)	D_E (Reduction factor)
$F_L \le 0.6$	$0 \le X \le 10$ $10 < X \le 20$	0 1/3
$0.6 < F_L \le 0.8$	$0 \le X \le 10$ $10 < X \le 20$	1/3 2/3
$0.8 < F_L \le 1.0$	$0 \le X \le 10$ $10 < X \le 20$	2/3 1

Tokimatsu and Nomura (1991) carried out shaking table tests to study dynamic soil-pile interaction during liquefaction. The tests were conducted under several conditions of input wave motion, input acceleration, pile rigidity, pile diameter and soil density. The measured maximum bending moments were compared with two design methods: the seismic coefficient method and seismic deformation method.

In the seismic deformation method, horizontal loads induced by horizontal displacement of the ground (lateral spreading) are applied to a pile through soil springs as shown in Figure 2.12(b). Figure 2.14 compares the bending moment observed in the tests with those estimated by the seismic coefficient method and seismic deformation method. The test results showed that the maximum bending moment of the pile is better related to the ground deformation (seismic deformation method or lateral spreading) than to the acceleration on the superstructure (seismic coefficient method or inertia force).



Seismic coefficient method Seismic deformation method

Figure 2.14: Relationship between observed bending moment and estimated bending moment using the two methods after Tokimatsu and Nomura (1991).

Japanese Highway Bridge Specification (JRA 1996)

The Japanese design specification for Highway bridges was revised after the 1995 Kobe earthquake due to the extensive damage of bridges. It is reported that liquefaction-induced lateral spreading was the main cause, for example, MOC (1996), Tamura et al. 2000. As a result guidelines were introduced to take into account the forces due to liquefaction-induced ground movement. The design idealisation for liquefaction-induced forces is shown in Figure 1.6. As mentioned in section 1.4, the code advises practising engineers to check the design of piles against bending failure according to the pressure distribution shown in Figure 1.6. This pressure distribution was formulated by back-analysing some of the piled bridge foundations of the Hanshin expressway that were not seriously damaged, Yokoyama et al. (1997). This check against lateral spreading forces is additional to the requirements against inertia. However, the code says, in pp 78

"*In a case where the effects of lateral spreading are accounted, the effect of lateral spreading shall be provided as horizontal force to study the seismic performance of the foundation. But in this case, it shall not be necessary to simultaneously account for the inertia force produced by the weight of the structure.*"

Kawashima (2000) illustrates the background for such a design philosophy. He notes that, when the ground moves, the force associated with the ground movement applies to a part of foundation in contact with the moving ground. He argues that this is essentially a force mechanism and it is appropriate to idealise the foundation as a structure supported by soil springs and prescribe the movement of ground at the end of each spring. In designing foundations based on such an analytical model, it is important to accurately predict the ground movement. Since the evaluation of maximum ground movement is difficult, the pressure distribution approach is incorporated in the code.

Using dynamic centrifuge tests, Sato et al. (2001) measured earth pressure acting on a piled foundation behind a retaining wall during and after earthquake loading. The authors concluded that the JRA (1996) code over-predicts the lateral pressure in both liquefiable and non-liquefiable layers. The centrifuge test results of Dobry and Abdoun (2001) show similar order of magnitude pressures, as predicted by JRA (1996) code.

This code is subjected to criticism (see Haigh, 2002). His centrifuge results showed that JRA (1996) under-predicts the lateral pressure distribution during the peak transient phase but gives a reasonable prediction for the post earthquake residual flow. He also argues that:

"This also seems a strange design philosophy to adopt, as in almost all other cases, horizontal stresses are taken to be some multiple of vertical effective stresses (i.e. a K value) plus pore pressure, rather than a multiple of total stresses."

2.5.2 Eurocode8 (Part 5)

The Eurocode advises designers to design piles against bending due to inertia and kinematic forces arising from the deformation of the surrounding soil. It goes on saying:

"Piles shall be designed to remain elastic. When this is not feasible, the sections of the potential plastic hinging must be designed according to the rules of Part 1-3 of Eurocode 8".

