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ERRORS IN DESIGN LEADING TO PILE FAILURES DURING SEISMIC LIQUEFACTION

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ABSTRACT

Collapse of piled foundations in liquefiable soils has been observed in the majority of the recent strong earthquakes despite the fact that a large margin of safety is employed in their design. This paper critically reviews the current design methods and the underlying mechanism behind them. The current method of pile design under earthquake loading is based on a bending mechanism where the inertia and slope movement (lateral spreading) induce bending in the pile. This paper shows that this hypothesis of pile failure cannot explain some observations of pile failure. It has been identified that the current design codes of practice for pile design omit considerations necessary to avoid buckling of piles due to the loss of lateral soil support in the event of soil liquefaction, i.e. the structural nature of the pile is overlooked. A new design approach is proposed in this paper taking into account buckling effects.

INTRODUCTION

Failure of piled foundations has been observed in the aftermath of the majority of recent strong earthquakes. Permanent lateral deformation or lateral spreading is reported to be the main source of distress to piles, for example Abdoun and Dobry (2002), Finn and Fujita (2002), Dobry and Abdoun (2001), Hamada (2000, 1992a, 1992b), Goh and O'Rourke (1999) Tokimatsu et al. (1998, 1997, 1996). The down-slope deformation of the ground surface adjacent to the piled foundation seems to support this explanation. The current hypothesis of pile failure simply treats piles as beam elements and assumes that the lateral loads due to inertia and slope movement (lateral spreading) cause bending failure of the pile.

The Japanese Code of Practice (JRA 1996) has incorporated this understanding of pile failure and is shown in Figure 1. The code advises practising engineers to design piles against bending failure assuming that the non-liquefied crust offers passive earth pressure to the pile and the liquefied soil offers 30% of the total overburden pressure. Other codes such as the USA code (NEHRP 2000) and Eurocode 8, part 5 (1998) also focus on the bending strength of the pile.

According to the authors' knowledge, "Lateral Spreading" was first proposed as a possible failure mechanism of piled foundation in a report published by NRC (1985) and there has been limited debate over the validity of this mechanism. 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 Takahashi et al (2002), Haigh (2002), Berrill (2001), Tokimatsu et al. (2001). Hamada (2000) in the 12th World Congress on Earthquake Engineering concludes that permanent displacement of non-liquefied soil overlying the liquefied soil is a governing factor for pile damage.

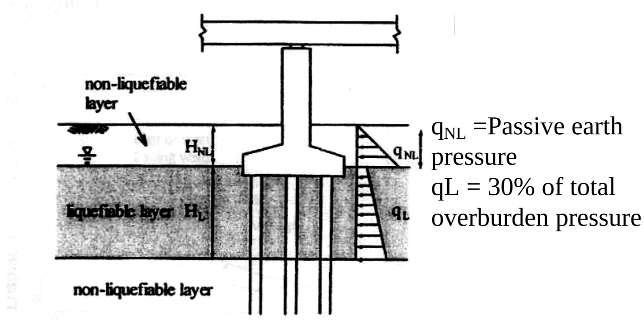


Fig 1. JRA (1996) code of practice showing the idealization for seismic design of bridge foundation.

WHY PILES STILL COLLAPSE DURING EARTHQUAKES?

Structural failure of piles (by formation of plastic hinges) passing through liquefiable soils has been observed in many recent strong earthquakes (see Figures 2 and 3). Figure 2 shows a case of plastic yielding of a pile from a past earthquake after Hamada (1992a). This suggests that the bending moments or shear forces that are experienced by the piles exceed those predicted by design methods (or codes of practice) and in some cases exceed the "Plastic Moment Capacity of the section (M_P)". All current design codes apparently provide a high margin of safety using partial safety factors, yet occurrences of pile failure in areas of seismic liquefaction are abundant. The overall safety factor against plastic yielding for a typical concrete circular pile, if designed in accordance to a code is of the order of 4. This is due to the multiplications of the partial safety factors on load (1.5), material (1.5 for concrete) and fully plastic strength factor (Z_P/Z_E = 1.67 for a circular section). Considering practical factors such as the minimum reinforcement requirements and minimum number of bars, the overall safety factor against plastic yielding may further increase by a factor of 2, thereby increasing the overall safety factor against plastic hinging to 8. This implies that the actual moments or shear forces experienced by the pile are 4 to 8 times those predicted by their design methods. It may be concluded that design methods may not be consistent with the physical mechanisms that governs the failure. In other words, something is missing in the current understanding.

