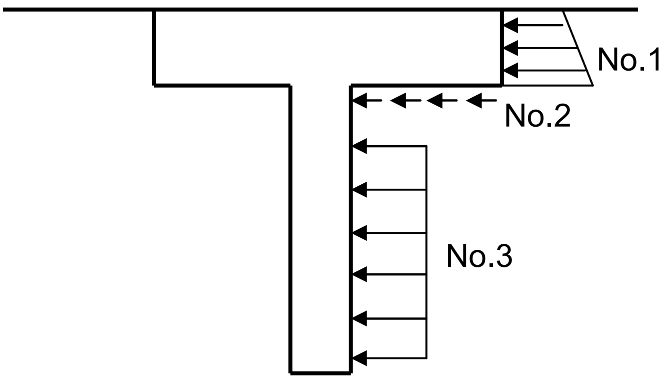
10) Laterally Loaded Vertical Piles

- Sources of Resistance

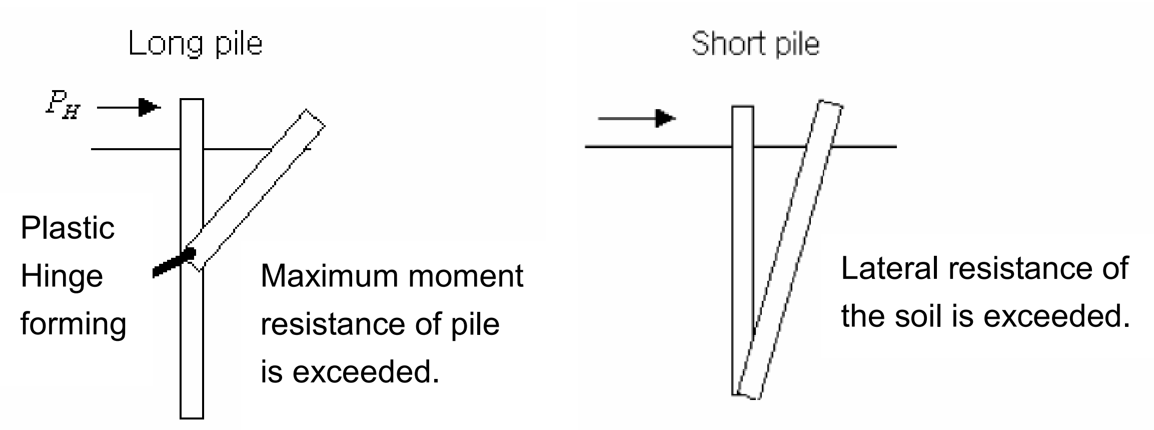


① Passive resistance against the side of pile cap. (Scouring or artificial excavation can eliminate its effect.)
② Shearing resistance along bottom of pile cap and soil interface.
(Settlement of soil beneath cap may eliminate its effect.)
③ Moment and shear resistance of pile itself.

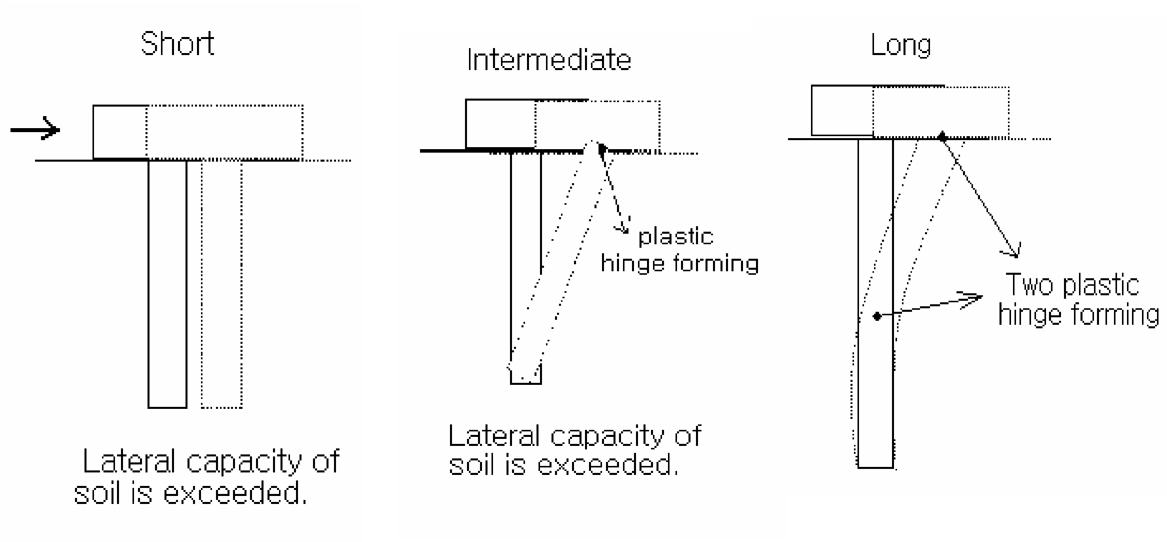
→ Use ③ for determining lateral capacity.

