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Chapter 55
Pile-group design

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Pile design often focuses on ultimate geotechnical capacity. When piles are located within a pile group, a wide range of additional factors need to be considered, including the deformation of the pile group and the structural forces induced in the piles and pile cap. Interaction between adjacent piles depends on several factors, including the direction of applied loads. When large horizontal loads are applied a complex range of interaction effects may need to be considered. Computer-based analytical methods are commonly used; nevertheless, it is important to use simplified methods of analysis both for preliminary design and to judge the reliability of output from computer-based methods. Pile-group deformation is usually more critical than group capacity, and, hence, the selection of appropriate deformation moduli is particularly important. Case history data and predictions from nonlinear modelling are compared with linear elastic methods. The delegation of design responsibility for pile-group design requires careful judgement and is also discussed.

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55.1. Introduction

The design of single piles is discussed in Chapter 54 *Single piles*, where the main focus is the assessment and verification of the ultimate geotechnical capacity of the pile. For pile groups, a wider range of factors will need to be considered. The design must consider the overall performance of the whole foundation system (**Figure 55.1**). It is often the case, especially for larger pile groups, that pile-group deformation and the structural forces (bending moments and shear forces) induced in the piles are the main design issues, rather than ultimate geotechnical capacity. A pile group will stress a much larger zone of soil than a single pile (**Figure 55.2**), and this will be extremely important when considering how to develop an appropriate analytical model and input parameters, and assessing potential failure and deformation mechanisms. For example, if there is a soft compressible layer at depth below the pile toe, it may have little effect on single-pile behaviour, but could cause excessive pile-group deformation or even collapse.

The overall behaviour of pile groups is affected by their geometry (length, width, etc.) and number of piles within a group, n. Pile groups have been classified by Basile (2020) by the number of piles in a group: 'small' ($n < 5$ piles), 'medium' (5–25 piles) and 'large' ($n > 25$ piles). This classification is particularly useful when considering pile-group capacity (i.e. ultimate limit state (ULS) design) – refer to

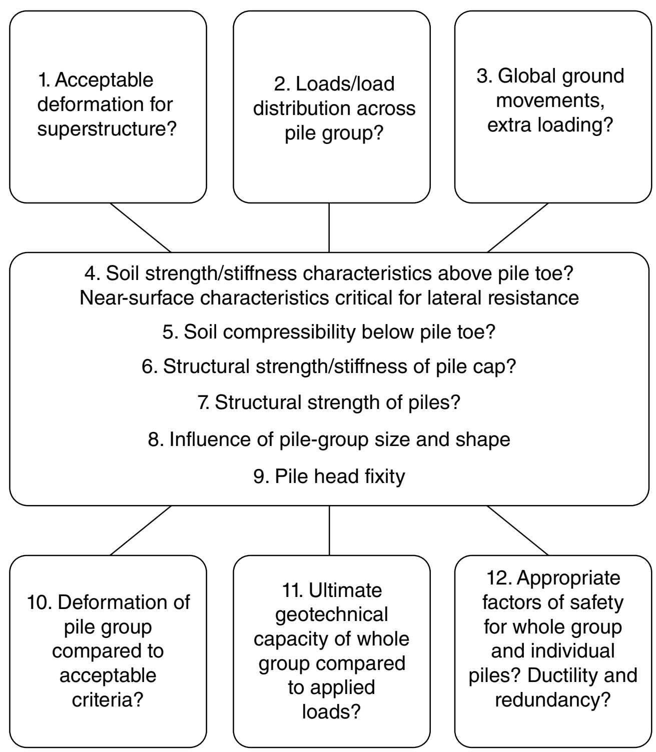


Figure 55.1 Design considerations for pile groups (Modified from O'Brien and Bown, 2010)

Section 55.2. The design of ‘medium’ and ‘large’ pile groups is typically controlled by deformation (i.e. serviceability limit state (SLS)) and structural design considerations – refer to Sections 55.3–55.6. A geometrical parameter which gives more general and powerful insights into group behaviour (both ULS and SLS) is the pile-group aspect ratio R (Randolph and Clancy, 1993), defined as $R = (ns/L)^{0.5}$, where n is the number of piles in the group and s and L are the pile spacing and length, respectively (both in metres). When the pile-group aspect ratio is large (**Figure 55.3(a)**), then most of the applied load is likely to be shed into the ground through compression of the deposits below the pile-toe level. In contrast, if R is small (**Figure 55.3(b)**), then most of the applied load is likely to be shed into the ground through shear around the pile-group perimeter. Randolph (1994) suggests that a pile group is ‘large’ if $R > 4$ and ‘small’ if $R < 2$ (therefore ‘medium’ applies if R is 2–4). Viggiani *et al.* (2012) define ‘large’ pile groups as those with a width, B, greater than pile length, L, whereas ‘small’ pile groups are those where $B < L$. Viggiani *et al.* provide several practical examples of how a simple appreciation of pile-group geometry (using the above classifications) can facilitate an appropriate choice of analytical methods and improved pile-group design.

In modern engineering practice, pile-group analysis usually relies upon commercially available software. However, most software has a number of limitations, namely

- soil behaviour is often assumed to be linear elastic
- the influence of soil layering below the pile-toe level may be poorly simulated.

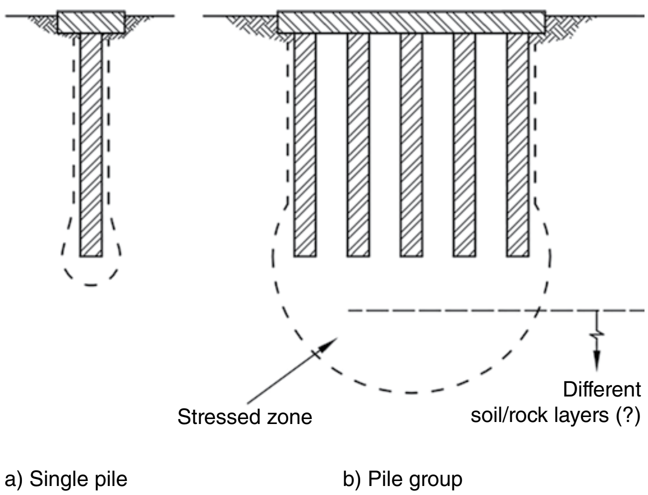


Figure 55.2 Comparison of influence zones beneath single piles and pile groups

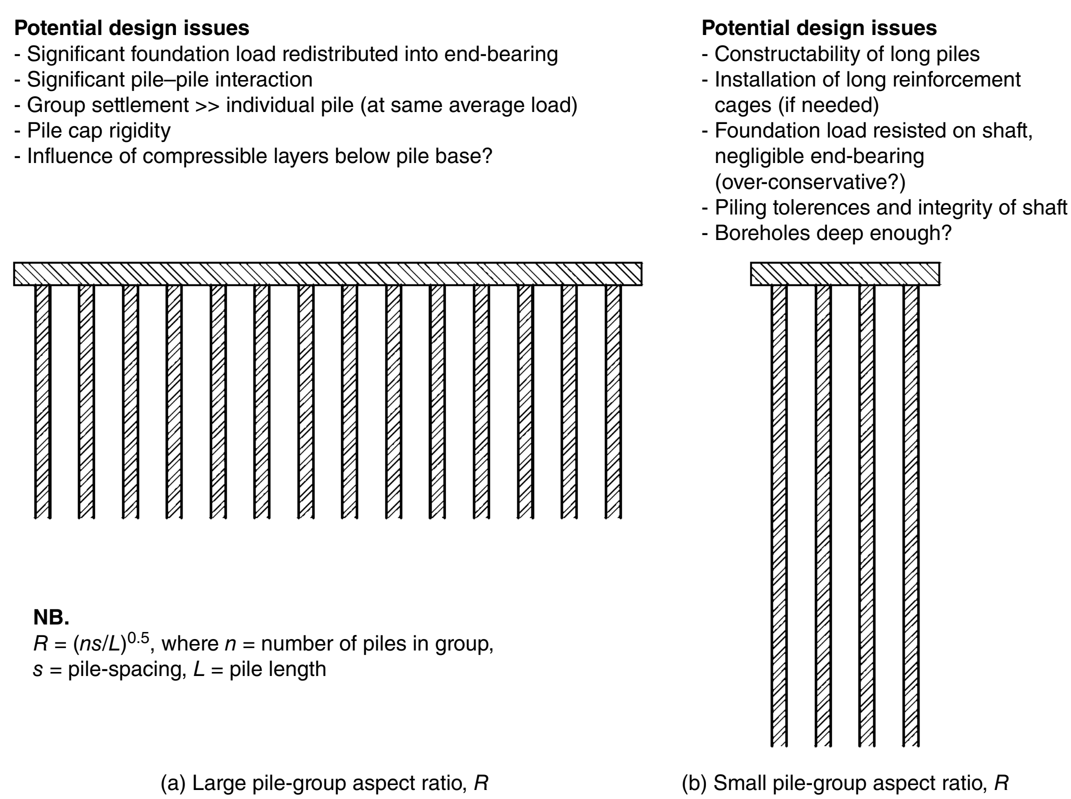


Figure 55.3 Pile-group aspect ratio, *R*, and potential design issues

Analytical methods for pile groups are often complex; therefore, the main intent of this chapter is to highlight

- the key factors that will affect pile-group behaviour
- several simplified analytical methods that can be useful either for preliminary design or to check computer output
- when linear elastic methods may give output that is appropriate for practical design, and when more complex methods are necessary
- how selected outputs, in particular axial load distributions, can be interpreted to facilitate more economic design.

55.2. Pile-group capacity

Independent calculations should be made of ultimate capacity based upon

(*a*) the sum of the individual pile capacities (based on capacity of isolated piles, see Chapter 54 *Single piles*) (**Figure 55.4(a)**)

(*b*) failure of the 'block' that encloses the perimeter of the pile group (**Figure 55.4(b)**)

(*c*) failure of rows of piles (**Figure 55.4(c)**).

For vertically loaded pile groups, usually only (*a*) and (*b*) will need to be checked; failure mode (*c*) becomes more important if the spacing between pile rows is variable or if the horizontal or moment loading is relatively large.

Most design codes offer little guidance on pile-group design, which often leads to over-conservative design. It is not necessary to provide the same geotechnical compressive ground resistance to peak axial load ratio for, say, a corner pile of a large pile group as is required for a single pile or small pile group (defined here as $n < 5$ piles). If the pile cap has adequate structural strength and stiffness, and the pile group has five or more piles (and, therefore, can be considered fully redundant (NCHRP, 2004)), then the designer should not be concerned about **local** geotechnical 'factors of safety' for individual piles within a group. As a highly loaded pile in a medium or large group attempts to settle more than adjacent piles, load redistribution occurs through the pile cap, which then reduces local peak loads and leads to a more uniform load distribution across a group. The important limit states to check are then

- the structural strength and stiffness of the pile cap and structural capacity of the single piles
- that the SLS pile-group deformation is acceptably small for the superstructure
- the durability of the foundations for the intended design life of the structure
- potential effects of global ground movements on the foundations (refer to Chapter 57 *Global ground movements and their effects on piles*)
- if relevant, earthquake effects such as liquefaction potential (refer to Chapter 60 *Foundations subjected to cyclic and dynamic loads*)
- if relevant, dynamic or cyclic response to wind or wave actions (refer to Chapter 60 *Foundations subjected to cyclic and dynamic loads*).

The failure of pile groups is rare; however, the following situations require particular care.

(*a*) When large global horizontal ground deformations develop around a pile group, usually due to out-of-balance loading on a soft clay or peat layer (**Figure 55.5(a)**), either due to placing fill or excavating adjacent to the pile group.

(*b*) When a layer of soft clay underlies a thin competent layer, and end-bearing piles are toed into the competent layer (**Figure 55.5(b)**).

(*c*) When end-bearing piles are toed in rock, which contains either dipping clay-filled discontinuities or voids (**Figure 55.5(c)**). Voids may arise in a rock mass either due to natural solution features or due to historic mining activity.

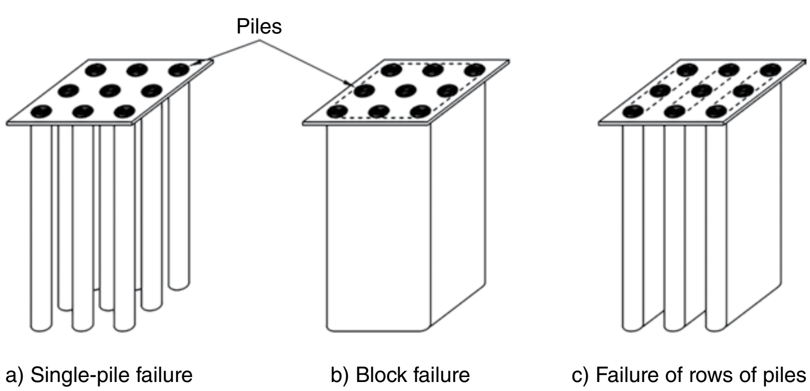


Figure 55.4 Failure modes for pile groups (Reproduced from Fleming *et al.*, 2009)

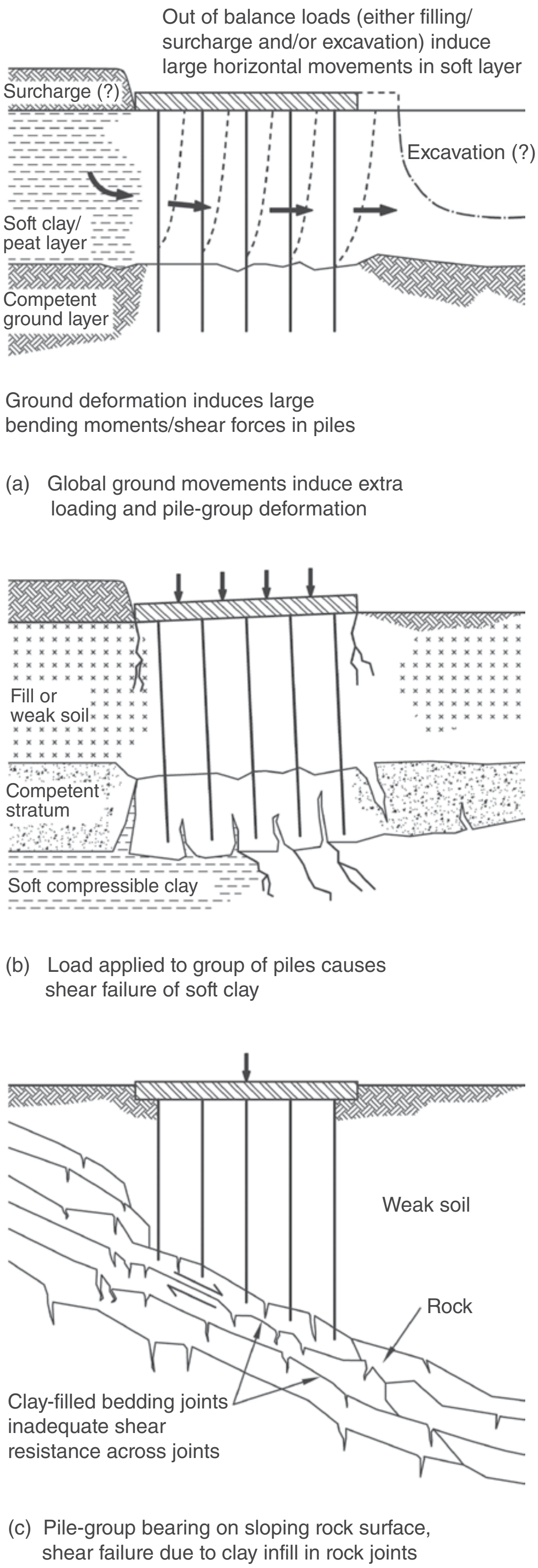


Figure 55.5 Pile-group failure, some high-risk situations

In the authors' experience, (*a*) is the most common cause of pile-group failure (usually due to inadequate structural pile strength). This has occurred when the pile designers or constructors did not appreciate the potential impact of global ground movements and this failure mode was completely overlooked. Chapter 57 *Global ground movements and their effects on piles* discusses the influence of global ground movements (both vertical and horizontal) on pile-group behaviour. This failure mode can often be avoided either by appropriate ground improvement (Chapter 59 *Design principles for ground improvement*) or by appropriate construction planning and sequencing. For example, installing the piles *after* the fill or excavation, adjacent to the pile group, has been completed. The analytical methods used to assess ultimate pile-group capacity are based on conventional bearing capacity theory (refer to Chapters 21 *Bearing capacity theory* and 53 *Shallow foundations*). Hence, for *vertical loading of a pile group in clay* the ultimate capacity of the block of soil encompassed by the group is

$$Q = 2L(B_1 + B_2)s_{u,av} + s_{u,b}N_cB_1B_2 \tag{55.1}$$

where L is the pile length or embedded depth in competent founding strata, B_1 is the pile-group breadth, B_2 is the pile-group width (i.e. B_2 is the larger of the plan dimensions for a rectangular pile group), $s_{u,av}$ is the average undrained shear strength around the group perimeter, $s_{u,b}$ is the undrained shear strength beneath the group (the average between the pile-toe level and $0.5B_1$ below the toe level for homogeneous conditions), and N_c is the bearing capacity factor allowing for appropriate depth, width and shape (refer to **Table 55.1**, which summarises the appropriate bearing capacity and shape factors). Total stress methods are commonly used for clays. However, effective stress methods can be utilised (refer to Chapter 54 *Single piles*) and are recommended if deep

	N_c	
L/B_1	$B_2/B_1 = 1$	$B_2/B_1 \geq 10$
0.25	6.7	5.6
0.50	7.1	5.9
0.75	7.4	6.2
1.00	7.7	6.4
1.50	8.1	6.8
2.00	8.4	7.0
2.50	8.6	7.2
3.00	8.8	7.4
≥ 4	9.0	7.5

For intermediate B_2/B_1 ratios, use linear interpolation.

Table 55.1 Bearing capacity factors for pile groups (After Viggiani *et al.*, 2012)

excavations (say in excess of 4 m) are constructed above or adjacent to the pile group (since the excavation would lead to a reduction in the clay's mobilised strength in the long term).

The influence of an underlying compressible layer on the ultimate capacity of the group (acting as a block or equivalent pier, refer to Section 55.5.4) can be assessed (based on studies by Matsui, 1993) as

$$Q_b = Q_{u2} \quad \text{for} \; \frac{h_c}{d_{eq}} \leq 0.5 \tag{55.2}$$

$$Q_b = Q_{u1} \quad \text{for} \; \frac{h_c}{d_{eq}} \geq 3 \tag{55.3}$$

and for z_c/d_{eq} between 0.5 and 3

$$Q_b = Q_{u2} + \left(\frac{0.4h_c}{d_{eq}} - 0.2\right)(Q_{u1} - Q_{u2}) \tag{55.4}$$

where Q_b is the ultimate bearing capacity of the base, Q_{u1} is the ultimate bearing capacity of the upper layer, Q_{u2} is the ultimate bearing capacity of the underlying weak compressible layer, h_c is the depth of the underlying layer below the pile-group toe level and d_{eq} is the equivalent pier diameter (**Figure 55.6**).

If the upper layer is sand or gravel then Q_{u1} can be assessed by using conventional bearing capacity theory based on effective stress parameters (refer to Chapters 21 *Bearing capacity theory* and 53 *Shallow foundations*) together with appropriate shape and depth correction factors. For weak rocks, the nature and spacing of discontinuities in the rock mass need to be considered, as outlined in Chapter 53 *Shallow foundations*. For clays, conventional bearing capacity theory can be used, together with appropriate shape and depth correction factors (Equation 55.5)

$$\text{Ultimate bearing pressure } q_b = N_c s_{u,b} \tag{55.5}$$

where N_c is the appropriate bearing capacity factor, allowing for depth and shape correction factors (**Table 55.1**) and $s_{u,b}$ are defined above.

For pile groups subject to horizontal or moment loading and underlain by relatively weak layers, the potential for edge

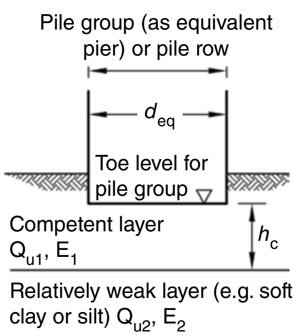


Figure 55.6 Pile group (or row) founded on a competent layer over a weak layer (After Matsui, 1993)

failure should be checked by assessing the bearing resistance beneath the outer (more heavily loaded) pile row. It is usually sufficient to calculate an average bearing pressure at the toe level beneath the row (based on the mobilised end-bearing pressure). The factor of safety against local failure can then be checked against Equations 55.2–55.5.

