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Revision Memo

- 12/02** Editorial Revision. Highlights of this revision are:
1. Included additional nomenclature to equations
 2. Added Metric Units
 3. Corrected editorial mistakes



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SCOPE

This subsection covers the structural analysis and design of foundations for process vessels and equipment. Machinery foundations and foundations for tanks are covered in other subsections of this Design Practice.

REFERENCES

GLOBAL PRACTICES

- GP 4-6-1 Reinforced Concrete Foundations, Anchor Bolts and Grout
- GP 4-7-1 Piles and Sheet Piling
- GP 4-1-1 Concrete Design and Construction

OTHER REFERENCES

- ACI 318, Building Code Requirements for Reinforced Concrete
- ACI 340.1R, Design Handbook, Volume 1
- ACI 349, Appendix B, Steel Embedments
- Bowles, Joseph, *Foundation Analysis and Design*, New York, McGraw-Hill Book Company, 1988.
- Federal Highway Administration, *Drilled Shafts: Construction Procedures and Design Methods*, Dallas, ADSC: The International Association of Foundation Drilling, 1988.
- Poulos, H. G., Davis, E. H., *Pile Foundation Analysis and Design*, New York, John Wiley and Sons, 1980.
- ASCE, Wind Loads and Anchor Bolt Design for Petrochemical Facilities, 1997.

FOUNDATION TYPES

Foundations are used to transmit loads from process equipment and structures to the underlying soil or rock. Foundations are usually placed at some depth below the ground surface into firm, competent soil or rock. Common foundation types for process equipment and structures are spread footings, piles, and drilled piers or caissons. Examples of these are shown in [Figure 1](#).

SHALLOW FOUNDATIONS

Shallow foundations include spread footings, combined footings and mats. A spread footing is a foundation that supports a single load, such as a tall vertical pressure vessel or individual column of multi-column support structure. It generally consists of a pad or base and a pedestal. The pedestal acts like a column and transfers loads to the pad. As its name suggests, spread footings are used to "spread" loads so that the soil stress intensity (pressure) is reduced to a value that the soil can safely carry.

Spread footings are generally square or rectangular but they may also be circular or octagonal. Octagonal footings are widely used to support vertical vessels and stacks with circular bases. Circular footings are rarely used because of difficulties in forming them. The base of the spread footing is normally placed below the frost line and at a level where the soil or rock has adequate strength and settlement characteristics.

Combined footings are special types of spread footings that are used to support two or more columns located in the same plane. This type of footing is frequently used to support horizontal drums or heat exchangers.

Another type of shallow foundation is a mat foundation. A mat is a large concrete slab used to support two or more columns. Mats are generally used when the base soil has low bearing capacity and/or the column loads are so large that more than 50 percent of the area is covered by conventional spread footings.

DEEP FOUNDATIONS

Piles are structural members with small cross-sectional areas compared to their length. They are generally used when shallow subsurface soils are too weak or too compressible to provide adequate support for the structure or process equipment. Piles transfer loads to deeper more suitable soil or rock. This transfer may be by vertical distribution of the load along the pile shaft, using friction, or by a direct application of load to a lower stratum through "end-bearing" of the pile tip. However, except when the pile penetrates an extremely soft soil to a solid base, load transfer is actually a combination of side resistance and point bearing. Piles are usually constructed of steel, concrete, or timber.

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Drilled piers are deep foundations which are constructed by drilling a hole, adding reinforcing, and back-filling the cavity with concrete. Drilled piers with straight shafts are also sometimes referred to as drilled shafts, drilled caissons, or bored piles. If the base is enlarged with an under-reaming tool, the foundation is referred to as a belled pier, belled caisson or an under-reamed foundation.

Drilled piers are normally used as deep foundations, but they can also be used as a shallow foundation, particularly those with enlarged bases. The diameter of drilled piers usually range between 1.5 ft (0.46m) and 12 ft (3.6m). Because they can be drilled in large sizes, single large-diameter belled piers can sometimes be used in place of a group of driven piles.

DESIGN CONSIDERATIONS

GEOTECHNICAL INVESTIGATION

A soils investigation should be performed to obtain the appropriate soil parameters to aid in the selection, design, and construction of the foundation system. In most cases, the investigation consists of making borings into the soil or rock underlying the site, sampling, and then testing the samples in the laboratory or in the field to obtain the soil strength and other geotechnical characteristics. A general reconnaissance of the area is also sometimes included as part of the investigation, noting such things as drainage conditions and the presence old pipelines or abandoned foundations which may require removal or relocation. If the site is in an area that is not heavily populated or industrialized, topographic and geologic maps available from government agencies can be helpful in assessing the general environment around a site. Aerial photographs are often used to identify landslide and fault zones, and buried stream channels, organic soils, etc., which are to be avoided if possible.