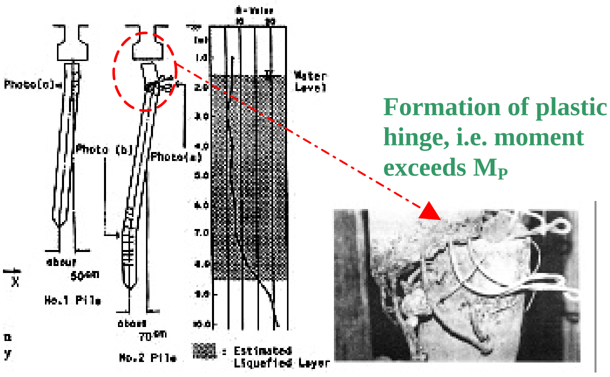
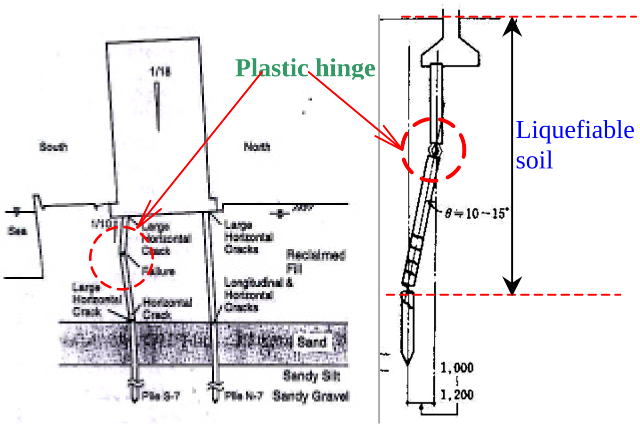


Figure 2: Failure of piles in NFCH building during the 1964 Niigata earthquake, Hamada (1992a).



(a) (b)

Figure 3:Failure of piled buildings; (a) A collapsed building after the 1995 Kobe earthquake, showing the hinge formation after Tokimatsu et al. (1997); (b): Failure piles of the NHK building after Hamada (1992b).

LIMITATIONS OF THE CURRENT DESIGN METHODS

Structurally, piles are slender columns with lateral support from the surrounding soil. Generally, as the length of the pile increases, the allowable load on the pile increases primarily due to the additional shaft friction but the buckling load (if the pile were laterally unsupported by soil) decreases inversely with the square of its length following Euler's formula. Figure 4 shows a typical plot for the variation of allowable load (P) and buckling (P_{cr}) load of a pile (if unsupported) against length of the pile. The pile in the above example has a diameter of 300mm (typical pile dimension in 1964 Japan) and is passing through a typical liquefied soil. The allowable load (P) is estimated based on conventional procedure with no allowance for liquefaction.

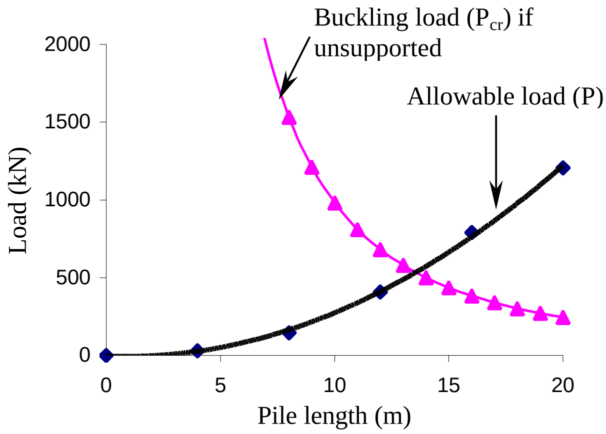


Figure 4: Allowable load and buckling load of a typical pile (if unsupported).