- 5 Potential Failure Modes

  - Free headed piles



- Fixed(restrained) pile head



i) Ultimate Soil Capacity

●

⇒ Brom's method

a) Cohesive soils

- Simplified distribution of the ultimate static resistance (P_{ult}) (Fig 17.8 & 17.9)
- Free head piles

$$D_{\min} = \sqrt{\frac{FV(e + 1.5B + 0.5f)}{2.25Bs_u}} + 1.5B + f$$

$$f = \frac{FV}{9Bs_u}$$

($F \equiv$ Factor of safety=3.0, e=M/V)

- Restrained head piles

$$D_{\min} = \frac{FV}{9Bs_u} + 1.5B$$

b) Cohesionless soils

$P_{ult} = 3\sigma_v^{'} K_P D$ $\Rightarrow$ triangular distribution in a uniform soil (Fig 17.10 & 17.11)

- Free head piles

$$F = \frac{0.5\gamma' B D_{\min}^3 K_p}{V(e + D_{\min})} \quad (\approx 3.0)$$

- Restrained head piles

$$D_{\min} = \sqrt{\frac{FV}{1.5\gamma' B K_p}}$$

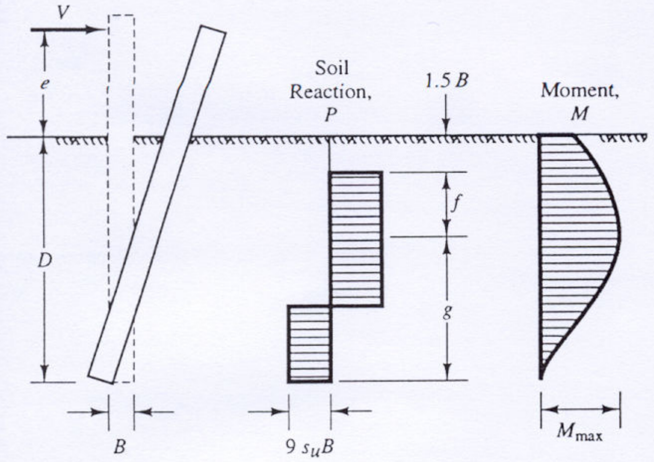


Figure 17.8 Soil reactions and foundation moments for free-head foundations in cohesive soil (Adapted from Broms, 1964a; Used with permission of ASCE).

