Raft Foundation on Clay/Plastic silt

Chapter 18

Book: Foundation Engineering – Peck, Hanson, Thornburn, 2nd edn.

CHAPTER 10

Footing and Raft Foundations

10.1. Types of Footings

A footing is an enlargement of the base of a column or wall for the purpose of transmitting the load to the subsoil at a pressure suited to the properties of the soil. A footing that supports a single column is known as an *individual column footing*, an *isolated footing* or a *spread footing*. The footing beneath a wall is known as a *wall footing* or a *continuous footing*. If a footing supports several columns, it is called a *combined footing*. A particular form of combined footing commonly used if one of the columns supports an exterior wall is a *cantilever footing*. The various types are illustrated in Fig. 10.1.

10.2. Historical Development

Footings undoubtedly represent the oldest form of foundation. Until the middle of the nineteenth century, most footings consisted of masonry. If they were constructed of stone cut and dressed to specific sizes, they were known as *dimension-stone footings*. In contrast, *rubble-stone footings* were constructed of pieces of random size joined by mortar. Masonry footings were adequate for most structures until the development of tall buildings with heavy column loads. Such loads required large and heavy footings that occupied valuable basement space.

In the earliest attempts to enlarge the areas of footings without increasing weight, timber grillages were constructed and conventional masonry footings built on them. In 1891 a grillage consisting of steel railroad rails embedded in concrete was devised as an improvement over the timber grillage (John Wellborn Root, Montauk Block, Chicago). The rail grillage was an important forward step because it saved much weight and increased space in the basement. Within the following decade, railroad rails were superseded by steel I beams that occupied slightly more space but that were appreciably more economical of steel. Typical grillage foundations of timber, railroad rails, and steel I beams are shown in Fig. 10.2.

The steel I beam proved admirably suited for the construction of cantilever footings. These were introduced in 1887 almost simultaneously in two buildings in Chicago. One of these early footings is illustrated in Fig. 10.3.

With the advent of reinforced concrete shortly after 1900, grillage footings were almost entirely superseded by reinforced-concrete footings, which are still the dominant type.

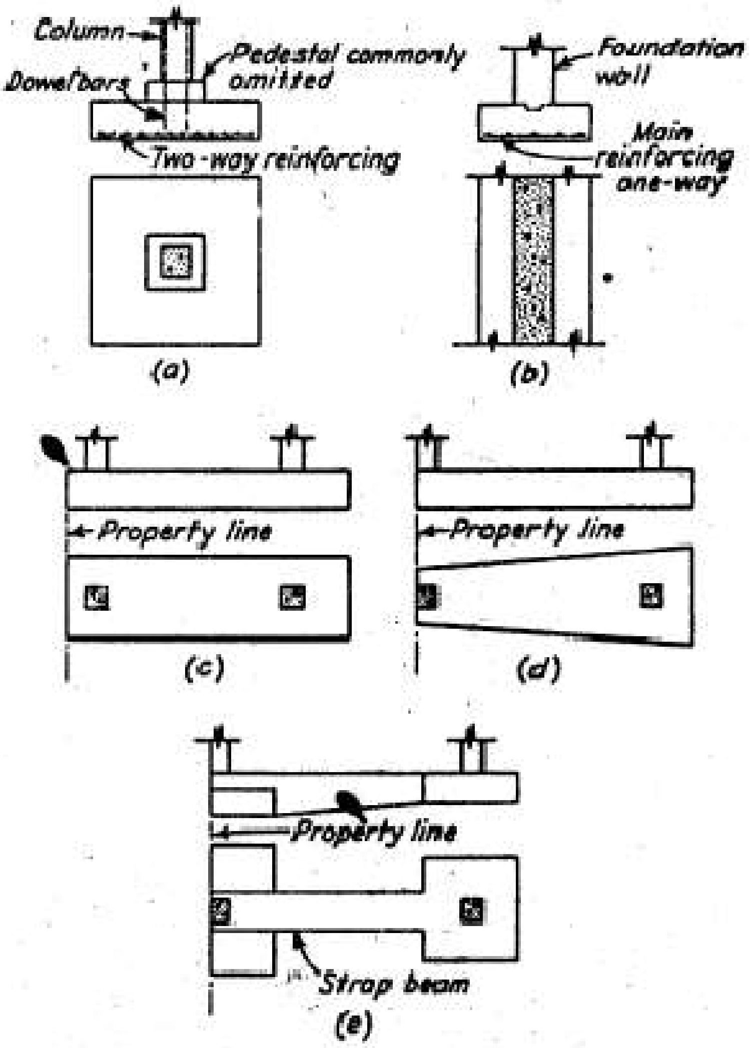


FIGURE 10.1. Types of footings. (*a*) Individual column footing. (*b*) Wall footing. (*c*) and (*d*) Combined footings. (*e*) Cantilever footing.

10.3. General Considerations

In temperate latitudes footings are commonly located at a depth not less than that of normal frost penetration. In warmer climates, and especially in semiarid regions, the minimum depth of footings may be governed by the greatest depth at which seasonal changes in moisture cause appreciable shrinkage and swelling of the soil.

The elevation at which a footing is established depends on the character of the subsoil, the load to be supported, and the cost of the foundation. Ordinarily the footing is located at the highest level where adequate supporting material may be found. In some instances, if an especially firm layer is encountered at greater depth, it may be more economical to establish the footing at a lower elevation because the area required for the footing is smaller.

The excavation for a reinforced-concrete footing should be kept dry so that the reinforcement can be set and held in its proper position while the concrete is being placed. To do this in waterbearing soil it may be necessary to pump either from sumps or from a previously installed system of drains. Forms are usually required around the sides of the footing. The necessity for pumping and for supporting the sides of the excavations in which the footings are placed may add appreciably to the cost of a footing foundation.