The process for conducting a site investigation as well as the sampling tools and tests are discussed in [Section XXIX-B](#), Soil Data, Site Preparation and Earthwork.

UNDERGROUND FACILITIES

The presence of underground facilities such as utilities, sewers, pipe conduit tunnels, etc., which might be impacted by or have animapact on the foundation system needs to be considered. The zone of influence for a foundation is often approximated as occurring along a cone extending on a 1:1 slope downward and outward from the perimeter of the base of the foundation. Generally, the placing of new foundations over underground facilities located within this zone is avoided unless it can be shown by analysis that there will be no detrimental effects to both the underground facilities and the foundation system. Such an evaluation should be performed by a specialist in this field and is beyond the scope of this Design Practice.

SIZE AND STRENGTH

Foundations are proportioned to provide adequate support to all applied loads without overstressing the surrounding soil. Generally, the foundation's overall dimensions are governed by the strength and settlement characteristics of the soil and its strength is governed by the distribution of internal forces considering the applied loads and interaction with the surrounding soil. The subsequent sections of this Design Practice discuss these considerations for each foundation type.

SPREAD FOOTINGS

This section covers spread footings that support a single column or pedestal. The design and analysis of other types of shallow foundations such as mats are usually carried out with the aid of a specialized computer programs and are not covered in this Design Practice.

BEARING CAPACITY

The soil beneath a shallow foundation, such as a spread footing, must be capable of carrying the loads placed upon it without shear failure, and resulting settlements must be tolerable for the particular process equipment or structure supported. The limiting unit pressure that a soil can carry is referred to as "bearing capacity" and it is a function of the shear strength of the soil and foundation size and depth.

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The shear strength of a soil is usually determined from the results of insitu and/or laboratory tests. The net ultimate bearing capacity, q_{ult} , is the total unit pressure that the soil at a given depth can support in excess of the pressure that exists at that depth due to weight of soil. This value is generally determined by a geotechnical engineer. The allowable bearing capacity, q_{all} , represents the maximum bearing pressure that the foundation will be allowed to transmit to the soil. It is the value used by the designer to establish the dimensions of a footing. It is obtained by dividing q_{ult} by a factor of safety:

$$q_{all} = \frac{q_{ult}}{F.S.} \quad \text{Eq. (1)}$$

The factor of safety used in bearing capacity analysis is generally 2.5 to 3.0 unless a special effort is made to define the loads or soil strength parameters more accurately than standard practice. On clay soils, an increase in loading over a short period of time will generally not cause any settlement, since changes in pore pressures will not occur rapidly. This is one reason a temporary decrease in the factor of safety (e.g., 2.0) is sometimes allowed for transient loadings on clay soils.

Bearing Capacity - Theory and Equations

The bearing capacity, q_{ult} , of a soil is computed by the addition of three components, all of which are functions of the friction angle ϕ of the material. The components are:

1. The cohesion and friction of a weightless material carrying no surcharge.
2. The friction of a weightless material with the addition of a surcharge.
3. The friction of a material possessing weight and carrying no surcharge.

The bearing capacity equation is:

$$q_{ult} = c N_c + \gamma D_f N_q + 1/2 \gamma B N_\gamma, \text{ for a continuous footing} \quad \text{Eq. (2)}$$

where:	q_{ult}	=	ultimate bearing capacity, ksf (kPa)
	c	=	cohesion, ksf (kPa)
	D_f	=	depth of footing, ft (m)
	γ	=	unit weight of the soil, kips/ft ³ (kN/m ³)
	B	=	footing dimension, ft (m)
	$N_c, N_q, \text{ and } N_\gamma$	=	bearing capacity factors with respect to cohesion, surcharge, and unit weight, respectively.

The bearing capacity equations for square and circular footings based on the theoretical equations adjusted empirically to fit test data are given below:

$$\text{Square} \quad q_{ult} = 1.3 c N_c + \gamma D_f N_q + 0.4 \gamma B N_\gamma \quad \text{Eq. (3)}$$

$$\text{Circular} \quad q_{ult} = 1.3 c N_c + \gamma D_f N_q + 0.3 \gamma B N_\gamma \quad \text{Eq. (4)}$$

Figure 2 shows the values of N_c , N_q , and N_γ for different values of friction angle ϕ .