Eurocode 8 (Part 5) says

"Potential plastic hingeing shall be assumed for:

- *a region of 2d from the pile cap*
- *a region of ± 2d from any interface between two layers with markedly different shear stiffness (ratio of shear moduli > 6)*

where d denotes the pile diameter. Such region shall be ductile, using proper confining reinforcements ……".

2.5.3 NEHRP (2000) Code

Figure 2.15 shows an illustration from the NEHRP code of practice. The code in page 203 notes

"If an unloaded pile were placed in the soil, it would be forced to bend similar to a pile supporting a building.

The primary requirement is stability, and this is best provided by piles that can support their loads while still conforming to the ground motions and, hence the need for ductility".

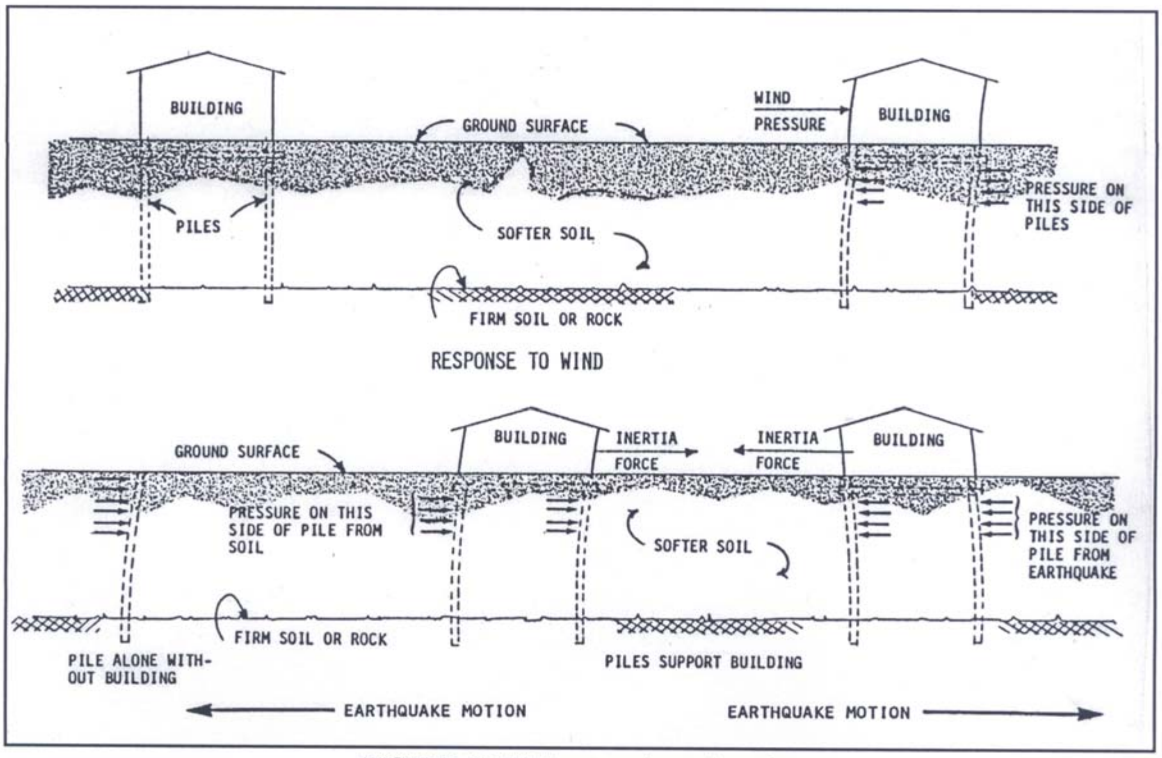


FIGURE C7.4.4 Response to earthquake.