Historically, there was some concern about the capacity of a group of piles relative to the sum of individual pile capacities, and the term group 'efficiency' has been used in some textbooks. This concept has led to confusion. It should be noted that the efficiency factors quoted are based on a limited set of small-scale model tests, and general experience (together with full-scale tests, e.g. O'Neill *et al*., 1982) indicates that there is little merit in using efficiency factors. For piles driven into sand, the group capacity is likely to be much higher than the sum of individual capacities. However, settlement considerations will usually be critical, and the actual ultimate capacity does not generally govern design. For bored piles (in sands or clays) it could be argued (with some justification) that there would be an overall relaxation of horizontal ground stress around the pile group (thus reducing the pile-group capacity). However, this will be counterbalanced by the fact that shear failure would occur mainly through an undisturbed soil mass (rather than through the highly disturbed or remoulded ground adjacent to an individual pile shaft). In summary, therefore, the use of group efficiency factors is *not* recommended for calculations of ultimate group capacity.

For pile groups subject to horizontal loading, the most likely geotechnical failure mode is shown in **Figure 55.4(c)**; commonly this will be more critical than the sum of individual pile capacities due to the 'shadowing' effect of adjacent piles in a row (**Figure 55.7**). The failure of pile rows under horizontal loading is discussed by Fleming *et al*. (2009).

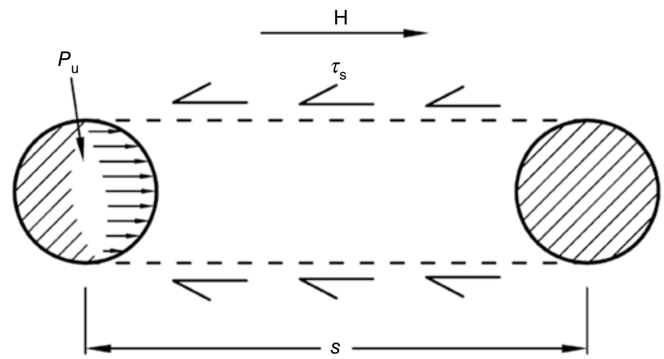
For practical purposes the ultimate lateral capacity of a pile group can be estimated as the lesser of

(*a*) the sum of the ultimate capacities of individual piles in the group (Chapter 54 *Single piles*)

(*b*) the ultimate capacity of an equivalent block containing the piles and soil

(*c*) the sum of the ultimate capacities of the pile rows.

For (*b*), this failure mode checks if the shear resistance of the soil between the piles is less than the limiting resistance of an individual pile (which then controls the limiting resistance provided by the soil in the wake of other piles in a group); only the 'short-pile' case is relevant (Fleming *et al*. 2009), and failure mode (*b*) is only likely to be critical when there are a large number of closely spaced and relatively short piles. For this scenario, the soil shear strength would only be ignored across a depth equal to 1.5 times the pile diameter. Pile-group failure under a lateral load will usually involve rotation as well as horizontal translation of the group. The ultimate geotechnical capacity will then be a function of both the axial and lateral



NB: The limiting force per unit length for piles in the wake of another pile is:

$$P_u < 2s\tau_s$$

where P_u is the limiting lateral resistance provided by the soil for an individual pile, τ_s is the ultimate shear resistance provided by the soil between piles in a group and s is the pile spacing. P_u and τ_s are evaluated as appropriate for fine- or coarse-grained soils (see Chapter 54 *Single piles*).

Figure 55.7 Failure and deformation mechanism ('shadowing') for a pile row under a horizontal load

capacities, with piles behind the axis of rotation failing in tension and piles in front of the axis of rotation failing in compression. In situations involving large horizontal or moment loading, the induced structural forces (bending moment, shear force, axial forces, tension or compression) and horizontal or rotational movements will usually be more critical than the ultimate geotechnical capacity. Pile-group deformation and induced structural forces will be discussed later in this chapter.

55.3. Pile-to-pile interaction: vertical loading

When piles are constructed close to one another, their subsequent load–deformation behaviour is affected by the presence of the neighbouring piles. **Figure 55.8(a)** provides a simple example of these 'pile-to-pile' interaction effects; it shows four piles under equal vertical loading (by way of a flexible pile cap). Each pile settles under the load and a zone of soil settlement develops around each pile. These soil settlement zones overlap and interact; hence, the middle piles settle more than the outer piles (and all the 'group' piles settle more than an individual isolated pile). If the pile group has a rigid pile cap (**Figure 55.8(b)**) each pile has to settle by an equal amount (because the pile cap is rigid) but the axial loads in the piles will now vary across the group, with the outer piles developing higher axial loads than the middle piles (because the outer piles are stiffer than the inner piles and, hence, attract more load). The pile-to-pile interaction and how it affects both pile-group deformation and the redistribution of forces across piles (and into the pile cap) within a group is of fundamental importance. In practice, the pile-to-pile interaction depends on a large number of factors (**Table 55.2**).

For vertical loading, many of the factors given in **Table 55.2** lead to a reduction in group interaction effects; this contrasts with horizontal loading, where group interaction effects may increase.

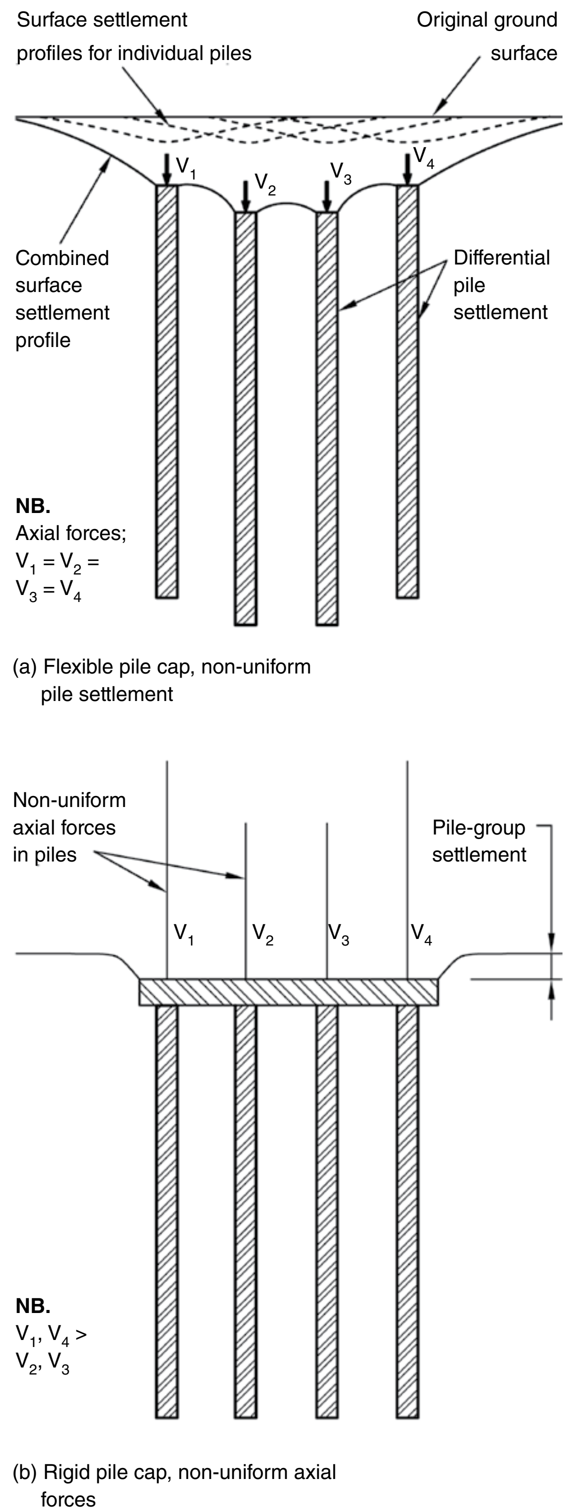


Figure 55.8 Effects of pile-to-pile interaction, vertical loading

Factor	Vertical loading		Horizontal loading	
	Group settlement ratio	Non-uniformity of axial loads	Group deformation	Stresses in piles
Increase of ground stiffness with depth	–	–	– (if $< L_c$)	– (if $< L_c$)
Ground layering, relatively stiff layer at depth below pile toe	–	–	NE	NE
End-bearing pile (compared with friction pile)	–	–	NE	NE
Ground layering, relatively compressible layer at depth below pile toe	+	+	NE	NE
Ground, nonlinear stress–strain behaviour	–[a]	–[a]	– or + [b]	– or + [b]
Pile installation[c]	–	–	NE	NE
Pile-group layout	NE	NE	+	+
Pile spacing ($s > 3d$)[d] ($s < 3d$)	– +	– +	– +	– +
Increasing displacement	–	–	+	+
Near-surface soils ($< 6d$) (if relatively weak layer)	NE	NE	+	+
Pile cap stiffness reduces	NE	–	NA	NA
Pile head fixity, fixed to free head condition	NA	NA	+	Depends on pile location in group

Key
– Leads to a reduction in interaction effects (e.g. smaller group settlement, more uniform axial loads); + leads to an increase in interaction effects (e.g. larger group deformation, more non-uniform axial loads and stresses in piles); L_c is the critical pile length for horizontal loading; NE, negligible effect; NA, not applicable.
[a] Particularly important for large pile groups;
[b] depends on selected 'elastic modulus', typical outcome. If the selected 'elastic' stiffness is an intermediate or large strain value, then the calculated bending moments and pile-group displacements are likely to be overestimated (refer to **Figure 55.29**);
[c] the locally disturbed zone around a pile reduces interaction effects;
[d] interaction effects reduce significantly as spacing increases, typically negligible at spacings > $6d$.

Table 55.2 Pile interaction, influential factors

In the context of assessing pile-group settlement, two parameters are useful

$$\text{Settlement ratio}, R_s = \frac{\text{Average group settlement}}{\begin{array}{c}\text{Settlement of single pile}\\ \text{at same average load as}\\ \text{a pile in the group}\end{array}} \tag{55.6}$$

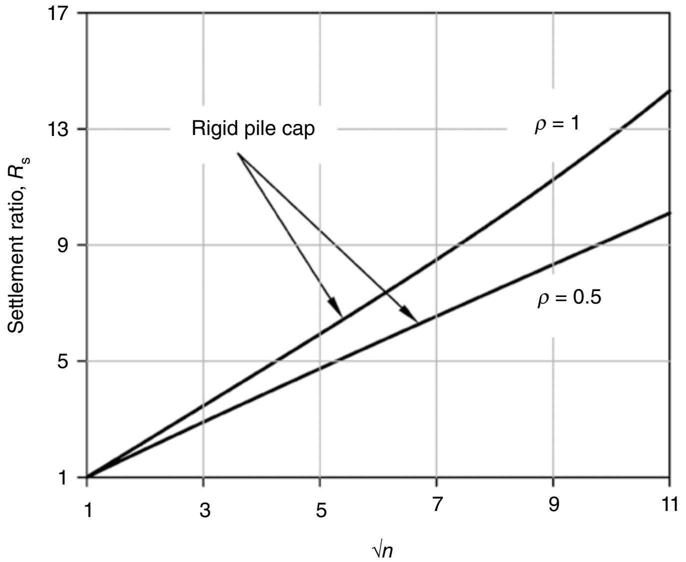
$$\text{Group reduction factor}, R_g = \frac{\text{Average group settlement}}{\begin{array}{c}\text{Settlement of single pile}\\ \text{at same total load as}\\ \text{the group}\end{array}} \tag{55.7}$$

Group reduction factor, R_g, is a conceptually difficult parameter; nevertheless, it is useful when comparing the performance of different pile groups. R_g is simply related to R_s by $R_s = nR_g$, where n is the number of piles in the group. The value of R_g varies between $1/n$ and 1.0. An important practical conclusion from various studies (e.g. Butterfield and Douglas, 1981) is that the precise layout of the piles has a negligible influence on the group settlement ratio R_s; for example, a square pile group has the same value of R_s as a rectangular or circular group, at the same average pile spacing.

Two of the most important factors which influence R_s are

- the variation of ground stiffness with depth (which may be expressed in terms of the shear modulus G or Young's modulus E)
- the influence of a rigid (or a relatively much stiffer or stronger) layer at some depth below the toe level of the pile group.

The influence of these factors is illustrated in **Figures 55.9** and **55.10**. **Figure 55.9** compares the values of R_s for a pile group in a soil layer that has constant stiffness with depth ($\rho = 1$) against a group in a soil layer that has stiffness increasing linearly with depth ($\rho = 0.5$). In both cases the soil layer is infinitely deep. It can be seen that the degree of interaction (and the value of R_s) is much less when the stiffness increases linearly with depth (which occurs commonly in practice, in many geologies, due to the combined effects of increasing effective stress and reductions in weathering with depth). **Figure 55.10(a)** plots a correction factor, F_h, against the ratio



NB.
ρ = 1, Young's (or shear) modulus is constant with depth; ρ = 0.5, Young's (or shear) modulus increases linearly with depth; n = number of piles in group; $R_s = W/W_s$, W = pile-group settlement, W_s = single-pile settlement (under same average load)

Figure 55.9 Influence of soil stiffness profile on pile-group settlement (Reproduced from Fleming *et al.*, 2009)

of compressible layer thickness, h, to the pile length, L. F_h is defined as

$$F_h = \frac{R_s \text{ for finite layer of depth } h}{R_s \text{ for infinitely deep layer}} \tag{55.8}$$

This effect becomes more significant for larger pile groups, with F_h values of 0.6–0.8 for a wide range of scenarios, reducing to 0.4 for some larger pile groups.

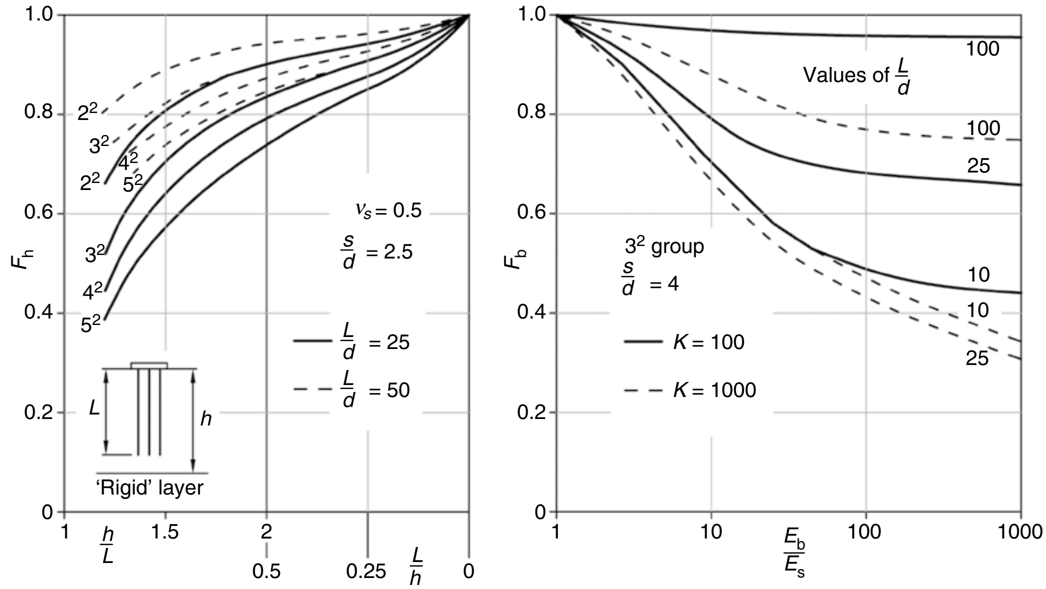
Figure 55.10(b) shows the potential difference in R_s value (indicated by the correction factor, F_b) for friction piles 'floating' in an infinitely deep homogeneous layer compared with end-bearing piles toed into a layer of varying stiffness, relative to the overlying strata (E_b/E_s). From Figure 55.10(b) it can be seen that R_s decreases (i.e. the pile-to-pile interaction decreases, and, similarly, the axial load distributions become more uniform) as the relative stiffness of the bearing layer increases. The effect is most significant for short stiff piles.

In practice, piles are often installed in geological sequences that exhibit marked variations in stiffness with depth, or 'layering'. Although this is a common situation, computer software is often not able to accurately model the influence of these layers on pile-group behaviour. Figure 55.11 illustrates two different situations. First, Figure 55.11(a) shows the increase or decrease in pile-group settlement (for pile groups increasing in size up to 121 piles, pile spacing of 3d and pile length of 25 m) due to underlying layers of lower or greater stiffness, respectively. Figure 55.11(b) shows the increase in settlement due to an underlying layer of lower stiffness (for pile groups with up to 64 piles, pile spacing of 4d and pile length of 15 m). Figure 55.11 shows that

- the effect of the underlying layer is more significant for larger pile groups
- for large pile groups, settlement may be increased by a factor of three or more when underlain by a more compressible layer
- if underlain by a relatively stiff layer, pile-group settlement may be more than halved.

Therefore, it is essential that ground investigations are deep enough to adequately characterise the ground beneath pile groups, and proper consideration is given to the effects of deeper layers on pile-group performance (noting that commercial software may not facilitate this, and some of the methods outlined in this chapter may need to be implemented). Axial load distributions across a pile group will be similarly affected: more compressible underlying layers will lead to more non-uniform axial loads across the group, whereas stiffer underlying layers will lead to more uniform axial loads across the group.