Bearing Capacity - Sands

The bearing capacity of sand depends on the angle of internal friction, ϕ , of the sand, and the location of the water table. However, bearing capacity does not ordinarily govern the design of footings on sand, unless the sand is loose, or the footing is very narrow. Settlement is ordinarily the controlling factor for footings on sand.

The value of ϕ is obtained either from a correlation of ϕ vs. standard penetration test (SPT-N values) or from direct shear or triaxial tests. For most sands, $c = 0$, so the $c N_c$ term drops out of the equation. If the groundwater table is at a depth equal to B below the base of the footing, it does not affect bearing capacity. If it is located at the base of the footing, q_{ult} is reduced by 50%. The reduction varies linearly to 0% at B below the footing. If the groundwater table is above the base of the footing, the buoyant unit weight, should be included in the calculation of the surcharge.

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Bearing Capacity - Clays

Bearing capacity often governs the design of footings on clay. For clays with an angle of internal friction $\phi_c = 0$, the bearing capacity equation becomes: $q_{ult} = c N_c + \gamma D_f N_q$ (i.e., $N_\gamma = 0$, see [Figure 2](#)). The $\gamma D_f N_q$ term is usually very small compared to $c N_c$ and thus is often neglected.

Factors Influencing Bearing Capacity - Local Shear

One factor that controls bearing capacity in some cases is a phenomenon known as local shear. For loose sands or soft clays, the failure paths in the soil may develop somewhat differently and cause localized shear under smaller loads than would be expected using conventional bearing capacity theory. Bearing capacity factors for local shear and guidelines which are sometimes used for local shear analysis are given on [Figure 2](#) (dashed lines for local shear).

BEARING PRESSURE

The pressure distribution beneath most footings is rather indeterminate because of the interaction of the footing rigidity with the soil type, state, and time response to stress. For this reason, it is common practice to assume a linear pressure distribution beneath spread footings.

Footings are often subjected to both vertical loads and moments resulting from horizontal loads (e.g., wind loads). Application of an overturning moment causes an increase in the applied bearing pressure under one end of the footing, and a decrease under the other end.

The calculation of pressure on the soil beneath a foundation subjected to vertical load and overturning moment is directly comparable to the calculation of stress in a structural member caused by axial force combined with bending. The combination of vertical load (P) and overturning moment (M) is equivalent to the resultant vertical load acting at an eccentricity, e , from the centroid of the footing, with e defined as:

$$e = \frac{M}{P} \quad \text{Eq. (5)}$$

when: $e \leq B/6$ for a square or rectangular footing, the resultant vertical load acts within the middle 1/3 of the base width B , and the soil beneath the footing is in full compression, ft (m)
 $e > B/6$ then the soil under the footing will only be partially in compression, ft (m)
 M = overturning moment, kip*ft (kN*m)
 P = vertical load, kips (kN)

For the resultant located within the middle 1/3, the bearing pressure distribution is calculated as follows:

$$q = \frac{P}{BL} \left(1 \pm \frac{6e}{L} \right) \quad \text{Eq. (6)}$$

where: q = bearing pressure, ksf (kPa)
 L = footing length, ft (m)
 B = footing width, ft (m)

For the resultant located outside the middle 1/3:

$$q_{min.} = 0$$

$$q_{max.} = \frac{2P}{3B \left[\left(\frac{L}{2} \right) - e \right]} \quad \text{Eq. (7)}$$

The soil pressure beneath a footing with moments applied about two axes (x and y) can be computed using the following if the soil under the footing is fully in compression:

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$$q = \frac{P}{BL} \left(1 \pm \frac{6e_x}{L} \pm \frac{6e_y}{B} \right) \quad \text{Eq. (8)}$$

where: e_x = eccentricity of vertically applied footing load, about the x axis, ft (m)
 e_y = eccentricity of the vertically applied footing load, about the y axis, ft (m)

The calculation of the bearing pressure under octagon shaped footings is similar to that of square or rectangular footings, except that the geometry for octagon footings is somewhat more complex. The nomograph shown on [Figure 3](#) can be used to compute the bearing pressures for octagon footings.