Figure 2.15: NEHRP (2000) Code of Practice

2.6 Recent research into the effect of lateral spreading on pile foundations

Following the large-scale damage to piled structures, such as buildings, bridges and port facilities, during the 1995 Kobe earthquake research into the failure mechanisms of pile foundations was intensified. Permanent lateral deformation or lateral spreading is reported to be the main source of distress to piles, for example Dobry and Abdoun (2001), Tokimatsu (1999), Hamada (1992a, 1992b). Hamada (2000) concludes that permanent displacement of non-liquefied soil overlying the liquefied soil is a governing factor for pile damage.

Based on the assumption that lateral spreading is the cause of failure, research into this pile failure mechanism has been conducted by various researchers, such as Sato el al. (2001), Takahashi et al (2002), Haigh (2002), Berrill (2001), Tokimatsu et al. (2001). A summary of some research publications in the aftermath of 1995 Kobe earthquake is presented in Table 2.3. A noteworthy paper by O'Rourke et al. (1994) is also included.

Table 2.3: Research work on mechanism of pile failure during (1995 – 2002)

Research group and reference	Aspect of the problem investigated.	Conclusions	Remarks
Haigh (2002) University of Cambridge	Transient lateral forces experienced by the pile during lateral spread down a slope. Rigid and flexible piles were used. No axial load was applied.	JRA (1996) code (Figure 1.6) is un-conservative by 300% in transient phase but gives reasonable predictions at residual values.	Conclusions arrived through centrifuge tests using the stress cells measurements. Near field pore pressures were also measured.
Berrill et al. (2001) University of Canterbury, New Zealand	Good performance of pile foundations of Landing bridge.	The chief threat to piled foundations comes from the non-liquefied crust and not from the drag force of the liquefied soil.	Limit equilibrium analysis was used Similar conclusions were reached by Tokida et al (1993), Vargas and Towahata (1995).
Sato et al (2001), National Research Institute for Earth science and disaster prevention, Japan	Lateral forces experienced by a pile foundation near a quay wall during lateral spread.	The forces predicted by JRA (1996) are over-conservative.	Conclusions arrived through centrifuge tests using stress cells measurements. The measurements showed no trends.
Tokimatsu et al (2001) Tokyo Institute of Technology, Japan	The p-y behaviour (i.e. relation between subgrade reaction and relative displacement between soil and pile) during soil liquefaction.	If the pile pushes the soil the subgrade reaction is co-related with relative soil pile displacement, and If soil pushes the pile the subgrade reaction can be co-related with relative velocity between pile and soil.	Shaking table tests were carried out using a large-scale laminar box.
Ramos et al. (2000) R.P.I (New York)	Effect of superstructure's horizontal stiffness on the bending moments induced on the pile by lateral spreading.	Simple limit equilibrium approach using the pressure distribution suggested by Dobry and Abdoun (1998) gives a good prediction of the bending moment in the pile.	Centrifuge tests were carried out on end-bearing piles in absence of axial load. The bending moments in the pile were measured using a pair of strain gauges.

Table 2.3 (continued): Research work on mechanism of pile failure during (1995 – 2002)