10.4. Allowable Soil Pressures

In the earliest days of foundation engineering the area of a footing was selected

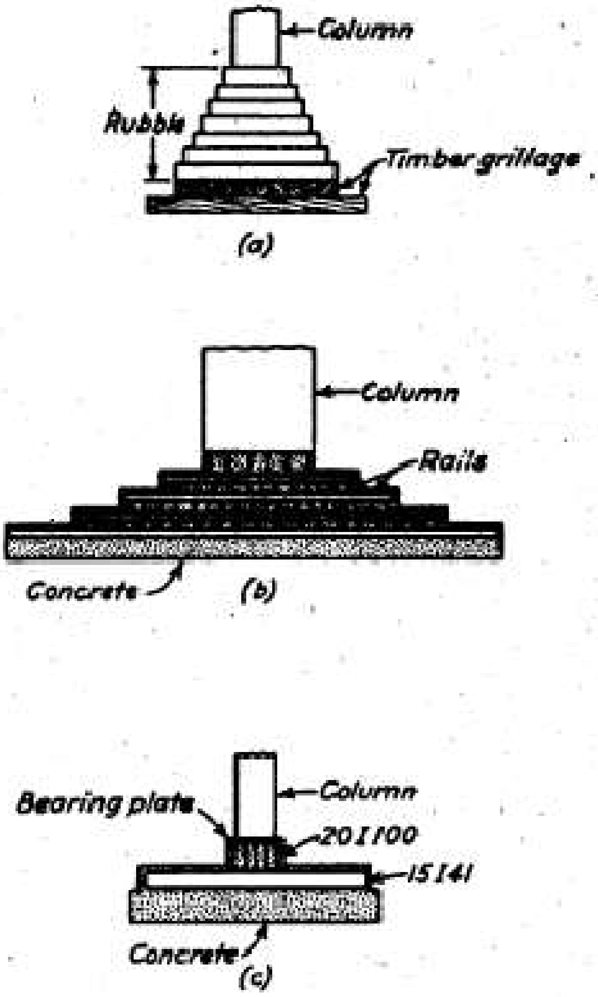


FIGURE 10.2. Historical development of grillage foundations of (*a*) timber, (*b*) railroad rails, (*c*) steel I-beams.

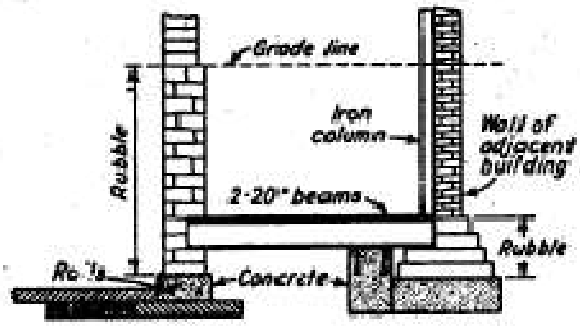


FIGURE 10.3. Cantilever footing supporting exterior column of Auditorium Building, Chicago, 1887.

according to the judgment of the engineer on the basis of his experience. In most localities simple rules of thumb developed. For example, in some parts of the United States the width of a continuous footing in feet was made equal to the number of stories in the structure. No attempt was made to provide larger footings for the support of heavier loads.

In the early 1870s the proportioning of footings was placed on a more rational basis. Progressive engineers of that day recommended that the areas of footings on a given site be made proportional to the loads on the footings and that the center of gravity of the load on each footing be made to coincide with the centroid of the footing. It was believed that the settlements of all footings beneath a structure would be equal and that no footing would tilt if these recommendations were conscientiously followed. Furthermore, it was believed that for each soil there existed a specific pressure under which the settlements of the various footings would not exceed reasonable values. This pressure, known as the *allowable soil pressure*, was generally specified in the building code or ordinances of the city in which the construction was to be located.

With the development of soil mechanics, it became evident that the safety or the settlement of a footing depended on many factors besides the pressure exerted on the subsoil. Nevertheless, the concept of an allowable soil pressure is so convenient that it has been retained in modern foundation engineering, but with modifications and limitations dictated by the improved state of our knowledge. These modifications and limitations constitute a large part of the information contained in Part C.

10.5. Combined Footings

If the loads from several columns are transmitted to the same footing, the footing should be proportioned so that its centroid coincides with the center of gravity of the column loads under normal conditions and so that the maximum pressure beneath the footing does not exceed the safe soil pressure under the most severe loading. Combined footings are customarily used along the walls of buildings at property lines where the footing for a wall column cannot extend outside the limits of the structure, Figs. 10.1*c*, 10.1*d*, and 10.1*e*. Under these circumstances, the wall footing is usually combined with an interior footing by one of the three methods shown.

10.6. Raft Foundations

A *raft* or *mat* foundation is a combined footing that covers the entire area beneath a structure and supports all the walls and columns. Wherever the building loads are so heavy or the allowable soil pressure so small that individual footings would cover more than about half the building area, a raft foundation is likely to be more economical than footings.

Ordinarily, rafts are designed as reinforced-concrete flat slabs. The downward loads on the raft are the loads from the individual columns or walls. If the center of gravity of the loads coincides with the centroid of the raft, the upward load is regarded as a uniform pressure equal to the sum of the downward loads divided by the area of the raft. The weight of the raft is not considered in the structural design because it is assumed to be carried directly by the subsoil. Since his method of analysis does not take into account the moments and shears caused by differential settlement, it is customary to reinforce the raft more heavily than required according to the analysis.

Raft foundations are also used to reduce the settlement of structures located above highly compressible deposits. Under these conditions, the depth at which the raft is established is sometimes made so great that the weight of the structure plus that of the raft is wholly compensated by the weight of the excavated soil. The settlement of the structure is then likely to be insignificant. Where complete compensation is impracticable, a shallower raft may be acceptable if the net increase in load is small enough to lead to tolerable settlements.

If the column loads are not more or less uniformly distributed or if the subsoil is such that large differential settlements would tend to develop, large rafts must be stiffened to prevent excessive deformations. This stiffening has been accomplished by using partitions as the stems of T beams connected to the raft (Fig. 10.4*a*), by constructing a cellular or rigid-frame foundation (Fig. 10.4*b*), or in some instances by utilizing the stiffness of a reinforced-concrete superstructure. The larger the foundation, the more expensive these procedures become; often pile or pier foundations are preferable.

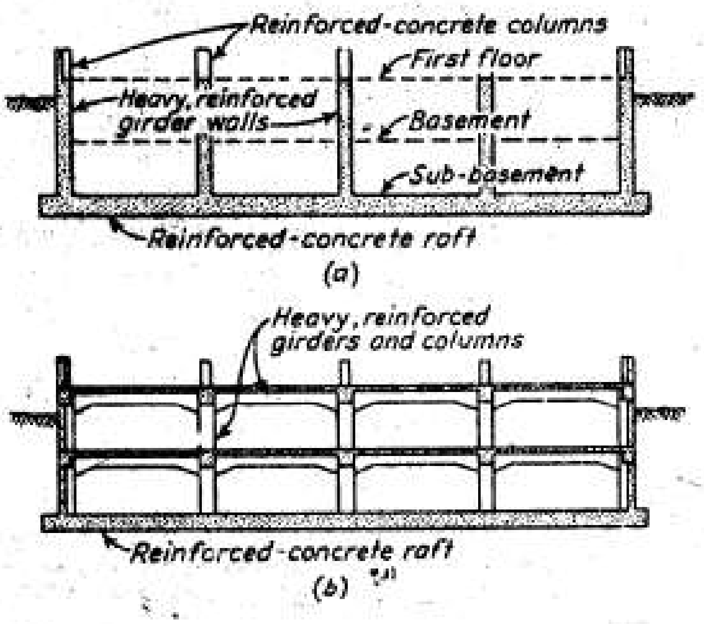


FIGURE 10.4. Methods of stiffening large raft foundations. (*a*) Use of ribs or walls as T beams. (*b*) Rigid-frame construction.