SETTLEMENT

Types of Settlement

Settlement accompanies an increase in the vertical stress in a soil deposit, and is usually due to loads placed on the deposit by fill or foundations, or due to lowering of the groundwater level. It can be broken into three components, namely immediate or distortional settlement, consolidation settlement, and secondary compression settlement. Distortional settlements occur almost instantaneously under application of load, and are primarily due to distortion within the soil mass. They are generally not elastic, but are calculated using elastic theory in some cases. Consolidation is a hydrodynamic process, occurring as water in the pores of a saturated soil is expelled under the pressure applied by the surface loading. The rate of consolidation and the accompanying settlement depend on the rate at which water can be expelled from the voids. Secondary compression settlements occur as the soil skeleton yields or compresses under the action of an applied load.

The three components of settlement occur in both granular and cohesive soils. However, in granular soils, water moves out of the voids almost instantaneously, and most of the settlement occurs as the load is applied. Because the settlements occur rapidly, they are often estimated using empirical methods or elastic methods which predict total settlement and make no attempt to predict the separate components.

For cohesive soils, distortional settlements are computed using elastic theory. Consolidation settlement occurs when a stress increase causes an increased pressure in the pore fluid. As the fluid flows out of the soil pores consolidation settlement results with its magnitude predicted by consolidation theory. Secondary compression is often of minor importance compared to consolidation settlements and can be neglected in those cases. It can be expected to be significant primarily in organic soils such as peats.

In foundation design, all components of settlement should be considered. In preparing Company specifications which deal with settlement, it should be understood that any calculation of magnitude of settlement is only an estimate and should be treated as such. Two factors should be kept in mind:

1. There are practical limitations in sampling and testing soils which lend a degree of uncertainty to design parameters determined from tests.
2. Field conditions may be different from conditions used in predicting the rate of settlement. Such a prediction should serve as a guideline for monitoring foundation performance, and should be recognized as an estimate. Rate of settlement can be predicted with fair accuracy by a geotechnical engineer who has experience with the soil deposit being analyzed, or by performing instrumented field load tests.

Differential Settlement

Calculated settlement-versus-load data as developed by most soils firms are for uniform settlements from uniformly applied vertical loading. However, uniform settlement of a rigid foundation by itself is usually not an item of concern. The problem is generally with the differential settlement that occurs over a large non-rigid foundation, or between separate foundations. This influences superstructure design stresses and necessary flexibility in interconnected piping.

Differential settlement can develop from one or more of the following causes:

1. Reported variations in soil properties over large areas for which more than one set of data on settlement versus load are provided.
2. Some degree of variation in soil properties over large areas even when conditions may appear sufficiently uniform to justify only one set of data.
3. Variations in applied unit pressures, or variations in footing sizes subjected to the same unit pressure.
4. Variations in the soil reaction to the applied pressure, as a function of whether the footing is flexible or rigid and whether the soil is cohesive, cohesionless or mixed.
5. Close spacing of footings, creating an overlap in the soil reaction forces (pressure bulbs).



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6. The effects of vibratory or cyclic loading.
7. The effect of unbalanced forces, such as wind, earthquake or process surging.
8. The factor of time with respect to the amount of settlement that occurs during erection, water test (if any) and operation.

Since soil conditions are never perfectly uniform, predicted values of settlement can represent average conditions only. As such, they may be inaccurate for large areas by 25 percent. Thus, it is possible that differential settlement over large areas can amount to as much as 50 percent of the average value. Over short distances, the degree of variation is usually less, about 10 to 15 percent, with maximum differentials of 20 to 30 percent.

These are generalizations; each site must be studied on its own merits by examination of the reported soils data, calculations and discussions with the soils consultant. Settlement calculated from theory generally errs on the conservative side when the soils are relatively uniform. This is less likely to be true for soils that are variable; the judgment of the individual making the analysis is important for proper assessment.

Within the confines of a building or equipment structure, variations in applied unit pressures, or variations in footing sizes subjected to the same unit pressure, can create differential settlement that adversely affects stresses in framing members. For buildings or other structures with a nominal degree of superstructure flexibility, the effects of differential settlement are generally not critical when total settlements do not exceed 1.0 in. Concern exists with rigid wall materials and rigid framing when total settlement values are higher, since stresses from differential settlement may be imposed that cannot economically or practically be provided for in the design.