Pile-group analyses often assume that the soil behaviour is linear elastic and that pile installation has no effect on soil properties. Both of these assumptions, as noted in Table 55.2, lead to an overestimation of pile-to-pile interaction effects for vertical loading. Figure 55.12 illustrates the influence of nonlinear elastic against linear elastic assumptions for soil behaviour; Figure 55.12(a) indicates that modelling the soil behaviour as linear elastic will tend to overpredict the soil settlement adjacent to a pile. Figure 55.12(b) plots contours of soil strain around a pile (derived from a sophisticated nonlinear soil model), which indicates that most of the soil adjacent to the pile experiences strains of less than 0.01%, whereas close to the pile there are small zones of soil experiencing larger strains. Hence, the mobilised soil stiffness close to the pile will be considerably lower than that mobilised further away from the pile, at any given depth below the ground surface. This effect will be exacerbated by the influence of pile installation. For either bored piles or driven piles there will be a zone of soil immediately adjacent to the pile that will be intensely disturbed and remoulded. Figure 55.13 compares predictions from a range of different software with measured pile-group settlements (Poulos, 1989). Figure 55.13(a) indicates that although some software could accurately predict single-pile settlement, the group settlement was overpredicted – that is, interaction effects, or the equivalent R_s value, were overpredicted. Figure 55.13(b) plots R_s against the number of piles in the group. This indicates that conventional linear elastic analysis overpredicts R_s and this overprediction becomes worse as the pile-group size increases. Relatively good predictions are



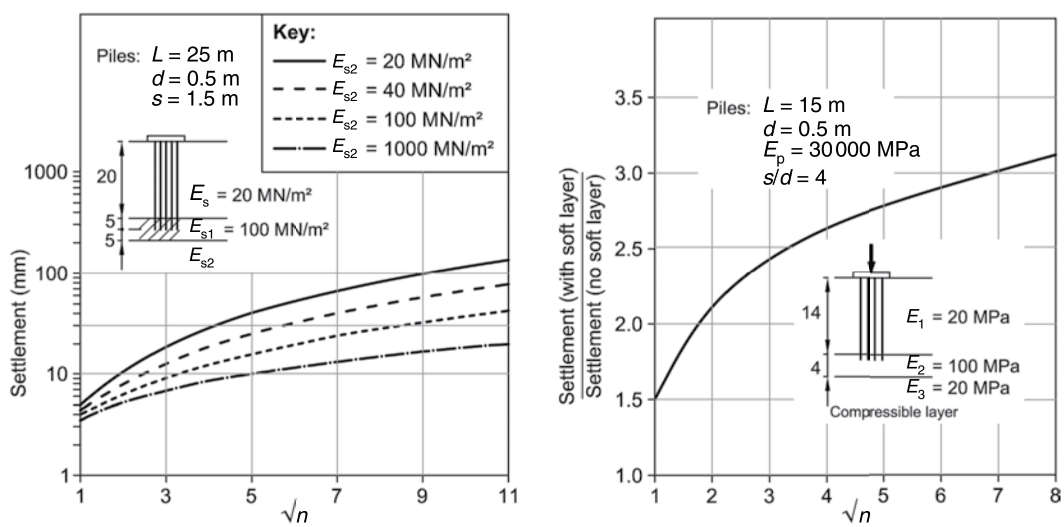
NB.
h = thickness of soil layer, L = pile length,
d = pile diameter, s = pile spacing,
3^2 = 3x3 pile group

NB.
K = relative pile/soil stiffness, $K = (E_p RA)/E_s$,
E_p = pile Young's modulus, RA = area ration (= 1.0 for solid pile) = $A_p/(0.25\pi d^2)$; A_p = area of pile wall thickness

(a) Reduction coefficient F_h for effect of finite layer thickness, compared with infinite layer

(b) Reduction coefficient F_b for effect of bearing stratum Young's modulus, E_b, to overlying soil modulus, E_s

Figure 55.10 Influence of finite layer thickness and bearing stratum stiffness on pile-group settlement (Reproduced from Poulos and Davis, 1980)

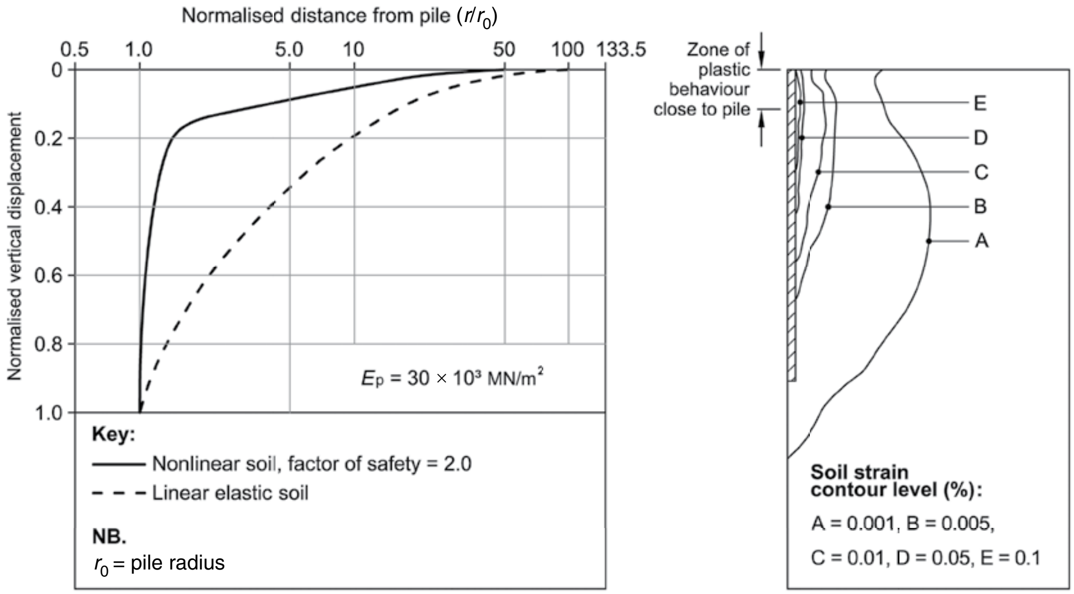


NB.
n = number of piles in group

(a) Effect of underlying compressible layer Young's Modulus, E_{s2}, on group settlement

(b) Effect of underlying compressible layer on group settlement

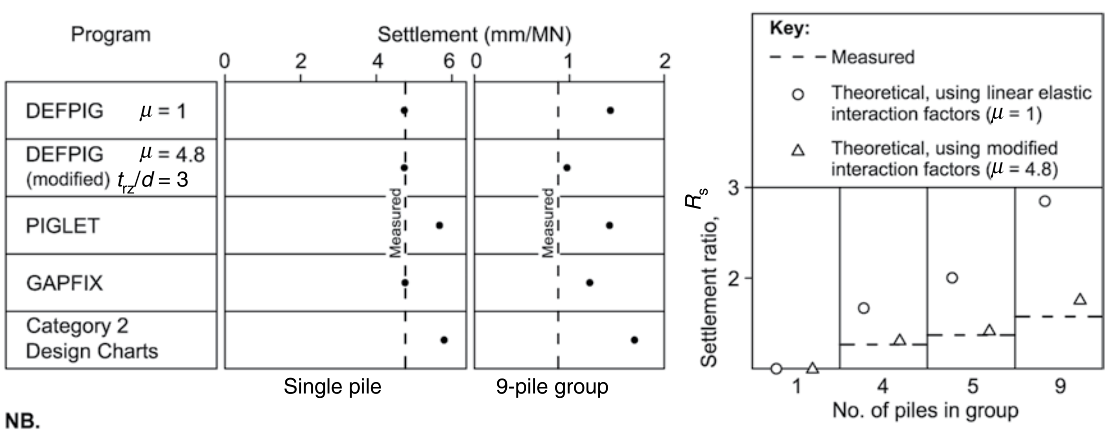
Figure 55.11 Influence of soil layering on pile-group settlement (Reproduced from (a) Poulos, 2005, with permission of ASCE; (b) Poulos *et al.*, 2001) made for a modified analysis in which the soil between the piles was assumed to be stiffer than that immediately adjacent to the piles (**Figure 55.13(b)**) (O'Neill *et al*., 1982); a Young's modulus of 750s_u was assumed for the thin zone of remoulded soil adjacent to the piles, and a Young's modulus of about 2500s_u for the soil beyond the remoulded zone (equivalent to the modulus at very small strain derived from cross-hole geophysics data).



(a) Profiles of surface settlement adjacent to a pile 30 m long

(b) Contours of deviatoric strain around a pile 30 m long when factor of safety = 2.0

Figure 55.12 Influence of soil nonlinearity on pile-to-pile interaction (Reproduced from Jardine *et al*., 1986)



NB.
$\mu = \frac{E_{sm}}{E_{sr}}$; E_{sm} =Young's modulus of soil beyond t_{rz}; E_{sr} = Young's modulus of remoulded soil adjacent to pile; t_{rz} = thickness of remoulded zone; E_s varies linearly across t_{rz} between E_{sr} and E_{sm}; d = pile diameter.

(a) Effect of analysis method on single pile and group settlements

(b) Theoretical and measured group settlement behaviour (tests of O'Neill *et al*., 1982)

Figure 55.13 Influence of analysis method on single-pile and pile-group settlement (Reproduced from Poulos, 1989)

For end-bearing piles toed into a relatively stiff layer, Figure 55.14 shows that pile axial loads rapidly tend towards a more uniform distribution as the stiffness of the bearing stratum increases; with V/V_{av} less than 1.2 for E_b/E_s values in excess of 10, where V is the axial load on an individual pile and V_{av} is the total vertical load applied to the group divided by the number of piles. Measurements of axial loads in pile groups are rare; however, Mandolini *et al.* (2005) have collated the available data, shown in Figure 55.15. Typically, the ratio of corner to centre pile axial loads is between about 2.0 and 2.5 for common pile spacings of about 2.5–4 times the pile diameter. It is apparent that pile axial loads become more uniform with increasing pile spacing, and interaction effects become negligible for pile spacings of 6–8 times the pile diameter. Table 55.3 gives measured axial load ratios (V_{max}/V_{min}) for a few specific case histories. At Stonebridge Park, given the large pile-group aspect ratio, a much larger value of V_{max}/V_{min} would have been expected. In this case, the pile cap was relatively flexible, which led to more uniform pile axial loads. For the Cargliano Bridge, the end-bearing piles led to a relatively low value of V_{max}/V_{min}. The pile-group test reported by Koizumi and Ito, with friction piles in a deep compressible layer and a rigid pile cap, led to relatively high values of V_{max}/V_{min}.

Figure 55.16(a) summarises the results of a study by O'Brien and Bown (2010), which plots the ratio of corner to centre pile axial loads against pile-group size. V_{max} is the maximum predicted axial load, which for square pile groups

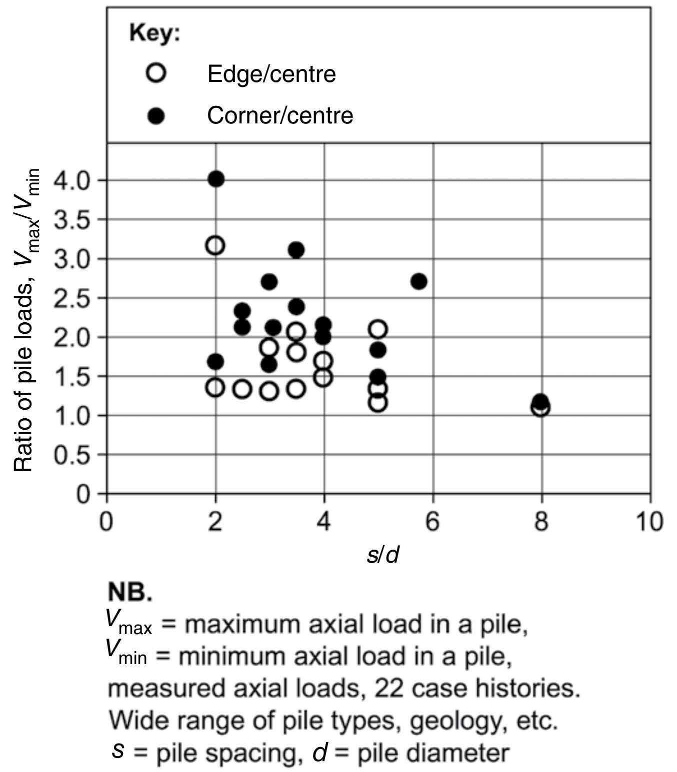


Figure 55.15 Measured axial load distributions, pile groups (Reproduced from Mandolini *et al.*, 2005)

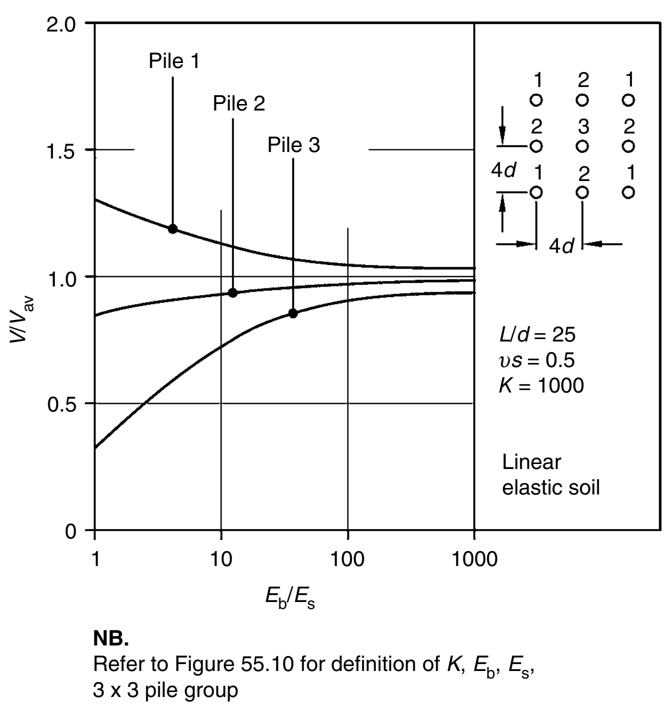


Figure 55.14 Influence of bearing stratum stiffness on axial load distribution (Reproduced from Poulos and Davis, 1980)

occurs in the corner piles. V_{min} is the minimum predicted axial load, which occurs in the piles close to the centre of the pile group. This shows that linear elastic analyses overestimate the load redistribution across the group when compared with nonlinear analyses and case history data, and that this discrepancy increases with increasing pile-group size.

For large groups, if pile axial loads are calculated by linear elastic methods then it is inevitable that corner piles will have an apparently low geotechnical 'factor of safety' even when the overall group factor of safety is two or more (Figure 55.16(b)). This can be a problematic issue when considering how to apply code requirements. This is discussed in Section 55.10.

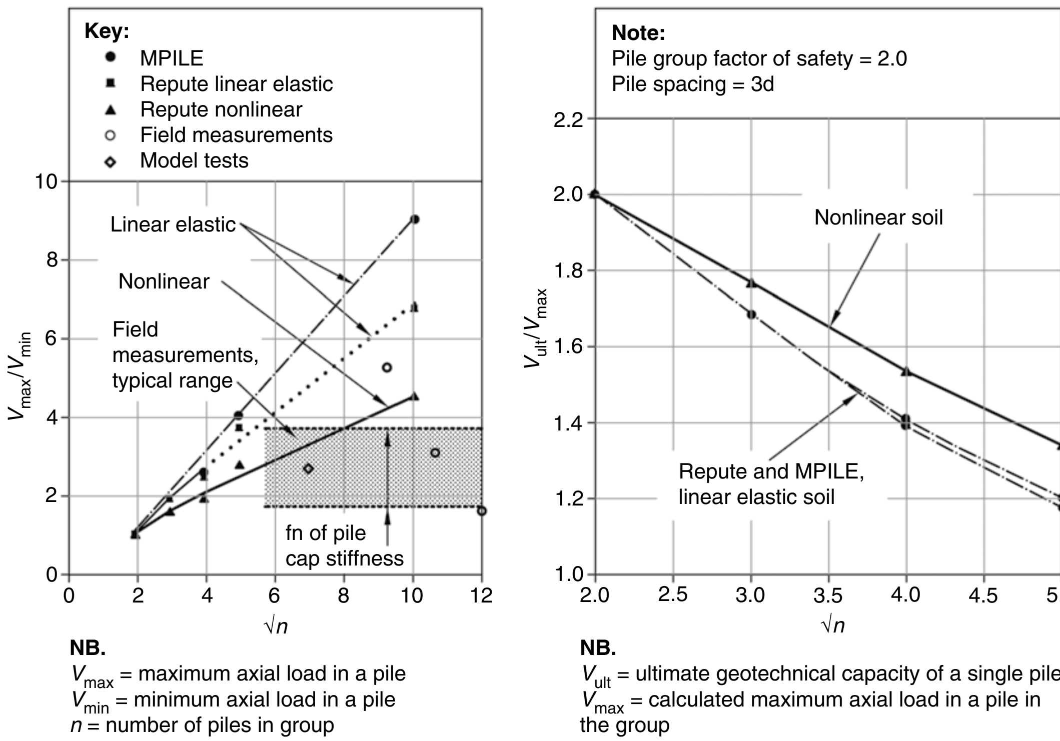
55.4. Pile-to-pile interaction: horizontal loading

For pile groups subject to horizontal load the main design issue is usually the structural demands (bending moment, shear force, etc.) on the piles relative to their structural strength. Mandolini *et al.* (2005) summarise several studies on pile-group behaviour when horizontal loads are applied. Pile-to-pile interaction under a vertical load is fully developed at relatively small displacements, and group interaction effects tend to reduce at larger displacements (ground nonlinearity, including local yield, becomes more focused in a thin zone around the pile–soil interface). In contrast, under horizontal loading, group interaction effects amplify as the pile-group

Pile group	Ground profile	Pile type	Pile length: m	Pile diameter: m	Number of piles	Spacing: m	Pile-group aspect ratio	Axial load ratios		Pile cap stiffness
								C/I	*E/I*	
Stonebridge Park (Cooke *et al.*, 1981)	Deep layer of overconsolidated clay (London Clay)	Bored (friction)	13	0.45	351	1.61	6.6	2.2	1.7	Flexible
Cargliano Bridge (Mandolini *et al.* 2005)	Interbedded soft clays and silts over dense sand and gravel	Driven tubular (end-bearing)	48	0.38	144	1.14	1.1	1.6	1.3	Intermediate
Pile-group test (Koizumi and Ito, 1967)	Deep layer of overconsolidated clay	Driven closed-end (friction)	5.5	0.3	9	0.9	1.2	2.6	1.9	Rigid

C, corner pile axial load; *I*, internal pile axial load; *E*, edge pile axial load.

Table 55.3 Examples of measured axial load distributions

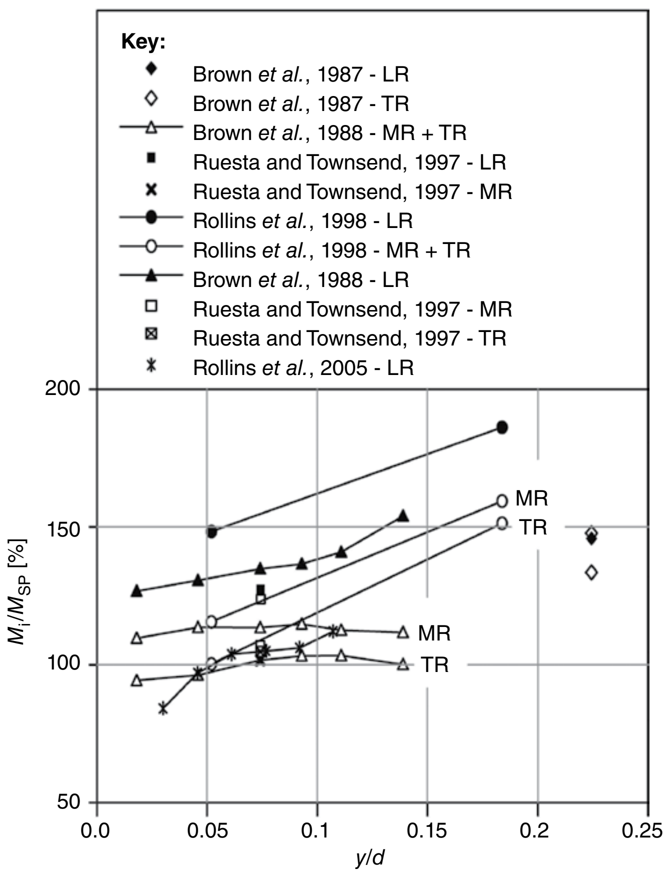


(a) Axial load ratio (V_{max}/V_{min}). Linear and nonlinear soil models and field measurements

(b) Perimeter pile factor of safety (V_{ult}/V_{max}); linear and nonlinear soil models, for overall pile group factor of safety = 2.0

Figure 55.16 Axial load distribution for linear and nonlinear soil models against pile-group size (Reproduced from O'Brien and Bown, 2010)

displacement increases (and ground nonlinearity – e.g. zone of passive yield in front of a pile – increases as horizontal displacement increases). Hence, structural demands across a pile group tend to become more non-uniform. Figure 55.17 provides some measured data from instrumented pile groups and centrifuge tests (Mandolini and Viggiani, 2005) for free head conditions. The ratio between the maximum bending moment in different piles in a group and in a single isolated pile is plotted against the pile-group displacement (the ratio is derived for the same average horizontal load per pile). The moments in the group piles are generally larger than the single pile, and the ratio increases with increasing displacement. Larger bending moments develop in the leading row (LR) piles compared to either the middle (MR) or trailing row (TR) piles (**Figure 55.17**); this phenomenon has been attributed to 'shielding' or 'shadowing' effects between adjacent piles in the same row (refer to **Figure 55.7**). This means that for horizontal loading, different group configurations (e.g. square, rectangular, circular) will exhibit different group interaction effects even if the number of piles and average spacing are the same (this contrasts with group interaction effects under a vertical load, which are practically unaffected by the group configuration for a given number of piles and similar average spacing). Hence, for piles under horizontal loading it is potentially unsafe to assess the bending moments in piles within a group from an assessment of single-pile behaviour; in addition the assumption of linear elastic behaviour is generally inappropriate (refer to **Figure 55.31**). The use of p–y curves is well established for predicting the behaviour of single piles under horizontal loads. However, the use of p–y multipliers to account for group interaction effects is of more doubtful validity because of the large number of complicating factors that can affect real pile-group behaviour under horizontal loading.



NB.
M_i = maximum bending moment in pile in a group; M_{sp} = maximum bending moment in a single isolated pile; LR = piles in a leading row MR = piles in middle of a row; TR = trailing piles

Figure 55.17 Ratio of bending moments for piles in a group and single isolated pile against displacement (Reproduced from Mandolini *et al.*, 2005)

55.5. Simplified methods of analysis

55.5.1 General comments

There are four simplified methods that may be used to assess pile-group settlement

- pile-group settlement ratio
- elastic interaction factors
- equivalent pier
- equivalent raft.

Each method has distinctive advantages and disadvantages (summarised in **Table 55.4**). Simplified analysis methods should always be used to check computer-based methods, which may have serious limitations in certain situations.

There are no simplified methods for checking axial load distributions across a group. However, **Figures 55.15** and **55.16** indicate the ranges of V_{max}/V_{min} that should be reasonably anticipated for friction piles (noting that linear elastic methods can lead to gross overestimates of axial loads in perimeter/corner piles; **Figure 55.16**). **Figure 55.14**, for piles toed into stiffer strata, indicates that V_{max}/V_{min} is close to 1. **Tables 55.2** and **55.3** indicate the factors that may influence axial load distribution.