If unsupported over a length of 10 metres or more, these columns will fail due to buckling instability and not due to crushing of the material. During earthquake-induced liquefaction, the soil surrounding the pile loses its effective confining stress and can no longer offer sufficient lateral support. The pile may now act as an unsupported column prone to axial instability. This instability may cause it to buckle sideways in the direction of least elastic bending stiffness under the action of axial load. In this case the pile may push the soil and it may not be necessary to invoke lateral spreading of the soil to cause a pile to collapse. This is established through a study of case histories and centrifuge tests and is summarized in the next section. The current design codes of practice overlook this consideration, which is the main point of this paper.

Inconsistency of the current understanding with observed seismic pile failure at liquefiable sites

This section highlights the shortcomings of the current understanding of pile failure in the light of well-documented case histories.

1. Had the cause of pile failure been lateral spreading, the location of the plastic hinge would have been expected to occur at the interface of liquefiable and non-liquefiable layer as this section experiences the highest bending moment. It is often seen that hinge formation also occurs within the top third of the pile as seen in Figures 2,3 and 5.
2. Figure 5 shows the failure of the Showa Bridge. The failure is widely accepted as being due to lateral spreading of the surrounding soil (see, for example (Hamada, 1992a), (Ishihara, 1993)). As can be seen from Figure 5, piles under pier no P_5 deformed towards the left and the piles of pier P_6 deformed towards the right (Fukoka, 1966). Had the cause of pile failure been

due to lateral spreading the piers should have deformed identically in the direction of the slope. Furthermore, the piers close to the riverbanks did not fail, whereas the lateral spread is seen to be most severe at these places.

To summarise, the limitations of the current hypothesis of pile failure i.e. lateral spreading 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.
2. The effect of axial load as soil liquefies is ignored in this hypothesis.
3. Some observations of pile failure cannot be explained by the current hypothesis.
4. It is suggested by Bhattacharya (2003), that the pile foundation of Showa Bridge, which is considered safe based on the current JRA (1996) code, actually failed by buckling during the 1964 Niigata earthquake.

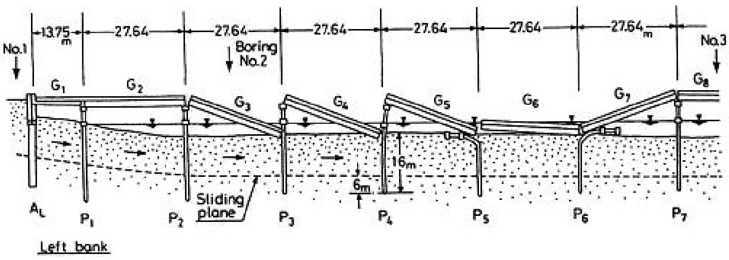


Figure 5:Failure of Showa Bridge; (a): Photograph of the failure after NISEE; (b): Schematic of the falling of the decks after Takata et al (1965)

RESEARCH ON BUCKLING AS AN ALTERNATIVE MECHANISM OF PILE FAILURE

Extensive research work has been carried in the Cambridge Geotechnical Research Group, (see Bhattacharya et al. 2002, 2003, Bhattacharya, 2003) to understand whether buckling instability can be a possible failure mechanism of pile foundation in areas of seismic liquefaction. Dynamic centrifuge tests, in-depth study of case histories and analytical studies form the basis of this investigation. This section summarises some of the important conclusions from the above study.

Study of case histories

Fifteen reported cases of pile foundation performance during earthquake-induced liquefaction were studied and analysed as listed in Table 1. Six of the piled foundations were found to survive while the others suffered severe damage. Emphasis is given to the slender nature of the piles. Accordingly, the concept of "effective length of piles in the liquefiable region (L_{eff})" is introduced to normalise the different boundary conditions of pile tip and pile head (see Figure 6). A parameter "minimum radius of gyration of the pile section (r_{min})" is also introduced to represent piles of any shape (square, tubular, circular). This parameter is used by structural engineers to study buckling instability of slender columns and is given by $r_{\min} = \sqrt{\frac{I}{A}}$, where I is the second moment of area; and A is the cross sectional area of the pile.