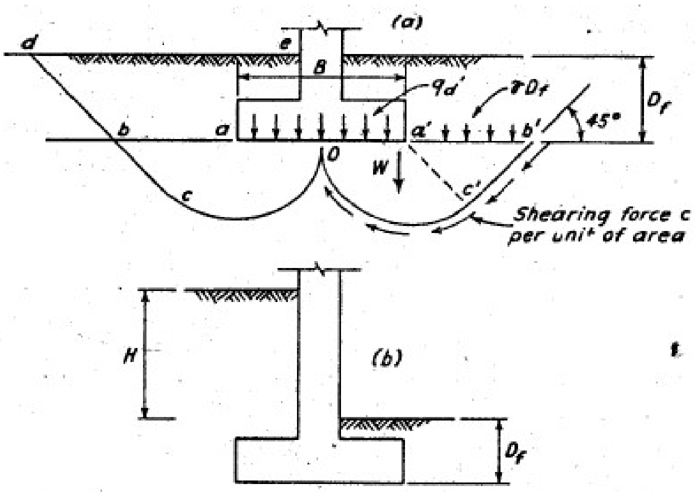


FIGURE 18.1. (*a*) Cross section through long footing on clay, showing basis for computation of ultimate bearing capacity. (*b*) Section showing D_f for footing with surcharge of different depth on each side.

18.2. Footings on Clay

The gross ultimate bearing capacity, q_d' of a long footing of width B can be obtained by considering the equilibrium of the soil wedge oc'a'b' as shown in Fig.18.1. From this analysis it is found that

$$q_d' = cN_c + \gamma D_f \qquad 18.1$$

The *net ultimate bearing capacity* q_d is defined as the pressure that can be supported at the base of the footing in excess of that at the same level due to the surrounding surcharge; hence

$$q_d = q_d' - \gamma D_f$$

and

$$q_d = cN_c \qquad 18.2$$

Safe Soil Pressure. Under dead load plus the maximum live loads that can normally be expected, the factor of safety against a bearing-capacity failure should be on the order of 3. The allowable soil pressure q_a (Art. 10.4) may, therefore, be taken as one third the net ultimate soil pressure (eq. 18.2)

$$q_a = \frac{cN_c}{3} \qquad 18.3$$

or

$$q_a = \frac{q_u N_c}{6} \qquad 18.4$$

For a given footing we can determine q_{net} and then find the factor of safety, F

$$q_{net} = \frac{Q}{A} - \gamma D_f = q_b - \gamma D_f \qquad F = \frac{cN_c}{q_b - \gamma D_f}$$

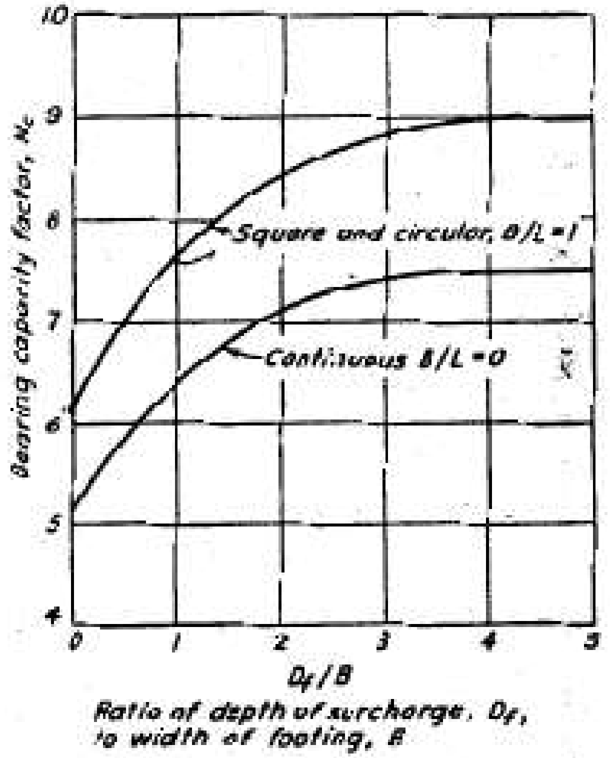


FIGURE 18.2. Bearing capacity factors for foundations on clay under $\phi = 0$ conditions (after Skempton, 1951).

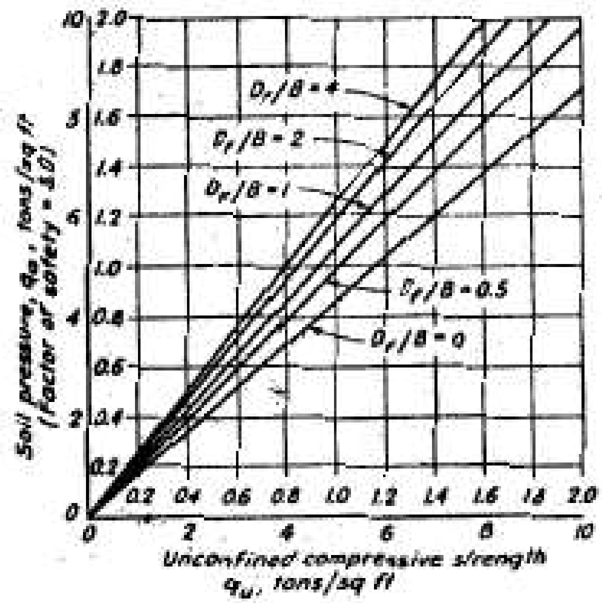


FIGURE 18.3. Net allowable soil pressure for footings on clay and plastic silt, determined for a factor of safety of 3 against bearing capacity failure ($\phi = 0$ conditions). Chart values are for continuous footings ($B/L = 0$); for rectangular footings, multiply values by $1 + 0.2\ B/L$; for square and circular footings, multiply values by 1.2.