It should be noted that the use of structural analysis software (which typically simulate pile behaviour as independent elastic springs) is not recommended, since pile–pile interaction is not modelled. Poulos and Davis (1980) give examples of some of the errors that can be produced.

55.5.2 Pile-group settlement ratio, R_{Se} (empirical correlations)

Mandolini *et al.* (2005) proposed an empirical correlation between the pile-group settlement ratio R_{Se} and the pile-group aspect ratio, based on an analysis of 63 case histories (for pile groups in varying geologies, and with varying pile length, type, diameter, etc.), which is shown in **Figure 55.18**. Equations have been developed for the upper-bound, best-estimate and lower-bound settlements.

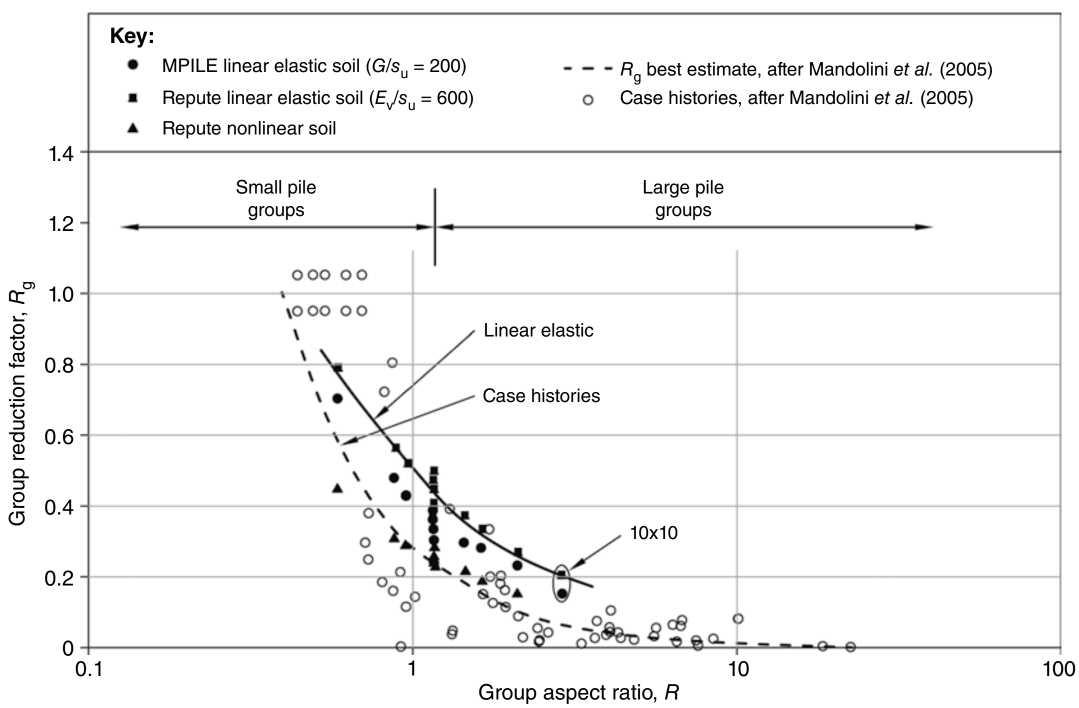
$$\text{Best estimate } R_{Se} = \frac{a}{(R)^b} = \frac{0.29}{(R)^{1.35}}(n) \qquad (55.9)$$

where n is the number of piles in the group, the pile-group aspect ratio $R = (ns/L)^{0.5}$ and R_{Se} is the empirical pile-group settlement ratio.

$$\text{Pile-group settlement, } W = R_{Se}W_s \qquad (55.10)$$

Simplified method	Advantages	Disadvantages
Empirical settlement ratio	Very quick and simple. Best suited for friction piles, in deposits with increasing strength and stiffness with depth Useful as a 'sense' check of more complex methods	The influence of specific geological features, pile or ground properties cannot be assessed Potentially unsafe if weak or compressible strata underlie the bearing stratum. Potentially over-conservative if a relatively stiff layer underlies the bearing stratum
Elastic interaction factors	Influence of varying pile length, diameter and spacing can be quickly checked Best suited for friction piles in deposits with increasing strength and stiffness with depth	Cannot directly check influence of underlying strong or weak layers (refer to **Figures 55.10** and **55.11**) Care needed in amplifying single-pile settlement for large groups, **use initial tangent pile stiffness**, rather than secant; otherwise likely to be over-conservative
Equivalent pier	Well suited for pile groups with relatively small aspect ratio, $R < 3.0$ If using elastic solutions for single pile, then is quick to use Flexible, method can also be used within sophisticated numerical models, in axisymmetric mode	Not appropriate for large pile-group aspect ratios, $R > 3.0$ Inappropriate if pile lengths in group vary significantly
Equivalent raft	Most appropriate simplified method for checking influence of strong or weak layers at depth	Over-conservative for pile groups with a small aspect ratio, $R < 3.0$ Significant judgement needed to assess appropriate equivalent raft level and dimensions

Table 55.4 Pile-group settlement: advantages and disadvantages of simplified methods



NB.
Settlement ratio, $R_s = nR_g$, n = number of piles in group, R_g = group reduction factor. Group aspect ratio, $R = (ns/L)^{0.5}$, s = pile spacing, L = pile length. G = soil shear modulus, E_v = Vertical Young's modulus of soil, s_u = undrained shear strength

Figure 55.18 Group reduction factor R_g against group aspect ratio R; field measurements and parametric study (Data taken from Mandolini *et al.*, 2005, and O'Brien and Bown, 2010)

where W_s is the single-pile settlement under the average working load of piles within the group (Q/n) and Q is the total vertical load applied to the pile group.

The single-pile settlement W_s can be calculated by using either relevant pile test data or Fleming's method for single-pile settlement (Fleming, 1992). The latter has been found to be reliable for a wide range of pile types and ground conditions (refer to Chapter 54 *Single piles*). The upper- and lower-bound values of R_{Se} are

$$\text{Upper-bound } R_{Se} = \frac{0.5}{R}\left[1 + \frac{1}{3R}\right]n \tag{55.11}$$

$$\text{Lower-bound } R_{Se} = \frac{0.17}{(R)^{1.35}}(n) \tag{55.12}$$

Hence, application of the above equations enables a rapid estimate to be made of the likely range of pile-group settlement.

Experience indicates that Equation 55.9 is generally appropriate for friction piles with soils with strength and stiffness increasing gradually with depth; Equation 55.11 applies if relatively deep and uniform soils underlie the pile group; and Equation 55.12 applies if the soil or rock beneath the group exhibits a rapid increase in strength and stiffness close to the toe level of the pile group.

55.5.3 Elastic interaction factors (design charts)

Elastic interaction factors have been widely published (for example Poulos and Davis, 1980). However, many of the early solutions relied on unrealistic assumptions (e.g. an infinite elastic medium with constant Young's modulus, rigid pile, etc.) and overestimated group interaction effects. Randolph (1994) has outlined a simple and practical approach to improve the reliability of elastic interaction factors, which is shown in **Figure 55.19**. The initial stiffness derived from the single-pile load–settlement response is given by the tangent line OA (and will be a function of the small strain stiffness of the soil). Plastic strain, which will develop at higher load levels, leads to the offset AB. This plastic strain will develop locally adjacent to the pile shaft. Group interaction effects under vertical load are predominantly due to elastic, rather than plastic, strains. Therefore, the pile-group settlement ratio (derived from linear elastic theory) R_{St} is only applied to the elastic component of the single-pile settlement. Hence, the elastic pile-group settlement is shown as OC in **Figure 55.19** where OC = R_{St}OA and the total pile-group settlement OD = OC + CD (with CD = AB). For most conventionally designed pile groups, which will have overall factors of safety well in excess of 2.0, the plastic component of settlement will be relatively minor. Back-analysis of a wide range of case histories by Mandolini and Viggiani (1997) confirms that Randolph's approach is usually far more realistic than applying the group settlement ratio R_{St} to the single-pile secant stiffness at the average working load of the piles within the group.

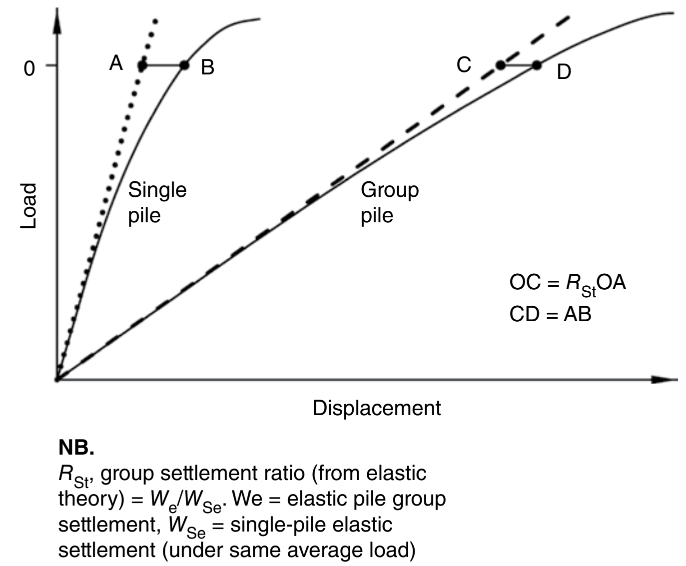


Figure 55.19 Applying elastic theory to pile interaction and nonlinear load–settlement behaviour for vertical loading (Reproduced from Randolph, 1994)

To a reasonable approximation, the group settlement ratio is (Fleming *et al*., 2009)

$$R_{St} = n^e \tag{55.13}$$

and

$$W = R_{St}W_{Se} + W_{Sp} \tag{55.14}$$

where W is the pile-group settlement, R_{St} is the group settlement ratio derived from linear elastic theory, W_{Se} is the elastic settlement under the average working load of piles within the group (Q/n), n is the number of piles in the group, Q is the total vertical load applied to the group and W_{Sp} is the plastic settlement of a pile at Q/n (estimated from a pile test or from a Fleming (1992) analysis for single-pile settlement at the average load for a pile in the group).

For piles working mainly in shaft friction, e is typically between 0.3 and 0.5. Based on Randolph (1994), **Figure 55.20** gives a set of design charts; the exponent e is given by a base value, e_1 (depending on the pile slenderness ratio L/d) and four correction factors, c_1–c_4

$$e = e_1c_1c_2c_3c_4 \tag{55.15}$$

where c_1 is a function of the pile stiffness to the ground stiffness, E_p/G_b, and E_p is Young's modulus for the pile. G_b is the ground shear modulus at the pile-toe level, c_2 is a function of the pile spacing, s, to the pile diameter d; c_3 is a function of the rate of increase of the ground shear modulus with depth,

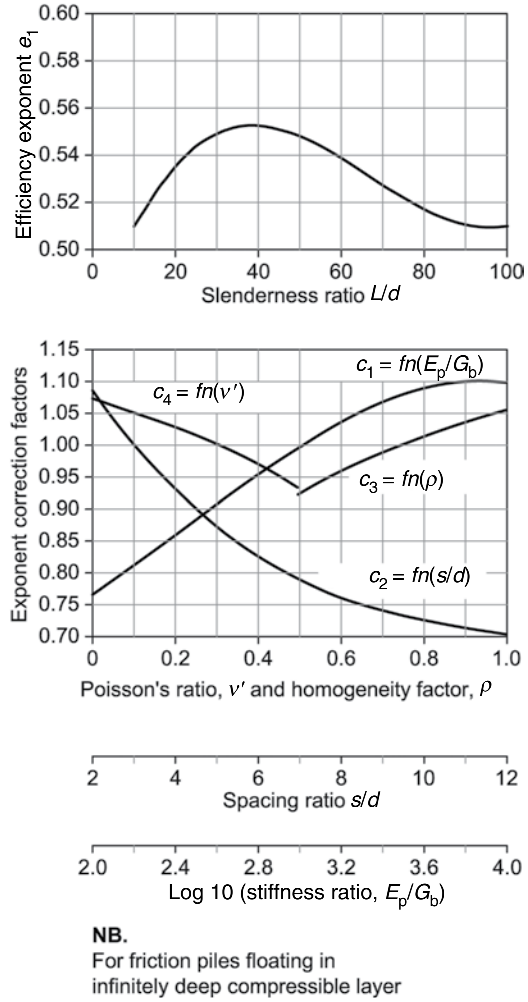


Figure 55.20 Design charts (linear elastic soil) for pile-group settlement ratio, R_{St} (Reproduced from Fleming *et al.*, 2009)

$\rho = G_{av}/G_b$; G_{av} is the ground shear modulus halfway down the pile; and c_4 is a function of Poisson's ratio, ν'. The value of e_1 is based on $E_p/G_b = 1000$, $s/d = 3$, $\rho = 0.75$ and $\nu' = 0.3$. These charts assume an infinitely deep elastic layer. Reference to **Figures 55.10** and **55.11** allows judgements to be made about the influence of a finite layer thickness and stiffer or weaker layers below the pile toe. Poulos (1989) indicates that for friction piles at typical working loads (factor of safety in excess of 2.0), plastic deformation would be expected to only increase single-pile settlement by 10–25% compared with the elastic response derived from the initial pile stiffness.

55.5.4 Equivalent pier and equivalent raft

Equivalent pier: the equivalent pier approach replaces the pile group by a buried 'cylinder' of appropriate dimensions and stiffness (**Figure 55.21**). Solutions for calculating the single-pile settlement can then be used to calculate the settlement of the equivalent pier (provided due account is taken of the compressibility of the cylinder, the stiffness of the soils below the base and the shear stiffness of the interface around the sides of the cylinder). Alternatively, the settlement of the equivalent pier can be simulated within an axisymmetric numerical model.

For a pile-group of area A_g, the diameter of the equivalent pier d_{eq} is

$$d_{eq} = \left(\frac{4}{\pi} A_g\right)^{0.5} = 1.13(A_g)^{0.5} \tag{55.16}$$

The equivalent Young's modulus E_{eq} of the pier is

$$E_{eq} = E_{s,av} + (E_p - E_{s,av})(A_p/A_g) \tag{55.17}$$

where E_p is Young's modulus of the pile, $E_{s,av}$ is the average Young's modulus of the soil penetrated by the piles, A_p is the total cross-section area of the piles in the group and A_g is the total plan area encompassed by the pile group.

The elastic solutions for settlement of a single pile presented by Randolph (1994), which allow for the relatively low aspect ratio of equivalent piers (for typical pile-group geometries) can be conveniently programmed in modern spreadsheets to facilitate rapid analyses of pile-group settlement. Alternatively, the nonlinear approach proposed by Fleming (1992) can be used (refer to Chapter 54 *Single piles*), although it should be noted that the strength and stiffness properties for the base and shaft resistance around the equivalent pier should mainly be representative of the undisturbed soil mass rather than the disturbed remoulded soil immediately adjacent to an individual pile.

The influence of an underlying compressible layer on the load–settlement behaviour of the equivalent pier can be assessed (based on studies by Poulos, 2005) from **Figure 55.22**. **Figure 55.22** provides a plot of the base stiffness reduction factor as a function of h_c/d_{eq} and E_2 and E_1 (where E_2 and E_1 are the Young's moduli of the upper and lower layers, respectively) – refer to **Figure 55.6**. The base stiffness reduction factor is defined as the ratio of the base stiffness with the underlying layer E_2 to the base stiffness of E_1 only (i.e. with no underlying layer present).

Equivalent raft: the equivalent raft method has been used extensively for estimating pile-group settlement and is described in many textbooks. The pile group is replaced by a raft foundation acting at a representative depth below the surface with some equivalent dimensions. Many variants are available; however, the approach by Tomlinson (1987) is relatively simple and widely used and is shown in **Figure 55.23**. As noted by Poulos *et al.* (2001), this method relies on considerable engineering judgement to assess the representative depth and appropriate raft dimensions, which depend on an understanding of the relevant load transfer mechanism for the pile group under consideration. Once the equivalent raft is established, then the settlement can be computed using the analysis methods used for shallow foundations.

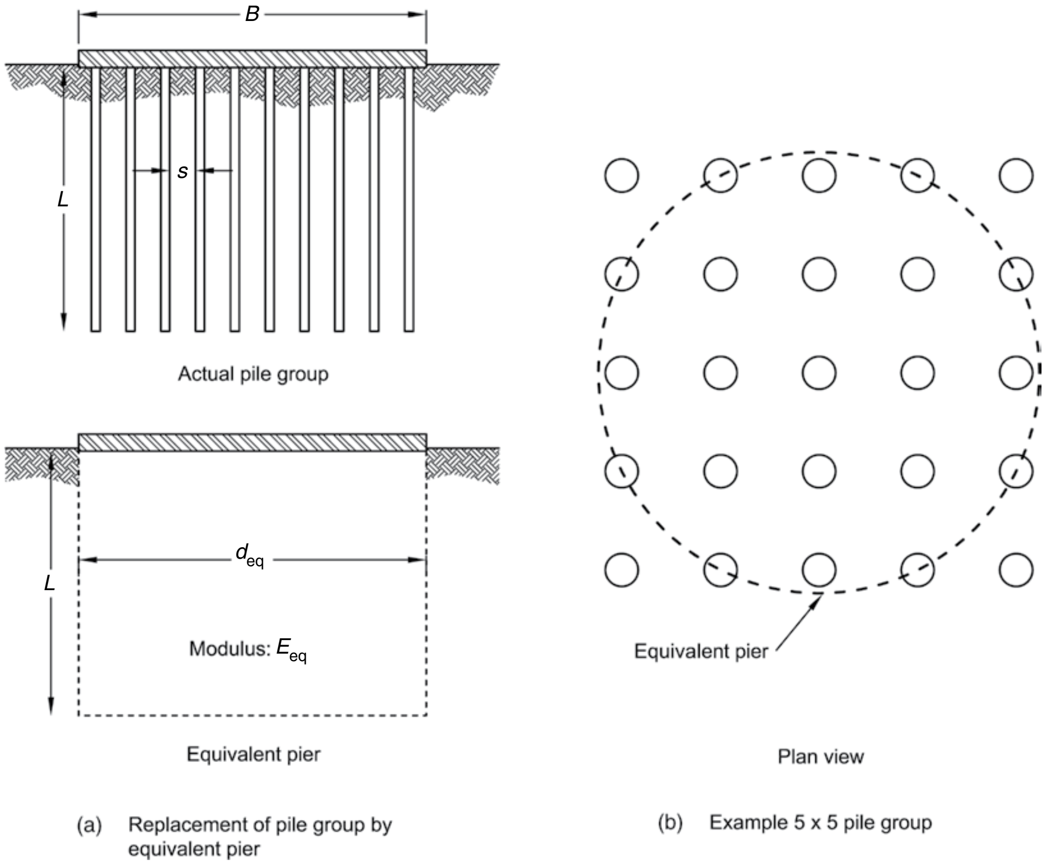


Figure 55.21 Replacement of pile group by an equivalent pier

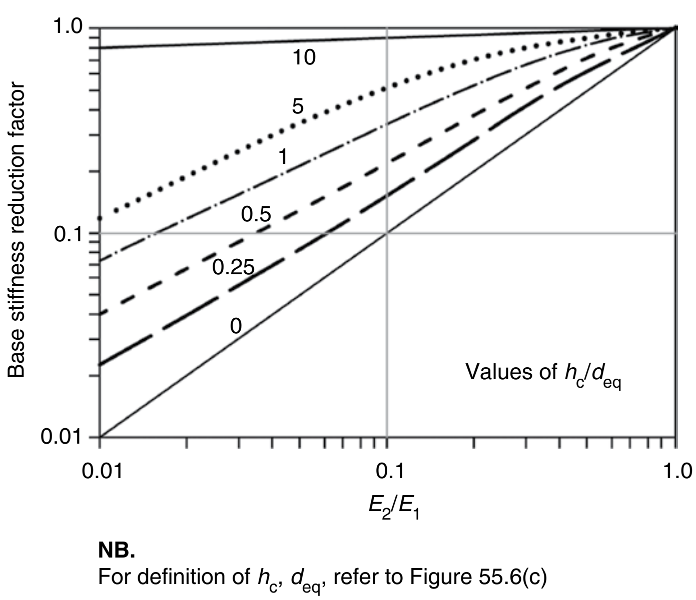


Figure 55.22 Base stiffness reduction factor for pile group (equivalent pier) underlain by compressible layers (Reproduced from Poulos, 2005, with permission of ASCE)

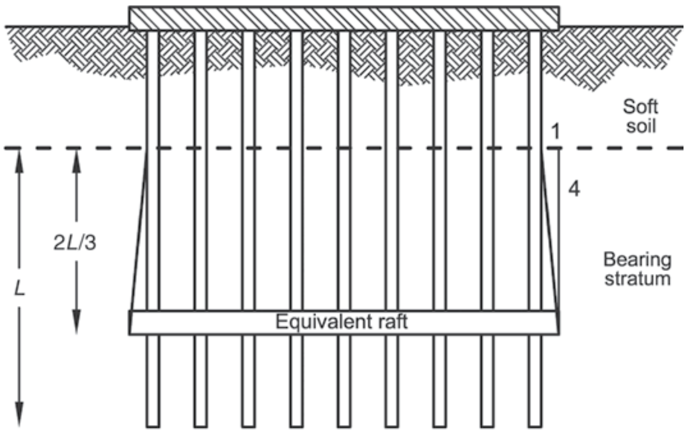
The average pile-group settlement W_{av} is then

$$W_{av} = W_{raft} + W_{ec} \tag{55.18}$$

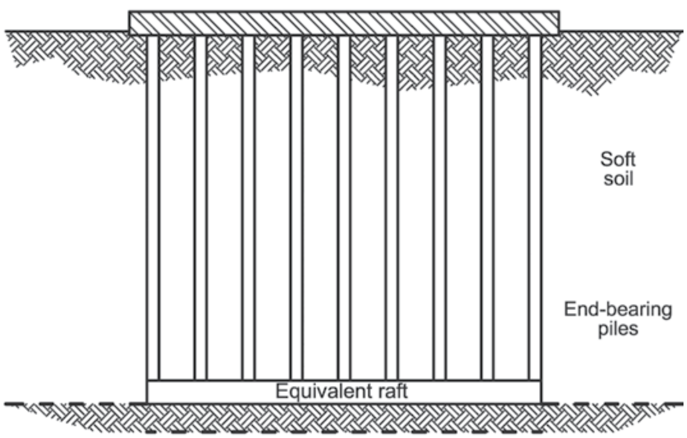
where W_{ec} is the elastic compression of the piles above the equivalent raft level (i.e. acting as free-standing columns). Assuming elastic soil behaviour

$$W_{raft} = F_{av} F_D q \sum_{i=1}^{n} \left(\frac{I_E}{E_{sec}} \right) h_i \tag{55.19}$$

where W_{raft} is the equivalent raft settlement, q is the average pressure applied to the raft, I_E is an influence factor (refer to **Figure 55.24**) for the ith layer, h_i and E_{sec} are the layer thickness and secant Young's modulus, respectively, for the ith layer, F_D is a depth correction factor (as derived by Fox, refer to **Figure 55.25**) and F_{av} is equal to 0.8 (to correct from centre line settlement to average raft settlement). The embedment depth of the raft should be assumed to be the depth below the competent bearing stratum, rather than the ground surface (**Figure 55.23**).