Figure 7 plots L_{eff} against the r_{min} of the piles listed in Table 1 with identification of their performance during earthquakes. A line representing a slenderness ratio (L_{eff}/r_{min}) of 50 is drawn and it distinguishes poor pile performance from good performance. This line is of some significance in structural engineering, as it is often used to distinguish between "long" and "short" columns. Columns having slenderness ratios below 50 are expected to fail in crushing whereas those above 50 are expected to fail by bending due to buckling instability. Thus, the analysis suggests that pile failure in liquefied soils is similar in some ways to the failure of long columns in air. The lateral support offered to the pile by the soil prior to the earthquake is removed during liquefaction.

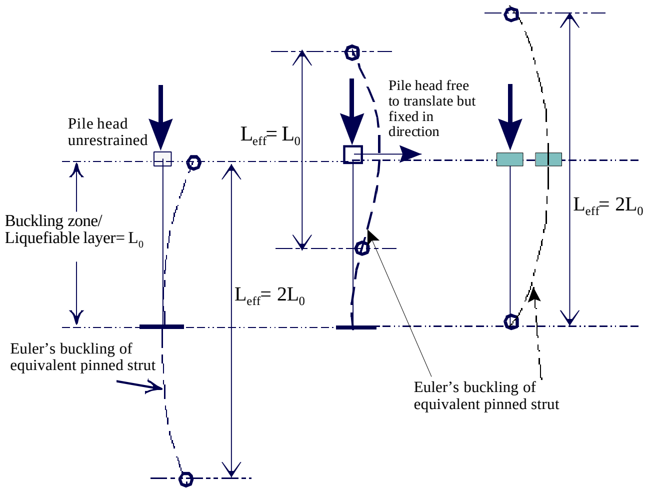


Figure 6: Concept of effective length of pile in liquefiable soil

Stability analysis of elastic columns shows (Timoshenko and Gere, 1961) that lateral deflections caused by lateral loads are greatly amplified if the axial load approaches the elastic critical load P_{cr}. In the presence of an axial load of magnitude 65% of P_{cr}, the sway deflections and bending strains will be 3 times those of small deflection theory. In most practical situations such enhanced strains also lead to degradation of the elastic stiffness of the column, bringing down the critical load and causing collapse. It can be shown that a slenderness ratio of 50 signifies (P/P_{cr}) below 0.35 for steel and 0.15 for concrete, Bhattacharya (2003). In each case, the expected amplification due to the combined action of lateral and axial loads is negligible. This suggests that for piles having slenderness ratio below 50, lateral loads – if properly accounted for in simple bending calculations - cannot lead a pile to fail prematurely.

This is consistent with the fact that piles in laterally spreading soil (Marked A through F in Figure 7) having slenderness ratio below 50 did not collapse. It is proposed in this paper that piles in liquefiable soil should be maintained below a slenderness ratio of 50 to avoid buckling instability.

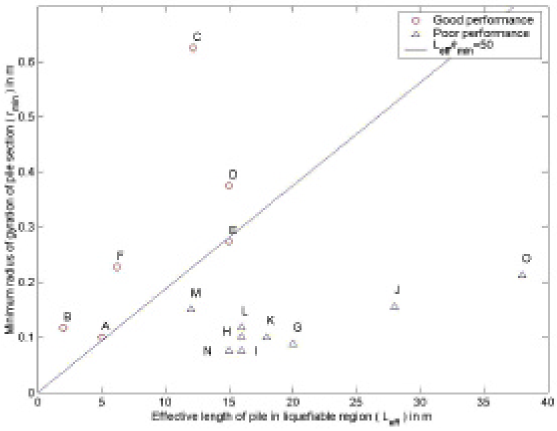


Figure 7: Effective length (L_{eff}) and r_{min} of the piles studied.