Although foundations, including paving slabs for materials storage, may be subjected to uniform loading, the soil reaction is not uniform. This can lead to non-uniform settlement, with resulting objectionable bending when the footing or slab is large and lacking in rigidity.

When two adjacent footings are closely spaced, the overlap of the soil reaction forces may be sufficient to create a tilting of the footings towards each other. This is generally of little significance when total predicted settlements do not exceed 1.0 in. When a number of footings are closely spaced, the settlement is greater than from isolated footings, due to the area loading effect that creates an overall, larger and deeper pressure bulb of soil stresses. Unbalanced forces, such as wind and earthquake, can result in a foundation tilt when the soil is cohesionless and only of moderate relative density. It can also occur even with somewhat higher densities (70%) when there are large differences in applied pressure.

Settlement Versus Time

The rate of settlement for cohesive soils is commonly calculated based on consolidation tests of representative soil samples from the site. Consolidation occurs when excess pressure in water occupying the voids in the soil is dissipated, as water is forced to flow out of the loaded region. The surface displacements accompanying the volume change are called consolidation settlements. The expelled pore water must go to the ground surface, or to some buried aquifer, or to adjacent non-loaded areas. The rate at which the pore water escapes, and thus, the time rate of consolidation and settlement, are functions of the size of the pores present and the actual path and distance the pore water must travel.

In cohesive soils, the pores are small so that settlement (consolidation) under loading is a function of time which may involve many years.

In cohesionless soils, the pores are relatively large and settlement of soil at less than maximum density will occur rapidly during loading.

In mixed soils, such a sandy or silty clay, clayey or silty sand, or stratified conditions involving layers of both cohesionless and cohesive soils, the rate of settlement (consolidation) will be somewhere between the extremes of the previously indicated uniform soil types.

The amount of settlement must always be related to appropriate design time periods, such as erection, testing and operation, with the latter normally considered as the total settlement during the service life.

Settlement - Sands

Settlements in sands are normally calculated using an elastic method wherein a modulus is used; by conducting small-scale field tests and adjusting for foundation size effects; or by empirical methods. Empirical methods can require a great deal of experience to calculate settlements with reasonable accuracy, and are beyond the scope of this Design Practice. Regarding elastic methods, two formulas are presented below.

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$$\text{Settlement (S)} = I q B \frac{1 - \mu^2}{E} \quad \text{Eq. (9)}$$

where: S = Settlement, ft (m)
I = an influence factor which depends on the geometry and stiffness of the foundation
q = unit pressure, kips/ft² (kPa)
B = smallest dimension of the foundation, ft (m)
μ = Poisson's ratio
E = elastic modulus, kips/ft² (kPa)

Reasonable values for E can sometimes be obtained from the following empirical formulas which use results from Standard Penetration (SPT) and Cone Penetration tests.

$$E = 10 (N + 15) \text{ ksf}; \quad N = \text{SPT blow count} \quad \text{Eq. (10)}$$

$$E = 3 q_c \text{ (units of } q_c\text{);} \quad q_c \text{ is cone resistance} \quad \text{Eq. (11)}$$

A more accurate value for E can be obtained from the stress-strain curves for triaxial tests, or from plate load test data.

The settlement equation assumes that the soil is thick in comparison to the size of the foundation and thus contains no term for the depth or thickness of the soil layer in question. For small foundations this assumption is generally valid; however, for large foundations, use of this equation could overestimate settlements considerably. In cases where the foundation size is large in comparison with the thickness of the compressible layer, the following equation is useful:

$$S = L \frac{\Delta q}{E} \quad \text{Eq. (12)}$$

where: L = thickness of the compressible stratum, ft (m)
Δq = average stress change over thickness L, kips/ft² (kPa)
E = average modulus over thickness L, kips/ft² (kPa)

In areas where relatively uniform sand deposits exist, field tests consisting of loading a plate and observing stress vs. deformation characteristics are sometimes used to determine settlement. Typically a plate loading test utilizes a 1 ft (0.3m) square "foundation." A common formula for computing settlement of the full-sized foundation (a square shape is assumed) using the data from 1 ft square plate test is provided below.

$$S = S_1 \left(\frac{2B}{B + 1} \right)^2 \quad \text{Eq. (13)}$$

where: S = settlement for the full-sized foundation at design bearing pressure, in (mm)
S₁ = settlement of the plate loaded to that same pressure, in (mm)
B = width of the full-sized foundation, ft (m)