(a) Primarily friction piles



(b) Primarily end-bearing piles

Figure 55.23 Replacement of pile group by an equivalent raft

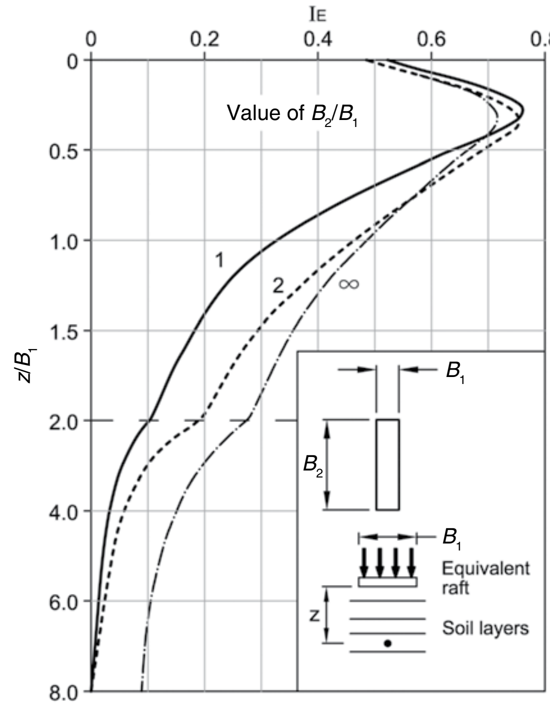


Figure 55.24 Equivalent raft settlement using elastic theory; strain influence factor against depth (Data taken from Randolph, 1994)

When using the equivalent raft analogy for calculating pile-group settlement, care is required to ensure input parameters and the calculation method are relevant; for example, using parameters derived from observations of shallow foundation settlement is likely to be inappropriate. In general, the use of elastic methods, such as Equation 55.19 (e.g. Poulos, 1993), is likely to be suitable provided the strata at, and below, the pile-toe level are competent (i.e. stiff heavily overconsolidated clays, dense sands, etc.). Elastic analyses will *not* be appropriate if a layer of normally or lightly overconsolidated clay is present at depth below the pile-toe level, *and* the pile-group bearing pressures below pile-toe level exceed the clay's preconsolidation pressure.

The main advantage of the equivalent raft method is that it enables the influence of softer or stiffer soil layers below the toe level of a pile group to be assessed in a simple and transparent way. This is particularly important in situations where softer soil layers may exist at some level below the pile group; as noted above, some commercially available software cannot analyse the situation reliably and **a separate check is essential**.

55.5.5 Which is appropriate, equivalent raft or equivalent pier?

If the piles are mainly dependent on shaft friction and the pile group is relatively deep and narrow, the resistance mobilised around the perimeter of the equivalent pier is likely to substantially exceed the applied working load (i.e. SLS load). Hence, the load reaching the base of the pile group is likely to be a small proportion of the applied load, which will be mainly dissipated in shear around the pile-group perimeter. For this situation, an equivalent pier will be appropriate. In contrast, if the piles are mainly end-bearing or the pile group is relatively wide, then a substantial proportion of the applied working load will need to be resisted at the base of the pile group. In this case the equivalent raft will be an appropriate analogue for analysis purposes.

The choice between using either an equivalent raft or equivalent pier is mainly dependent on the overall pile-group aspect ratio or normalised width: **Figure 55.26** illustrates this

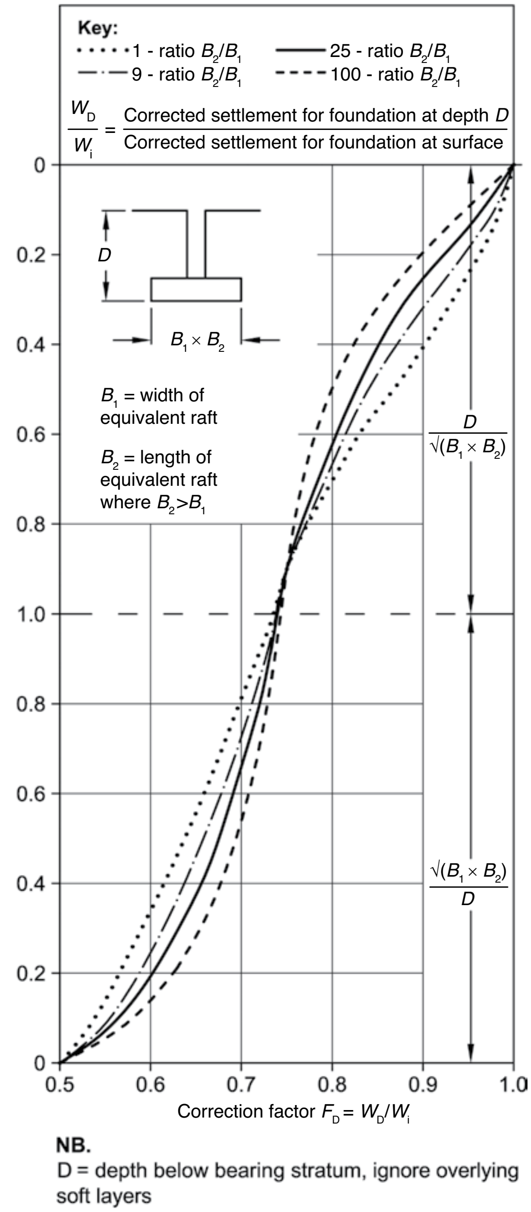


Figure 55.25 Equivalent raft settlement using elastic theory; Fox's depth correction factor (Data taken from Randolph, 1994)

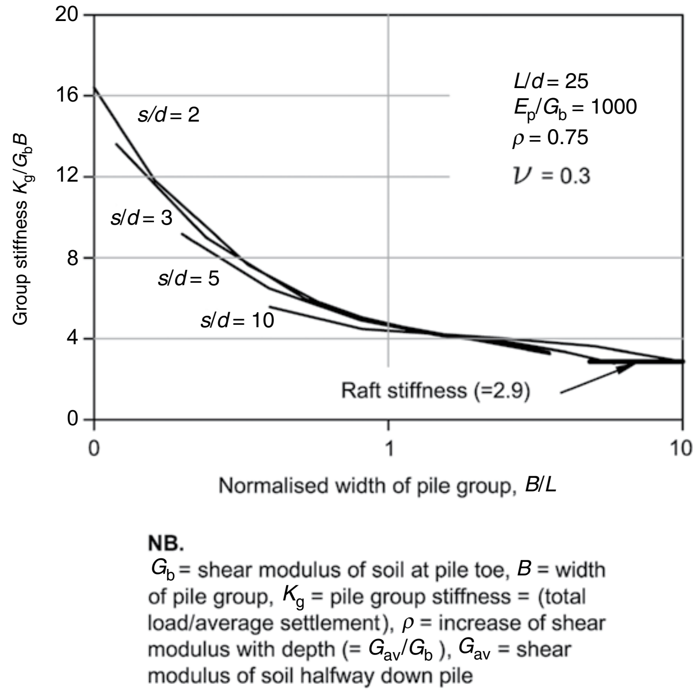


Figure 55.26 Normalised stiffness of pile groups and equivalent raft against normalised width (Reproduced from Randolph, 2003)

quite clearly. When the normalised pile-group width (B/L; B is the group width and L is the pile length) is less than 1.0, the normalised pile-group stiffness is much higher than that for the equivalent raft. Hence, the equivalent raft approach would tend to be over-conservative and the equivalent pier should be used. Once the normalised width exceeds 1.0, then the pile-group stiffness tends towards that of an equivalent raft. Randolph (1994) has suggested that, for friction piles, if the pile-group aspect ratio exceeds about 3 or 4, then an equivalent raft is appropriate, whereas for lower values of R an equivalent pier is more appropriate (and then the accuracy of the equivalent pier is likely to be within 20% of more rigorous solutions (Randolph, 2003), which is perfectly satisfactory for practical design purposes).

55.6. Pile cap analysis and differential settlement

Pile-group analysis commonly assumes that the pile cap is perfectly rigid or, occasionally, that it is perfectly flexible. Most pile-group analysis software cannot carry out analyses with a pile cap of intermediate stiffness. While this may be a reasonable assumption for small groups of piles, it becomes increasingly inaccurate as the size of the group increases. The flexibility of the pile cap has a significant influence on the computed load distribution to individual piles.

Basile (2020) states that for small to medium-sized groups (up to 25 piles), the assumption of a rigid pile cap is generally reasonable for practical purposes. However, this assumption becomes increasingly inaccurate for larger pile groups. Pile cap stiffness can be assessed by calculating the raft–soil stiffness ratio (K_{rs} – see Section 53.6.7 and **Figure 55.27**). Basile (2020) provides further guidance on selecting an appropriate value for ground stiffness and dealing with layered soils. Limits for K_{rs} equal to 0.1 and 1.5 are suggested for design purposes for which pile caps may be considered as 'practically flexible' and 'practically rigid', respectively (**Figure 55.27**). Axial loads for pile groups with a 'practically flexible' pile cap can be assumed to be uniform (assuming a uniform applied load).

The decision to use pile groups is often driven by the wish to reduce settlement, in particular to reduce differential settlement, in order to reduce the risk of superstructure damage

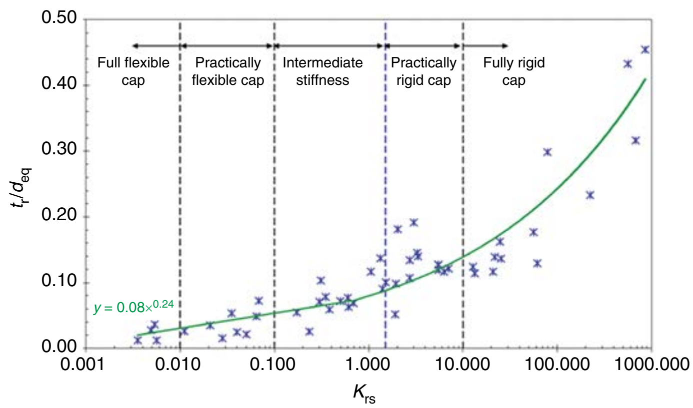


Figure 55.27 Pile cap–soil stiffness ratio K_{rs} and relationship with pile cap thickness t_r/equivalent diameter d_{eq} (After Basile 2020)

(refer to Chapter 52 *Foundation types and conceptual design principles*). Hence, a check on likely differential settlement will be important, especially for large pile groups or sensitive structures.

Pile-group differential settlement depends on the pile-group aspect ratio and on the pile cap stiffness. Randolph and Clancy (1993) have shown that the normalised differential settlement of pile groups, with a fully flexible pile cap, is mainly a function of the pile-group aspect ratio R. Normalised differential settlement is reasonably independent of the precise number of piles, spacing and so on. Hence, for a flexible pile cap

$$\Delta W_{flex} = f(R/4)W_{av} \quad (\text{for } R \leq 4) \tag{55.20}$$

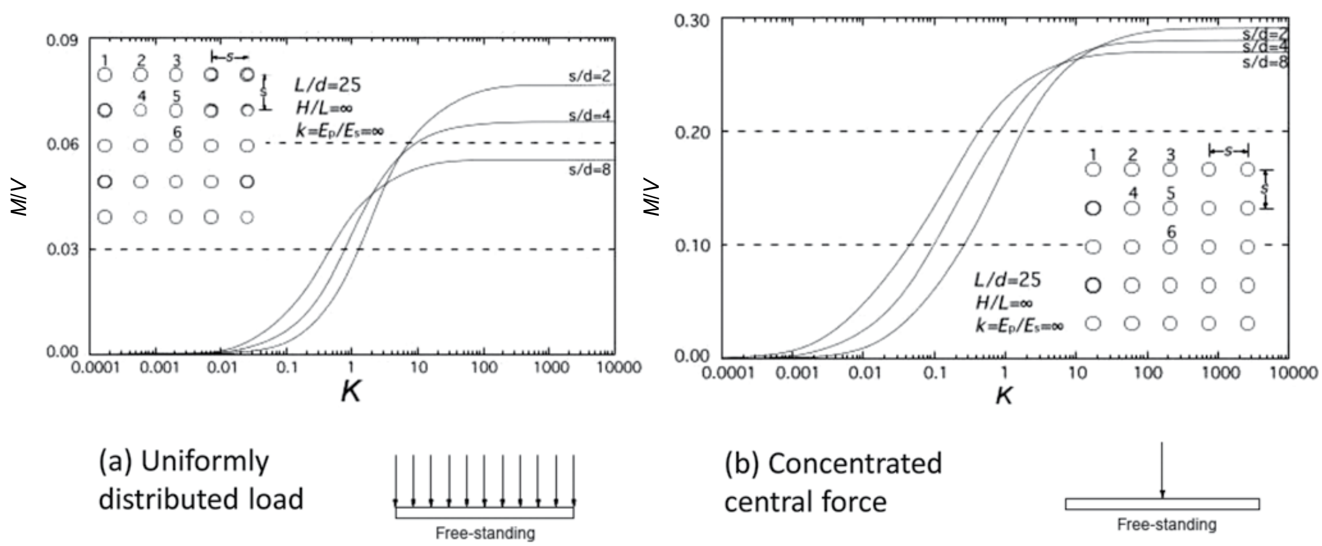
$$\Delta W_{flex} = fW_{av} \quad (\text{for } R > 4) \tag{55.21}$$

with $f = 0.3$ for differential settlement between the centre and mid-side, $f = 0.5$ for centre to corner; ΔW_{flex} is the differential settlement across a flexible pile cap, R is the pile-group aspect ratio and W_{av} is the average pile-group settlement. The pile-group differential settlement ΔW is

$$\Delta W = F_R \Delta W_{flex} \tag{55.22}$$

where ΔW_{flex} is derived from Equations 55.20 or 55.21, and F_R is a correction factor dependent on the pile cap rigidity (varying from 1.0 for fully flexible to 0 for fully rigid), which can be assessed from solutions given in Chapter 53 *Shallow foundations* (Section 53.6.7).

The pile cap rigidity also significantly influences the bending moments induced in the pile cap, especially for medium and large pile groups, as indicated in **Figure 55.28** (for a 5 × 5 pile group). The type and location of applied loads onto the pile cap also affect the induced bending moment (comparing **Figures 55.28(a)** for a uniformly distributed load with **55.28(b)** for a load applied at the centre of the pile cap). Viggiani *et al.* (2012) have shown that simple analytical methods and the 'strut and tie' (STM) analogy are adequate for structural design of pile caps for 'small' pile groups ($n < 5$ piles); however, these methods will usually be inadequate for 'large' pile groups. For large pile groups, more sophisticated analysis (e.g. a 'shell and beam model') of interactions between the pile cap, piles and the ground may facilitate more economic design. Significant economies may be possible if large pile groups are designed as piled rafts (refer to Chapter 56 *Rafts and piled rafts*), with the ground resistance below the pile cap (or raft) taken into account.



NB: where M is the maximum bending moments at the centre of the raft (bending moments are positive when the lower edge of the raft is subject to tension), V is the total vertical load applied, K is relative raft–soil stiffness, E_p is the Young's modulus of the pile, E_s is the Young's modulus of the soil, H is the depth to rigid layer, L is the pile length, d is the pile diameter and s is the pile spacing.

Figure 55.28 Pile cap bending moments as a function of pile cap rigidity (Reproduced from Viggiani *et al.*, 2012)

55.7. Time-dependent settlement

Time-dependent settlement of pile groups under foundation loading can occur due to

- primary consolidation, due to dissipation of excess pore pressure, in clays and silts (Hooper, 1979; Katzenbach *et al*., 2000)
- settlement due to creep, in sands, gravels and weak rock (Mandolini and Viggiani, 1997).

The pile-group aspect ratio and load transfer mechanism are important factors in controlling the amount of time-dependent settlement. For small aspect ratios when load transfer is primarily due to shear, the time-dependent settlement at working load would be expected to be small. Studies by Poulos (1993) indicate that consolidation settlement would be less than about 15% of total settlement. In contrast, compression of the ground below the pile-group base may become increasingly important when the group aspect ratio increases to values in excess of 3 (or the B/L value exceeds 1). Mandolini *et al*. (2005) compared observations of 'end of construction' settlement with observations of time-dependent movements for rafts and piled rafts in Frankfurt and London Clay (both stiff overconsolidated plastic clays). End of construction settlement (which probably includes some primary consolidation settlement) varied between 50% and 75% of the total, with the ratio increasing as B/L reduced (from 2.5 to 1.0). Poulos *et al*. (2001) suggest that creep settlement is unlikely to be significant at working loads (when the group factor of safety exceeds 1.4).

A separate issue that can cause relatively large time-dependent settlement is when large groups of full displacement (closed-end) piles are driven at close spacings into soft clays, silts or peats. Significant heave and lateral displacement can occur during pile driving. The shearing and remoulding of soft soils can generate substantial excess pore pressures, which will lead to large settlement of the heaved clay as the pore pressures dissipate. If the pile group is end-bearing, then negative skin friction forces will result; if it is acting in friction, then large pile-group settlements can develop (Adams and Hanna, 1970; Bjerrum, 1967; Brzezinski *et al*., 1973). This will be independent of, and additional to, any settlement due to foundation or structure loading.

55.8. Optimising the pile-group configuration

Once the decision has been made to use pile groups, some thought will be needed on how to optimise the layout of the pile group in terms of the length, diameter and spacing of the piles. This will be dependent on a number of factors.