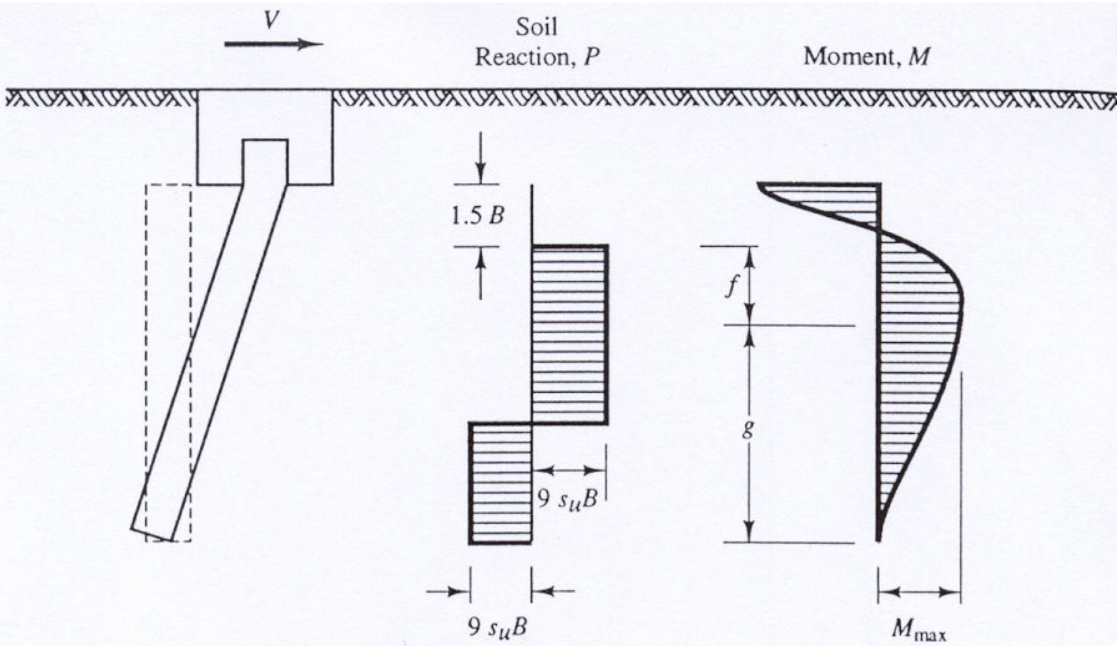


Figure 17.9 Soil reactions and foundation moments for restrained-head foundations in cohesive soil (Adapted from Broms, 1964a; Used with permission of ASCE).

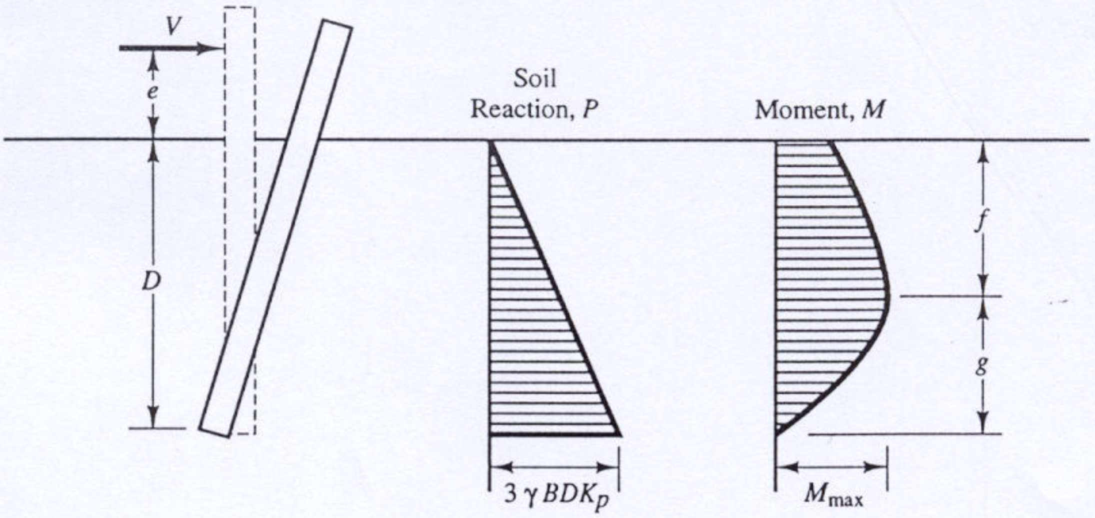


Figure 17.10 Soil reactions and foundation moments for free-head foundations in cohesionless soil (Adapted from Broms, 1964b; Used with permission of ASCE).

