Chapter 7: Design method

7.1 Introduction

Through the analysis of reported case histories, geotechnical centrifuge tests and analytical studies, it has been demonstrated in earlier chapters that buckling is a possible failure mode of piled foundations in areas of seismic liquefaction. As discussed in section 6.4, influences such as lateral loading due to slope movement, inertia effects due to early shaking or out-of-line straightness, cause lateral deflections which are severely amplified if the axial load is permitted to approach 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, all current design methods, such as JRA (1996), NEHRP (2000) or Eurocode 8 (1998), focus on the bending strength of the pile and overlook considerations necessary to avoid buckling in the event of soil liquefaction.

In this chapter a new framework for designing pile foundations in liquefiable deposits is proposed. The principal aim of this framework is to provide a design methodology that takes into consideration all the identified pile failure mechanisms. Reported case histories are used to validate the design method. An example of pile design using the proposed method is also given to illustrate the new methodology.

7.2 Distinguishing between bending and buckling

In design, beam bending and column buckling are approached in two different ways. Piles have erroneously been designed as cantilever beams.

Bending is a stable mechanism, i.e. if the lateral load is withdrawn, the pile comes back to its initial configuration provided the yield limit of the material has not been exceeded.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 (moment for first yield, M_Y; or plastic moment capacity, M_P) of the member 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 amounts 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 to avoid buckling. To avoid buckling, there should be a minimum pile diameter depending on the depth of the liquefiable soil.

7.3 Possible failure mechanisms identified

Section 3.5 describes the worst loading condition of a single pile. In this section, an attempt is made to extend the discussion for the worst loading conditions to be experienced by a piled structure. Figure 1.9 shows a typical time history of shear stress, excess pore pressure, displacement of ground and soil stiffness during an earthquake after Yasuda and Berrill (2000). In the figure, two time intervals are identified.

Interval 1 is the time interval between the soil being fully liquefied and the time at which lateral spreading starts, whereas interval 2 relates to the time interval during lateral spreading.

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 an 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.

During 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 7.1. 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 7.1. 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 3.7b and denotes the buckling zone. A procedure for estimating D_F is shown in section 7.5.4.

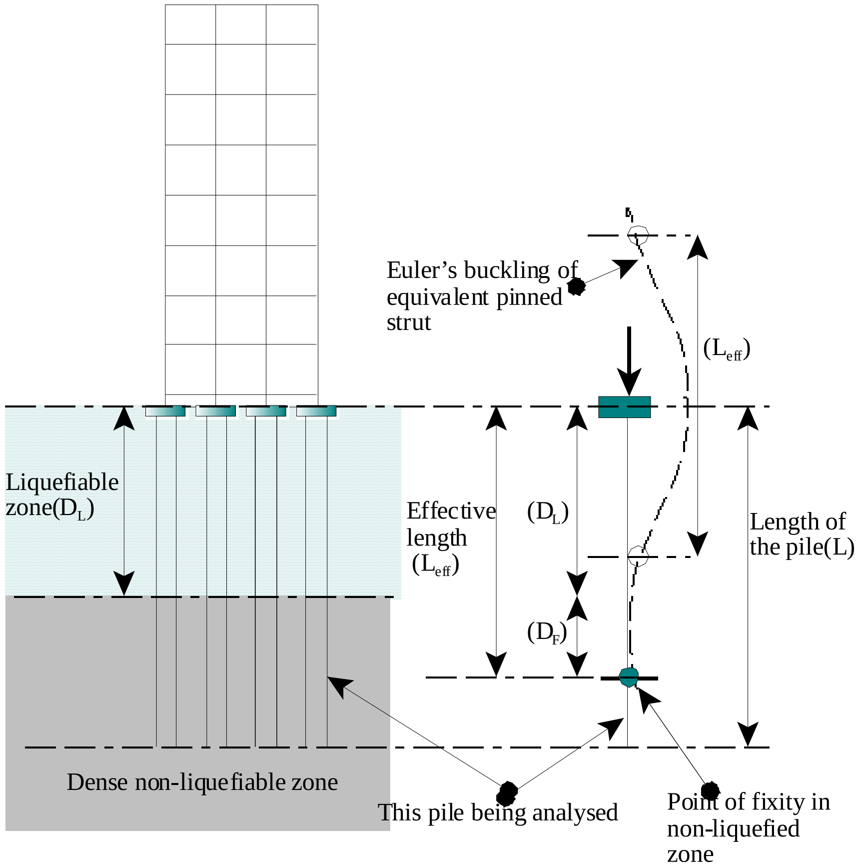

Figure 7.1: Idealisation of buckling instability of piled foundation.

During time interval 2, the piled foundation may experience additional drag due to lateral spreading of the soil. The drag will induce additional bending moments in the piles which may then fail plastically as shown in Figure 7.2.

Thus, the design method should safeguard the piles against:

1. Buckling failure due to unsupported pile carrying axial loads in liquefied soil.
2. Formation of a collapse mechanism due to additional lateral spreading forces.
3. Excessive settlement leading to serviceability failure.

The existing design method normally safeguards piles against settlement failure and failure due to lateral spreading in the absence of pile axial loads. But the research presented in this thesis shows that engineers should also concentrate on the buckling mode of failure for the safe design of piled foundations susceptible to seismic forces.

