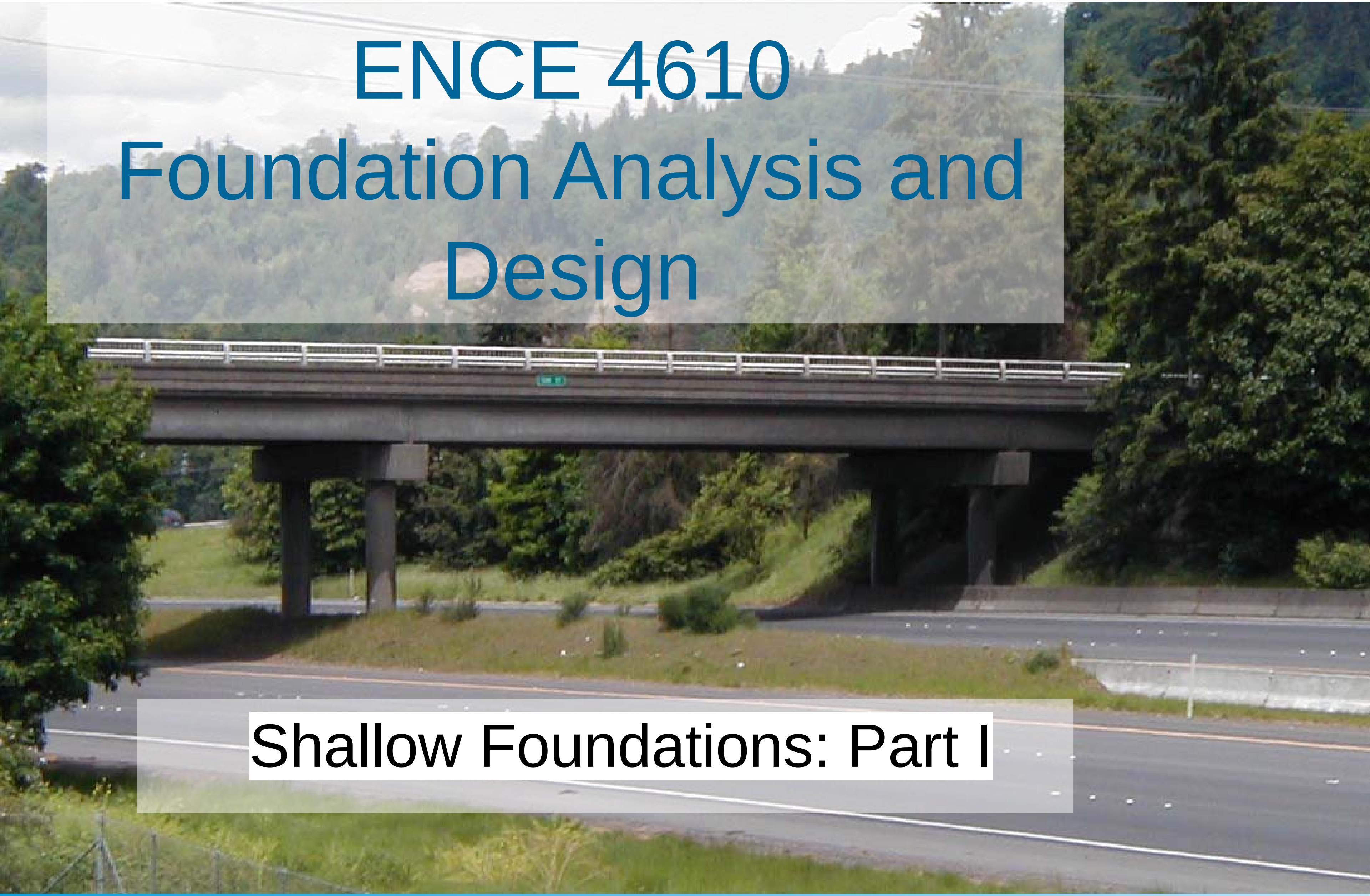
# ENCE 4610
# Foundation Analysis and Design



## Shallow Foundations: Part I

# Topics for Shallow Foundations, Part I

- Types of Shallow Foundations
- Spread Footing Design Concept and Procedure
- Bearing Capacity
  - Failure Mechanisms
  - Bearing Capacity Equation Formulation
  - Bearing Capacity Correction Factors
- Other Items
  - Local or Punching Shear
  - Factors of Safety
  - Practical Aspects of Bearing Capacity Formulations
  - Presumptive Bearing Capacities

# Types of Shallow Foundations

- Shallow foundations are usually placed within a depth D beneath the ground surface less than the minimum width B of the foundation
- Shallow foundations consist of:
  - Spread and continuous footings
  - Square, Rectangular or Circular Footings
  - Continuous footings
  - Ring Foundations
  - Strap Footings
- Wall footings
- Mats or Rafts