Centrifuge tests

The central aim of the centrifuge tests was to verify if fully embedded end-bearing piles passing through saturated, loose to medium dense sands and resting on hard layers buckle under the action of axial load alone if the surrounding soil liquefies in an earthquake. This would verify the proposed hypothesis of pile failure arising from the study of case histories. Details of the test can be seen in Bhattacharya et al (2002).

During earthquakes, the predominant loads acting on a pile are axial, inertial and those due to lateral movement of the soil (lateral spreading). The failure of a pile can be because of any of these load effects or a suitable combination of them. The centrifuge tests were designed in level ground to avoid the effects of lateral spreading. Twelve piles were tested in a series of four centrifuge tests including some which decoupled the effects of inertia and axial load. Table 2 summarises the performance of the piles along with the load effects acting. Axial load (P) was applied to the pile through a block of brass fixed at the pile head. With the increase in centrifugal acceleration, the brass weight imposes increasing axial load in the pile. The packages were centrifuged to 50-g and earthquakes were fired during the flight. The effect of axial load alone was studied by using a specially designed frame to restrain the head mass against inertial action.

Table 1: Case histories studied

ID in Fig 7	Case History and Reference	Pile section/ type	L_0^* (m)	L_{eff} (m)	r_{min} (m)
A	10 storey-Hokuriku building, Hamada (1992a)	0.4m dia RCC	5	5	0.1
B	Landing bridge, Berrill et al (2001)	0.4m square PSC	4	2	0.1
C	14 storey building, Tokimatsu et al (1996)	2.5m dia RCC	12.2	12.2	0.6
D	Hanshin expressway pier, Ishihara (1997)	1.5m dia RCC	15	15	0.4
E	LPG tank 101, Ishihara (1997)	1.1m dia RCC	15	15	0.3
F	Kobe Shimim hospital, Soga (1997)	0.66m dia steel tube	6.2	6.2	0.2
G	N.H.K building, Hamada (1992a)	0.35m dia RCC	10	20	0.1
H	NFCH building. Hamada (1992a)	0.35m dia RCC hollow	8	16	0.1
I	Yachiyo Bridge Hamada (1992a)	0.3m dia RCC	8	16	0.1
J	Gaiko Ware House, Hamada (1992b)	0.6m dia PSC hollow	14	28	0.2
K	4 storey fire house, Tokimatsu et al (1996)	0.4m dia PSC	18	18	0.1
L	3 storied building at Kobe university, Tokimatsu et al (1998)	0.4m dia PSC	16	16	0.1
M	Elevated port liner railway, Soga (1997)	0.6m dia RCC	12	12	0.2
N	LPG tank –106,107 Ishihara (1997)	0.3m dia RCC hollow.	15	15	0.8
O	Showa bridge, Hamada (1992a)	0.6m dia steel tube.	19	38	0.2

L_o= Length of the pile in the liquefiable zone; L_{eff} = Effective length of the pile in the liquefiable zone following Figure 6, PCC = Pre Stressed Concrete; RCC = Reinforced Cement Concrete.

Table 2: Summary of pile performance in the centrifuge tests

Test ID	Pile ID	Max load (P) N	P/A MPa	P/P_{cr}	Load effects	Remarks
SB-02 Pile length = 160mm A=9.7 mm^2	1	768	79	0.97	Axial + Inertia	Failed
	2	642	65	1.01	Axial + Inertia	Failed
	3	617	63	0.97	Axial + Inertia	Failed
SB-03 Pile length = 180mm A = 11.2 mm^2	4	294	26.3	0.5	Axial + Inertia	Did not collapse
	5	220	19.7	0.35	Axial + Inertia	Did not collapse
	6	113	10.1	0.22	Axial + Inertia	Did not collapse
SB-04 Pile length = 180mm A = 11.2 mm^2	7	610	54.5	1.04	Axial	Failed
	8	872	78	1.48	Axial	Failed
	9	2249	201	0.25	Axial	Did not collapse
SB-06 Pile length = 180mm A = 11.2 mm^2	10	735	65.6	1.25	Axial	Failed
	11	269	24	0.46	Axial + Inertia	Did not collapse
	12	441	39.4	0.75	Axial + Inertia	Failed