18.3 Rafts on Clay

Ultimate Bearing Capacity. The net ultimate pressure that can be sustained by the soil at the base of a raft on a deep deposit of clay or plastic silt may be obtained in the same manner as for footings on clay (Art. 18.2). The quantity q_a in eq. 18.2 is the pressure at the elevation of the base of the raft in excess of that exerted by the surrounding surcharge. Likewise, in eq. 18.4 and in Fig. 18.3, q_a is a net soil pressure. By increasing the depth of excavation, the pressure that can safely be exerted by the building is correspondingly increased. This can be accomplished by increasing the number or depth of basements. On the other hand, the area of a raft cannot usually be enlarged appreciably in an attempt to reduce the soil pressure because it is not feasible to extend a raft more than a few feet beyond the building proper. Therefore, if a raft foundation is to be constructed at a site underlain by clay too soft to provide support at the normal basement level, the only practical method to provide the required factor of safety is to lower the elevation of the raft.

Safe Soil Pressure for Rafts on Clay. In proportioning footings on clay (Art. 18.2), the net ultimate bearing capacity is divided by the factor of safety to obtain a net allowable soil pressure. For a factor of safety equal to 3, this procedure results in eqs. 18.3 and 18.4 and in Fig. 18.3. The same principles are applicable to rafts on clay. Accordingly, the factor of safety, in terms of net soil pressures, may be expressed as

$$F = \frac{cN_c}{q_b - \gamma D_f} \qquad 18.5$$

where q_b is the *gross soil pressure* or *contact pressure* produced at the base of the raft by the weight of the building and live load, and which in the denominator of eq. 18.5 is reduced to a net pressure by subtracting the weight of the surrounding surcharge, γD_f.

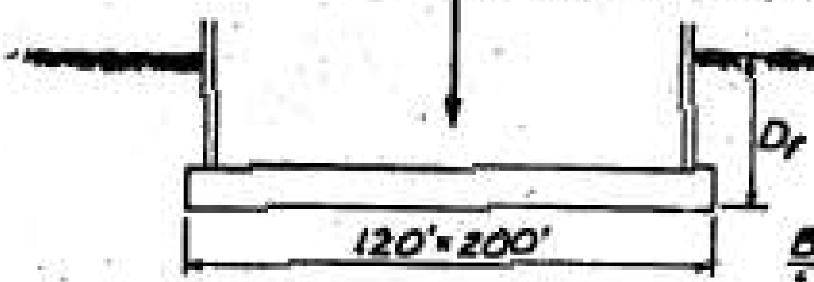
According to eq. 18.5, the factor of safety is very large for rafts established at such depths that γD_f is nearly equal to q_b. When these terms are equal the raft is said to be a *fully compensated foundation*. The theoretical factor of safety against failure of the subsoil under these circumstances is infinite, provided no uncertainty is involved in the estimate of loads or in the action of γD_f. However, even if γD_f is fully effective, an increase in the gross soil pressure q_b, possibly caused by unanticipated additional loads on the raft, reduces the degree of compensation; the decrease in the factor of safety, moreover, is out of proportion to the increase in loads. Equation 18.5 also shows that an error in estimating the weight of the structure or the live load has a greater influence on the factor of safety for a weak clay than for a strong one.

As for footings on clay (Art. 18.2), the factor of safety against failure of the soil beneath a raft on clay should not be less than 3 under normal loads, or less than 2 under the most extreme loads. Therefore, Fig. 18.3 may be used to obtain the allowable net soil pressure for rafts on clay. The values from Fig. 18.3 may be multiplied by appropriate ratios to convert the pressures to those corresponding to factors of safety other than 3. Since N_c and, consequently, the allowable soil pressure are somewhat influenced by the depth of surcharge, the determination of D_f for the partial compensation required to attain a desired factor of safety is, strictly speaking, a trial procedure. However, the first trial based on an assumed D_f/B is ordinarily sufficiently accurate.

DP 18-2
Raft on Clay
Sh. 1 of 3

Raft Design, General Data.

DL = 35,000#
Normal LL = 21,600#



$\gamma = 115$ pcf

$\frac{B}{L} = \frac{120}{200} = 0.6$

Deep soft clay, average $q_u = 0.3$ Tsf

$$q_b = \frac{35,000 + 21,600}{120 \times 200} = \frac{56,600}{24,000} = 2.36 \text{ Ksf}$$

Determine D_f for full compensation

$\gamma D_f = q_b$ or $115 D_f = 2360$

$$D_f = \frac{2360}{115} = \underline{20.5' \text{ for full compensation}}$$

Determine D_f for F=3 (partial compensation)

$\gamma D_f = q_b - q_a$ Since F=3, Use Fig. 18.3

Assume $\frac{D_f}{B} = 0.1$

For full compensation
$D_f/B = 20.5/120 = 0.17$

$$1 + 0.2\frac{B}{L} = 1 + 0.2\left(\frac{120}{200}\right) = 1.12$$

$q_a = 0.26 \times 1.12 = 0.29$ Tsf $= 580$ psf

1 Ton = 2000 lb

$$D_f = \frac{2360 - 580}{115} = \frac{1780}{115} = \underline{15.5' \text{ for } F = 3}$$

$$F = \frac{cN_c}{q_b - \gamma d_f} \rightarrow q_b - \gamma d_f = \frac{cN_c}{F} = q_a \rightarrow d_f = \frac{q_b - q_a}{\gamma}$$

DP 18-2
Raft on Clay
Sh. 2 of 3

Raft Design.

Determine Factor of Safety (D_f = 15.5')

(a) If q_b Increases 25% ($2360 \times 1.25 = 2950$ psf)

Use eq. 18.5 $F = \frac{3 \times 580}{2950 - 1780} = \frac{1740}{1170} = 1.49$

Fs =3,
q_a =580 kPa

For d_f =15.5 ft
γd_f =1780 psf

(b) If q_b Increases 50% ($2360 \times 1.5 = 3540$ psf)

$$F = \frac{1740}{3540 - 1780} = \frac{1740}{1760} = 0.99$$

Determine D_f for Factor of Safety = 1.0

Assume no change in q_b = 2360 psf

Use eq. 18.5 $1.0 = \frac{1740}{2360 - 115 D_f}$

$$D_f = \frac{2360 - 1740}{115} = 5.4'$$

Determine Factor of Safety (D_f = 15.5 & q_b = 2360 psf)