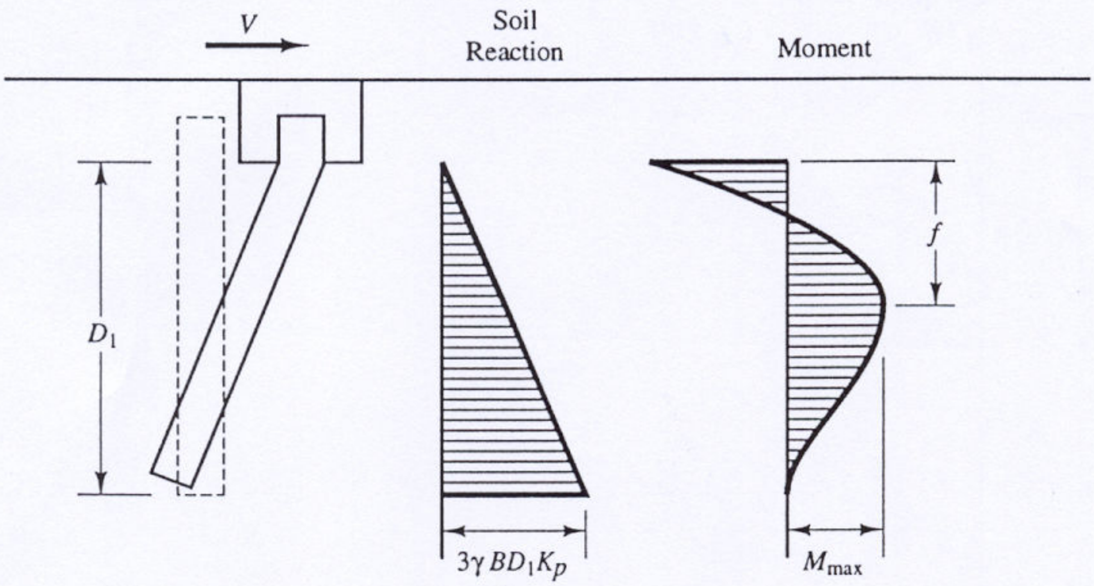


Figure 17.11 Soil reactions and foundation moments for restrained-head foundations in cohesionless soil (Adapted from Broms, 1964b; Used with permission of ASCE).

ii) Ultimate structural capacity

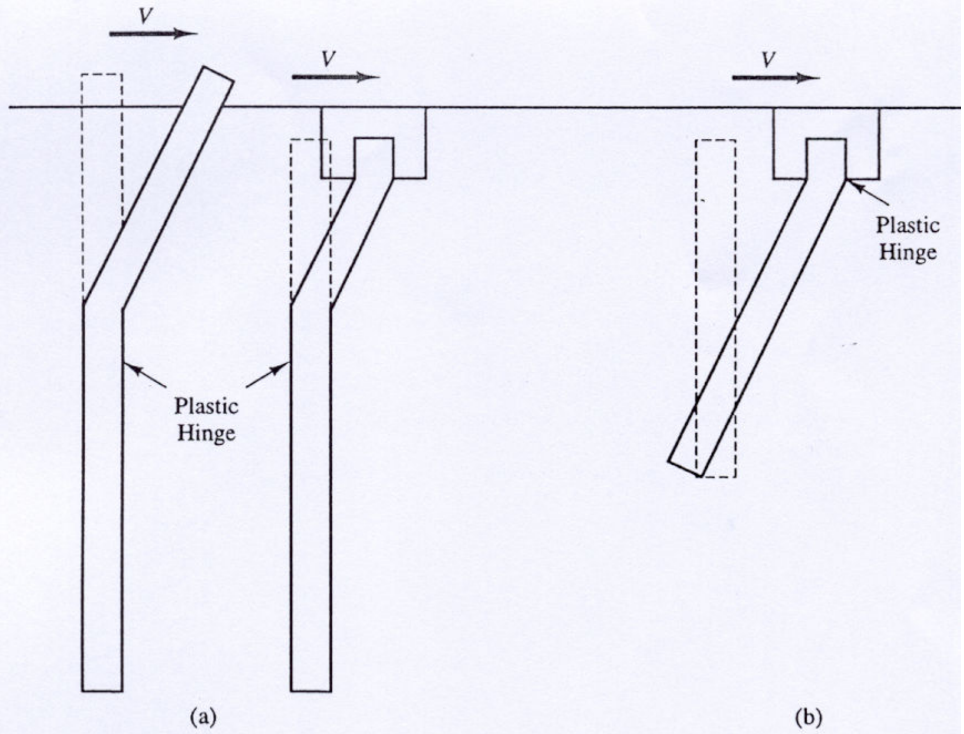


Figure 17.12 (a) Structural failure occurs in free-head and relatively long restrained-head foundations when a plastic hinge forms at some depth below the ground surface; (b) Shorter restrained-head foundations fail when a plastic hinge forms immediately below the pile cap (Adapted from Broms, 1965; Used with permission of ASCE).