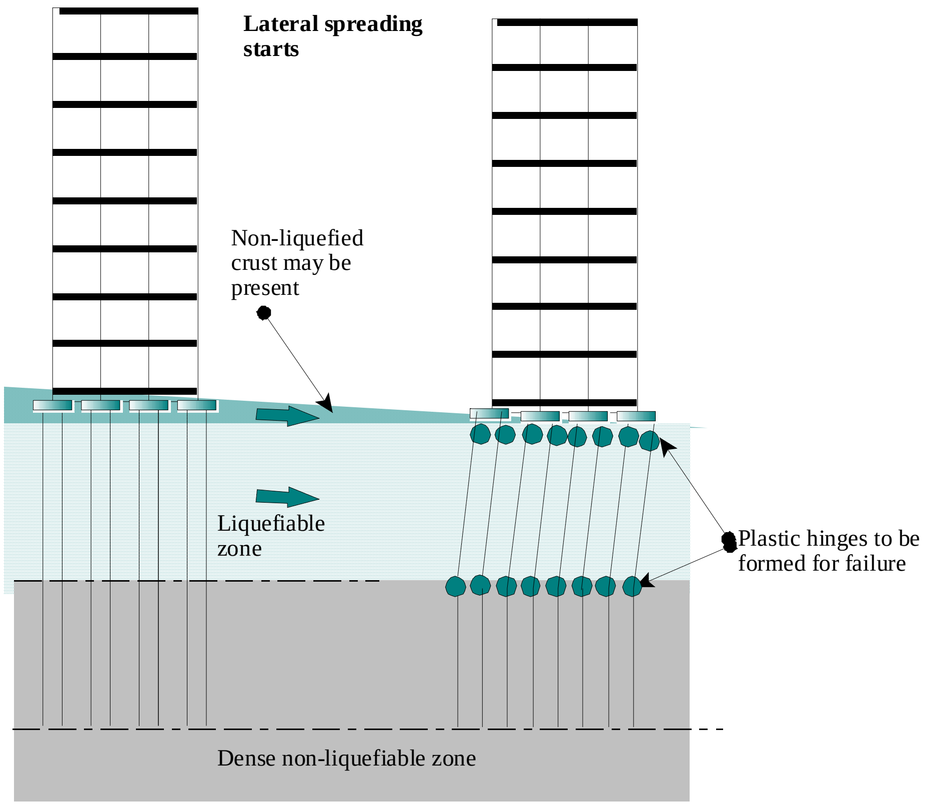

Figure 7.2: Collapse mechanism of piled foundation during lateral spread

7.4 Proposed design criteria for piled foundations

Several failure criteria can be found in the literature to determine the failure load of an axially loaded pile. Most commonly, the failure criteria refer 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 the failure of the soil surrounding and underlying the pile. The design criteria are obtained either by using an appropriate factor of safety on failure, or based on some serviceability limit state for the structure in consideration.

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 to set up criteria for the design of piled foundations in seismic areas encompassing both structural and serviceability criteria.

The proposed design criteria for piles are as follows:

- During the entire earthquake, the pile should always be in stable equilibrium, and the amplitude of displacement should be such that no section of the pile should reach the 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 section 3.5.

7.5 Proposed design approach

The design process should ensure the following:

1. Avoid pile buckling under the action of axial loads.
2. Avoid lateral displacement amplification effects leading to plastic deterioration of pile stiffness, due to the axial loads (Figure 6.2).
3. Avoid any 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 piles as "columns carrying lateral loads" i.e. "beam column" type structural elements. The present method also assumes that the piled foundation is fixed at some depth in a non-liquefiable hard layer. The liquefied soil provides no lateral support to the pile but may create lateral loads.

7.5.1 Effects of axial load

The axial force in the column member of a framed structure has two effects on the collapse. It may cause premature failure to the column member due to instability and will reduce the value of the full plastic moment of resistance.

Avoid buckling instability

To ensure stability of the pile during liquefaction, the part of the pile in liquefying soil should first be checked against Euler's buckling. The practical way of dealing with Euler's buckling is to reduce the allowable load based on slenderness ratio as shown in Figure 7.3. In the figure σ_f denotes the failure stress using Rankine's formula (see equation 3.4). This ensures the stability of the pile at full liquefaction under the action of axial load. However, this does not safeguard the pile foundation against combined axial and lateral loads.