If q_u decreases to 0.25 Tsf

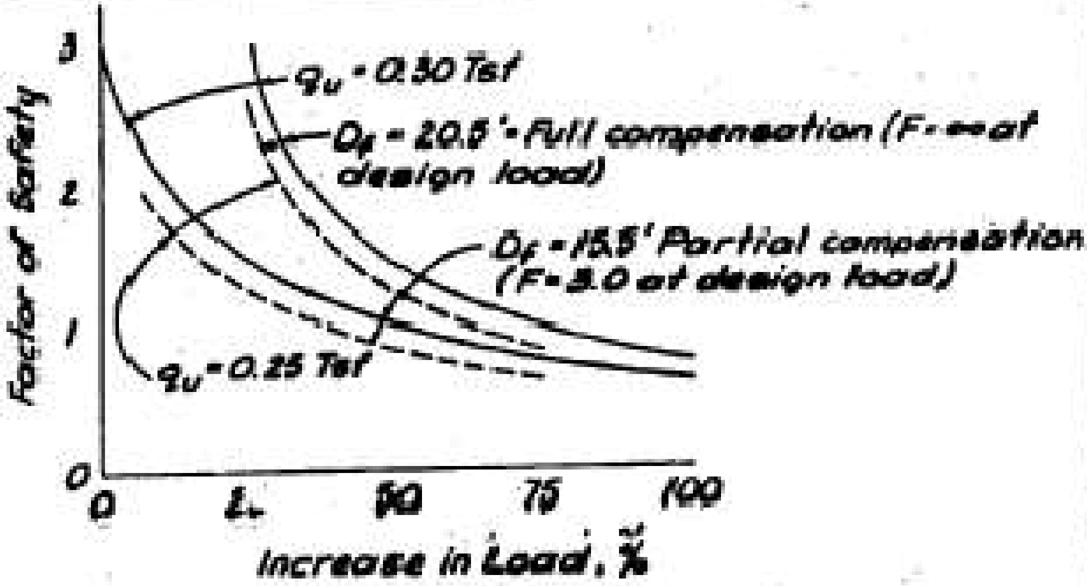
Fig. 18.3 q_a = 493 psf

Use eq. 18.5 $F = \frac{3 \times 493}{2360 - 1780} = \frac{1479}{580} = 2.55$

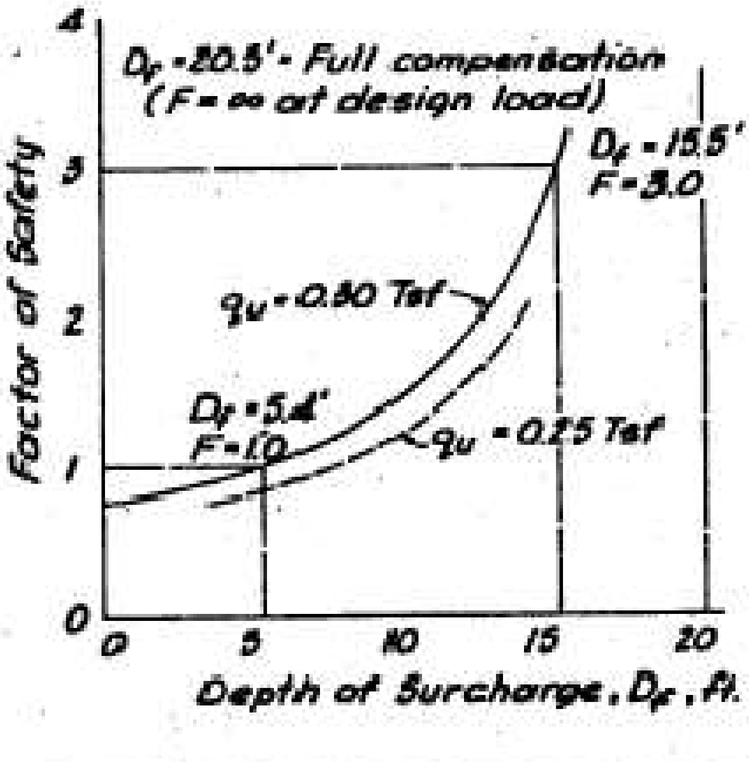
Check by direct proportion, $F = 3.0 \times \frac{0.25}{0.30} = 2.50$

Raft Design.

Effect of Increase of Load on Factor of Safety



Effect of Depth of Raft on Factor of Safety



ILLUSTRATIVE DESIGN. DP 18-2. RAFT ON CLAY

Various principles regarding full and partial compensation and the relationships to the factor of safety of rafts on clay are illustrated in DP 18-2. The combination of loads chosen for design is the weight of the building plus normal live load. For this combination, a depth D_f of 20.5 ft is required for full compensation, whereas a depth of 15.5 ft provides a factor of safety of 3.

The computations in DP 18-2 show that, for a depth of 15.5 ft, an unforeseen increase in loads of only 25 per cent would reduce the factor of safety of the foundation approximately 50 per cent. At the same depth of foundation, the factor of safety would be further disproportionately reduced to slightly less than unity if the loads were increased 50 per cent.

The importance of the surrounding surcharge D_f is also illustrated in DP 18-2. It is seen that the factor of safety decreases to unity when the surcharge is reduced from 15.5 to 5.4 ft. If D_f were reduced to zero, the factor of safety for the design loads would equal 0.74, and failure would occur.

FS =1740/2360 = 0.74
q_{ult} 3*580 =1740 psf
q_b =2360 psf

For any change in the strength of the clay, all values of the factor of safety computed in DP 18-2 change proportionately. Such changes are illustrated by the curves on Sh. 3 for different values of q_u.

Raft Foundation on Sand

Chapter 19

Foundation Engineering, Peck, Hanson, Thornburn; 2nd edn

Raft foundation is also a shallow foundation like a footing and let's first see the bearing capacity of a footing on sand!

the gross ultimate bearing capacity may be expressed as

$$q_d' = \tfrac{1}{2} B\gamma N_\gamma + \gamma D_f N_q \qquad 19.1$$

and the *net ultimate bearing capacity* as

$$q_d = q_d' - \gamma D_f$$
$$= \tfrac{1}{2} B\gamma N_\gamma + \gamma D_f (N_q - 1) \qquad 19.2$$

In these equations, N_γ and N_q are dimensionless *bearing-capacity factors* depending primarily on ϕ. They may be evaluated by means of the chart, Fig. 19.5.