Research group and reference	**Aspect of the problem investigated.**	**Conclusions**	**Remarks**
Hamada (2000) Waseda University, Japan	Characterisation of the external forces from the flowing liquefied soil and non-liquefied crust on model piles under 1-g condition. 26mm polycarbonate pipe was used as model pile.	The force from flowing liquefied soil on a model pile can be estimated as a drag force against a cylindrical object in a viscous flow. The external force from non-liquefied soil overlying the flowing liquefied soil governs the deformation of the pile.	He notes that the deformation of the model pile can be simulated by a beam and soil spring model where the displacement of the soil is forced into the pile through the soil spring.
Wilson et al. (2000) University of California (Davis)	Lateral p-y resistance of liquefied soils. Centrifuge tests were carried out on single pile and pile-group supported structures in liquefied sand.	The lateral p-y resistance of liquefied soil is a complex phenomenon and is significantly affected by relative density, cyclic degradation, excess pore pressures, phase transformation behaviour, prior displacement history and loading rate.	They note that there is considerable uncertainty in any simplified representation of p-y characteristics of liquefied soil and this uncertainty must be allowed for in design.
Goh and O'Rourke (1999) Cornell University	Development of numerical code to calibrate the centrifuge test results of Abdoun (1997). The model uses a tri-linear strain softening p-y curves. The curves are obtained from FLAC analysis of a pile being displaced through a cohesive material. The undrained shear strength in the curve initially rises to a peak value and then falls linearly with plastic deviatoric strain until a residual shear strength is achieved.	The strain softening p-y model provides excellent predictions of the measured peak and residual moments. The computed soil pressure distribution agrees well with the JRA (1996) code.	Dynamic effects are not included. Haigh (2002) extended the above model to a pseudo dynamic p-y-u analysis where u is the pore pressure rise.

Table 2.3 (continued): Research work on mechanism of pile failure during (1995 – 2002)

Research group and reference	**Aspect of the investigated.**	**Conclusions**	**Remarks**
Dobry and Abdoun (1998) R.P.I (New York)	Pressure distribution acting on a pile during lateral spreading. The pressure distribution was back calculated from the bending moments.	The maximum bending moment in a pile could be obtained by applying an inverted triangular distribution having a value of 17.7kPa at the surface to zero at 6m depth.	Limit equilibrium analysis was used to fit the bending moment data of instrumented piles obtained in centrifuge tests.
Abdoun (1997) RPI (New York)	Bending moments acting on single piles and pile groups in layered soils.	Non-liquefied crust exerts approximately passive pressure to a pile foundation.	A series of centrifuge tests were carried out in two-layer system and three-layer system.
Liu and Dobry (1995)	Stiffness degradation of liquefied soil due to excess pore pressure rise so as to develop a dynamic p-y curve for design of pile foundations.	They derived a dimensionless degradation coefficient against r_u (ratio of excess pore pressure to that causing full liquefaction). This coefficient can be multiplied to p-y curves for static tests to generate dynamic p-y curves.	A series of centrifuge tests were carried out.
O'Rourke et al (1994) Cornell University	Analytical studies of pile response to lateral spread using the computer code B-STRUCT. Dimensionless plots were developed.	Three failure mechanisms viz. excessive bending, buckling and soil flow were recognised under lateral spread condition. Each of the mechanisms is a function of relative stiffness between soil and pile and the axial load carried by the pile.	The pile was modelled as a series of beam element whereas the surrounding soil is modelled as transverse and longitudinal bilinear spring-slider elements.

2.7 Critical review of the current understanding of pile failure mechanisms in seismic liquefaction

This section highlights the limitations of the current hypothesis of pile failure i.e. lateral spreading. Particular attention is given to the clauses of JRA code. The limitations identified are:

1. This hypothesis of pile failure assumes that the pile remains in stable equilibrium (i.e. vibrates back and forth and does not move unidirectionally as in case of instability) during the period of liquefaction and before the onset of lateral spreading. In other words, the hypothesis ignores the structural nature of pile (see section 2.3.1).
2. The effect of axial load as soil liquefies is ignored in this hypothesis.
3. Some observations of pile failure (as mentioned in section 1.5) cannot be explained by the current hypothesis.
4. It is shown in section 2.7.1 that the pile foundation of Showa Bridge, which is considered safe based on the current JRA (1996) code, actually failed during the 1964 Niigata earthquake.

2.7.1 A case study: Failure of Showa Bridge after 1964 Niigata earthquake

This section describes the bridge and the resulting damage due to the 1964 Niigata earthquake. It is shown that the piles satisfied the criteria of JRA code i.e. had enough strength to resist the lateral spreading but they failed.