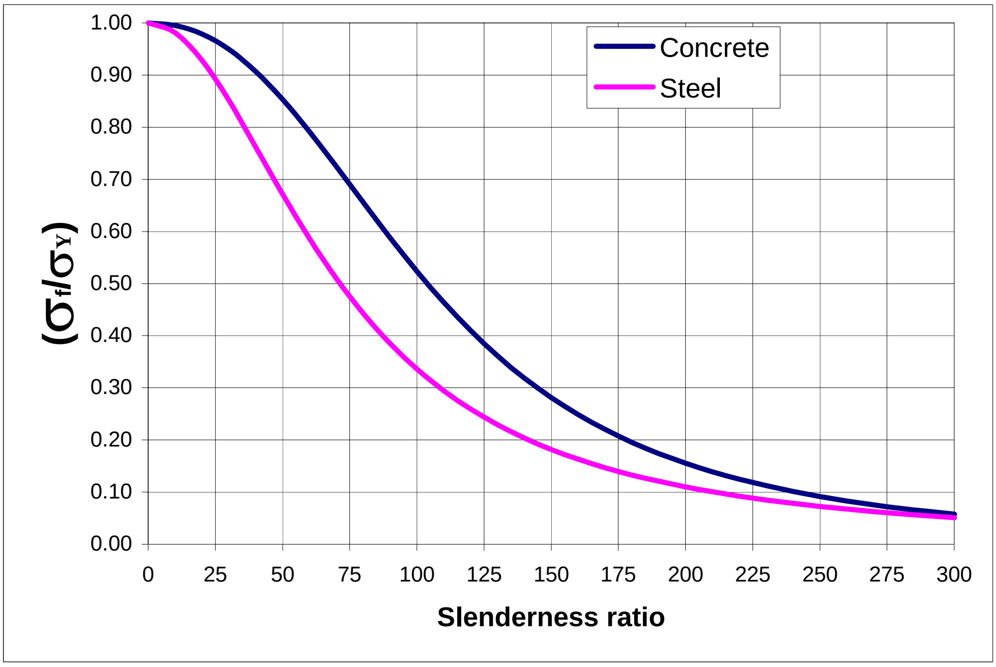

Figure 7.3:Plot of slenderness ratio against allowable stress for concrete and steel using Rankine's formula (equation 3.4).

Reduction of plastic moment capacity due to axial load

If a hinge forms under the combined action of bending moment (M) and axial load (P), the yield condition takes the form as shown by equation 7.1, Heyman (1996)

$$f(M,P) = \left(\frac{P}{P_Y}\right)^n + \left(\frac{M}{M_P}\right) = 1 \qquad (7.1)$$

where,

P_Y= Squash load in absence of bending, i.e. the element fails in pure compression by crushing of the material.

M_P = Plastic moment capacity in absence of axial load i.e. the element fails in pure bending.

Thus, the allowable bending moment (M) in a section for a certain axial load (P) can be obtained following equation 7.1. For a circular section, the interactive yield expression is given by equation 7.2

$$\left(\frac{P}{P_Y}\right)^{\frac{3}{2}} + \left(\frac{M}{M_P}\right) = 1 \qquad (7.2)$$

Figure 7.4 shows the plot of the line (given by equation 7.2), which is often termed as "yield surface for a plastic hinge under bending and thrust". Any point within the yield surface would imply that the stress in the section has not exceeded the yield stress. However this analysis assumes that the structural member is elastically stable.

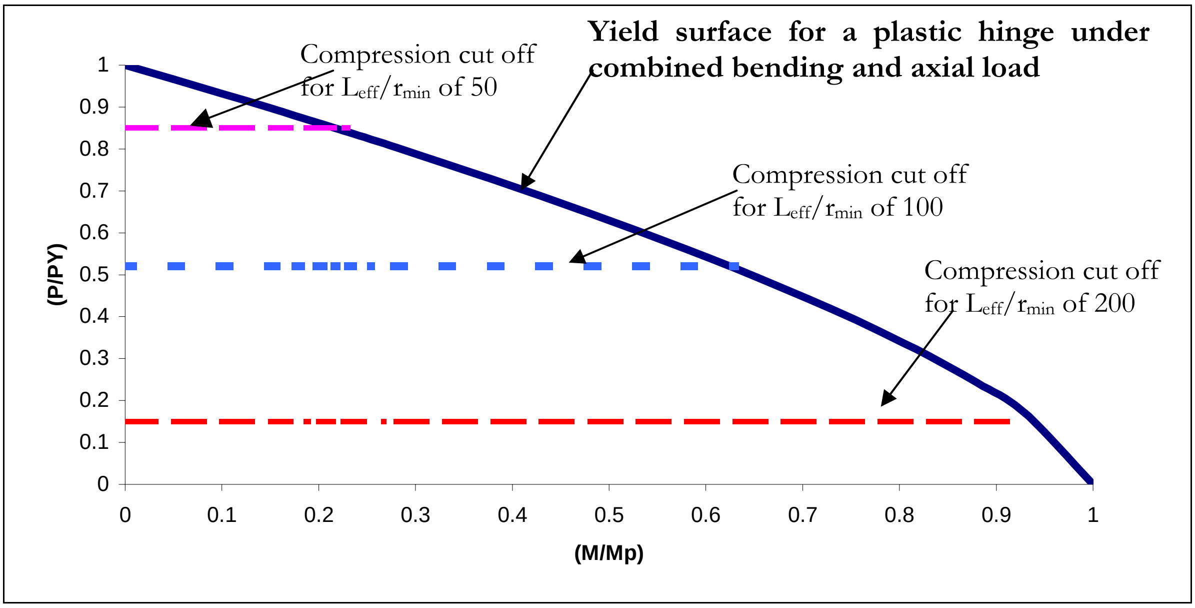

Figure 7.4: Yield surface for a plastic hinge for a circular solid pile section under bending and thrust.

7.5.2 Lateral displacement amplification effects

As discussed in chapter 6, if δ_0 is the deflection of a pile due to lateral loads as shown in Figure 7.5, the final deflection (δ) gets amplified in the presence of axial load following equation 6.1, which is graphically shown in Figure 6.2.