As can be seen from Table 2, axial loads applied to the piles ranged from 22% to 148% of Euler's elastic critical load (P_{cr}) treating piles as long columns neglecting any support from the soil. It immediately becomes obvious from the table that the piles having P/P_{cr} ratio greater than 0.75 failed (see Figure 8). The loads in the piles marked 7 8 and 10 were purely axial. The pile heads were restrained in the direction of shaking (no inertia effects) and the piles buckled transversely to the direction of shaking. It must also be remembered that the piles were carrying the same load (load at which it failed) at 50-g and were stable before the earthquake. The stress in the pile section is well within the elastic range of the material (less than 30% of the yield strength) but it failed as the earthquake was fired. This confirms that the support offered by the soil was eliminated by earthquake liquefaction and that the pile started to buckle in the direction of least elastic bending stiffness.

Thus we must conclude, if the axial load is high enough (P/P_{cr}=0.75) it may not be necessary to invoke lateral spreading of the soil to cause a pile to collapse and piles can collapse before lateral spreading starts once the surrounding soil has liquefied.
Figure 9 shows the surface observations of the piles after test SB-02. It may be noted that the head of the piles rotated. This is quite similar to the visual observations of the collapsed piled building in laterally spreading soil after the 2001 Bhuj earthquake.

Figure 8: Some failed piles in the centrifuge tests. The tests were carried out in level grounds to avoid lateral spreading.

Figure 9:Replication of the failure; (a): Failed piles in the centrifuge test carried out in level ground; (b): Collapsed piled Kandla Tower after 2001 Bhuj earthquake in laterally spreading soil, after Madabhushi et al (2001).

NEED OF A NEW APPROACH FOR PILE DESIGN IN AREAS OF SEIMIC LIQUEFACTION

It has been demonstrated in earlier sections of the paper that buckling is a possible failure mode of piled foundations in areas of seismic liquefaction. Lateral loading due to slope movement, inertia or out-of-line straightness increases lateral deflections, which in turn reduces the buckling load. These lateral load effects are, however, secondary to the basic requirements that piles in liquefiable soils must be checked against Euler's buckling. In contrast, the current design methods focus on the bending strength of the pile.

Distinguishing between bending and buckling

In design, bending and buckling are approached in two different ways. Bending is a stable mechanism, i.e. if the lateral load is withdrawn; the pile comes back to its initial configuration, provided the yield limit of the material has not been exceeded. This failure mode depends on the bending strength of the member (moment for first yield, M_Y; or plastic moment capacity, M_P) under consideration.
On the other hand, buckling is an unstable mechanism. It is sudden and occurs when the elastic critical load is reached. It is the most destructive mode of failure and depends on the

geometrical properties of the member i.e. slenderness ratio and not on the yield strength of the material. For example, steel pipe piles having identical length and diameter but having different yield strength [f_y of 200MPa, 500MPa, 1000MPa] will buckle at almost the same axial load but can resist different amount of bending. Bending failure may be avoided by increasing the yield strength of the material, i.e. by using high-grade concrete or additional reinforcements, but it may not suffice the conditions necessary to avoid buckling. To avoid buckling, there should be minimum pile diameter depending on the depth of the liquefiable soil.

Possible failure mechanisms identified

Figure 10 shows a typical time history of shear stress, excess pore pressure, displacement of ground and soil stiffness during an earthquake after Yasuda and Berrill (2001). In the figure, two time intervals are identified:

Interval 1 is the time interval between the soil being fully liquefied and lateral spreading yet to start, whereas interval 2 relates to the time interval when lateral spreading starts.

Before time interval 1, bending moments and shear forces are induced in the pile due to inertia forces. The available confining pressure around the pile is not expected to decrease substantially in this time interval. Here the behaviour of the pile may be approximately described as a beam on elastic foundation. At this stage, the pile will start losing its shaft resistance in the liquefied layer and shed axial loads downwards to mobilise additional base resistance. If the base resistance is exceeded, settlement failure of the structure will occur.