⇒ Brom's method

- Computing M_{max} using the ultimate soil resistance distribution

a) <u>Cohesive soils</u>

- Free head piles

$$M_{\max} = V(e + 1.5B + 0.5f)$$

- Restrained head piles

(Computed the moment immediately below the pile cap and at a depth of $1.5B + f$)

$$M_1 = 9Bs_u f(1.5B + 0.5f) - 2.25Bs_u g^2 \geq 0$$

$$g = D_{\min} - 1.5B - f$$

$$M_2 = \frac{1}{2}V(1.5B + 0.5f)$$

b) <u>Cohesionless soils</u>

- Free head piles

$$M_{max} = V(e + 0.67f) \quad \Leftarrow \quad f = 0.82\sqrt{\frac{FV}{B\gamma K_p}}$$

- Restrained head piles

$$M_1 = VD - 0.5\gamma BD^3 K_p \geq 0$$

$$M_2 = 0.67VD$$

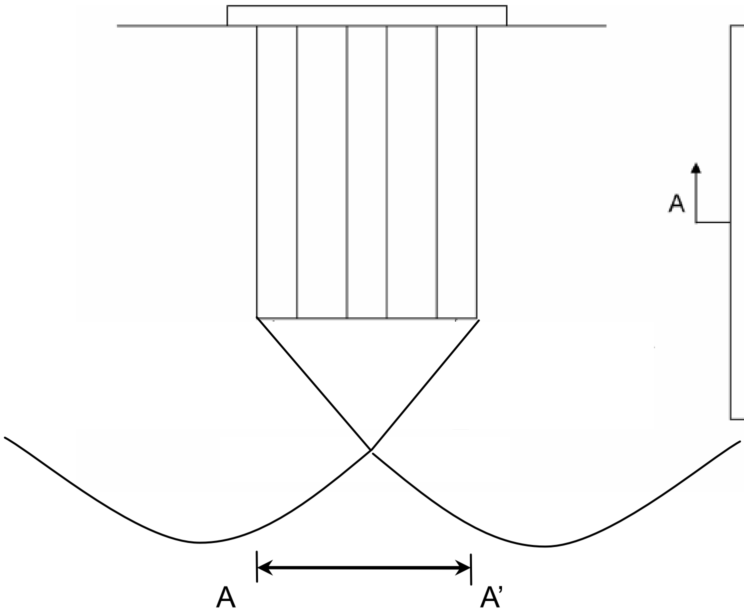
M_1 : Just below the pile cap
M_2 : At a depth "f" below the ground surface