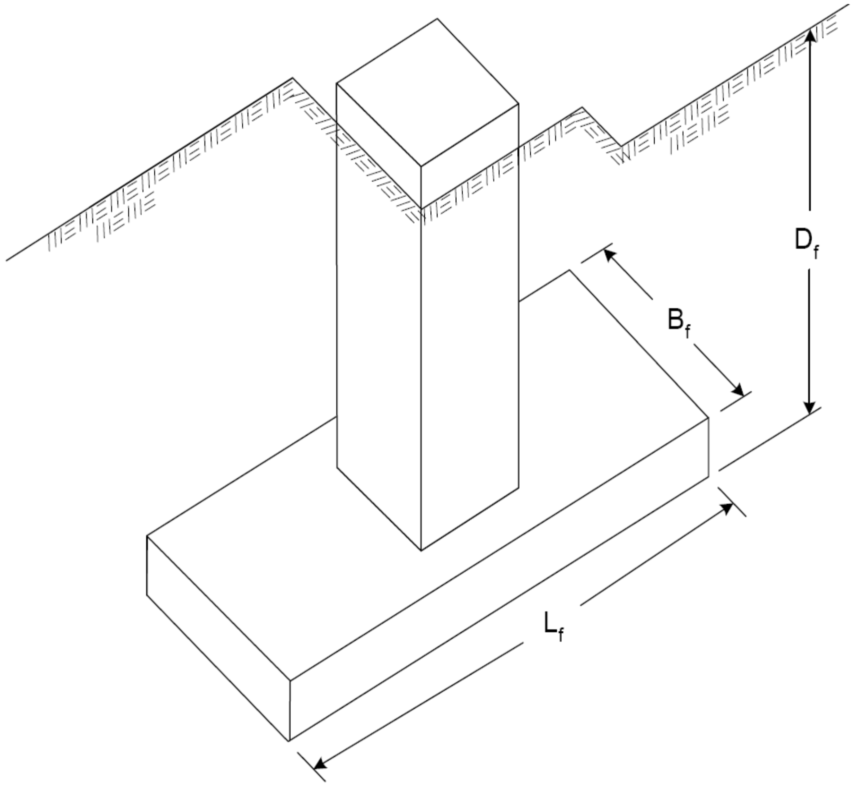



**Figure 8-1. Geometry of a typical shallow foundation (FHWA, 2002c, AASHTO 2002).**

# Footings

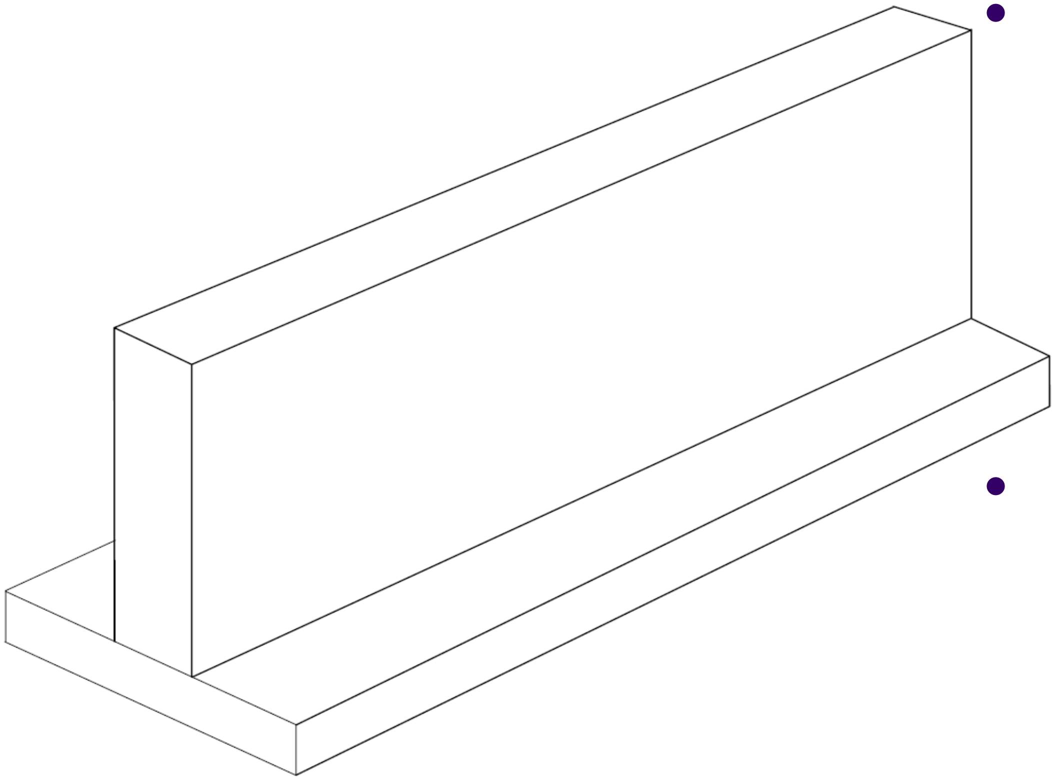


Figure 8-3. Continuous strip footing (FHWA, 2002c).

- A finite spread footing is a shallow foundation that transmits loads and has an aspect ratio of $1 \leq {}^{L}/_{B} \leq 10$
- A continuous spread footing is an "infinite" footing where ${}^{L}/_{B} > 10$ and the effects of L are ignored

# Abutment/Wingwall Footing

- A situation where the shallow foundation is a part of and acts with a retaining wall for both vertical load-bearing and horizontal loads of retained soil

# Combined Footing

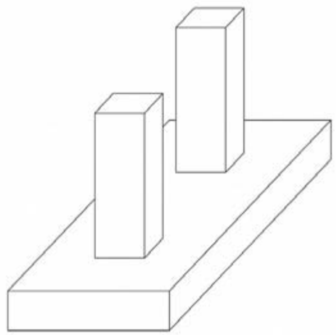


Figure 8-7. Combined footing (FHWA, 2002c).

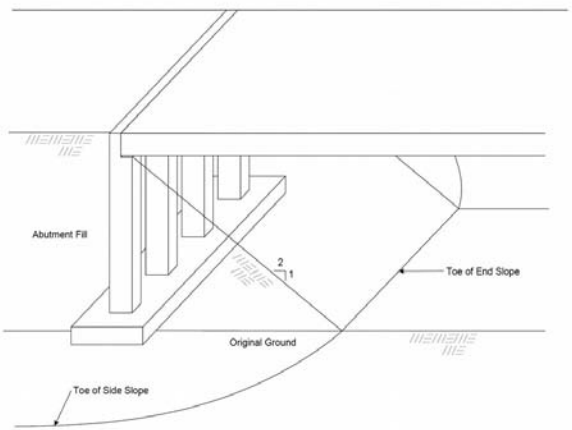



Figure 8-8. Spill-through abutment on combination strip footing (FHWA, 2002c).

- Combined footings are similar to isolated spread footings except that they support two or more columns and are rectangular or trapezoidal in shape (Figure 8-7). They are used primarily when the column spacing is non-uniform (Bowles, 1996) or when isolated spread footings become so closely spaced that a combination footing is simpler to form and construct. In the case of bridge abutments, an example of a combined footing is the so-called “spill-through” type abutment (Figure 8-8). This configuration was used during some of the initial construction of the Interstate Highway System on new alignments where spread footings could be founded on competent native soils. Spill-through abutments are also used at stream crossings to make sure that foundations are below the scour depth of the stream.