From Figure 6.2, it may be observed that in the presence of lateral loads, as the axial load reaches 70% of the Euler load the deflection increases nonlinearly and ultimately becomes infinitely large which is essentially the onset of buckling. It can also be noted that at $P/P_{cr} = 0.65$ the amplification of initial deflection is 3 times. This may lead to large deformation problems and "small deflection theory" is no longer valid. In most practical situations such enhanced deformations also lead to degradation of the elastic stiffness of the column, bringing down the critical load. The amplification due to the lateral loads can be avoided by maintaining the (P/P_{cr}) ratio below 0.35. This would allow a factor of safety of about 3 against buckling instability.

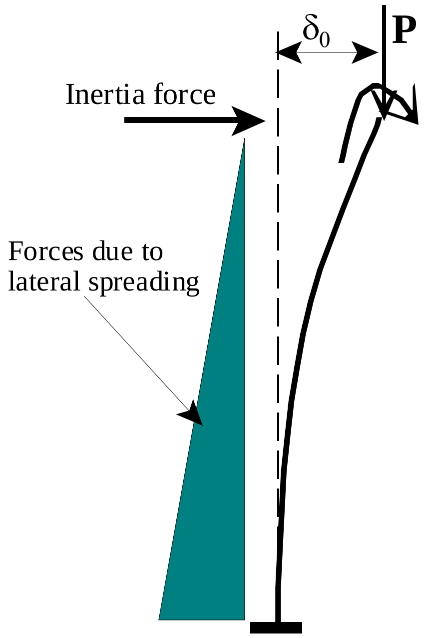

Figure 7.5:Generalised loading on a pile.

In the study of case histories in chapter 3, the slenderness ratio of 50 distinguished the good performance of piles from the poor ones. Piles having a slenderness ratio less than 50 survived the earthquakes even in laterally spreading ground, whereas nearby piled foundations having higher slenderness ratios failed. Experimental and numerical studies carried by Aberle (2000) showed that for solid "beam-column" members with slenderness value below 50, the second order effects (P-Δ) could be neglected and the lateral loads can be accounted for in simple bending calculations.

Significance of the slenderness ratio of 50

From Figure 7.3, it may be noted that for a concrete section with a slenderness ratio of 50, σ_f/σ_y = 0.85 which results in σ_f/σ_{cr} = 0.15 following equation 3.4. Similarly, for steel at a slenderness ratio of 50 the value of σ_f/σ_{cr} = 0.35. The ratio P/P_{cr} being the same as σ_f/σ_{cr} it can be concluded that a slenderness ratio of 50 signifies (P/P_{cr}) below 0.35 for steel and 0.15 for concrete. Thus following Figure 6.2, for slenderness ratio below 50 the pile would be stable under any lateral loading and no significant amplification of lateral deflection is possible.

For concrete piles at a slenderness ratio of 50 the ratio σ_f/σ_y = 0.85 (see Figure 7.3) and it must be remembered that P/P_Y is same as $\sigma_f/\sigma_{y.}$ It may then be concluded from Figure 7.4 that almost the entire region (90-95%) within the yield surface can be utilised and a very small portion at the top is compression cut off.

Thus it is proposed to keep slenderness ratio of piles in liquefiable zone within 50 which would ensure that the piles will not only be stable but also the P-Δ effect can be safely ignored. The design of piles can be carried out as beam (the effect of axial load can be ignored) with the moment of resistance reduced due to the effect of axial load following equation 7.1.

7.5.3 Check against collapse due to lateral loads (inertia, transient and residual) and axial loads

As the slenderness ratio of the piles is proposed to be maintained below 50, lateral loads – however large they may be – would not significantly amplify the lateral deflections. The effects of axial load can then be ignored in this analysis of the collapse mechanism due to lateral loads. However, the plastic moment capacity (M_P) of the pile needs to be reduced to take into the effect of axial load as discussed in section 7.5.1.

In this step, different feasible collapse mechanisms will be assumed. The lateral load required for each mechanism to be formed will be determined. An example is shown by considering a single pile from Figure 7.2. The free body diagram of the pile at failure is shown in Figure 7.6. The pressure distribution is assumed to be triangular following JRA (1996). It is also assumed based on the JRA (1996) code that inertia and lateral spreading forces do not act together (section 2.5.1). The lateral collapse load can be calculated from the moment equilibrium

Collapse load= $H_{collapse} = 6.\dfrac{M_P{}'}{L_{eff}}$ (7.3) where

$M_P{}'$= Reduced M_P due to the axial load following equation 7.1

The design method should ensure that the lateral load (due to inertia, transient and residual) does not exceed $H_{collapse}$ with an appropriate factor of safety.

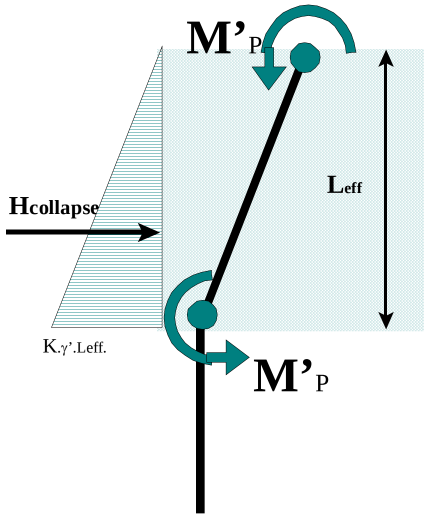

Figure 7.6: Collapse mechanism for a single pile in Figure 7.2.