At time interval 1, slender piles will be prone to axial instability, and buckling failure may occur, enhanced by the actions of the lateral disturbing forces. A simple model is shown in Figure 11. For practical purposes, it may be assumed that the pile is virtually fixed at some depth in the non-liquefiable hard layer, shown by (D_F) in Figure 11. D_F can be estimated using Fleming et al (1992). Thus, the unsupported zone can be taken as ($D_L + D_F$) where D_L is the depth of liquefiable layer. ($D_L + D_F$) corresponds to L_0 in Figure 6 and denotes the buckling zone.

During time interval 2, the piled foundation experiences additional drag due to lateral spreading of the soil (transient forces and residual forces). Haigh (2002) showed that the transient forces can be quite high compared to the residual forces. His centrifuge results showed that the transient forces are 3 times the forces predicted by JRA (1996). These drags (transient or residual) will induce bending moment in the pile as shown in Figure 12.

Thus, the design method should safeguard the piles against:

1. Buckling failure due to unsupported pile in liquefied soil.
2. Formation of a collapse mechanism due to lateral spreading forces (transient and residual).
3. Excessive settlement leading to failure due to serviceability limit state.

The existing design method normally safeguards piles against settlement failure and failure due to lateral spreading. But it becomes obvious that the engineers should also concentrate on the buckling mode of failure for safe design of piled foundations susceptible to seismic forces.

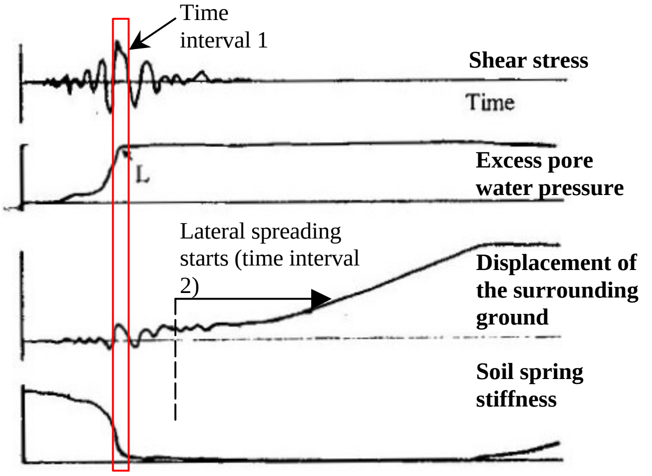


Figure 10: Typical time history of events after Yasuda and Berrill (2001).

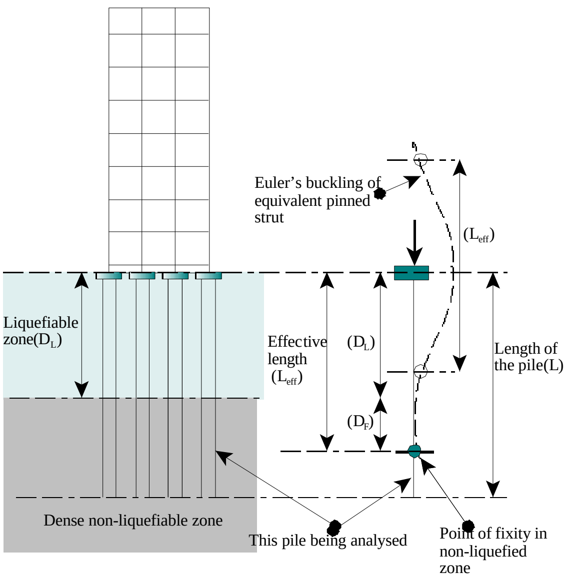


Figure 11: Idealisation of buckling instability of piled foundation.