- *A shallow foundation would have adequate bearing capacity, but is it likely to suffer excessive settlement?* If a shallow foundation such as a raft has adequate bearing capacity and the requirement is to control settlement, differential settlement or raft bending moment, then a piled raft is likely to be most cost-effective; refer to Chapter 56 *Rafts and piled rafts*. A piled raft would usually comprise a small number of widely spaced piles, normally located beneath the superstructure columns.
- *A shallow foundation would have inadequate bearing capacity?* A conventional pile design approach should be used. A single pile may be adequate (Chapter 54 *Single piles*), although for heavily loaded foundations a deep large-diameter pile may be needed. This may occasionally be advantageous – for example, landing an underbridge to a railway during a short blockage, for designing out pile caps in a floodplain or for some offshore applications. An alternative may be to form a small pile group by installing several smaller piles and a pile cap. This is usually advantageous since the mobilisation costs for a smaller piling rig would be lower. The overall cost–benefit of different piling options will be dependent on several site-specific issues (refer to Chapter 52 *Foundation types and conceptual design principles*), in particular site access and headroom. A holistic approach to site development is needed in terms of the requirements for all foundations across the site and matters such as overall construction sequence and programme, and the available space for foundation construction plant and for building pile caps.
- *Site geology*: if there is a distinctive competent bearing stratum at depth and the piles act predominantly as end-bearing piles, then the appropriate pile length will be obvious, and the main design variables will be diameter and spacing. Alternatively, if the bearing stratum exhibits a gradual increase of strength or stiffness then the benefit of increasing the pile length, rather than the number or diameter of piles, may not be immediately obvious; this is discussed in more detail below.
- *Applied loads*: if high horizontal loads will be applied, then increasing the pile diameter will generally be more effective than increasing the pile length. A critical pile length L_C can be defined (Fleming *et al*., 2009) for lateral loading. L_C/d depends on the relative ground to pile stiffness, but is typically between $6d$ and $10d$ (where d is the pile diameter). Piles longer than L_C will not reduce lateral pile deflection. Structural strength will often be the critical issue for laterally loaded pile groups, and structural strength rapidly increases with increasing pile diameter. For friction piles, if vertical loading and associated pile-group settlement is the key consideration, then increasing the pile length will usually be more cost-effective than increasing the pile diameter. For friction piles there is a critical pile length L_t beyond which further increases in length will not reduce settlement (Fleming *et al*., 2009). L_t is approximately $1.5(E_p/G_b)^{0.5}d$, where E_p is Young's modulus for the pile concrete, G_b is the shear modulus at the pile toe and d is the pile diameter. For friction piles, an equally important consideration is the pile spacing. Provided ultimate group capacity is adequate, a relatively small number of widely spaced piles can provide similar pile-group stiffness to a larger number of more closely spaced piles. The design charts, shown in **Figure 55.20**, can facilitate a rapid comparison between the effectiveness of varying any of pile-group length, diameter and spacing on pile-group settlement performance. Pile spacing is often chosen to be $3d$ for conventionally designed pile groups; increasing the pile spacing to $4d$ or $5d$ may be worthy of consideration since pile interaction effects will be substantially reduced. For friction piles, spacings below $3d$ are usually inappropriate because pile interaction effects become far more significant and the potential benefits (in terms of bearing capacity and settlement) are likely to be negligible relative to the increased costs. For end-bearing piles minimum pile spacing is less of a concern for loss of efficiency.

In general, on the basis of observed performance (e.g. Mandolini *et al*., 2005) it is reasonable to assume that pile-to-pile interaction is negligible for practical purposes at pile spacings of between 6*d* and 8*d* for both vertical and horizontal loading. At these spacings, the pile design can be based on single-pile behaviour; refer to Chapter 54 *Single piles*.

Raking piles: raking piles are sometimes used to support foundations subject to horizontal loads, and they can provide a very stiff support system. Although horizontal deformation will be relatively small compared with vertical piles of the same diameter (under an equivalent horizontal load), the use of vertical piles is usually favoured for several practical and technical reasons.

- Construction tolerances tend to be far poorer for raking piles than for vertical piles. The relevant tolerances are the position and alignment tolerances – that is, comparing the intended toe position with its actual position.
- Construction problems, such as installation damage for driven piles or potential integrity problems for bored cast in situ piles. Any potential problems associated with pile installation, such as groundwater inflows, pile bore instability, penetration into or through hard strata or obstructions, will all be far more challenging for raking than vertical piles. Because of this, raking piles will often be more expensive and slower to install than vertical piles.
- Raking piles should *not* be used in situations where there are likely to be large global ground movements, such as for bridge foundations on soft soils when adjacent approach embankments are likely to induce large vertical and horizontal ground movements (or, for similar reasons, adjacent to deep excavations); nor for heavily loaded foundations that are likely to undergo large settlement. In both situations the relative soil–pile movements are likely to induce bending stresses in the raking piles. In these circumstances, raking piles are vulnerable to being overstressed.

Raking piles were often used historically because of the lack of adequate analytical tools to assess the lateral load–deformation behaviour of vertical piles. Nowadays, there is a wide range of pile-group analysis software available to practitioners, which allows lateral load behaviour to be readily analysed. Therefore, for most circumstances, groups of vertical piles should be adequate. Nevertheless, there are some situations when raking piles can be cost-effective; one example is maritime applications when vertical loads may be negligible but there are substantial horizontal loads, such as for ship berthing facilities. Marine applications also tend to favour the use of driven piles, which are generally better suited to being installed at a rake than bored piles.

55.9. Information requirements for design and parameter selection

55.9.1 Ground profile

Golder and Osler (1968) described the performance of five pile groups (each with 32 piles, 20 m long), which were constructed to support an industrial facility. The piles were founded at a high level in a layer of dense sand. A pile test indicated a settlement of about 1 mm at the working load and pile-group settlement was expected to be less than 10 mm. Fifteen years after construction, the observed settlements were more than 70 mm and continuing to increase. The settlement of the pile groups at each end of the row of five groups showed significant tilt due to interaction with the adjacent pile groups. Subsequent analysis indicated that layers of compressible clay (about 3 m below the pile-toe level) were mainly responsible for the excessive pile-group settlements, which were predicted to reach 90 mm. About 80 mm, out of the total of 90 mm, was due to settlement of the underlying clay layers. This (and other) experience indicates that supporting a structure on a large pile group does not necessarily eradicate all risk of excessive foundation movement. It also highlights the importance of a proper understanding of the overall ground conditions and ground–structure interaction. Clearly, if considered in isolation, the results of a pile load test can be highly misleading.

The requirements for a proper desk study and comprehensive ground investigations are just as important for piled foundations as they are for other aspects of geotechnical engineering. For pile groups, an appreciation of the relative strength and compressibility of strata below the pile group is particularly important. A minimum of the pile-group width and, depending on the overall geology, up to 2–3 times the width should be investigated below the pile-toe level. Hence, deep boreholes may be necessary, together with careful logging and index testing at close centres, to check for weak layers within and below the anticipated bearing stratum. If lateral loading is significant, then the strength and compressibility of near-surface soils (typically the uppermost 6*d*, where *d* is the pile diameter) will be particularly important. This is challenging since near-surface soils can be very variable. Cost-effective investigations of near-surface soils will probably include some form of penetration or probing tests (refer to Chapter 47 *Field geotechnical testing*) to obtain rapid and cheap coverage across the site, together with several trial pits to facilitate interpretation.

55.9.2 Applied loading

The critical load case for pile-group design (especially for bridges and maritime structures) will often include a significant transient live load (due to, for example, vehicle impact or wind loading). For this type of short-term loading appropriate parameters (e.g. undrained strength and stiffness parameters for clays) should be used for pile-group analysis.

55.9.3 Mobilised ground stiffness, vertical loading

Randolph (1994) discusses the implications of nonlinear ground stiffness behaviour on both single-pile and pile-group settlement. **Figure 55.29** summarises the output from these analyses, plotted as normalised secant ground stiffness against factor of safety for a large pile group. The pile-group settlement remains relatively linear until the overall group factor of safety reduces below about 1.6. At a factor of safety of 2.0, the secant stiffness for the soil within the pile group is more than 80% of the initial tangent stiffness (which is close to the modulus mobilised at a strain amplitude of about 0.01% for many soil types). The relationship between choice of elastic modulus and pile-group size is shown schematically in **Figure 55.30**. **Table 55.5** provides guidance on the selection of the appropriate linear elastic moduli for single piles and pile groups of varying size at different factors of safety. Mandolini *et al.* (2005) report back-analyses of 48 pile groups across a range of geologies. Linear elastic analyses assuming very small strain stiffness (initial tangent stiffness from a pile test or derived from field geophysics G_{max} or G_0) generally gave estimated pile-group settlements within 20% of observed settlements. In contrast, a 'secant' stiffness approach (consistent with conventional practice, assuming a stiffness from a pile test at the working load, or using conventional empirical correlations for large strain stiffness) generally gave gross overestimates of pile-group settlement. Chapter 52 *Foundation types and conceptual design principles* gives some guidance on selection of G_{max}.

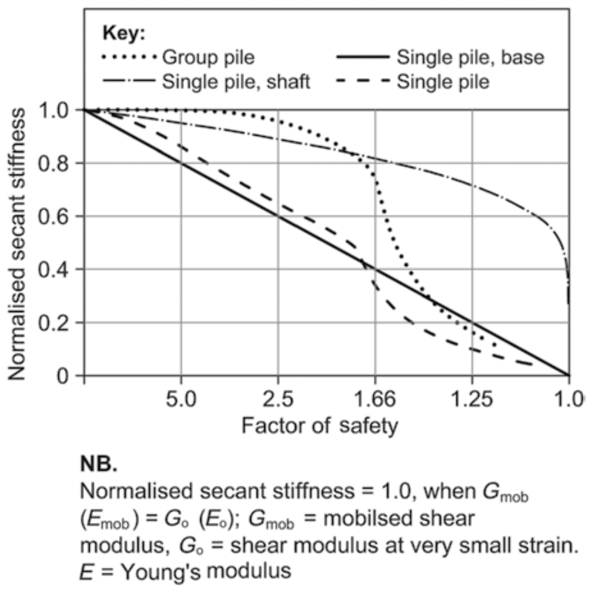


Figure 55.29 Normalised secant stiffness against factor of safety. Nonlinear settlement for a single pile and a pile group (Modified from Randolph, 1994)

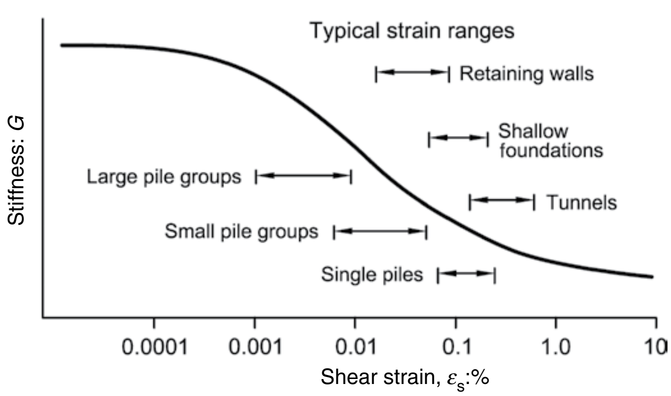


Figure 55.30 Influence of pile-group size on the selection of the appropriate 'elastic' modulus

Factor of safety	G/G_{max} Single pile	Medium pile group	Large pile group
>5.0	>0.6	>0.8	>0.9
3.0	0.45–0.6	0.7–0.9	>0.8
2.0	0.3–0.45	0.6–0.8	>0.8

Above values are for vertical loading and intended as preliminary guidance. A medium pile group has 5–25 piles; large pile group >25 piles. G_{max} is the shear modulus at very small strain for vertical shear. The values are relevant for linear elastic analysis and they assume the pile spacing is 3*d*; as spacing increases the guide values will be increasingly conservative.

Table 55.5 Tentative values of shear modulus ratio (G/G_{max}), linear elastic analyses

55.9.4 Mobilised ground stiffness, lateral loading

For lateral loading, pile-group performance (in terms of deformation and structural demands) can be far more strongly influenced by nonlinear material behaviours than that for vertical loading. For example, both nonlinear ground stiffness reduction with strain amplitude and reduction of structural bending stiffness (for reinforced concrete piles) can become significant as pile-group deformation increases. The potential importance of both of these nonlinear behaviours will depend on the pile head fixity and the overall shape and size of the pile group (**Figure 55.31**). For example, at one extreme if pile head fixity is close to a 'pinned' condition, or for small pile groups (or a single row of piles), then performance is likely to be strongly influenced by nonlinear material behaviours. Modern software (e.g. LPILE or RSPile) can readily analyse the effects of nonlinear ground behaviour through the use of *p*–*y* curves,

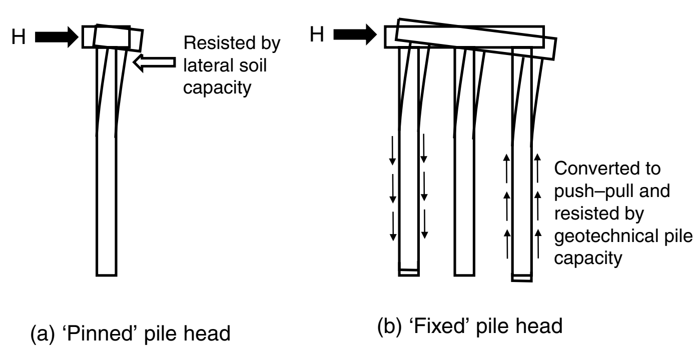


Figure 55.31 Comparison of pile head fixity on group behaviour

for which there is a significant empirical database (e.g. refer to Parkes *et al.*, 2018)).

At the other extreme, if the pile head is approaching a 'fixed' condition, then most of the applied lateral loading tends to be converted into a 'push–pull' action (that is, the applied lateral load is resisted by vertical pile resistance (compression and tension) across the group), which then reduces the practical effects of nonlinearity. If linear elastic analyses are being used, then it is recommended that a simple set of checks are implemented: first, the sensitivity of pile bending moments and group deformation to variations in shallow ground stiffness is checked (i.e. within a depth equal to the 'critical pile length' for lateral loading, L_c, typically the upper 6d to 10d, where d is the pile diameter), perhaps by doubling and halving ground stiffness compared with the design assumption; second, the structural demands are considered in the context of local yield of the pile material (e.g. concrete 'cracking moment').

For critical situations, specialist advice should be sought and more sophisticated nonlinear analyses may be required. Hardy and O'Brien (2006) discussed the influence of ground stiffness nonlinearity on the predictions of bending moment in pile groups subjected to horizontal loads. For single piles, back-analysis of vertical and lateral load tests highlighted the significant influence of nonlinearity on lateral pile performance (across the working load range, 'elastic' secant stiffness reduced to about half its initial value for vertical loading, whereas for horizontal loading it reduced to less than 15% of its initial value). **Figure 55.32** shows that for a medium-size pile group, the pile bending moments can be sensitive to the soil constitutive model (hyperbolic or linear elastic) and, for the linear elastic model, the magnitude of the selected moduli. If the design is substantially influenced by lateral load effects, then nonlinear analyses should be implemented (Mandolini *et al.*, 2005; O'Brien, 2007). These analyses may require specialist input and guidance, although software such as REPUTE includes a hyperbolic model which is quick to use for suitably experienced practitioners.

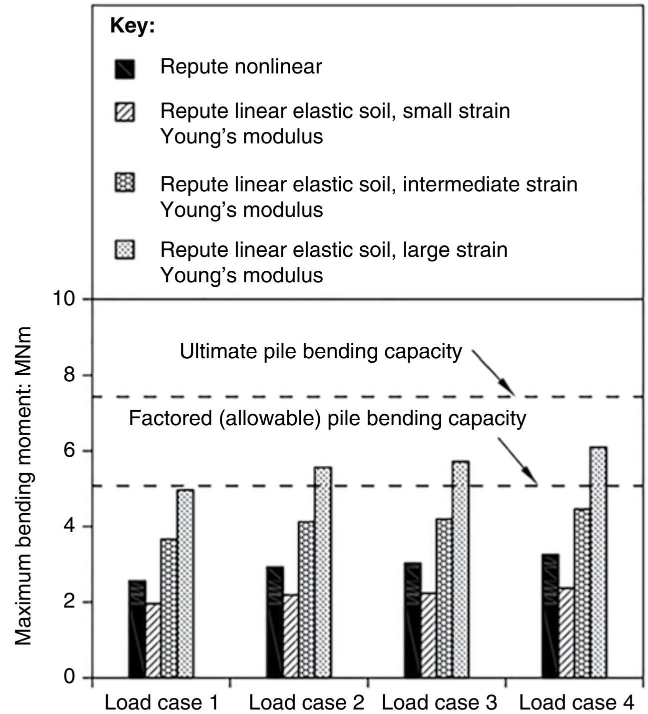


Figure 55.32 Influence of soil model and selected Young's modulus on the peak bending moment under horizontal loading (Reproduced from Hardy and O'Brien, 2006)

55.9.5 Ground stiffness characterisation for pile groups

When considering the selection of input parameters for pile-group analysis, and, in particular, comparing single-pile with group performance, it is helpful to consider the different zones of ground around a pile or pile group that will influence behaviour (**Figures 55.33** and **55.34**), and the relevant information that will be needed for making decisions. Five separate zones, and associated ground stiffness moduli, can affect behaviour.

- The stiffness of the soil or rock close to the pile shaft: this zone will strongly influence the settlement of a single pile at typical working loads. It will be strongly affected by the construction process (remoulding and horizontal stress changes), either due to pile boring or pile driving.
- The stiffness of the soil or rock at the pile toe: this zone is dominated by the construction process and is generally far more sensitive to the construction process than shaft stiffness. Base stiffness will be crucially important for end-bearing piles; however, for many piles (which often gain significant shaft resistance) the base stiffness only becomes influential at low factors of safety. For driven piles the base stiffness tends to be relatively high compared with the original ground stiffness. For bored piles, the base stiffness will tend to be lower, sometimes much lower, than the original ground stiffness and will be highly dependent on factors such as groundwater inflows, debris at the pile toe and the effectiveness of any base-cleaning activities (Fleming, 1992).
- The stiffness of the ground close to the ground surface: this can strongly affect pile behaviour under horizontal loading, especially for single piles and small pile groups. Ground investigations frequently overlook the characterisation of this zone, and it can be extremely variable. Also, site redevelopment (excavation and filling) will often change the nature of this zone. Pile concrete stiffness under lateral loads will tend to be lower (due to yield, creep and cracking) than that for compression.
- Ground stiffness between the piles: this zone has a dominant influence on pile-group settlement. This zone will not be significantly affected by construction activities. As noted earlier, the relevant modulus, for linear elastic analysis, will be close to the very small strain stiffness of the soil or rock, especially for medium and large pile groups.

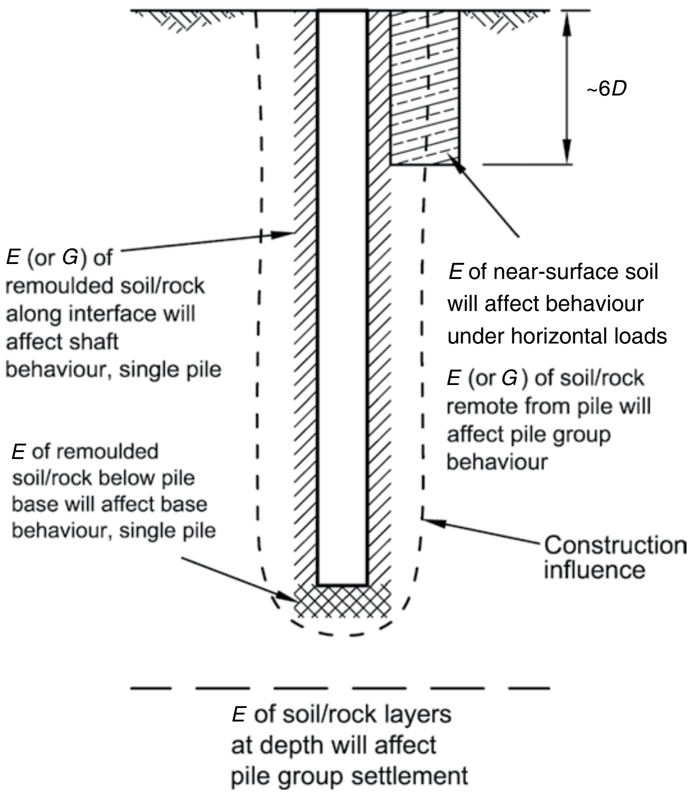


Figure 55.33 Different ground moduli that may influence single-pile and pile-group behaviour

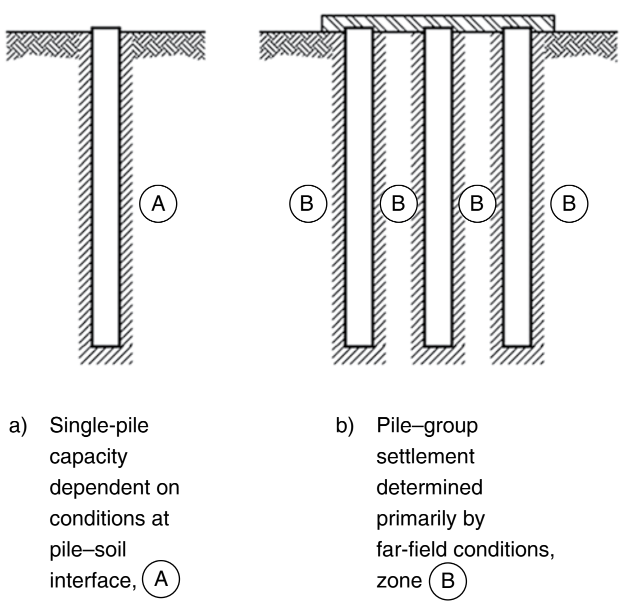


Figure 55.34 Influence of (a) the pile–soil interface on single-pile capacity and (b) far-field conditions on pile-group settlement (Reproduced from Randolph, 2003)

- Ground stiffness well below the pile-toe level: this zone will be unaffected by construction activities. It will not affect the behaviour of single piles, but will influence pile-group behaviour, especially for pile groups with a large group aspect ratio.