7.5.4 Point of fixity in non-liquefiable layer

Davisson and Robinson (1965) developed an approximate procedure for treating the problem of buckling of partially embedded piles. In the procedure, the partially embedded pile is represented as a freestanding pile with a fixed base and having an equivalent length (L_{eq}) calculated based on relative stiffness of soil and pile as shown in Figure 7.7. This procedure has also been widely accepted in practice: Tomlinson (1994).

The method involves the computation of stiffness factor T for a particular combination of pile and soil and as defined by equation 7.4.

Stiffness factor $T = \sqrt[5]{\dfrac{EI}{\eta_h}}$ (in units of length)………………(7.4) where,

EI = Stiffness of the pile

η_h = Soil modulus or constant of horizontal subgrade reaction having unit of force/length^3.

In this method, it is assumed that for granular soils, the soil modulus increase linearly with depth.

Depth to point of fixity is taken as 1.8 T as recommended by Davisson and Robinson (1965)

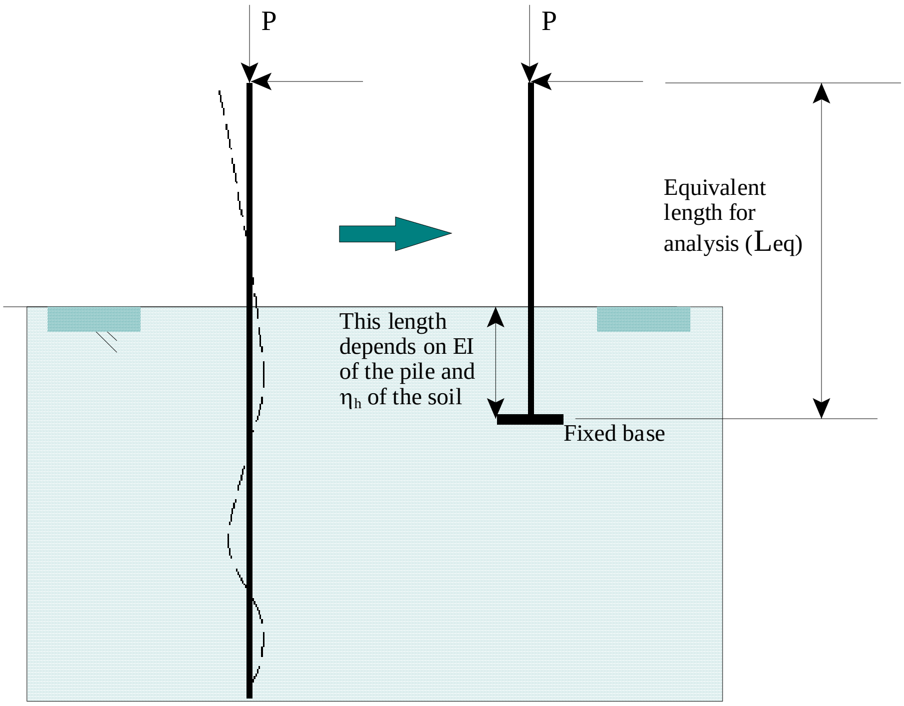

Figure 7.7: Conceptual model for design of partially embedded pile after Davisson and Robinson (1965).

This concept has been proposed to predict the point of fixity in a non-liquefiable hard layer. The depth of fixity (D_F) in Figure 7.1 is thus a function of pile stiffness and η_h of the non-liquefiable hard soil underlying the liquefiable soil. The slenderness ratio is a function of the depth of the liquefiable layer (D_L), depth of fixity (D_F) and the cross section of the pile (r_{min}). The allowable slenderness limit (L_{eff}/r_{min}) can be taken to be 50 as discussed earlier.

7.5.5 Proposed design chart for choosing pile diameter

Various simplifications are necessary in order to provide a simple and safe solution to this complex problem of pile-soil interaction. Very elaborate calculations are not justified because of various uncertainties such as non-homogeneity of natural soil, sensitivity of buckling to minor imperfections or lateral resistance of liquefied soil once the pile starts to buckle (section 6.10). It is very difficult to reproduce these factors in calculations.

In this section, an attempt has been made to carry out some simple calculations based on typical values of η_h for dense sandy soil. The value of η_h is taken to be 40MN/m^3 for 80% relative density sand following API (1993) code. The minimum thickness of steel pile is chosen from the API (1993) code as discussed in equation 2.2. Tables 7.1 and 7.2 show the depth of fixity for concrete and steel piles for different depths in liquefiable soil. The slenderness ratio is maintained at 50 as discussed earlier. Typical values show that the point of fixity lies between 3 to 6 times the diameters of the pile.

Figure 7.8 plots the minimum diameter of the pile required for a given of depth of liquefiable layer for steel and concrete piles. This ensures stability of the pile under axial load and lateral displacement amplification effects due to axial loads. A check is required against the formation of a collapse mechanism.