While this formula is well known and is commonly used, it often underestimates settlements, particularly for large foundations on loose sand. A formula which appears to provide more accuracy is:

$$\frac{S_a / B_a}{S_p / B_p} = \left(\frac{B_a}{B_p} \right)^n \quad \text{Eq. (14)}$$

where: S_a, S_p = settlement of footing (desired quantity) and plate measured, respectively, ft (m)
B_a, B_p = full-size footing and load plate dimension, respectively, ft (m)
n = coefficient depending on soil, with the following values in the absence of test values:

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For clay:	0.03 to 0.05
For sandy clay:	0.08 to 0.10
For dense sand:	0.40 to 0.50
For medium dense sand:	0.25 to 0.35
For loose sand:	0.20 to 0.25

Settlement - Clays

Settlement in clays can be "immediate" in the case of unsaturated clays, or time-dependent in the case of saturated or nearly saturated clays. Settlement in unsaturated clays (degree of saturation < about 90%) can be calculated using the elastic formula:

$$S = I q B \frac{1 - \mu^2}{E} \quad \text{Eq. (15)}$$

where: I = an influence factor which depends on the geometry and stiffness of the foundation
q = unit pressure, ksf (kPa)
B = smallest dimension of footing, ft (m)
μ = Poisson's ratio
E = elastic modulus, ksf (kPa)

which is the same formula as presented above for sands, with all terms in the formula as previously defined. [Figure 4](#) gives values of I, μ, and E for rough calculations. More accurate values of E can be obtained from the stress-strain curves from triaxial tests. Time-dependent settlements are calculated using consolidation theory.

One of the most important parts of consolidation settlement analysis is the determination of the stress history of the clay, i.e., whether it is normally consolidated or overconsolidated. A normally consolidated clay is presently experiencing the maximum load that it has ever experienced. In contrast, an overconsolidated clay has previously experienced a greater load than it is now experiencing. Since consolidation is accomplished by squeezing water out of the soil voids under pressure, the water content of the soil can be a preliminary indicator of behavior. The water content can be compared to the liquid limit and plastic limit obtained in the Atterberg limits test. The closer the water content is to the liquid limit the more likely the clay is normally consolidated. A water content close to the plastic limit generally indicates a soil which is overconsolidated. From the results of the consolidation tests, the preconsolidation pressure is determined by the Casagrande Construction. The preconsolidation pressure is the best estimate of the maximum load to which the clay has previously been loaded. Typically clays become overconsolidated by previous loads which have been removed (e.g., glaciers, sand dunes), or by drying (desiccation).

Another important step in the calculation of consolidation settlements is the determination of the stress increase due to the load, $\Delta\sigma'$. This is normally done using stress distribution charts based on the Boussinesq equations.

With a knowledge of preconsolidation pressure, the stress increase in the clay due to imposed loads, and parameters from the soil tests conducted, consolidation settlements can be computed. Applicable equations are presented below.

For a normally consolidated clay,

$$S = \frac{C_c H}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_o} \quad \text{Eq. (16)}$$

For an overconsolidated clay, where $\Delta\sigma' > (\sigma'_p - \sigma'_o)$

$$S = \frac{C_r H}{1 + e_o} \log \frac{\sigma'_p}{\sigma'_o} + \frac{C_c H}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_p} \quad \text{Eq. (17)}$$

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For an overconsolidated clay, where $\Delta\sigma' < (\sigma'_p - \sigma'_o)$

$$S = \frac{C_r H}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_o} \quad \text{Eq. (18)}$$

where: S = settlement, ft (m)
 C_c = compression index (measured in consolidation test)
 C_r = recompression index (measured in rebound cycle of consolidation test)
 e_o = initial void ratio
H = thickness of layer, ft (m)
 $\Delta\sigma'$ = effective stress increase due to applied load, ksf (kPa)
 σ'_o = in situ effective stress, ksf (kPa)
 σ'_p = preconsolidation pressure (determined from consolidation test results using Casagrande's construction), ksf (kPa)

A common problem associated with consolidation settlement of clays is computation of settlement with time. This problem is handled using the equation:

$$c_v = \frac{T_v}{t} H^2 \quad \text{Eq. (19)}$$

where: c_v = coefficient of consolidation, ft/day (m/day)
 T_v = time factor - related to degree of consolidation
t = time, days
H = length of flow path - usually 1/2 of the compressible layer thickness, ft (m)

In theory the development of consolidation settlement with time can be calculated. In reality, however, there are generally sand seams or other extra drainage layers which are not encountered in the soils investigation or cannot be taken into account in the calculations which will make the actual time rate of settlement faster than the calculated time rate of settlement.