Hence, the information required will involve a consideration of the following.

- Construction effects and the back-analysis of single-pile behaviour (e.g. England, 1999; Fleming, 1992). For laterally loaded piles, tests on single piles will usually be free-headed, whereas piles in a group will generally be fixed-headed. This different head fixity must be allowed for when comparing isolated piles and pile-group behaviour.
- Data from relevant case histories (e.g. Mandolini *et al.*, 2005). Case histories are particularly useful for calibration of software. Software calibration is particularly important for large pile groups or more sophisticated nonlinear analytical methods.
- Site-specific ground investigation data. Useful additional data (although seldom provided by routine investigations) would be measurements of G_0 or G_{max}. Modern empirical correlations (e.g. Atkinson, 2000) are helpful for the selection of the appropriate moduli. Chapter 47 *Field geotechnical testing* provides details of field methods for direct measurement of soil stiffness. Modern empirical correlations are helpful for the selection of the appropriate moduli and further guidance is given in Chapter 52 *Foundation types and conceptual design principles.*

55.9.6 Selecting nonlinear ground stiffness parameters for pile-group analysis

Some commercially available pile-group analysis software can simulate nonlinear ground stiffness. The mathematical formulations may vary (although simple power law or hyperbolic models are commonly used) and the inputs cannot necessarily be linked to fundamental or commonly measured ground properties. The ideal means of calibrating these software models is to curve-fit the software load–displacement curve against full-scale pile test data (either site-specific or published data for the relevant geology) for vertical or lateral pile tests as appropriate. In the authors' experience, in the absence of relevant pile test data, the following approaches can also be successful, if implemented with care.

- For vertical loading: use the Fleming (1992) method (for which a good empirical database is available) to generate an appropriate nonlinear load–settlement curve for a single pile, and then calibrate the software's nonlinear model against this. The calibrated model inputs can then be used for pile-group analysis.
- For horizontal loading: use relevant empirical *p*–*y* curves and calibrate against these for a single pile, prior to use for a pile-group analysis.

When developing the pile-group model, the issues highlighted in **Figures 55.33** and **55.34** also need careful consideration, especially for end-bearing piles, so realistic group behaviour is simulated (e.g. appropriate layering below the toe level of the pile group).

55.9.7 Single-pile capacity compared to pile-group deformation

Ultimate geotechnical capacity of a single pile is mainly dependent on local conditions at the pile–soil interface, which are strongly affected by construction methods and can exhibit significant variability (e.g. NCHRP, 2004). In contrast, pile-group deformation is determined primarily by far-field conditions (i.e. ground stiffness between and remote from the piles), and is relatively unaffected by construction processes (**Figure 55.34**). Hence, pile-group deformation can often be more reliably predicted than the ultimate capacity of a single pile (Randolph, 2003). This is in stark contrast to many other aspects of geotechnical engineering (such as predicting movements around retaining walls or shallow foundation movements), where predicting movement is perceived as more difficult than predicting ultimate capacity.

55.10. Some practical design considerations

55.10.1 Factors of safety

Burland (2006) highlighted the profound importance of understanding the mode of foundation failure mechanisms. Routine geotechnical and structural engineering practice is based on the assumption of ductile behaviour. The load–settlement behaviour of pile foundations is usually ductile. However, this is not always the case; for example, experience indicates that some rock socket piles can exhibit brittle behaviour, such as piles socketed into some rocks (e.g. limestone and sandstone) or end-bearing piles toed into thin competent strata and underlain by soft clays. If brittle behaviour is anticipated, then a cautious approach is essential. Rather than summing shaft friction and end-bearing capacity, it would be more appropriate to consider a deformation-based capacity (i.e. discounting some fraction of either shaft or end-bearing resistance). Preliminary pile tests to failure would clearly be helpful. Pile-group design would need to limit pile-group deformation to ensure brittle failure was not induced.

For more normal situations, when pile load–settlement behaviour is ductile, the selected factors of safety will be influenced by

- pile cap and substructure stiffness
- number of piles in the group (i.e. level of redundancy)
- code requirements
- nature and direction of applied loading (e.g. cyclic or monotonic loads, vertical or horizontal loads)
- analysis methods
- reliability and scope of ground investigations.

Table 55.6 gives a commentary on these issues.

Many codes and standards do not make specific statements about factors of safety for pile-group design. However, EC7 provides the following, in Clause 7.6.2.1 (the code uses the term ‘stiffness’ in the context of a structure’s ability to redistribute loads, whereas the term ‘rigidity’ is generally used in this chapter).

Factor	Comment
Pile cap and substructure stiffness	A stiff pile cap or substructure can redistribute axial loads; hence, the individual pile factor of safety is not significant. If the pile cap is flexible then differential settlement needs to be considered. If the pile group is small, individual pile factors of safety need to be considered.
Number of piles in the group	If more than five piles, then there is redundancy, and ‘failure’ of a pile within the group does not imply failure of the group (NCHRP, 2004). For large pile groups there is considerable redundancy.
Code requirements	Many codes do not discuss pile groups in detail and mainly focus on single piles. EC7 provides some guidance, see text. AASHTO (and NCHRP, 2004) gives guidance on reduced risk of failure associated with varying levels of redundancy.
Direction of loading	For horizontal and moment loading, carefully check structural strength of piles, pile cap and pile–pile cap connections. For large long-term moment loading, nonlinear analysis should be considered and risk of excessive group rotation in the long term (creep) assessed, based on holistic assessment of overall group behaviour.
Analysis method	For nonlinear methods reliable calibration of the model is important. Use simple methods to check factor of safety against failure. Computer-based methods are more appropriate for assessing deformation and structural demands on piles.
Reliability and scope of ground investigations	The most important factor to consider. It is especially important to verify the strength and stiffness of layers below the pile group. Near-surface materials are important for laterally loaded pile groups. The greatest uncertainty lies with establishing the geological model, the idealisation of the ground profile for analysis and the selection of appropriate geotechnical parameters.
Nature of loading	Guidance in this chapter is solely for pile groups with predominantly static, monotonic loading. Under prolonged cyclic loading, significant degradation of shaft resistance can occur, with associated substantial increases in deformation and reductions in ultimate capacity (Jardine, 1991).

Table 55.6 Factors that may influence the factor of safety for a pile group

- The stiffness and strength of the structure connecting the piles in the group **shall** be considered when deriving the design resistance of the foundation.
- If the piles support a stiff structure, advantage may be taken of the ability of the structure to redistribute load between the piles. A limit state will occur only if a significant number of piles fail

together; therefore a failure mode involving only one pile need not be considered.

- Special attention should be given to possible failure of edge piles caused by inclined or eccentric loads from the supported structure.

Hence, EC7 allows piles within a pile group to have ratios of ultimate geotechnical capacity to axial load to be below the partial factors required for a single isolated pile (for GEO ULS) provided appropriate checks are made to verify that the overall structural strength/stiffness of the pile cap, pile–pile cap connection and piles are satisfactory (to ensure load redistribution can occur). Hence, it is necessary to check that the pile cap can be considered to be stiff (or 'rigid') rather than flexible. Referring to **Figure 55.27**, the pile cap may be considered 'stiff' if $K_{rs} > 1.5$, and then if the structural strength of the pile cap, pile–pile cap connections and the piles are sufficient, geotechnical failure of the pile group can only develop if either the 'block' resistance is inadequate or the sum of individual pile capacities is inadequate (refer to **Figures 55.4(a)** and **55.4(b)**). In general, there are three options for verifying GEO ULS capacity.

(*a*) Sum the design resistance of all the piles, or the equivalent block, and check it exceeds the applied design loads. This is relatively simple if the pile group is subject to vertical loads, but more complex for combined loading.

(*b*) For pile groups subject to combined vertical, moment and lateral loads, input the **design resistance** of the piles and loads (i.e. relevant resistance and load factors applied) into a **nonlinear** pile-group analysis (i.e. using a hyperbolic model or similar) and check that there are piles with available 'spare' capacity to avoid an overall failure mechanism.

(*c*) For pile groups subject to combined vertical, moment and lateral loads, check that the design resistance of the leading row exceeds the sum of the mobilised axial loads.

Option (*c*) is quite conservative, since the 'spare' capacity available in the other rows in a group is ignored and the ability of the stiff pile cap to redistribute loads is also ignored (refer to **Figure 55.4(c)**). It will be particularly conservative if based on a linear elastic analysis, since the axial loads in the perimeter piles will be overpredicted. Nevertheless, the check is quick and simple to undertake. Option (*b*) can enable more economic pile groups to be designed. As noted in the case history example (and by O'Brien and Bown, 2010), there are major structures where some of the piles within the group have factors of safety of less than 1.0 (based on linear elastic analyses). However, when the pile cap is stiff and the pile response is ductile, then the overall pile-group stability is entirely satisfactory and pile-group deformation is relatively small. In practice, group GEO ULS capacity is seldom critical, and SLS and structural strength are usually the key criteria for pile groups, especially for medium to large pile groups. Therefore, it is important to consider the structural demands (e.g. axial loads, bending moments) across a pile group, together with pile group deformation.

55.10.2 Structural checks and pile head fixity

The structural strength of piles, pile cap and pile–pile cap connections should always be checked and be sufficient to resist the induced axial forces, bending moments and shear forces. This is particularly important for pile groups subject to large moment or horizontal forces, since inadequate structural strength in any part of the overall foundation system could lead to a brittle and progressive collapse of the structure.

The concept of free and fixed head piles in Section 54.6.5 is explained further by considering partial fixity provided by the pile cap in small pile groups. Small pile groups (< 5 piles) are very common for supports to single columns in buildings and partial fixity or moment restraint at the pile head can be considered to improve the economy of the structural design of the piles. When designing for large accidental impact actions the need to consider partial fixity to achieve a practical solution is particularly important.

Some geotechnical software assumes a fully fixed head condition which may be relevant for larger pile groups, but this can generate an excessively large negative bending moment at the top of the pile which does not occur in partially fixed small pile groups. Soil–structure interaction software including, but not limited to, LPILE and RSPile provide functionality to apply a moment restraint at the head of the pile for the predicted level of fixity. The method to calculate the rotational stiffness is illustrated in **Figure 55.35** and relies on a prediction of pile settlement or heave which could be made by use of Fleming's method for single-pile settlement (Fleming, 1992) or from pile load testing. A case history to illustrate this method is included in Section 55.12.

55.11. Pile-group design responsibility

Two options are normally considered: contractor's design or engineer's design.

(*a*) *Contractor's design:* this is commonly used, especially for small building projects. The engineer employed by the client as the overall project designer should provide the piling tenderers with relevant information, including site history, ground conditions, applied structural loads and acceptance criteria for pile loading tests. The contractor is responsible for detailing the appropriate pile length, diameter and reinforcement to resist the specified loads. The pile cap is usually designed by the overall project designer.

(*b*) *Engineer's design:* under this arrangement design responsibility lies with the overall project designer, who

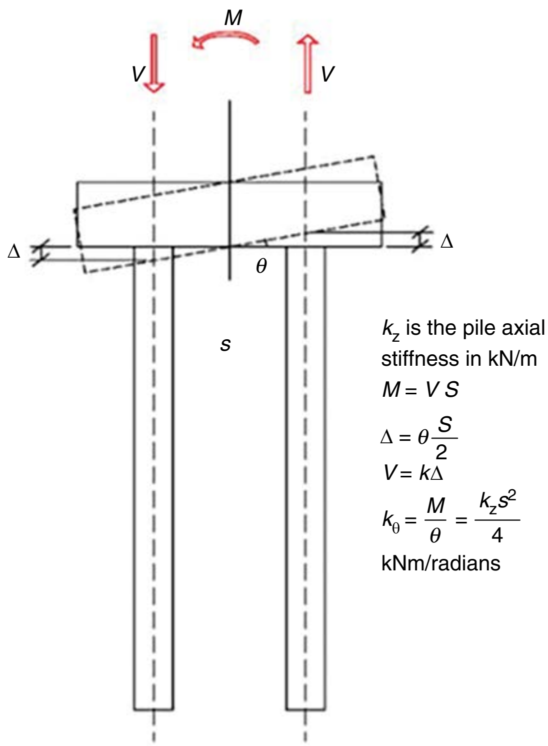


Figure 55.35 Rotational fixity of twin piles by pile cap (Reproduced from Raison and Egan 2016)

would design and detail the pile lengths, reinforcement and so on. This arrangement is often followed for major civil engineering projects, such as bridges or large building projects. The contractor is responsible for workmanship and method of construction.

There are advantages and disadvantages with both options. The specialist piling contractor will have a lot of expertise on how to best utilise his materials, plant and labour. He may also have local experience on the likely pile capacities that can be achieved in particular geological sequences. Therefore option (*a*) will favour those situations where the geotechnical capacity of a single pile is the dominant consideration, where buildability is very challenging (due to poor access, etc.) or if particular proprietary piling systems are well suited to local conditions. Option (*a*) is less appropriate if pile-group deformation is critical or if there are broader ground–structure interaction problems (due to complex loading or if global ground movements are a concern). A particular problem with option (*a*) for pile-group design is the appropriate specification of pile loads and pile test acceptance criteria. For example, axial loads will not be uniform across the group, and the stresses induced in the piles and pile cap will be a function of the pile length, diameter and so on and the pile cap bending stiffness. The deformation of the pile group is dependent on several factors and not just the settlement characteristics of an isolated pile. Hence, the overall designer needs to have a good understanding of these ground–structure interaction issues before realistic and sensible specification requirements for pile capacity and settlement (for an isolated pile) can be developed. In many cases, if design responsibility is to be split then it is more appropriate to split responsibility at the top of the pile cap level rather than at the top of the pile level. The overall designer then only needs to specify the appropriate loads at the pile cap level, and the acceptable limits for overall pile-group deformation and differential settlement across the pile cap. Option (*b*) will also be more appropriate for large pile groups or for pile groups underlain by compressible layers (where single-pile performance cannot be easily correlated with pile-group performance). Option (*b*) would also facilitate more economic solutions, such as piled rafts (refer to Chapter 56 *Rafts and piled rafts*).

55.12. Case histories

Two case histories are discussed: first, pile-group analyses and observed performance are discussed for a large pile group; second, analysis for a small pile group is outlined which facilitated more economic design.

55.12.1 Large pile group

This case history is for the Emirates Twin Towers in Dubai (which has been described in detail by Poulos and Davids, 2005). The ground conditions mainly comprise interbedded calcareous sandstone and siltstone. The measured unconfined compressive strength is variable and typically between about 0.5 MN/m^2 and 1.5 MN/m^2. The stiffness at very small strain was measured by cross-hole geophysical tests, and shear moduli varied between 2 and 3.5 GN/m^2. A summary of the ground profile is given in **Table 55.7**. The twin towers were founded on two triangular-shaped pile groups comprising 92 and 102 1.2 m dia. piles, 40 m long. The estimated ultimate geotechnical capacity of a single pile was about 42 MN.

Based on the published data, two different analyses have been carried out, the first using a computer-based method and the other using a simple empirical method. The computer-based analysis, using the software REPUTE, was used with two different models (linear elastic and nonlinear hyperbolic) to simulate the load–displacement characteristics of the piles. The input parameters are summarised in **Table 55.8**. The load–settlement behaviour of a single pile was modelled initially and calibrated against preliminary pile test data for a 0.9 m dia., 40 m long pile (**Figure 55.36**) prior to modelling the entire pile group. For the single-pile calibration, Fleming's method (Fleming, 1992) was used to derive reasonable nonlinear parameters for use in the hyperbolic model.

The calculated pile-group axial load distributions and settlement given by the two different analytical models are

Strata number	Strata designation	Typical depth of strata base: mbgl	Description
1	Silty sand	5	Uncemented calcareous silty sand, loose to moderately dense
2	Sand	10	Variably and weakly cemented calcareous silty sand
3	Calcareous sandstone	29	Calcareous sandstone, slightly to highly weathered, well cemented
4	Cemented sand (?)	35	Calcareous silty sand, variably cemented, with localised well-cemented bands
5	Calcareous siltstone and conglomerate	56	Variably weathered, very weakly to moderately well cemented
6	As Unit 5	70	As Unit 5

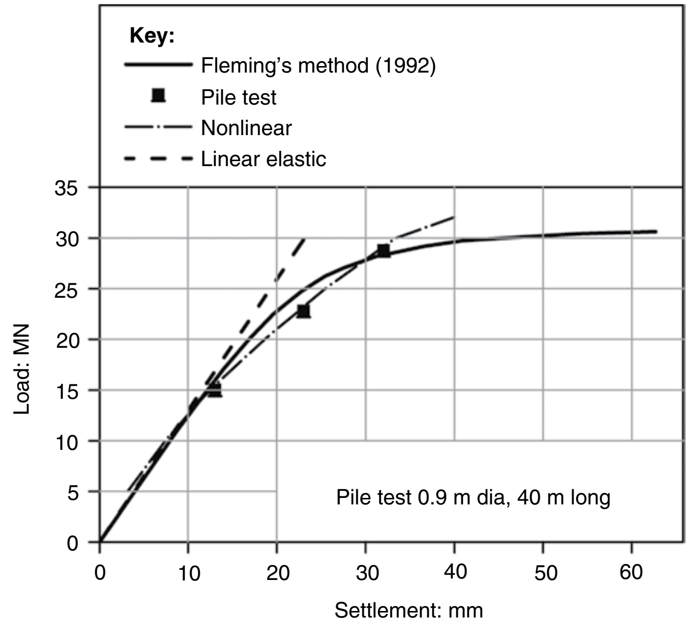
mbgl, metres below ground

Table 55.7 Case history: ground profile

Strata	Linear elastic[a]	Nonlinear[b]	
	Secant Young's modulus: MN/m²	Tangent Young's modulus (small strain): MN/m²	Interface strength τ: kN/m²
Silty sand	15	$20 + 8Z$	$20 + 5.5Z$
Sand	50		
Calcareous sandstone	250	$1500 + 40Z$	$150 + 12Z$
Cemented sand (?)	50		
Calcareous siltstone and conglomerate	200	$3000 + 50Z$	500

[a] Linear elastic model, pile base stiffness, $E' = 40$ MN/m².
[b] Nonlinear model, hyperbolic constants, $R_{shaft} = 0.65$, $R_{base} = 0.99$, maximum shaft friction = 500 kN/m², maximum base pressure = 2700 kN/m² and pile base modulus $E_0' = 750$ MN/m² (E_0 = small strain tangent stiffness).
Z is the depth below the ground surface.