Table 7.1: Depth of fixity for concrete piles

Depth in liq layer (m)	**Dia (D) m**	**(I) m^4**	**E (MPa)**	**Soil Mod (MN/m^3)**	**T m** (Eq 7.4)	**Depth of fixity (1.8T) (m)**	**L_{eff} (m)**	**L_{eff}/r_{min}**
10	1.1	0.072	2.25E+04	40	2.10	3.77	13.77	50.1
12	1.4	0.188	2.25E+04	40	2.54	4.57	16.57	47.4
14	1.6	0.322	2.25E+04	40	2.83	5.09	19.09	47.7
16	1.8	0.515	2.25E+04	40	3.11	5.59	21.59	48.0
18	2	0.785	2.25E+04	40	3.38	6.09	24.09	48.2
20	2.2	1.149	2.25E+04	40	3.65	6.57	26.57	48.3

Table 7.2: Depth of fixity for steel piles

Depth in liq layer (m)	**Dia (D) m**	**"t" (mm) API code**	**(I) m^4**	**E GPa**	**Soil Mod (MN/m^3)**	**T m** (Eq 7.4)	**Depth of fixity (1.8T) (m)**	**L_{eff} m**	**r_{min} m**	**L_{eff}/r_{min}**
10	0.75	14	0.0022	210	40	1.63	2.93	12.93	0.3	49.70
12	0.9	16	0.0043	210	40	1.87	3.36	15.36	0.3	49.15
14	1	17	0.0063	210	40	2.02	3.63	17.63	0.3	50.72
16	1.2	19	0.0123	210	40	2.30	4.14	20.14	0.4	48.23
18	1.3	20	0.0165	210	40	2.44	4.39	22.39	0.5	49.47
20	1.4	21	0.0216	210	40	2.58	4.64	24.64	0.5	50.53

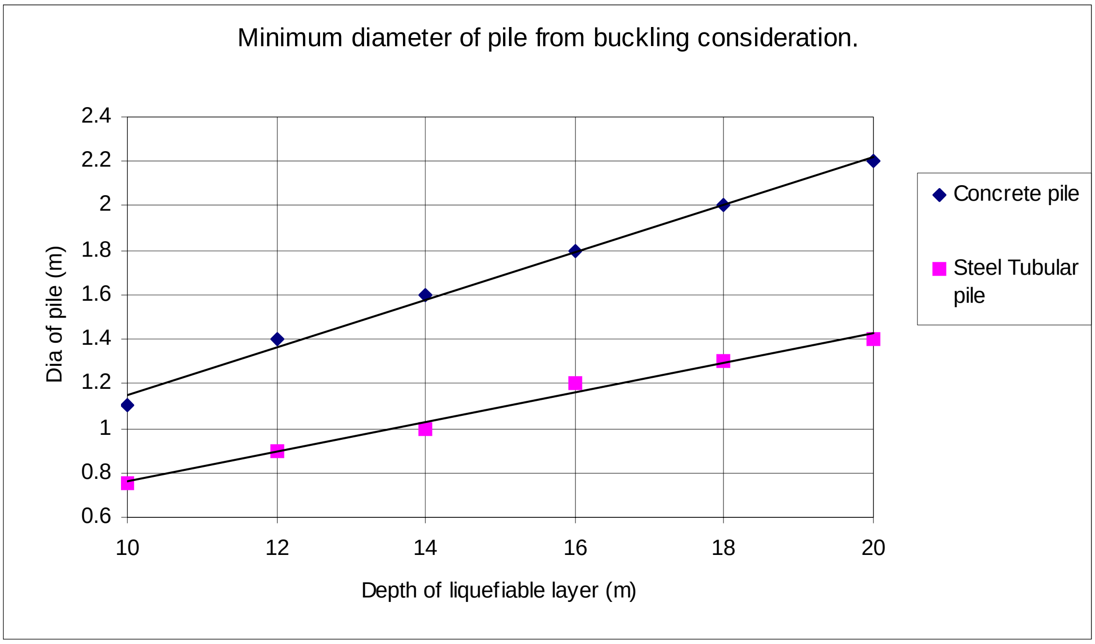

Figure 7.8: Plot of minimum diameter of pile required

7.5.6 Allowable lateral load for the piles having slenderness ratio 50

This section attempts to make an estimate of the allowable lateral spreading load that can be carried by the pile having slenderness ratio of 50.

Following Figure 7.7

$$H_{collapse} = \frac{1}{2}(K.\gamma'.L_{eff}).L_{eff}.D = \frac{6M_P^{'}}{L_{eff}} \quad ....(7.5) \text{ where}$$

D = diameter of the pile.

K= lateral earth pressure coefficient.

On simplification

$$K = \frac{12M_P^{'}}{(L_{eff})^3.\gamma'.D} \quad ....(7.6)$$

For M40 concrete piles, yield strength for 0.0035 strain = 17.84MPa (0.446 f_{ck}), the allowable earth pressure coefficient before collapse is tabulated in Table 7.3 for different depths of liquefiable layer. In the calculation the minimum diameter is chosen based on stability requirements i.e. Table 7.1.

For steel tubular piles a similar calculation is done and is shown in Table 7.4, the yield strength of steel is assumed to be 500MPa.