PROPOSED DESIGN CRITERIA FOR DESIGN OF PILED FOUNDATIONS IN AREAS OF SEISMIC LIQUEFACTION

Several failure criteria can be found in the literature to determine the failure criteria for an axially loaded pile. Most commonly, the failure criteria refers to the load at which settlement continues to increase without any further increase of load, or the load causing a gross settlement of 10% of the least pile width. Essentially, these criteria are based on failure of soil surrounding and underlying the pile. The design criteria are obtained either by using an appropriate factor of safety on failure or are based on serviceability limit state (settlement) for the structure in consideration.

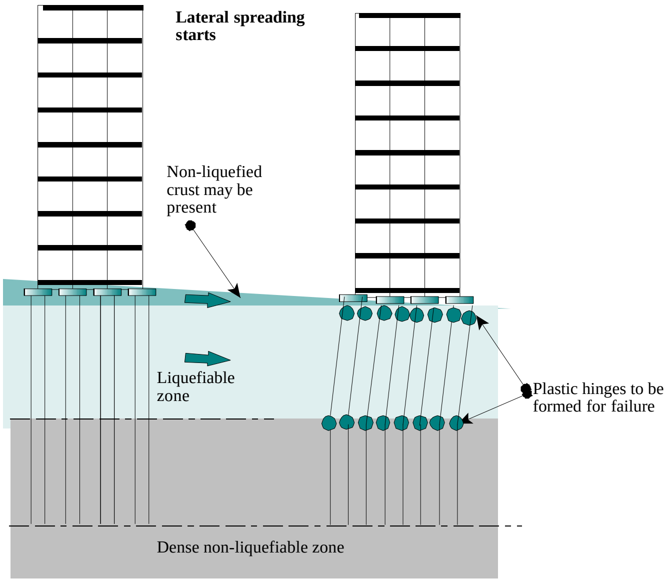


Figure 12:Collapse mechanism of piled foundation due to lateral spread.

There are no additional design criteria for piles in liquefiable soil even though structural failures of piles are abundant in almost all strong earthquakes. There is a need for setting up criteria for design of piled foundations in seismic areas based on both structural as well as serviceability point of view. The proposed design criteria for piles are as follows:

- During the entire earthquake, the pile should always be in stable equilibrium, the amplitude of vibration should be such that no section of the pile should have an ultimate limiting strain for the material, for example 0.0035 for concrete piles. This automatically ensures that no plastic hinge will form and no cracks will open up.
- The settlement of the piled foundation should be within acceptable limits for the structure. It may be noted that the pile will lose its shaft resistance in the liquefiable region as the soil liquefies, and have to settle as discussed in earlier section.

PROPOSED DESIGN APPROACH

The design process should ensure the following:

1. Avoid pile buckling under the action of axial loads ($P < P_{cr}$).
2. Avoid lateral displacement amplification effects leading to instability, due to the axial loads. P/P_{cr} should be about 0.35, which provides a safety margin of 3 on buckling.
3. Avoid plastic collapse mechanism formation due to lateral spreading loads (transient and residual).
4. Avoid excessive settlement due to the loss of shaft resistance in the liquefiable zone.

The design approach proposed here is based on idealising pile as "columns carrying lateral loads" i.e. "beam column" type structural element. Liquefied soil provides no lateral support to the pile but offers lateral load. It is also assumed that the piled foundation is fixed at some depth in non-liquefiable hard layer. Typical values show that the point of fixity lies between 3 and 6 times the diameter of the pile. It is also proposed to keep slenderness ratio of piles in the buckling zone ($D_L + D_F$) within 50 which would ensure that the piles will not only be stable but also the amplification effects can be safely ignored. The design of piles can then be carried out as beams (the effect of axial load can be ignored) with the moment of resistance reduced due to the effect of axial load.

CONCLUSIONS

The current understanding of pile failure is based on a bending mechanism where lateral spreading and inertia induce bending moment of the pile. This hypothesis treats pile as a beam element. It has been shown that the current understanding of pile failure overlooks the structural nature of pile. The current design codes needs to address buckling of piles due to loss of soil support owing to liquefaction. Criteria have been proposed for the design of piles in liquefiable soils. To avoid buckling instability of piles it has been recommended to keep the slenderness ratio of piles in the buckling zone below 50.

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