Description of the bridge and damage features

The bridge was built over river Shinano and was completed just a month before the earthquake (Fukuoka, 1996). The bridge had a width of 24 m and total length of 307 m. The foundation consisted of a row of 9 piles connected laterally as shown in Figure 1.7. After the earthquake five girders fell into the river. Figures 1.8 and 1.2(b) illustrate the failure of the bridge. Figure 2.16 shows the post earthquake failure investigation and recovery of the damaged pile along with the soil investigation data. Table 2.4 summarises the design data of the pile.

Table 2.4: Design data of pile

Length	25 m
External diameter	609 mm
Internal diameter	591 mm
Material	Steel
E (Young's Modulus)	210 GPa

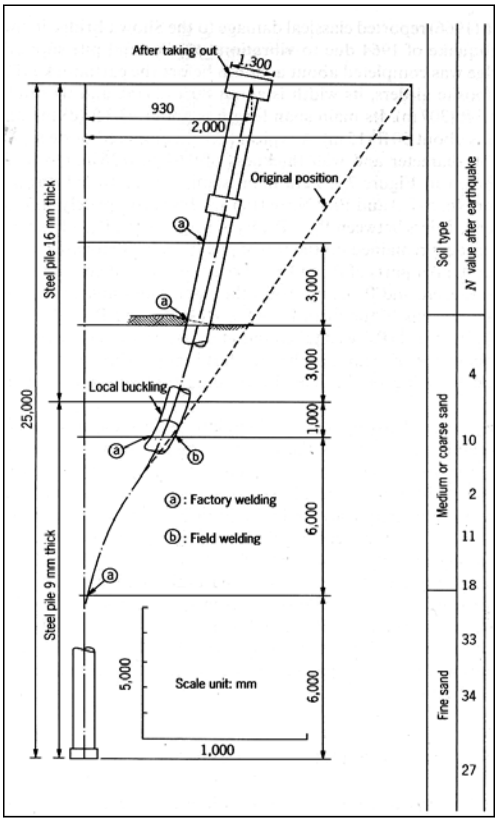


Figure 2.16: Post earthquake recovery and deformation of the pile from Showa Bridge, after Fukuoka (1966).

Eyewitness report

Reliable eyewitnesses report, "*the girders began to fall somewhat later, perhaps about 0 to 1 minute after the earthquake motion ceased*" (Hamada, 1992a). The reason for failure based on current conventional theory, as described by Hamada (1992a) is

"*The ground on the left bank and in the riverbed liquefied as a result of the earthquake motion and moved toward the river centre. The ground displacements continued even after the earthquake motion ceased, until the excess pore pressure dissipated. The permanent ground displacement on the* *left bank reached several metres, substantially deforming the foundation piles and causing the girders to fall*'.

Calculation based on JRA (1996) code

The photographs shown in Figures 1.2(b) and 1.7 show that the failed piles were fully submerged in water and hence a non-liquefied crust is unlikely to exist. Figure 2.17 shows the loading diagram based on the JRA (1996) code. The calculation below estimates the maximum moment based on JRA code.