Eqn 19.1 is obtained considering equilibrium of soil zone beneath the footing as shown in Fig.19.4

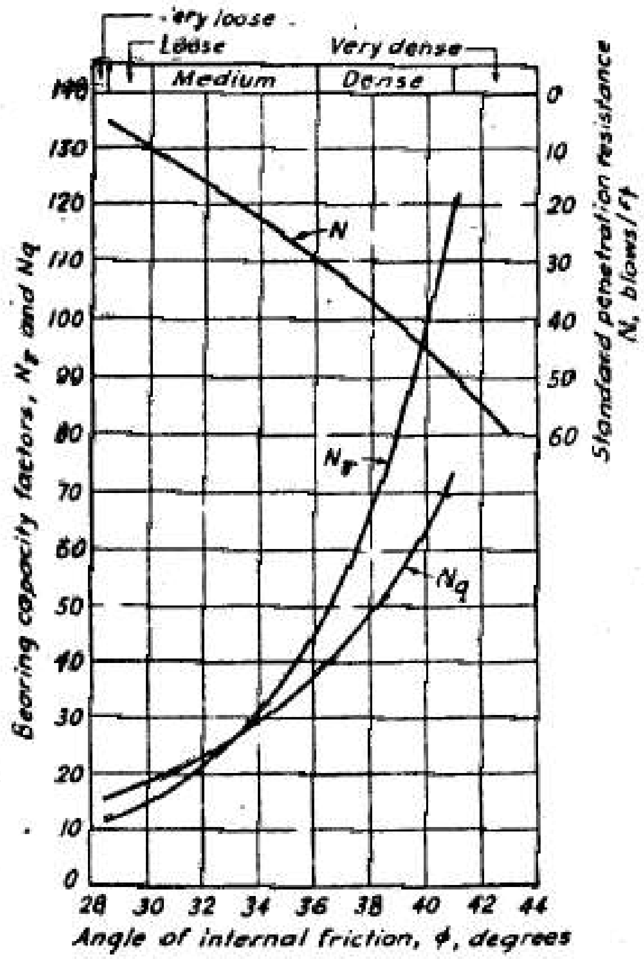


FIGURE 19.5. Curves showing the relationship between bearing-capacity factors and ϕ, as determined by theory, and rough empirical relationship between bearing capacity factors or ϕ and values of standard penetration resistance N.

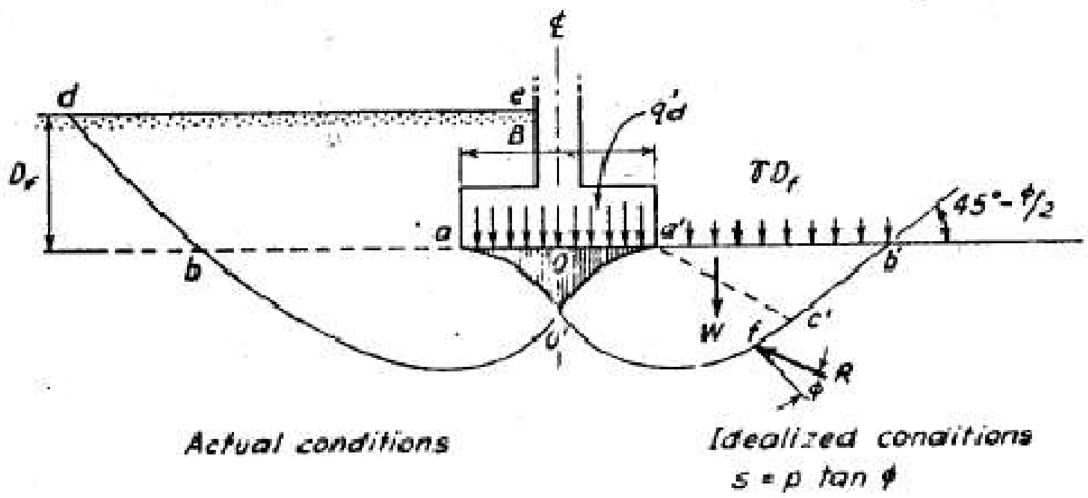


FIGURE 19.4. Cross section through long footing on sand showing (left side) pattern of displacements during bearing capacity failure, and (right side) idealized conditions assumed for analysis.

Equation 19.2 can be expressed in the form

$$q_d = \left[\frac{\gamma N_\gamma}{2} + \gamma(N_q - 1)\frac{D_f}{B}\right] B \quad 19.3a$$

and, for a given factor of safety F against a bearing-capacity failure,

$$q_a = \frac{q_d}{F} = \left[\frac{\gamma N_\gamma}{2} + \gamma(N_q - 1)\frac{D_f}{B}\right]\frac{B}{F} \quad 19.3b$$

For a particular value of D_f/B and a given deposit of sand, the expression within the brackets is a constant. Thus, the relation between the width of footing and the net soil pressure q_a for a given factor of safety can be expressed in a plot such as Fig. 19.3 as a family of straight lines radiating from the origin. Each line corresponds to a sand having a different N-value. The initial branches of the curves in Fig. 19.3 have been drawn to provide a factor of safety of 2. If the soil pressures indicated by these lines are not exceeded, runaway settlement of a footing is precluded.

The horizontal lines on the right side of the three parts of Fig. 19.3 corresponds to a particular N-value and indicates the soil pressure corresponding to a settlement of 1 in. The lines are drawn for the condition that the water table is at great depth. The necessary correction for other positions is considered later.

The width B in Fig. 19.3 may be taken as the side of a square footing, the smaller dimension of a rectangular footing, the width of a long continuous footing, or the diameter of a circular footing.