# Mat Foundations

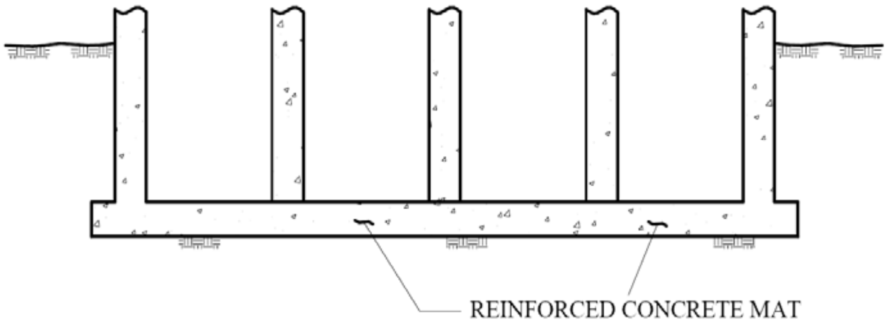
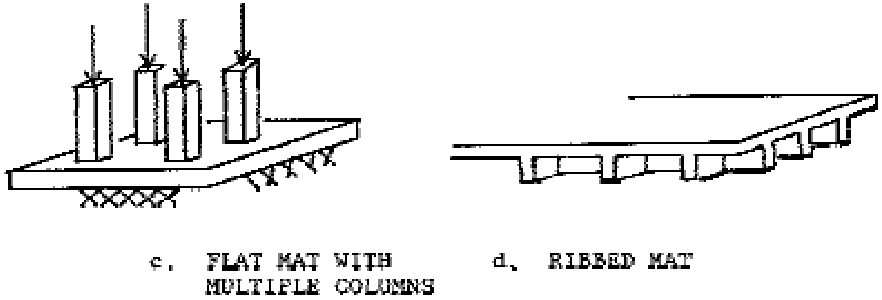


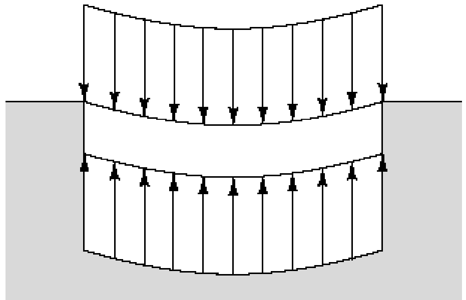

Figure 8-9. Typical mat foundation (FHWA, 2002c).



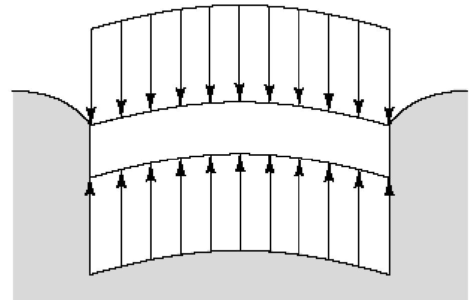

- A mat is continuous in two directions capable of supporting multiple columns, wall or floor loads. It has dimensions from 20 to 80 ft or more for houses and hundreds of feet for large structures such as multi-story hospitals and some warehouses
- Ribbed mats, consisting of stiffening beams placed below a flat slab are useful in unstable soils such as expansive, collapsible or soft materials where differential movements can be significant (exceeding 0.5 inch).

# Bearing Pressure Distribution Concentric Loads

Flexible foundation on clay



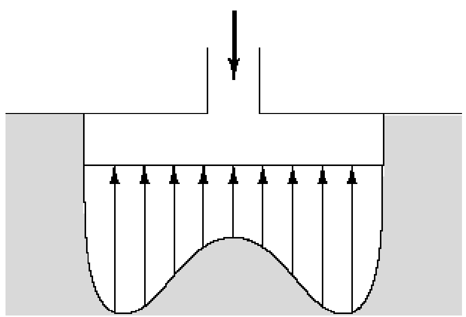
(a)



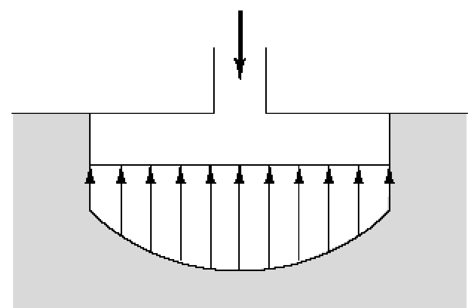
(b)

Flexible Foundation on Sand

Rigid foundation on clay

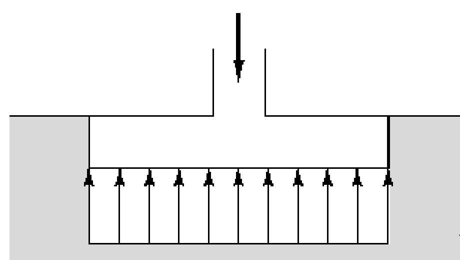


(c)



(d)

Rigid Foundation on Sand



(e)