Additional settlement, called secondary compression settlement, may occur after excess pore water pressures are completely dissipated (after primary consolidation is complete). Secondary compression settlement is caused by shifting or deformation of the soil skeleton. Fracture of some soil particles might also occur. Secondary compression settlement is generally only significant in organic soils such as peat, or in deep deposits of soft clay. To calculate secondary compression settlement, the following formula may be used.

$$ps = c \log_{10} \frac{t_2}{t_1}, \quad \text{Eq. (20)}$$

where: ps = secondary consolidation settlement, ft (m)
 t_2 = time at which secondary compression settlement is desired, days
 t_1 = time corresponding to 100% of primary consolidation, days
c = coefficient of secondary compression, ft (m)

c is determined from a consolidation test wherein a soil sample is loaded to the expected foundation pressure, and the load is held for a long period of time.

It should be noted that even though lab test results are mentioned as input to the above equations laboratory test data can be misleading. Better estimates of magnitude and rate of settlement usually result if field data from past projects can be used.

Recommended Settlement Limits

Recommended settlement limits are presented in [Table 1](#) and guidelines based on experience are given below for piping, buildings, structures and equipment. In each case, the discussion considers three types of soils: cohesionless, cohesive and mixed.

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Piping - Flexibility in piping should be provided for differential settlement between interconnected equipment items and buildings not all on the same foundation, to accommodate the difference in predicted total settlement plus 0.25 times the average of the two predicted values. This design settlement value may be reduced with certain soils, as noted below. The design settlement should reflect any anticipated additional settlement expected in operation from vibratory or cyclic loading of the interconnected items.

Piping flexibility should be provided to cover settlement over the full service life of the connected building or equipment items, unless economics favors installing a smaller degree of flexibility and making corrections in operation. However, such corrections should not be scheduled during the first 5 years or so of plant operation, since this much time is needed to check actual versus predicted settlement and to permit orderly scheduling of needed corrections.

1. For purely cohesionless soils and highly overconsolidated cohesive soils, for which settlement occurs coincident with load application, the amount of settlement during equipment erection and testing is usually in the range of 40 to 80 percent of the total. Hence, an incentive exists for separating this value from the settlement expected during operation when nominal flexibility is not sufficient for the overall anticipated settlement.
2. For normally consolidated or slightly overconsolidated cohesive soils, the amount of settlement during equipment erection, plus that for the short period of water testing, is usually small with respect to that occurring during the long-time period of operation. Hence, piping flexibility must consider the total anticipated settlement.
3. With mixed soils, for which settlement with time falls between the above extremes, a study of the settlement versus time predictions is necessary to determine whether exclusion of settlements during erection and testing would significantly reduce the costs of piping flexibility requirements.

Buildings, Structures and Equipment - Recommended settlement limits for such items are listed in [Table 1](#). These are not to be used without judgment; the values are guides only. Comments concerning these recommendations are as follows:

1. For cohesionless soils, the recommended settlement limit is usually related to the erection condition, when the major portion of the load is the erected weight. Pumps, compressors and turbo-generators are in this category; also ordinary buildings for which the live load is a small percentage of the total load. For other items, for which the water-test or operating-fluid weight is usually significant, settlement is given for the test / operating condition. In some cases the latter is coupled with the erection settlement in order that the differential settlement on additional loading will not create undue stress increases in framing members.
2. For cohesive soils, the operating condition will govern, since settlement during erection and water test will be small for the time period involved.
3. With mixed soils, except for towers and drums, the operating condition should also govern, even though a larger portion of the total settlement will occur during erection and water test than with purely cohesive soils. For towers and drums on mixed soils, one should investigate whether water test or operation governs, since with some mixed soils the settlement due to water test may be more than that which develops from use of the operating weight alone. This normally occurs when the water test weight greatly exceeds the operating weight and soils are predominately cohesionless. In actuality, this represents an unusual condition, since settlement on completion of water test with mixed soils rarely exceeds 70 percent of the total from the operating weight alone. However, it merits investigation.