Table 7.3: Allowable lateral earth pressure for RCC piles

Depth of liquefiable layer (m)	Dia of pile (m)	M_P kNm	L_{eff} (m)	Allowable lateral earth pressure Coefficient (K)
10	1.1	3957	13.77	2.06
12	1.4	8158	16.57	1.92
14	1.6	12178	19.09	1.64
16	1.8	17340	21.59	1.43
18	2	23786	24.09	1.27
20	2.2	31660	26.57	1.15

Table 7.4: Allowable lateral earth pressure for tubular steel piles

Depth of liquefiable layer (m)	Dia of pile (m)	M_P kNm	L_{eff} (m)	Allowable lateral earth pressure Coefficient (K)
10	0.75	3792	12.93	3.5
12	0.9	6252	15.36	2.87
14	1	8214	17.63	2.24
16	1.2	13251	20.14	2.02
18	1.3	16385	22.39	1.68
20	1.4	19968	24.64	1.43

In reality, M'_P in equation 7.6 will not only be restricted to the plastic moment of the pile (as used in the calculation presented in this thesis) but also will depend on the contributory stiffness of the other members in the pile cap. Berrill et al (2001) used this approach in the analysis of Landing Road Bridge (see Figure 3.10).

It must be noted that an increase in diameter by 10% will increase the lateral spreading load by 10% but the plastic moment capacity would rise by 30%.

7.6 Flow chart of the design method

This section of the chapter will outline the algorithm of the proposed design process.

Step 1

Use inclined piles to take any horizontal components of static load, e.g. retaining structures. Establish static axial loads and minimum eccentricity on all piles.

Step 2

From the site investigation data, identify any non-liquefiable crust, e.g. clay layer, that might be mobilised to translate laterally when r_u=1 beneath it. From the slope of the ground and seasonal variation of ground water table, identify whether ground spreading is possible.

Step 3

Identify the depth of any liquefiable layer, e.g. loose or medium dense sands or silts. Based on the material of pile and depth of liquefiable layer, choose a pile section following Figure 7.8. Estimate lateral pressures on the piles in these zones. Ignore skin friction and lateral support in these zones.

Assume that the piled structure does not move, and that the crust does move, creating passive earth pressures on walls and ultimate lateral pressures on piles. Calculate shear force H.

Step 4

For the chosen pile diameter, estimate the length of the pile required beneath the liquefiable zone to carry the axial load. Ignore the skin friction in the liquefiable layer. From economic considerations choose the number of piles and the length. However, a minimum depth of embedment is necessary in the hard layer for fixity of the pile i.e. to create moment restraint.

Step 5

Estimate the depth of fixity of the pile in the non-liquefiable hard layer based on the standard conventional procedure for partially embedded pile, for e.g. Davisson and Robinson (1965).

Create a pilecap with full moment fixity at the pile heads wherever possible. Estimate the equivalent length of pile for Euler's buckling considering the restraints offered by the pile cap and the zone of embedment beneath the liquefiable soil layer. Keep the slenderness ratio of the pile in the unsupported layer below 50.

Step 6

Assume possible collapse mechanisms, for e.g. Figure 7.6, and estimate the lateral load required to form the mechanisms. The reduction of plastic moment capacity due to axial load is to be taken into consideration. Calculate the factor of safety for the formation of plastic collapse mechanisms against lateral loads. If the pile section is inadequate, increase the plastic moment capacity of the section. The options are using higher-grade concrete or increasing the thickness of the steel pile section. If that is not feasible, tubular steel pile filled with concrete can be used or the diameter of the pile may be increased.

7.7 An example problem

An eight-storied building is considered as shown in Figure 7.9 to explain the proposed design method. It is founded on liquefiable soil having depth of 10m and underlying by hard soil having soil modulus of 40MN/m^3. Grade of concrete = 25MPa. In this example the lateral loads on the pile is estimated using JRA (1996). Detailed calculations are omitted in the interests of brevity.

Design loads

From detailed structural analysis and design calculations, the following data are extracted.

1. Typical axial load at each column base = 1800kN (Dead load + live load + appropriate dynamic earthquake loads)
2. Typical lateral inertia load at each column base = 50kN (from base shear) for the design earthquake.

Stability considerations

From Figure 7.8, the initial guess of the pile diameter = 1.1m

Depth of fixity = 3.77m (from Table 7.1).

Unsupported length = (10 + 3.77) m= 13.77m

Effective length = 13.77m [From the boundary conditions, Figure 3.6]

Slenderness ratio = 49.7 (less than 50, SAFE against buckling).