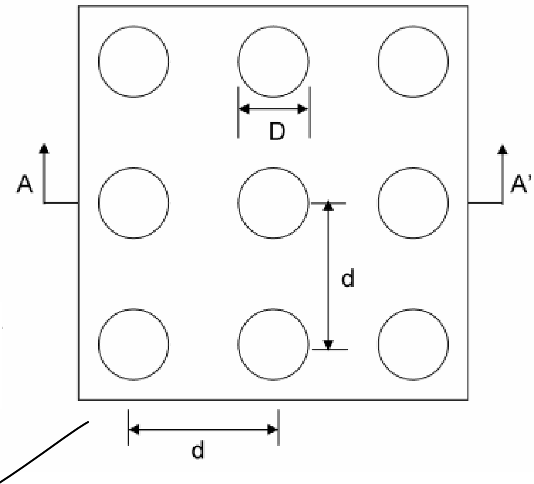
11) Group Piles

i) Bearing capacity of group piles

- <u>Differences in behavior between group piles & single pile</u>

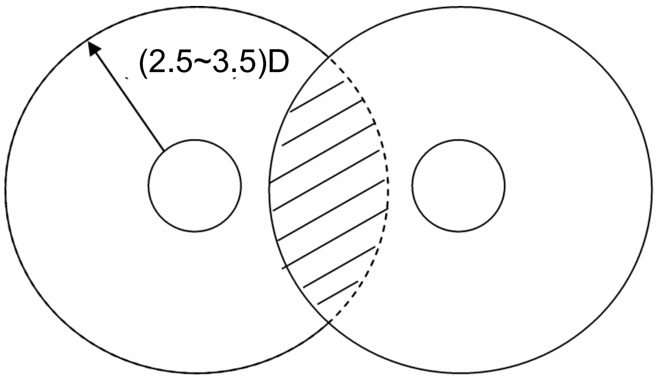
① Block failure potential *(Ideally but not practically)*

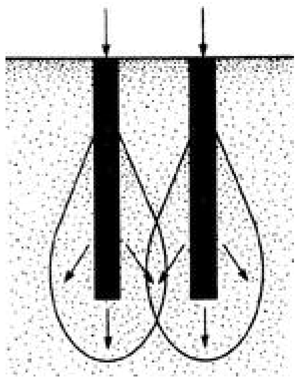




② Overlapping of displacement or stress fields of soils adjacent to piles

⇒ This may reduce or increase the load-bearing capacity of piles





③ The overlapping effect of pile installation (In particular, driven piles)

⇒

⇒

- Pile spacing in group piles
  - Practically : $d \geq 2.5D$

    Optimal spacing : $3.0D \leq d \leq 3.5D$

- The group efficiency $(\eta = Q_{u(group)} / \sum Q_{individual})$ depends on several factors;

  - The number, length, diameter, arrangement and spacing of the piles.
  - The load transfer mode (skin friction vs. end bearing).
  - The construction methods used to install the piles.
  - The sequence of installation of the piles.
  - The soil type.
  - The elapsed time since the piles were driven.
  - The interaction, if any, between the pile cap and soil.
  - The direction of the applied load.

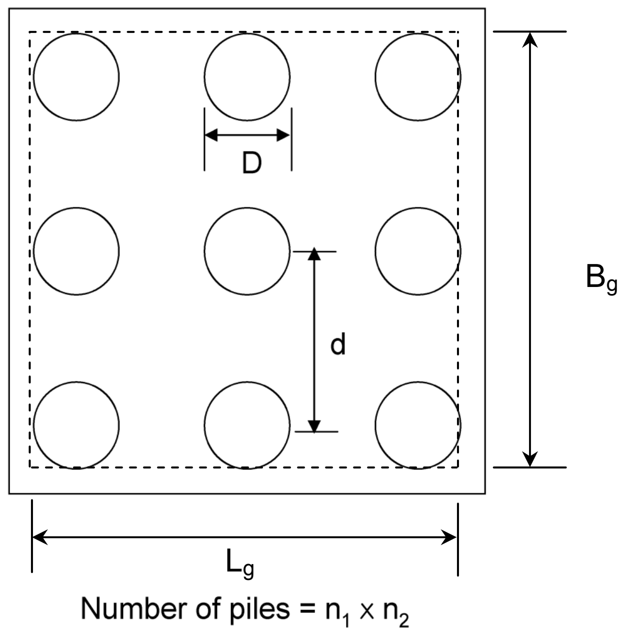
● Bearing Capacity of Pile Groups

→ Conservative (based on $Q_{group} \le \sum Q_{individual}$)

a) True end bearing pile

$$Q_{group} = n_1 n_2 Q_{individual\ pile}$$

(No interaction between piles)



b) All other cases

① Point bearing capacity

$$Q_{p(group)} = n_1 n_2 Q_{p(individual)}$$

② Frictional resistance

$$Q_{s(group)} = f_{av} P_{gruop} L \quad \text{……………..(1)}$$

$$P_{group} = 2(L_g + B_g)$$

or

$$Q_{s(group)} = n_1 n_2 f_{av} pL \quad \text{…………(2)}$$

⇒ Take minimum from (1) and (2).