STRUCTURAL DESIGN

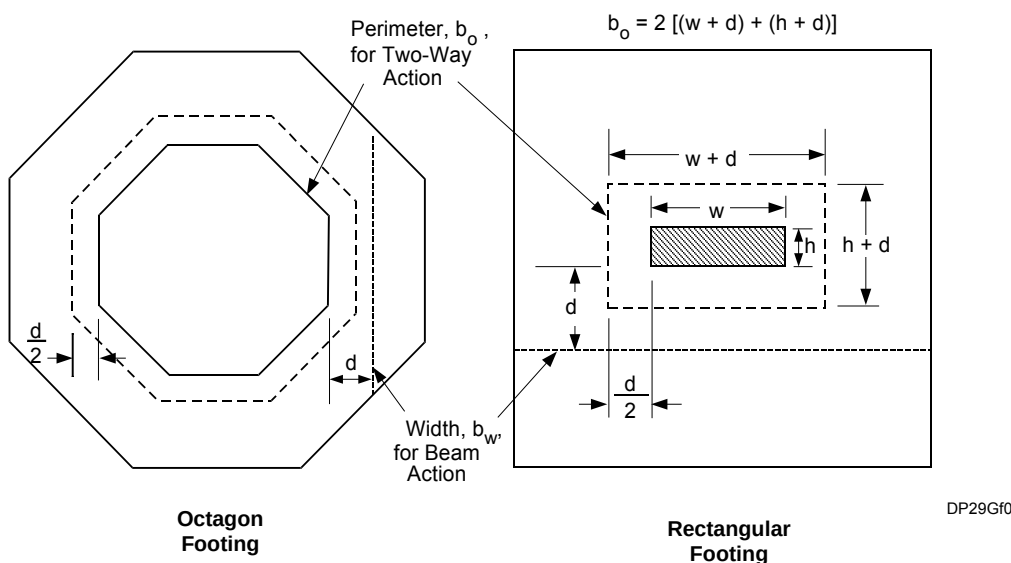
Once the plan dimensions of the footing and its depth have been established based on allowable bearing and settlement considerations, the contact pressures and other external loads are used to determine the thickness and reinforcement of the foundation components. Being made primarily of concrete, these components, namely the pedestal and base, should be designed in accordance with the latest revision of ACI-318, Building Code Requirements for Reinforced Concrete, for the most unfavorable combination of factored loads per **GP 4-1-3** and **GP 4-6-1**.

Shear Considerations

The thickness of the base of a single spread footing is generally governed by shear. Since such footings are subject to two-way action, i.e., bend in two major directions, their performance in shear is much like that of flat slabs in the vicinity of columns. However, in contrast to two-way floor and roof slabs, it is generally not economical to use shear reinforcement. Instead, increasing the thickness of the base of a footing is made large enough to satisfy shear requirements. Increasing the base thickness also has the beneficial effect of increasing the footing rigidity so that the assumption of linear bearing pressure is more likely to be obtained.

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Two different types of shear strength are distinguished in footings: two-way, or punching, shear and one-way, or beam, shear.



The critical sections for shear are illustrated below:

The critical section perimeter for two-way punching shear, b_o , completely encloses the column or pedestal and is located a distance $d/2$ from the outside face; d is the depth from the top of the base of the footing to the lower steel reinforcement. The critical section for beam shear, b_w , is located a distance d from the face of the column or pedestal.

Tests of footings have shown that for punching type shear failures the shear stress computed on the critical perimeter area is larger than in one-way action. The ACI allowables for beam and two-way action shear are as follows:

Customary

Metric

wide beam, v_c $= 2 \phi \sqrt{f'_c}$ $= \phi \sqrt{f'_c} / 6$ Eq. (21)

two-way, v_c $= \left(2 + \frac{4}{\beta}\right) \phi \sqrt{f'_c} / 6$ $= \left(1 + \frac{2}{\beta}\right) \phi \sqrt{f'_c} / 6$ Eq. (22)

where: v_c = shear, lbs (N)
 ϕ = 0.85 for shear
 β = pedestal length to width ratio; equals 1.0 for square, circular and octagon pedestals
 f'_c = concrete compressive strength, psi (MPa)

An example of the calculation of the shear stress in the critical sections for a rectangular footing is shown in **Example Problem No. 2** in the [Appendix](#).

SHEAR CRITICAL SECTIONS

BASE REINFORCING STEEL SIZE / DISTRIBUTION

Factored soil pressures are used to calculate the moment, M_u , at the critical section in the base of spread footings. (see ACI-318-02, par. 15.4). In most cases, the critical section is located at the face of the pedestal.