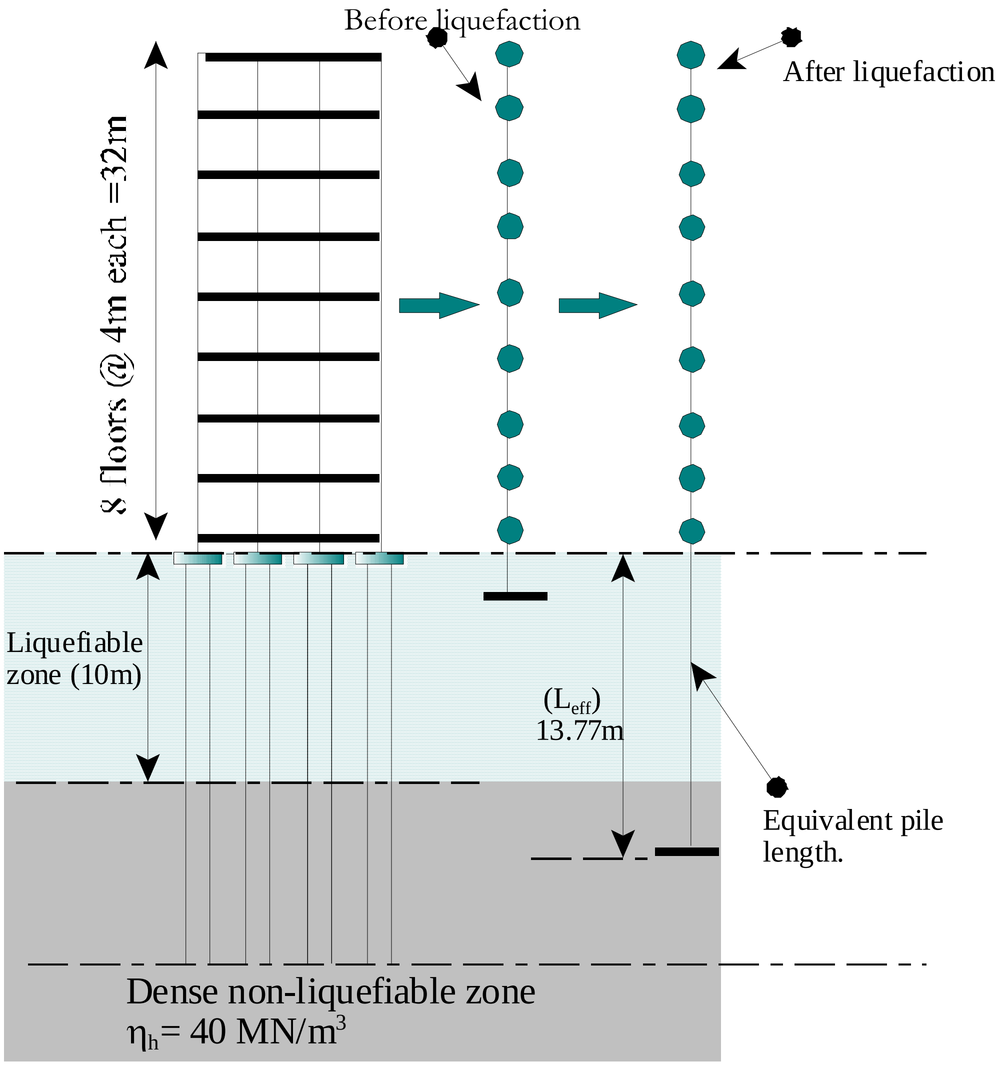

Figure 7.9: An example problem

Length of the pile for serviceability limit state

Here the shaft resistance in the top 10m is ignored.

For the type of soil the

- End bearing capacity of the pile is given by = $2450D^2$ kN, where D = dia. of the pile
- Shaft capacity in Layer II (Non-liquefiable layer) = (248L) kN, where L = length of the pile in non-liquefiable hard layer.

Choosing 1.1m dia pile,

END BEARING RESISTANCE = 2960 kN

SHAFT RESISTANCE

- For 6m embedment in hard layer = 1488 kN

- For 10m embedment in hard layer = 2480kN
- For 15m embedment in hard layer = 3720kN

Depth of fixity = 3.77m. Allowing a "Factor of Safety" of 1.5 the minimum embedment = 6m

Choosing 1.1m dia, 20m of total pile length. It gives ultimate load of (2480 + 2960) kN= 5440kN. Allowable load allowing a factor of safety of 3 = 5440/3 =1813kN.

Thus one pile (1.1m) is required per column.

Check against mechanism formation

For M25 grade concrete, assuming yield strength in bending = 11.2 MPa (0.446 f_{ck})

M_p of the section of the pile in absence of axial load = $\left(\frac{1.1^3}{6}\right) \times 11.2$ MPa = 2485kNm

P = 1800kN

P_Y= 15918 kN (assuming yield strength in compression = 16.75MPa)

From Figure 7.4, M_p'= available plastic moment capacity = 96% of M_p= 2385kNm

Considering a single pile-pilecap connection and assuming there is no non-liquefied crust (Figure 7.6), the lateral load required to form the mechanism is estimated below.

Collapse load ($H_{collapse}$)= $6\left(\frac{M_p'}{L_{eff}}\right)$ **= 1039kN,** following Figure 7.6.

The lateral spreading forces should not exceed the collapse load (1039kN)

Calculation of lateral spreading forces following JRA 1996

For 10m of liquefiable layer

Lateral pressure at the bottom of the pile = 30% of overburden pressure

= 0.3 × 10m × 18kN/m^3=54kPa.

Total load = 297kN

Thus Factor of Safety against residual lateral spreading loads = (1039/297) =3.49

For seasonal variation of water table the depth of non-liquefied crust can be estimated and thus the corresponding Factor of Safety.

7.8 Summary

A design method has been proposed for design of piles in liquefiable soils. This method ensures the stability of piled foundations during the entire earthquake. Provisions are kept for adequate stiffness and strength against bending for lateral spreading forces. It also takes care the serviceability limit state by controlling settlement during full liquefaction. It has been found that in areas of seismic risk, large diameter high modulus pile is better than a group of slender piles carrying the same axial load. A minimum diameter of pile is proposed depending on the depth of liquefiable zone and the material of pile.