- Alternative way to get $Q_{u(group)}$ for friction piles

$$\eta = \frac{Q_{u(group)}}{\sum Q_{u(individual)}} \le 1.0$$

▼ **TABLE 9.13** Equations for Group Efficiency of Friction Piles

Name	Equation
Converse-Labarre equation	$\eta = 1 - \left[\frac{(n_1 - 1)\,n_2 + (n_2 - 1)\,n_1}{90 n_1 n_2}\right]\theta$ where θ (deg) $= \tan^{-1}(D/d)$
Los Angeles Group Action equation	$\eta = 1 - \frac{D}{\pi d n_1 n_2}[n_1(n_2 - 1)]$ $+ n_2(n_1 - 1) + \sqrt{2}(n_1 - 1)(n_2 - 1)]$
Seiler-Keeney equation (Seiler and Keeney, 1944)	$\eta = \left\{1 - \left[\frac{11d}{7(d^2 - 1)}\right]\left[\frac{n_1 + n_2 - 2}{n_1 + n_2 - 1}\right]\right\} + \frac{0.3}{n_1 + n_2}$ where d is in ft

Notes

1) Clay : a)

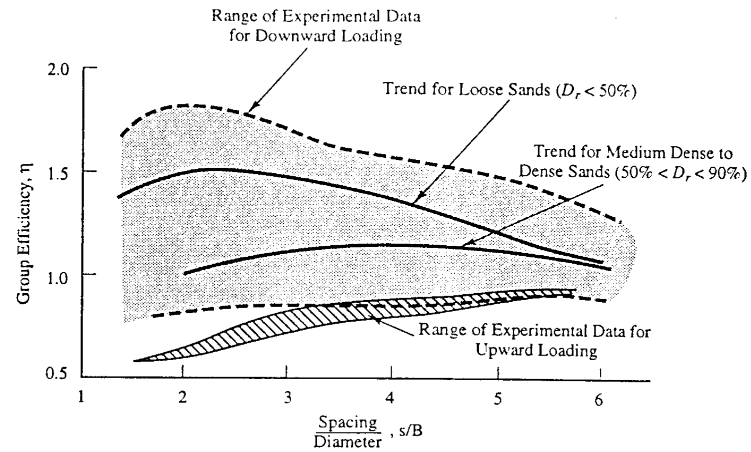
b)

c)

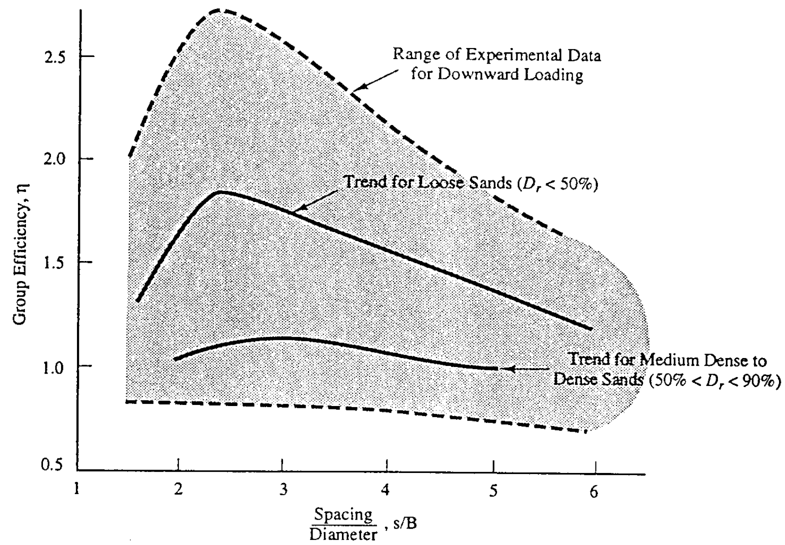
2) Sand :

3) Pile cap resting on soil contributes to the load bearing capacity, but its effect is neglected for design purpose. → (Fig 11.46)

4) Conclusively, the method to determine bearing capacity of pile groups is not well defined and conservative design rule is employed (group efficiency ≤ 1.0).