Simplified Distribution

# Shear Failure vs. Settlement in Allowable Bearing Capacity

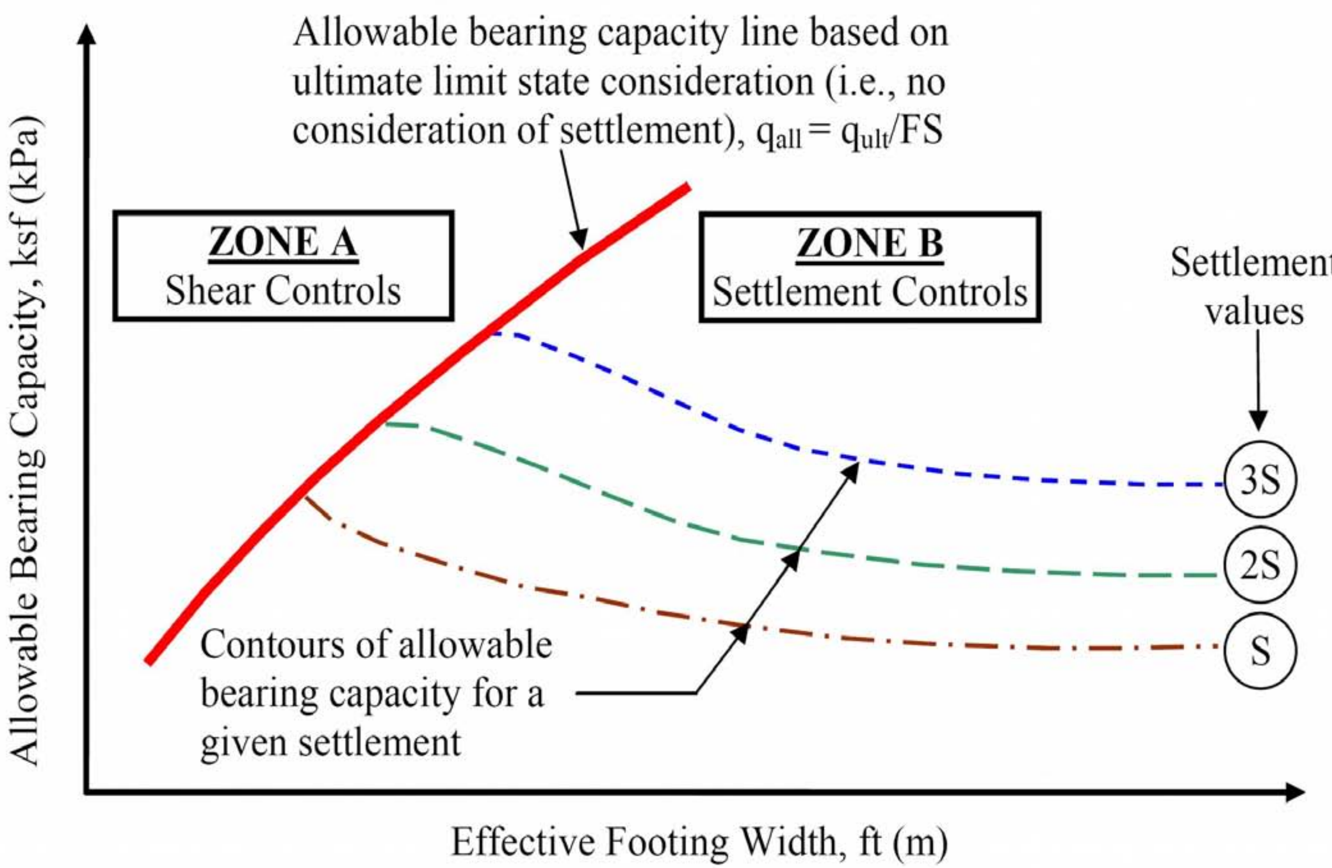


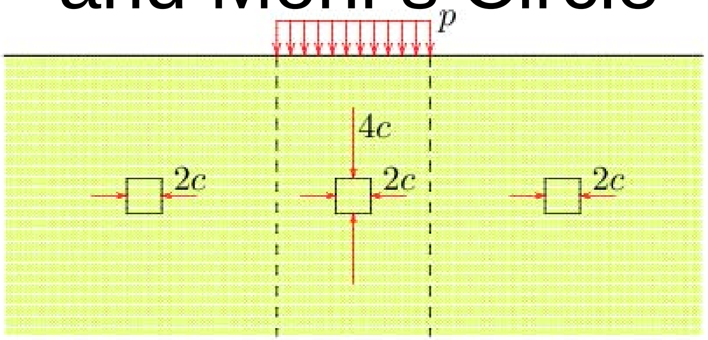

**Figure 8-10. Shear failure versus settlement considerations in evaluation of allowable bearing capacity.**

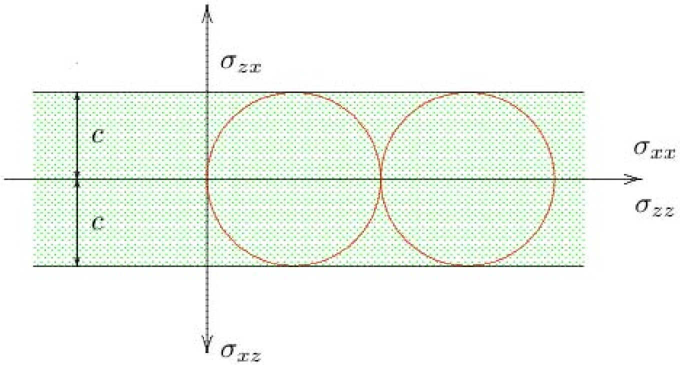
# Plasticity: Lower and Upper Bound Solutions

- The Problem
  - Bearing Capacity failure is a plastic failure of the soil along some failure surface
  - The problem of plastic failure of twofold:
    - Finding the failure surface along which the plasticity an ultimately failure takes place
    - Determining the failure state to which we should design, i.e. lower or upper bound
    - The first is done by determining which failure surface provides the “path of least resistance” of failure
    - The second is in part driven by uncertainty requirements in failure
- Review of Upper and Lower Bound Concept
  - Lower Bound: The true failure load is larger than the load corresponding to an equilibrium system. The system has failed in at least one place.
  - Upper Bound: The true failure load is smaller than the load corresponding to a mechanism, if that load is determined using the virtual work principle. The system has failed “in general.”
  - The idea is that the true solution is somewhere between the two
- We saw this when we went through unsupported cuts in purely cohesive soils
  - In that case, we had to consider both the shape of the failure surface and its location
  - For slopes, a circular failure surface was considered as the most likely failure surface
  - The actual surface could be located for simple slopes using theoretical considerations, but for more complex slopes (layered soils, water table, frictional soils) a trial and error solution was adopted
- In principle, only applicable to purely cohesive soils without friction, due to volume expansion considerations
  - Upper and lower bound theory can be extended to soils with a frictional component (or only a frictional component,) but the implementation is much more complicated
- We will begin by considering strip (infinite or continuous) foundations only) in cohesive soils