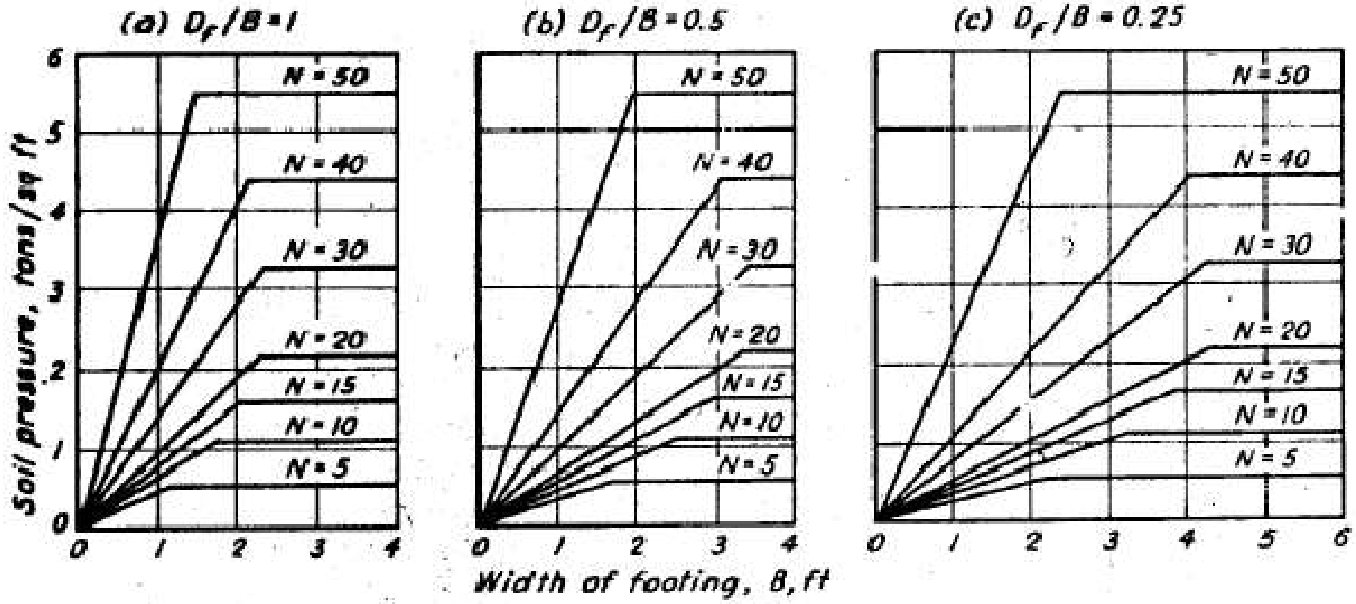


FIGURE 19.3. Design chart for proportioning shallow footings on sand.

Basis of the inclined and horizontal part in the design chat (Fig.9.3)

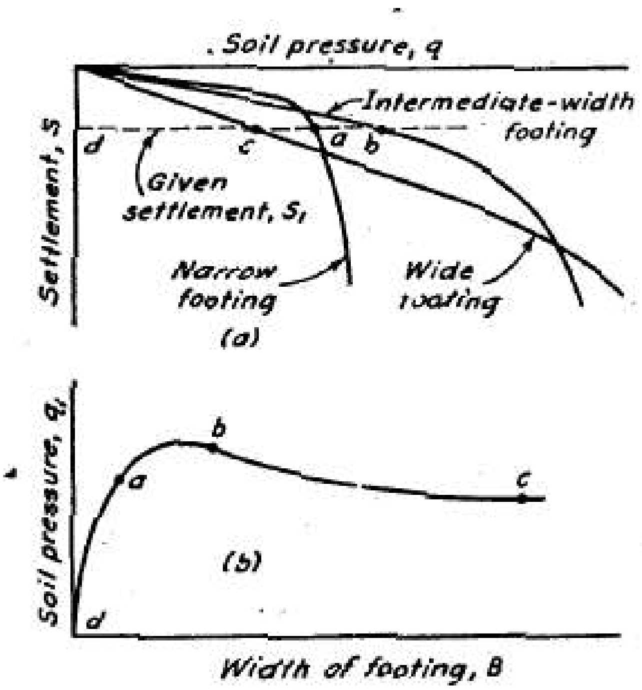


FIGURE 19.1. Relationships among soil pressure, width of footing, and settlements for footings of constant D_f/B ratio on sand of uniform relative density. (*a*) Load-settlement curves for footings of increasing widths B_a, B_b, and B_c. (*b*) Variation of soil pressure with width of footing for given settlement S_1.

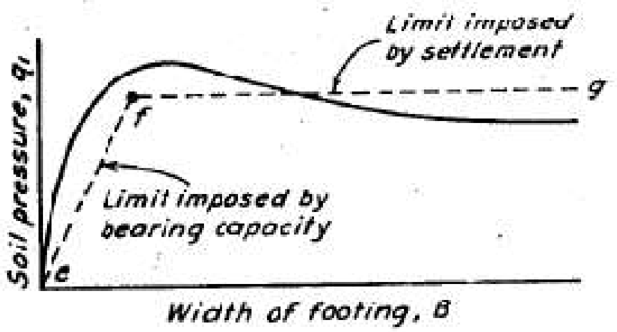


FIGURE 19.2. Actual relation (solid line) between soil pressure and width of footing on sand for given settlement S_1, and substitute relation (dashed lines) used as basis for design.

Corrections required for using Charts in Fig.9.3

The chart applies to shallow footings ($D_f \lesseqgtr B$) resting on a uniform sand for which γ = 100 lb/cu ft, and in which the water table is at too great a depth to influence the behavior of the footings. In view of the other approximations in the procedure, variations of γ from the assumed value of 100 lb/cu ft are inconsequential and may be neglected. On the other hand: (1) the *N*-values must sometimes be adjusted for the influence of the overburden pressure during the performance of the standard penetration test; (2) the variability of the deposit, as reflected in variation in the *N*-values from boring to boring, is usually appreciable and should be taken into account; and finally, (3) the influence of the water table, if shallow enough to affect the behavior of the footings, must be evaluated.

$$C_w = 0.5 + 0.5\frac{D_w}{D_f + B} \qquad 19.4$$

The correction factor for the presence of the water table is given by eq. 19.4.

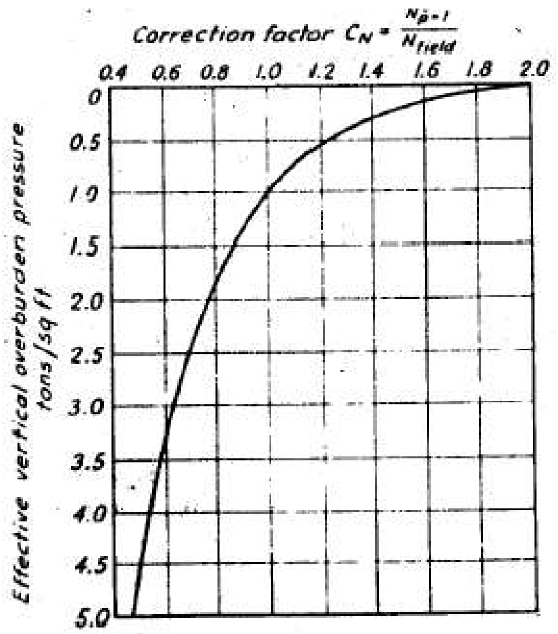


FIGURE 19.6. Chart for correction of *N*-values in sand for influence of overburden pressure (reference value of effective overburden pressure 1 ton/sq ft).

19.3. Rafts on Sand

Soil Pressure. Because of the large size of rafts compared to that of footings, the factor of safety against a bearing-capacity failure of the underlying sand is always very great. This can be seen from eq. 19.2. If the width of the raft is only 20 ft, the depth of foundation only 10 ft, and the number of blows equal to 10 or more, the ultimate bearing capacity on submerged sand exceeds 7 tons/sq ft. With increasing width of the raft or increasing relative density of the sand, the ultimate bearing capacity increases rapidly. Hence, the danger that a large raft may break into a sand foundation is too remote to require consideration.