(a)



(b)

Figure 14.4 Group efficiencies from tests of model pile groups in cohesionless soils subjected to vertical loads: (a) groups of 4 piles; (b) groups of 9 - 16 piles (Adapted from O'Neill, 1983; Used with permission of ASCE).

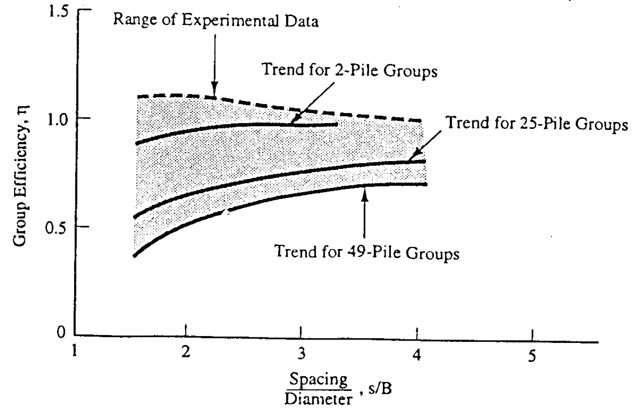


Figure 14.6 Group efficiencies from model pile groups in cohesive soils subjected to vertical loads (Adapted from O'Neill, 1983; Used with permission of ASCE).

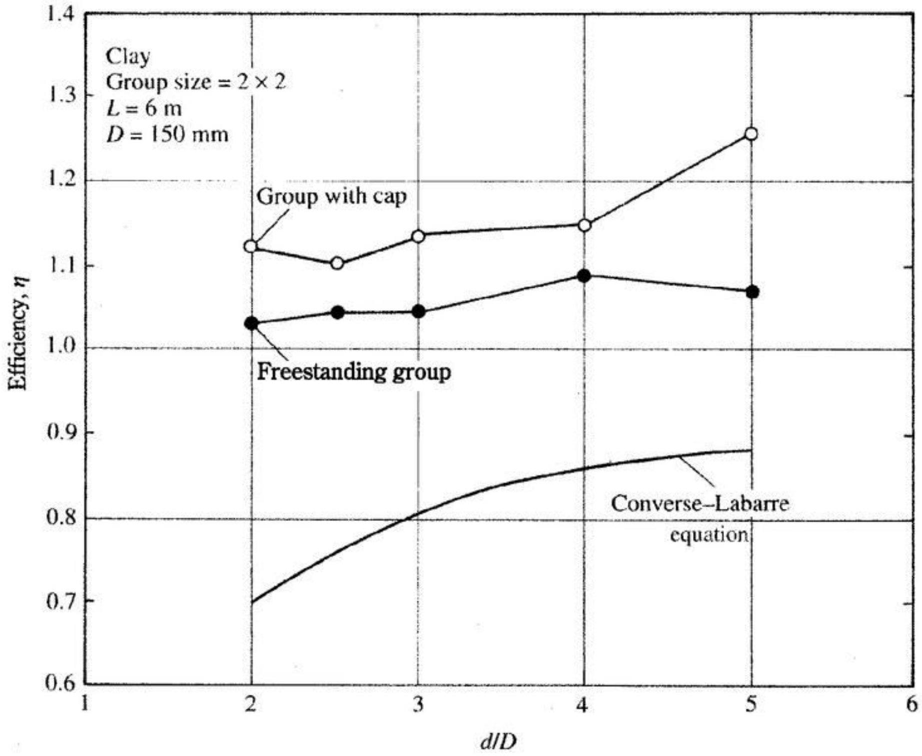


Fig 11.46 Variation of group efficiency with d/D (after Brand et al., 1972)

For pile design purpose, following comments can be drawn.

1. For driven group piles in sand with $d \geq 3D$, $Q_{g(u)}$ may be taken to be ΣQ_u , which includes the frictional and the point bearing capacities of individual piles.

2. For bored group piles in sand at conventional spacings $(d \approx 3D)$, $Q_{g(u)}$ may be taken (2/3~3/4) times ΣQ_u (frictional and point bearing capacities of individual piles).

* <u>Piles in rock</u>

Point bearing pile $\Rightarrow$ $Q_{group} = \sum Q_{individual}$ in case of

center to center spacing $> D + 300mm$