Table 55.8 Linear elastic and nonlinear input parameters used in the REPUTE analyses of the case history

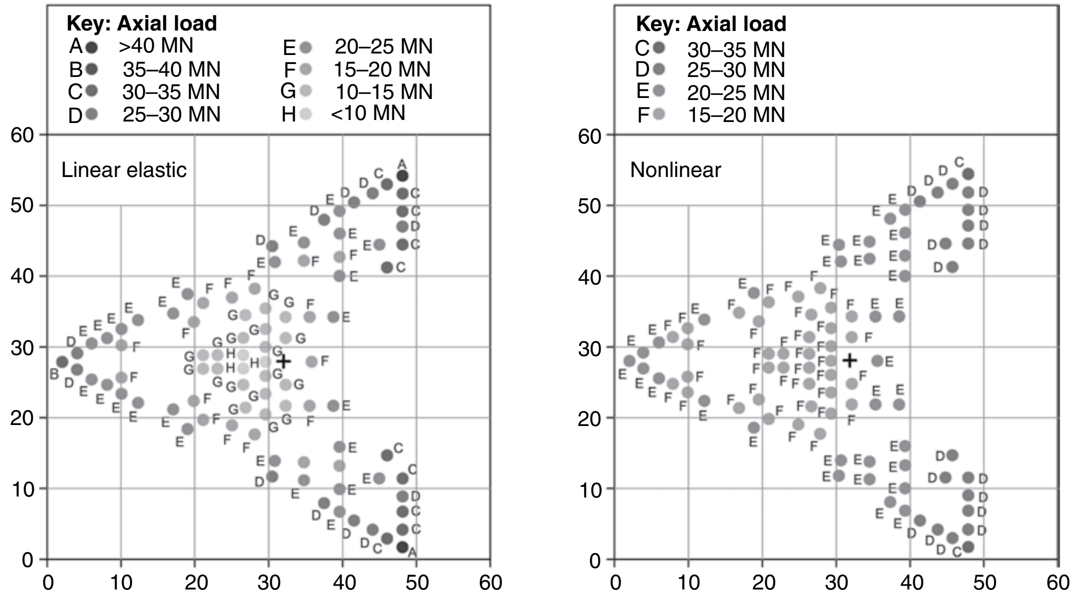


At a load of 15 MN, Geo factor of safety ~ 2.0	
Analysis/data	Settlement: mm
Linear elastic	11.6
Nonlinear	12.3
Pile test	12.9

Figure 55.36 Case history: pile test in weak rock

summarised in **Figure 55.37**. The linear elastic analysis predicts a highly non-uniform distribution of axial load; in particular, axial loads in the corners are especially large. The linear elastic analysis predicts maximum axial loads of 43 MN compared with a maximum of 31 MN from the nonlinear analysis. The ratio of corner to central pile axial load from the linear elastic analysis is about 5, and for the nonlinear analysis it is about 2. Hence, the local factor of safety for a pile in the group (based on linear elastic analysis) is less than 1.0, although the overall factor of safety for the pile group is about 2.0. This is similar to the results from boundary element analysis reported by Poulos and Davids (2005), where they state that 'a number of the piles reached their full geotechnical design resistance (i.e. the factor of safety for some piles was about 1.0), but the foundation as a whole could still support the ultimate design loads'. The pile-group settlement was observed to be about 8–10 mm after 70% of the load had been applied, and the estimated final settlement is 20–40 mm. The nonlinear analysis indicates a settlement of 25 mm, compared with 55 mm from the linear elastic analysis (**Figure 55.37**).

The pile-group aspect ratio is about 2.9 and Mandolini's empirical correlations for pile-group settlement (refer to Section 55.5.2) gives settlement ratios R_{Se} between 4 and 7. Based on the site geology (weak rock with stiffness increasing with depth) the best-estimate and lower-bound pile-group settlement ratios are relevant (Equations 55.9 and 55.12). The single-pile settlement at the average working load of 21 MN is about 10–11 mm. Hence, based on Mandolini's empirical method, the pile-group settlement would be expected to be about 40–75 mm. Although conservative relative to



1.2m dia. piles, ultimate capacity ~42 MN, Geo facor of safety for group ~2.0		
	Peak axial load	Group settlement: mm
Linear elastic	43 MN	55 mm
Nonlinear elastic	31 MN	25 mm
Estimated settlement (in situ measurements) ~20 to 40 mm		

Figure 55.37 Case history: pile group in weak rock; axial load distributions and settlement for the linear elastic and nonlinear models

observations and the estimated final value, it is more accurate than the designer's original estimate of about 90–140 mm (based on a linear elastic boundary element analysis). Poulos and Davids describe the two main reasons for the initial overprediction using boundary element analysis

- overestimation of pile–pile interaction
- underestimation of the ground stiffness for Unit 5 (**Table 55.7**).

Poulos and Davids produced revised analyses where the stiffness varied radially from each pile, so that the ground stiffness between the piles was five times stiffer than that adjacent to the piles (to values of about 2–2.5 GN/m^2 – that is, close to the geophysics, G_0, values), and the ground stiffness for Unit 5 was increased from 80 to 600 MN/m^2. This revised analysis indicated a settlement of 23–40 mm, (allowing for differential settlement across the pile cap). With the benefit of hindsight, it seems likely that the laboratory-measured stiffness for Unit 5 was far too low because of sample disturbance.

This case history emphasises the challenges in deriving realistic ground stiffness values, especially in weak rocks, and the potential errors even when sophisticated analyses are utilised. It also highlights the value of simple empirical methods to check computer output. It is likely that the axial loads around the perimeter of the pile group were high and the factor of safety against geotechnical failure for the perimeter piles was much lower than code values (nonlinear analyses predict 'local' factors of safety of about 1.3–1.4 for the perimeter piles). Nevertheless, the overall group factor of safety is code compliant (equal to about 2.0), the piles and pile cap have adequate strength and stiffness to redistribute forces across the group and therefore the overall pile-group performance is satisfactory.

55.12.2 Small pile group

Pile-group analyses are briefly described for a small pile group on the Werrington Grade Separation scheme, near Peterborough, UK. A new footbridge was constructed with one abutment very close to the live track and the risk assessment resulted in a need to design the foundations for an accidental horizontal impact force of 2000 kN acting at the top of the pile cap. Due to restricted space in the area of the foundation, a four-pile group was chosen with 900 mm dia. bored piles at 2D spacing, 11 m in length penetrating made ground over Oxford Clay, where strength improved with depth from a firm to a very stiff clay or extremely weak mudstone (**Figure 55.38**).

The worst-case orientation of impact for the pile design was 45° to the group, where the combined compressive and horizontal actions resulted in 2009 kN compression on the pile furthest from impact, a tension of −1134 kN on the nearest pile and +438 kN compression on the two middle piles. The vertical pile stiffness was based on a load of 2000 kN divided by an estimated pile settlement of 0.01 m, giving 'best estimates' for a vertical pile spring of k_z of 200 000 kN/m and a rotational spring k_θ of 162 000 kNm/radian, derived using the formula in **Figure 55.35**. **Figure 55.39** illustrates the differences in pile bending moment and deflection for the full range of pile head fixity.

Due to the lack of pile load test data to verify vertical stiffness, a sensitivity analysis was carried out assuming half and double the best estimate of applied moment restraint at the pile head in order to calculate the required reinforcement. The partially fixed laterally loaded pile analysis resulted in a 25% reduction in pile reinforcement and avoided steel congestion in the pile cap (all reinforcement had a clear spacing greater than 100 mm). Hence, buildability was improved and the design was safer to build.

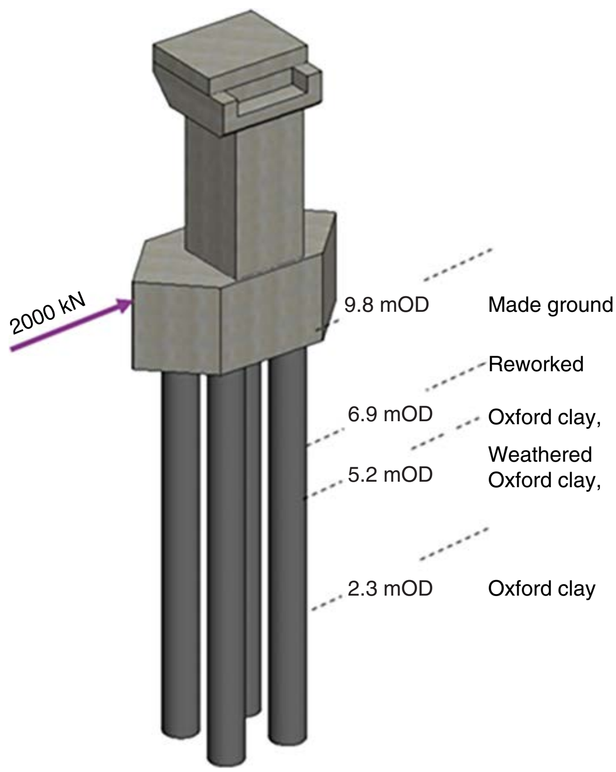


Figure 55.38 3D view of the pile group to the west abutment of Cock Lane Footbridge, subject to accidental derailment action

55.13. Overall conclusions

- The overall behaviour of pile groups is affected by their geometry. An important parameter is the pile-group aspect ratio (**Figure 55.3**). Where R is large, the group settlement will be many times larger than the settlement of an isolated pile (at the same average load). For pile groups with a large aspect ratio, $R > 3$, an 'equivalent raft' analogy is appropriate; where $R < 3$ an 'equivalent pier' analogy is more appropriate.
- The failure of pile groups is rare; the risk of failure increases when global movements develop around the group (typically due to out-of-balance loads on soft clays and peats); end-bearing piles toed into a thin competent layer are underlain by a weaker underlying strata; or piles are toed into steeply dipping fractured rock or rock containing voids.
- Pile–pile interaction, under vertical or horizontal loads, depends on many factors (**Table 55.2**), including pile spacing and the

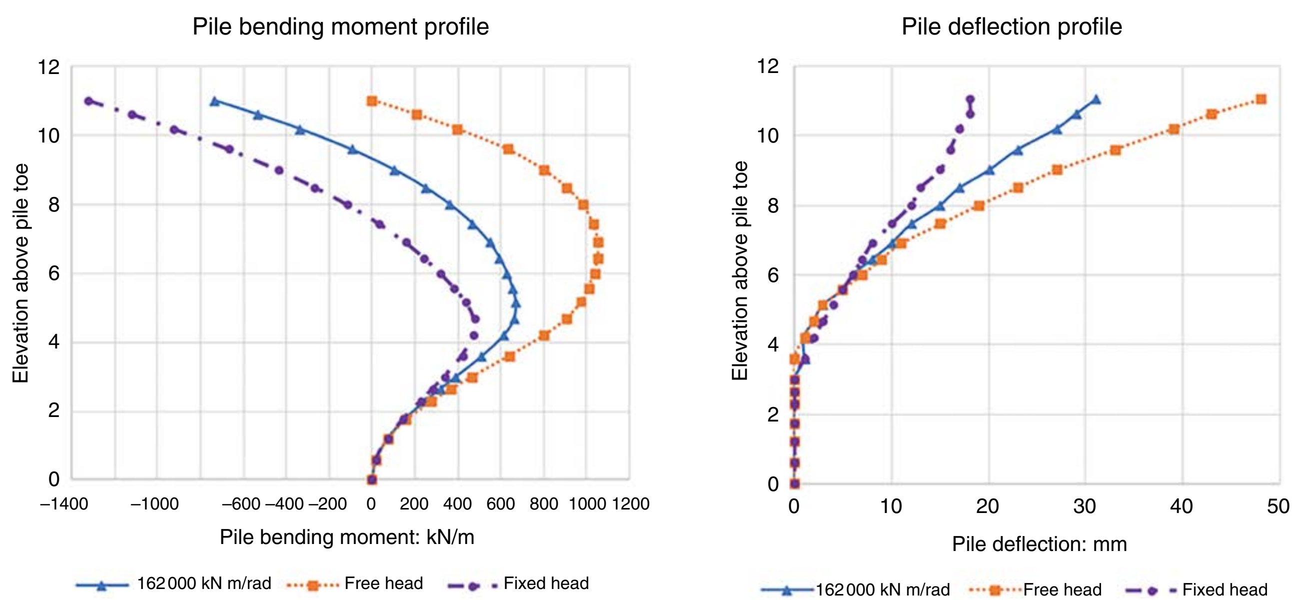


Figure 55.39 WALLAP single-pile analysis results for bending moment and deflection of a pile under different head fixity conditions

variation of ground stiffness with depth, especially 'hard' or 'soft' layers beneath the pile toe (for vertical loads) or adjacent to the pile head (horizontal loads). For horizontal loads, the pile head fixity is important, whereas for large pile groups under vertical loads the pile cap stiffness is important.

- For small- to medium-sized vertically loaded pile groups linear elastic analysis may be adequate for a practical design, although care is required to avoid over-conservatism. However, for larger groups or pile groups subject to large horizontal loads, linear elastic analyses are likely to be inappropriate. Simplified methods of analysis (based on empirical settlement ratios, elastic interaction factors or an equivalent pier) will often be sufficient to calculate the group settlement and should always be carried out as a check on more sophisticated methods.
- When considering the selection of input parameters for pile-group analysis, it is helpful to consider the five separate zones (and associated deformation moduli) that can affect behaviour (**Figures 55.33** and **55.34**): (*a*) the stiffness of the ground adjacent to the pile shaft; (*b*) the zone immediately below the pile toe; (*c*) the stiffness of the ground close to the pile head; (*d*) the ground stiffness between the piles; and (*e*) the ground stiffness at depth below the pile-toe level.
- The appropriate values for factors of safety selected for pile-group design depend on several factors – refer to **Table 55.6**. These include pile cap and substructure rigidity, the number of piles in a group (redundancy), code requirements, the nature and direction of loading, analysis methods and the reliability and scope of ground investigations. In addition, the ductility or brittleness of pile load–settlement behaviour is an important consideration.
- If design responsibility is split for pile-group design, then careful thought is required regarding the most appropriate point to divide responsibility (pile head or pile cap level). When pile-group deformation is critical, or if global ground movements or complex loads are applied, then pile and pile cap design should be considered together so that the soil–structure interaction is properly assessed.

References

Adams JI and Hanna TH (1970) Ground movements due to pile driving. *Proceedings of the Conference on the Behaviour of Piles, London, UK*. ICE Publishing, London, UK, pp. 127–133.

Atkinson JH (2000) Non-linear soil stiffness in routine design. *Géotechnique* **50(5)**: 487–508.

Basile F (2020) Influence of cap rigidity on pile group design. *Proceedings of the Piling 2020 Conference*. ICE Publishing, London, UK.

Bjerrum L (1967) Engineering geology of normally-consolidated marine clays as related to the settlement of buildings. *Géotechnique* **17(2)**: 83–117.

Brzezinski LS, Shector L, MacPhie HL and Vander Noot HJ (1973) An experience with heave of cast *in situ* expanded base piles. *Canadian Geotechnical Journal* **10(2)**: 246–260.

Burland JB (2006) Interaction between structural and geotechnical engineers. *The Structural Engineer*, 18 Apr.: pp. 29–37.

Butterfield R and Douglas RA (1981) *Flexibility Coefficients for the Design of Piles and Pile Groups. CIRIA Technical Note 108*. CIRIA, London, UK.

Cooke RW, Bryden Smith DW, Gooch MN and Sillet DF (1981) Some observations of the foundation loading and settlement of a multi-storey building on a pile raft foundation in London Clay. *Proceedings of the ICE, Part 1* **70**: 433–460.

England M (1999) *A Pile Behaviour Model*. PhD thesis, Imperial College, University of London, London, UK.

Fleming WGK (1992) A new method for single pile settlement prediction and analysis. *Géotechnique* **42(3)**: 411–425.

Fleming WGK, Weltman AJ, Randolph MF and Elson WK (2009) *Piling Engineering*, 3rd edn. Taylor & Francis, London, UK.

Golder HQ and Osler JC (1968) Settlement of a furnace foundation. *Canadian Geotechnical Journal* **5(1)**: 46–56.

Hardy S and O'Brien AS (2006) Nonlinear analysis of large pile groups for the New Wembley Stadium. *Proceedings of the 10th International Conference on Piling and Deep Foundations, Amsterdam, the Netherlands* (Bottiau LM and VanTol AF (eds)). Deep Foundations Institute, Hawthorne, NJ, pp. 303–311.

Hooper JA (1979) *Review of Behaviour of Piled Raft Foundations*. CIRIA Report 83. CIRIA, London, UK.

Jardine RJ (1991) The cyclic behaviour of large piles with special reference to offshore structures. In *Cyclic Loading of Soils* (O'Reilly MP and Brown SF (eds)). Blackie, Glasgow, pp. 174–248.

Jardine RJ, Potts DM, Fourie AB and Burland JB (1986) Studies of the influence of non-linear stress–strain characteristics in soil structure interaction. *Géotechnique* **36(3)**: 377–396.

Katzenbach R, Arslan U and Moormann C (2000) Piled raft foundation projects in Germany. In *Design Applications of Raft Foundations* (Hemsley JA (ed)). Thomas Telford, London, UK, pp. 323–391.

Koizumi Y and Ito K (1967) Field tests with regard to pile driving and bearing capacity of piled foundation. *Soil and Foundations* **3**: 30–53.

Mandolini A and Viggiani C (1997) Settlement of piled foundations. *Géotechnique* **47(3)**: 791–816.

Mandolini A, Russo G and Viggiani C (2005) Pile foundations: experimental investigations, analysis and design. *Proceedings of the 16th ICSMGE, Osaka, Japan*, vol. 1, pp. 177–213.

Matsui T (1993) Case studies on cast-in-place bored piles and some considerations for design. *Proceedings of BAP II, Ghent, Belgium*. Balkema, Rotterdam, the Netherlands, pp. 77–102.

NCHRP (National Cooperative Highway Research Program) (2004) *Load and Resistance Factor Design (LRFD) for Deep Foundations. Report 507.* Transportation Research Board, Washington, DC.

O'Brien AS (2007) Raising the 133 m high triumphal arch at the New Wembley Stadium, risk management via the observational method. *Proceedings of the 14th European Conference on Soil Mechanics and Geotechnical Engineering, Madrid, Spain*, vol. 2, pp. 365–370.

O'Brien AS and Bown A (2010) Pile group design for major structures. *Proceedings of the 11th International Conference on Piling and Deep Foundations, London, UK*. DFI, London, UK.

O'Neill MW, Hawkins RA and Mahar LJ (1982) Load transfer mechanisms in piles and pile groups. *Journal of the Geotechnical Engineering Division* **108(GT12)**: 1605–1623.

Parkes J, Castelli R, Zelenko B, *et al.* (2018) *Design, Analysis, and Testing of Laterally Loaded Deep Foundations that Support Transport Facilities. Geotechnical Engineering Circular FHWA-HIF-18-031.* Department of Transport, Washington, DC.

Poulos HG (1989) Pile behaviour: theory and application. *Géotechnique* **39(3)**: 365–415.
Poulos HG (1993) Settlement prediction for bored pile groups. In *Deep Foundations on Bored and Auger Piles* (van Impe WF (ed)). Balkema, Rotterdam, the Netherlands, pp. 103–189.
Poulos HG (2005) Pile behaviour: consequences of geological and construction imperfections. *Journal of Geotechnical and Geoenvironmental Engineering* **131(5)**: 538–563.
Poulos HG and Davids AJ (2005) Foundation design for the Emirates Twin Towers, Dubai. *Canadian Geotechnical Journal* **42**: 716–730.
Poulos HG and Davis EH (1980) *Pile Foundation Analysis and Design*. Wiley, New York, NY.
Poulos HG, Carter JP and Small JC (2001) Foundations and retaining structures: research and practice. *Proceedings of the XV International Conference on Soil Mechanics and Foundation Engineering, Istanbul, Turkey*, vol. 4, pp. 2527–2606.
Raison C and Egan D (2016) Design of piles: United Kingdom practice. *International Symposium on Design of Piles in Europe, Leuven, Belgium*, pp. 1–24.
Randolph MF (1994) Design methods for pile groups and piled rafts. *Proceedings of the 13th ICSMFE, New Delhi, India*, vol. 5, pp. 61–82.
Randolph MF (2003) Science and empiricism in pile foundations design. The 43rd Rankine Lecture. *Géotechnique* **53(10)**: 847–875.
Randolph MF and Clancy P (1993) Efficient design of piled rafts. *Proceedings of the 2nd International Geotechnical Seminar on Deep Foundations on Bored and Auger Piles, Ghent, Belgium*, pp. 119–130.
Tomlinson MJ (1987) *Pile Design and Construction Practice*, 3rd edn. Palladian, Farnham, UK.
Viggiani C, Mandolini A and Russo G (2012) *Piles and Pile Foundations*. Spon Press, London, UK.

It is recommended this chapter is read in conjunction with

- Chapter 19 *Settlement and stress distributions*
- Chapter 22 *Behaviour of single piles under vertical loads*
- Chapter 54 *Single piles*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*