On account of the large size of rafts, the stresses in the underlying sand are likely to be relatively high to considerable depth. Therefore, the influence of local loose pockets distributed at random throughout the sand is likely to be about the same beneath all parts of the raft, and the differential settlement is likely to be smaller than that of a footing foundation designed for the same soil pressure. Although it is not improbable that a single footing may rest entirely above a pocket of loose sand and experience large settlement, a loose pocket beneath part of a raft has a much smaller influence.

Because the differential settlements of a raft foundation are less than those of a footing foundation designed for the same soil pressure, it is reasonable to permit larger allowable soil pressures on raft foundations. Experience has shown that a pressure approximately twice as great as that allowed for individual footings may be used because it does not lead to detrimental differential settlements. For a soil pressure that produces a differential settlement of $\frac{3}{4}$ in., however, the maximum settlement of a raft may be about 2 in. instead of 1 in. as for a footing foundation.

The shape of the curve in Fig. 19.1*b* shows that the net soil pressure corresponding to a given settlement is practically independent of the width of the footing or raft when the width becomes large. The allowable net soil pressure for design may with sufficient accuracy be taken as twice the pressure indicated by the horizontal lines in Fig. 19.3. The corresponding relation between allowable net soil pressure and N is

$$q_a \text{ (tons/sq ft)} = 0.22N \quad (5 \leq N \leq 50) \qquad 19.5$$

The correction factor for the presence of the water table is given by eq. 19.4. For values of $N > 50$, the linear relation expressed by eq. 19.5 becomes somewhat unconservative. Moreover, N-values of this magnitude may be associated with the presence of gravel or boulders, or with cementation. Hence, they should be scrutinized carefully to permit judging whether the routine procedure described in this paragraph is applicable to the actual conditions.

The values of q_a from eq. 19.5, with appropriate corrections, serve as a rational basis for the design of a raft foundation on sand under most conditions encountered in the field. They may be increased somewhat if bedrock is encountered at a depth less than about one half the width of the raft.

If the average value of N after correction for the influence of overburden pressure is less than about 5, the sand is generally considered to be too loose for the successful use of a raft foundation. Either the sand should be compacted or else the foundation should be established on piles or piers.

The loads that should be considered in computing the gross soil pressure on the raft are the dead load of the structure including the raft, and the maximum live load that is really likely to be active. The surcharge due to the weight of the soil between the surrounding ground surface and the base level of the foundation is subtracted from the gross pressure to obtain the net soil pressure for comparison with the allowable soil pressure. That is, the net soil pressure at the base of the raft is

$$q_{net} = \frac{Q}{A} - \gamma D_f = q_b - \gamma D_f \qquad 19.6$$

where Q = total weight of structure plus live load

A = base area of raft

q_b = gross soil pressure or contact pressure at base of raft

A raft-supported building with a basement extending below water table is acted on by hydrostatic uplift or buoyancy equal to $\gamma_w(D_f - D_w)$ per unit of area. The beneficial effect of the buoyancy is automatically taken into account in calculating the net pressure, provided the total weight of the surcharge γD_f is used in eq. 19.6. In many instances, however, the settlement is governed by conditions during construction rather than by those prevailing after completion.

During construction of the substructure the water table is usually drawn down below the base of the raft. If it then rises to a higher level, the gross soil pressure is reduced by the uplift of the full head of water on the base. Simultaneously, the effective weight of the surcharge is reduced by the same amount. Hence, the actual net pressure is not influenced by the buoyancy. The allowable net pressure, however, is a function of the water-table correction.

If the soil pressure is chosen in accordance with the foregoing procedures and if the corrected value of N is not less than about 5, the differential settlements between adjacent columns on a raft foundation on sand will not exceed about $\frac{3}{4}$ in., provided the base of the raft is located at least 8 ft below the surrounding ground surface. Experience has shown that, if the surcharge is less than this amount, the edges of the raft settle appreciably more than the interior because of the lack of confinement of the sand.

ILLUSTRATIVE PROBLEM

A reinforced concrete structure 100 ft square is to be supported by a raft with its base 16 ft below the surrounding ground surface. The subsoil consists of sand to great depth. Five borings have been made at the site; the average *N*-values, corrected for the influence of the overburden pressure, are respectively 36, 30, 32, 35, and 33. The average unit weight of the sand is 114 lb/cu ft. While test-boring was in progress, the water level was at a depth of 5 ft. During construction the water level will be lowered to 20 ft, but upon completion of the structure the level will return to its original position. What total load, including the weight of raft, structure, and contents, may be supported at a settlement not to exceed 2 in.; that is, at a differential settlement not to exceed $\frac{3}{4}$ in.?

Solution. According to eq. 19.5, the allowable net soil pressure for a value of $N = 30$ would be

$$q_a = 0.22 \times 30 = 6.6 \text{ tons/sq ft}$$

if the water table were at great depth. The water table correction is

$$\begin{aligned} C_w &= 0.5 + 0.5 \frac{D_w}{D_f + B} \\ &= 0.5 + 0.5 \times \frac{5}{16 + 100} \\ &= 0.5 + 0.02 \\ &= 0.52 \end{aligned}$$

Hence, the allowable net pressure is $6.6 \times 0.52 = 3.4$ tons/sq ft. The surcharge $\gamma D_f = 114 \times 16/2000 = 0.91$ ton/sq ft. The corresponding gross pressure or contact pressure that would lead to a 2-in. settlement is, therefore, $3.4 + 0.9 = 4.3$ tons/sq ft, and the total weight that can be supported is $4.3 \times 100^2 = 43{,}000$ tons.

The total weight that will produce a settlement of 2 in. should, of course, be independent of whether the calculations are based on total or effective stresses. In the preceding paragraph a total-stress calculation is illustrated, in accordance with eq. 19.6. On the basis of effective stresses, the surcharge is only

$$\begin{aligned} \gamma D_w + \gamma'(D_f - D_w) &= \frac{1}{2000}\Big[114 \times 5 \\ &\quad + (114 - 62.5)(16 - 5)\Big] \\ &= 0.57 \text{ ton/sq ft} \end{aligned}$$

This value is $0.91 - 0.57 = 0.34$ ton/sq ft less than that calculated on the basis of the total weight of the surcharge, and the net capacity of the raft is decreased by the same amount. At the same time, however, the raft is buoyed up by the hydrostatic uplift on its base equal to

$$\begin{aligned} \gamma_w(D_f - D_w) &= \frac{1}{2000} \times 62.5(16 - 5) \\ &= 0.34 \text{ ton/sq ft} \end{aligned}$$

and the downward load of the structure can be increased by the amount of the buoyancy. Hence, the total weight that can be supported remains 4.3 tons/sq ft.