# Lower Bound Solution

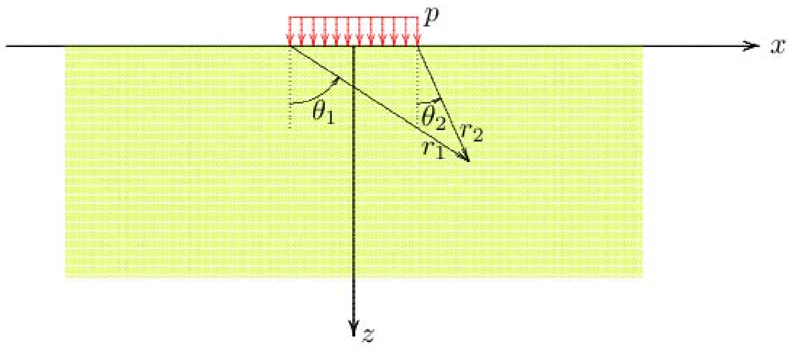
- By direct application and Mohr's Circle




$$p_c \geq 4c.$$

- By theory of elasticity




$$\tau = \frac{p}{\pi} \mid \sin(\theta_1 - \theta_2) \mid$$

The maximum value of $\mid \sin(\theta_1 - \theta_2) \mid$ is 1

$$p = \pi c.$$

- The more realistic lower bound

# Upper Bound Limit Equilibrium Method

# (Circular Failure Surface, Cohesive Soil)

$$\sum M_p = q_{ult} Bb \times \frac{B}{2} - \pi c Bb \times B - \sigma_0 Bb \times \frac{B}{2} = 0$$

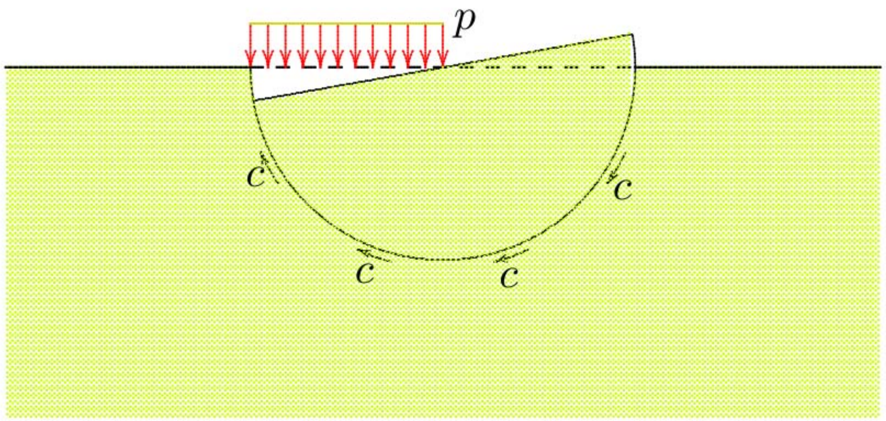



Figure 40.5: Mechanism 1.

$$q_{ult} = 2\pi c + \sigma_0$$

$$N_c = 2\pi \approx 6.28$$

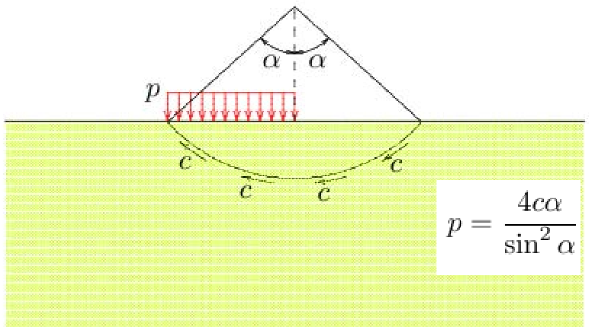
$$q_{ult} = N_c c + \sigma_0$$

Assume: No soil strength due to internal friction (cohesive soil,) shear strength above foundation base neglected

We add the effect of the weight of the soil (effective stress) acting on the top of the right side of the circle against rotation.

# More Realistic Upper Bound Case

- This is done by moving the centre of the failure surface upward




The smallest value is obtained for $\alpha = 1.165562$, or $\alpha = 66.78°$. The center of the circle then is located at a height $0.429a$. The corresponding value of $p$ is $5.52c$. This is an upper bound, hence

$$p_c \leq 5.52c. \quad (40.9)$$

So the solution is bounded by

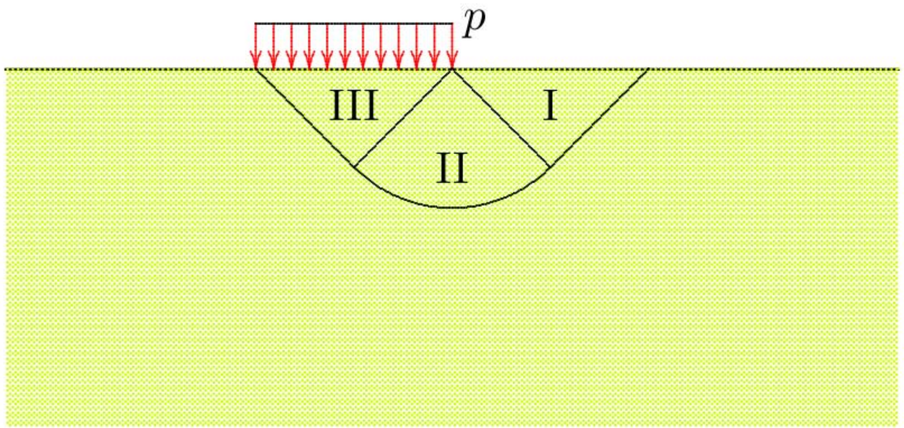
$$3.14c \leq p \leq 5.52c$$

- A circular failure surface is what we assumed for slope failure
- It is valid for very soft clays, and methods have been developed for use with these types of soils
- Soft clays are more subject to settlement
- We will not use these in this course

# Development of Prandtl Bearing Capacity Theory

- Application of limit equilibrium methods first done by Prandtl on the punching of thick masses of metal (materials with no internal frictional effects)
- Prandtl's methods first adapted by Terzaghi to bearing capacity failure of shallow foundations (specifically, he added the effects of frictional materials)
- Vesić and others (Meyerhof, Brinch Hansen, etc.) improved on Terzaghi's original theory and added other factors for a more complete analysis

- Note the three zones, the foundation fails along the lower boundary of these zones