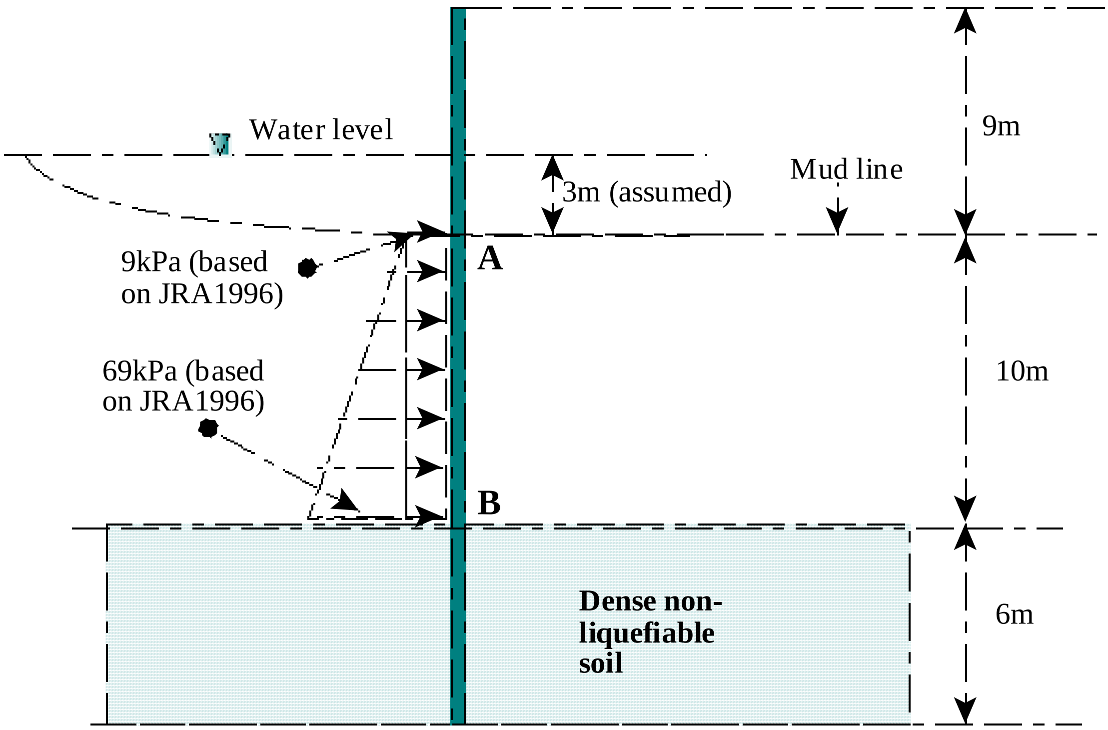


Figure 2.17: Schematic diagram showing the predicted loading based on JRA code.

Calculations

Assuming the bulk unit weight of soil is 20kN/m^3

Maximum lateral spreading pressure at mudline at point A in Figure 2.17 = 30% of total overburden pressure due to water = 0.3 × 10kN/m^3 × 3m = 9kPa.

Maximum lateral spreading pressure at 10m depth acting at point B in Figure 2.17 = 30% of total overburden pressure = 0.3 × (20kN/m^3 × 10m + 10kN/m^3 × 3m) = 69kPa.

Maximum moment, at point B in Figure 2.17, due to spreading force (trapezoidal loading)

= (0.5 × 60 kPa × 10m × 0.609m × 3.33m) + (9kPa × 10m × 0.609m × 5m) = 608kNm + 274kNm = 882kNm.

The plastic moment capacity of the section (9mm thick)

$$= \left(\frac{0.609^3}{6} - \frac{0.591^3}{6} \right) m^3 \times 500MPa = 1620kNm$$

Hence the calculated factor of safety against plastic bending failure= (1620/882)=1.84

Thus according to the JRA code the bridge should not have collapsed!

In addition the hinge formed at 4 m below the mud line (as can be seen from Figure 2.16) whereas the moment should be a maximum at 10 m depth based on JRA code and should be only 5% of the plastic moment of resistance at the location of the observed hinge.

2.8 Summary

The parameter r_{min} has been introduced to describe piles of any shape and section. The current understanding of pile failure is based on "lateral spreading" and treats pile as a beam element. Lateral spreading is the background mechanism being the JRA 1996 code. This hypothesis was first proposed in 1985 and there has been limited debate over the validity of this approach. Most research into liquefaction-induced pile failure has assumed that lateral spreading is the governing mechanism. It has been shown that this assumption cannot explain the failure of the pile foundations of the Showa Bridge – a bridge that collapsed only one month after the construction. The limitations of the current understanding are highlighted; in particular the structural nature of pile is overlooked.