# Assumptions for Bearing Capacity Methods

- Foundation-Soil Interface Assumptions
  - Foundation is very rigid relative to the soil
  - No sliding occurs between foundation and soil (rough foundation)
- Loading Assumptions
  - Applied load is compressive and applied vertically to the centroid of the foundation*
  - No applied moments present*
- Geometric assumption
  - Depth of foundation is less than or equal to its width
  - Foundation is a strip footing (infinite length)*
- Geotechnical Assumptions
  - Soil beneath foundation is homogeneous semi-infinite mass*
  - Mohr-Coulomb model for soil
  - General shear failure mode is the governing mode*
  - No soil consolidation occurs
  - Soil above bottom of foundation has no shear strength; is only a surcharge load against the overturning load*

* We will discuss "workarounds" to these assumptions

# Loads and Failure Zones on Strip Foundations

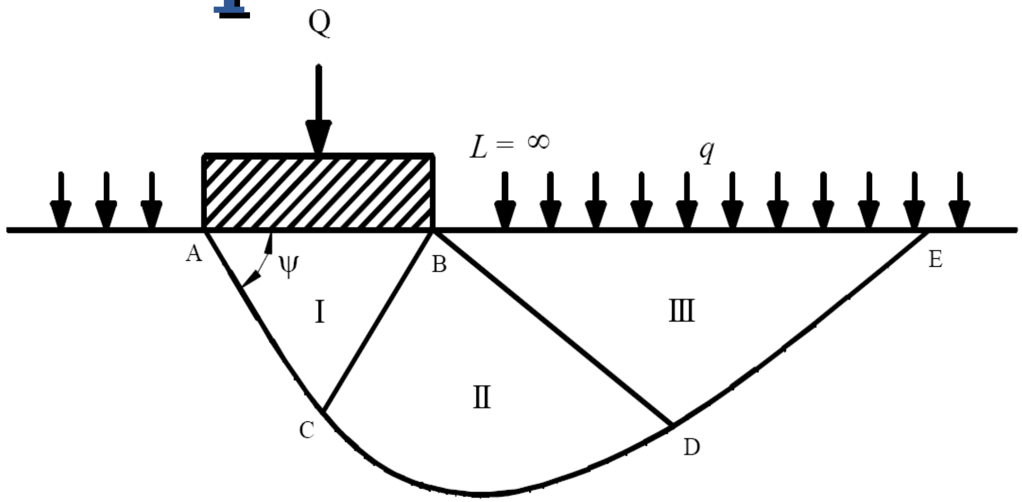
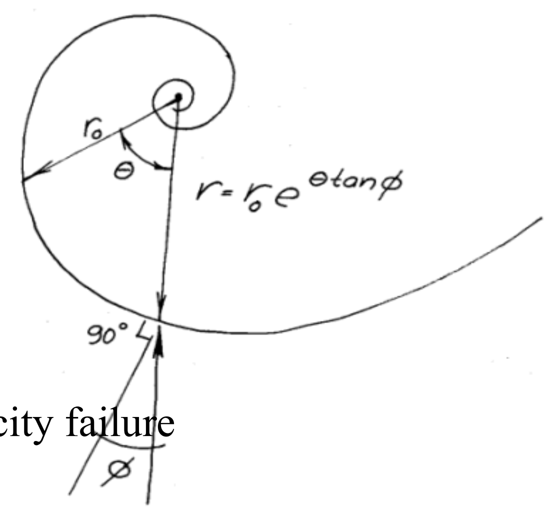



Figure 8-13. Boundaries of zone of plastic equilibrium after failure of soil beneath continuous footing (FHWA, 2002c).




Q = load/unit length on foundation
q = load/unit area due to effective stress at base of foundation
I, II, III = regions of failure in Prantdl theory for bearing capacity failure

# Basic Equation of Bearing Capacity

**8.4.2 Bearing Capacity Equation Formulation**

In essence, the bearing capacity failure mechanism is similar to the embankment slope failure mechanism discussed in Chapter 6. In the case of footings, the ultimate bearing capacity is equivalent to the stress applied to the soil by the footing that causes shear failure to occur in the soil below the footing base. For a concentrically loaded rigid strip footing with a rough base on a level homogeneous foundation material without the presence of water, the gross ultimate bearing capacity, $q_{ult}$, is expressed as follows (after Terzaghi, 1943):

$$q_{ult} = c\,(N_c) + q\,(N_q) + 0.5\,(\gamma)(B_f)(N_\gamma) \qquad \text{8-1}$$

"Cohesion" term "Surcharge" term Foundation soil "Weight" term

where: $c$ = cohesion of the soil (ksf) (kPa)

$q$ = total surcharge at the base of the footing = $q_{appl} + \gamma_a D_f$ (ksf) (kPa)

$q_{appl}$ = applied surcharge (ksf)(kPa)

$\gamma_a$ = unit weight of the overburden material above the base of the footing causing the surcharge pressure (kcf) (kN/m$^3$)

$D_f$ = depth of embedment (ft) (m) (Figure 8-1)

$\gamma$ = unit weight of the soil under the footing (kcf) (kN/m$^3$)

$B_f$ = footing width, i.e., least lateral dimension of the footing (ft) (m) (Figure 8-1)

$N_q$ = bearing capacity factor for the "surcharge" term (dimensionless)

$$= e^{\pi \tan\phi} \tan^2 (45^\circ + \frac{\phi}{2}) \qquad \text{8-2}$$

$N_c$ = bearing capacity factor for the "cohesion" term (dimensionless)

$$= (N_q - 1)\cot\phi \quad \text{for } \phi > 0^\circ \qquad \text{8-3}$$

$$= 2 + \pi = 5.14 \quad \text{for } \phi = 0^\circ \qquad \text{8-4}$$

$N_\gamma$ = bearing capacity factor for the "weight" term (dimensionless)

$$= 2\,(N_q + 1)\tan(\phi) \qquad \text{8-5}$$

# Basic Equation of Bearing Capacity

- Values of $N_c$, $N_q$ mostly the same. Values of $N_\gamma$ depend upon theor
- DIN/Brinch-Hansen:

$$N_\gamma = 2(N_q - 1)(\tan\phi')$$

- CFEM:

$$N_\gamma = 1.5(N_q - 1)(\tan\phi')$$

- Vesić:

$$N_\gamma = 2(N_q + 1)(\tan\phi')$$

- Vesić-AASHTO Factors:

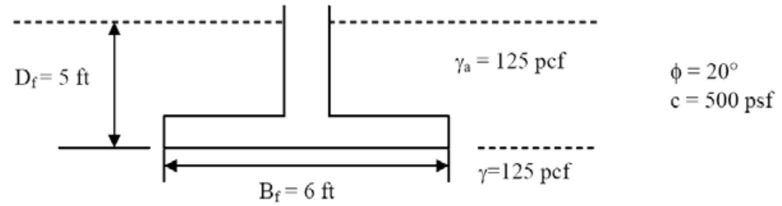
**Table 8-2**

**Bearing Capacity Factors (AASHTO, 2004 with 2006 Interims)**

| $\phi$ | $N_c$ | $N_q$ | $N_\gamma$ | $\phi$ | $N_c$ | $N_q$ | $N_\gamma$ |
|---|---|---|---|---|---|---|---|
| 0 | 5.14 | 1.0 | 0.0 | 23 | 18.1 | 8.7 | 8.2 |
| 1 | 5.4 | 1.1 | 0.1 | 24 | 19.3 | 9.6 | 9.4 |
| 2 | 5.6 | 1.2 | 0.2 | 25 | 20.7 | 10.7 | 10.9 |
| 3 | 5.9 | 1.3 | 0.2 | 26 | 22.3 | 11.9 | 12.5 |
| 4 | 6.2 | 1.4 | 0.3 | 27 | 23.9 | 13.2 | 14.5 |
| 5 | 6.5 | 1.6 | 0.5 | 28 | 25.8 | 14.7 | 16.7 |
| 6 | 6.8 | 1.7 | 0.6 | 29 | 27.9 | 16.4 | 19.3 |
| 7 | 7.2 | 1.9 | 0.7 | 30 | 30.1 | 18.4 | 22.4 |
| 8 | 7.5 | 2.1 | 0.9 | 31 | 32.7 | 20.6 | 26.0 |
| 9 | 7.9 | 2.3 | 1.0 | 32 | 35.5 | 23.2 | 30.2 |
| 10 | 8.4 | 2.5 | 1.2 | 33 | 38.6 | 26.1 | 35.2 |
| 11 | 8.8 | 2.7 | 1.4 | 34 | 42.2 | 29.4 | 41.1 |
| 12 | 9.3 | 3.0 | 1.7 | 35 | 46.1 | 33.3 | 48.0 |
| 13 | 9.8 | 3.3 | 2.0 | 36 | 50.6 | 37.8 | 56.3 |
| 14 | 10.4 | 3.6 | 2.3 | 37 | 55.6 | 42.9 | 66.2 |
| 15 | 11.0 | 3.9 | 2.7 | 38 | 61.4 | 48.9 | 78.0 |
| 16 | 11.6 | 4.3 | 3.1 | 39 | 67.9 | 56.0 | 92.3 |
| 17 | 12.3 | 4.8 | 3.5 | 40 | 75.3 | 64.2 | 109.4 |
| 18 | 13.1 | 5.3 | 4.1 | 41 | 83.9 | 73.9 | 130.2 |
| 19 | 13.9 | 5.8 | 4.7 | 42 | 93.7 | 85.4 | 155.6 |
| 20 | 14.8 | 6.4 | 5.4 | 43 | 105.1 | 99.0 | 186.5 |
| 21 | 15.8 | 7.1 | 6.2 | 44 | 118.4 | 115.3 | 224.6 |
| 22 | 16.9 | 7.8 | 7.1 | 45 | 133.9 | 134.9 | 271.8 |

# Bearing Capacity Example

**Example 8-1**: Determine the ultimate bearing capacity for a rigid strip footing with a rough base having the dimensions shown in the sketch below. Assume that the footing is concentrically loaded and that the total unit weight below the base of the footing is equal to the total unit weight above the base of the footing, i.e., in terms of the symbols used previously, $\gamma = \gamma_a$. First assume that the ground water table is well below the base of the footing and therefore it has no effect on the bearing capacity. Then, assume that the groundwater table is at the base of the footing and recompute the ultimate bearing capacity.




**Solution:**

Assume a general shear condition and enter Table 8-2 for $\phi$= 20° and read the bearing capacity factors as follows:

$N_c = 14.8$, $N_q = 6.4$, $N_\gamma = 5.4$. These values can also be read from Figure 8-15.

$$q_{ult} = c(N_c) + \gamma_a(D_f)(N_q) + 0.5(\gamma)(B_f)(N_\gamma)$$

$q_{ult}$ = (500 psf)(14.8) + (125 pcf) (5 ft) (6.4) + 0.5(125 pcf) (6 ft)(5.4)
= 7,400 psf + 4,000 psf + 2,025 psf
$q_{ult}$ = 13,425 psf

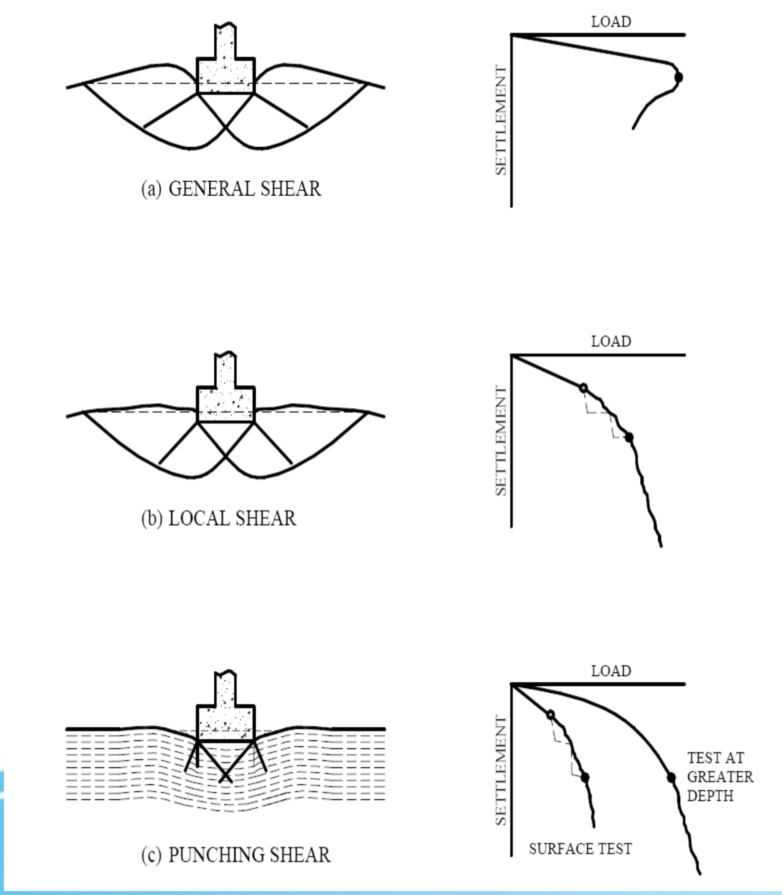
Effect of water: If the ground water table is at the base of the footing, i.e., a depth of 5 ft from the ground surface, then effective unit weight should be used in the "weight" term as follows:

$q_{ult}$ = (500 psf)(14.8) + (125 pcf) (5 ft) (6.4) + 0.5(125 pcf - 62.4 pcf) (6 ft)(5.4)
= 7,400 psf + 4,000 psf + 1,014 psf
$q_{ult}$ = 12,414 psf

Sections 8.4.2.1 and 8.4.3.2 further discuss the effect of water on ultimate bearing capacity.

# Types of Bearing Capacity

- Traditionally, bearing capacity has been classified as follows:
  - General Shear (the case upon which the theory is based)
  - Local Shear
  - Punching Shear
  - Which one takes place depends upon consistency or density of soil, which decreases from general to local to punching
- Generally, with softer soils, settlement tends to govern the design more than bearing capacity



# Soil Property Corrections for Local and Punching Shear

- Local and punching shear are accounted for by reducing the cohesion and/or friction angle of the soil
- Read entire section on how and when to use this reduction
- Also analyze for settlement

**8.4.5 Local or Punching Shear**

Several references, including AASHTO (2004 with 2006 Interims), recommend reducing the soil strength parameters if local or punching shear failure modes can develop. Figure 8-19 shows conditions when these modes can develop for granular soils. The recommended reductions are shown in Equations 8-11 and 8-12.

$$c^* = 0.67c \qquad 8\text{-}11$$

$$\phi^* = \tan^{-1}(0.67\tan\phi) \qquad 8\text{-}12$$

where: $c^*$ = reduced effective stress soil cohesion for punching shear (tsf (MPa))
$\phi^*$ = reduced effective stress soil friction angle for punching shear (degrees)

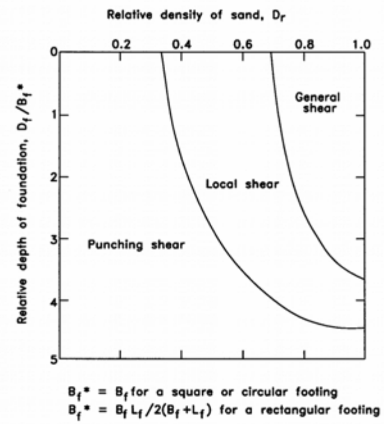



**Figure 8-19. Modes of failure of model footings in sand (after Vesic, 1975; AASHTO, 2004 with 2006 Interims)**

# Bearing Capacity Correction Factors

#### 8.4.3 Bearing Capacity Correction Factors

A number of factors that were not included in the derivations discussed earlier influence the ultimate bearing capacity of shallow foundations. Note that Equation 8-1 assumes a rigid strip footing with a rough base, loaded through its centroid, that is bearing on a level surface of homogeneous soil. Various correction factors have been proposed by numerous investigators to account for footing shape adjusted for eccentricity, location of the ground water table, embedment depth, sloping ground surface, an inclined base, the mode of shear, local or punching shear, and inclined loading. The general philosophy of correcting the theoretical ultimate bearing capacity equation involves multiplying each of the three terms in the bearing capacity equation by empirical factors to account for the particular effect. Each correction factor includes a subscript denoting the term to which the factor should be applied: "c" for the cohesion term, "q" for the surcharge term, and "γ" for the weight term. Each of these factors and suggestions for their application are discussed separately below. In most cases these factors may be used in combination.

The general form of the ultimate bearing capacity equation, including correction terms, is:

$$q_{ult} = cN_c s_c b_c + qN_q C_{wq} s_q b_q d_q + 0.5\gamma B_f N_\gamma C_{w\gamma} s_\gamma b_\gamma \qquad 8\text{-}6$$

where: $s_c$, $s_\gamma$ and $s_q$ are **shape correction factors**

$b_c$, $b_\gamma$ and $b_q$ are **base inclination correction factors**

$C_{w\gamma}$ and $C_{wq}$ are **groundwater correction factors**

$d_q$ is an **embedment depth correction factor** to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation. Recall that the embedment is modeled as a surcharge pressure applied at the bearing elevation. To be theoretically correct, the "q" in the surcharge term consists of two components, one the embedment depth surcharge to which the correction factor applies, the other an applied surcharge such as the traffic surcharge to which the correction factor, by definition, does not apply. Therefore, theoretically the "q" in the surcharge term should be replaced with ($q_a + \gamma D_f d_q$) where $q_a$ is defined as an applied surcharge for cases where applied surcharge is considered in the analysis;

$N_c$, $N_q$ and $N_\gamma$ are **bearing capacity factors** that are a function of the friction angle of the soil. $N_c$, $N_q$ and $N_\gamma$ can be obtained from Table 8-2 or Figure 8-15 or they can be computed by Equation 8-3/8-4, 8-2 and 8-5, respectively. As discussed in Section 8.4.3.6, $N_c$ and $N_\gamma$ are replaced with $N_{cq}$ and $N_{\gamma q}$ for the case of sloping ground or when the footing is located near a slope. In these cases the $N_q$ term is omitted.

The following sections provide guidance on the use of the bearing capacity correction factors, and whether or not certain factors should be used in combination.

# Shape Factors

**Table 8-4**

**Shape correction factors (AASHTO, 2004 with 2006 Interims)**

| Factor | Friction Angle | Cohesion Term ($s_c$) | Unit Weight Term ($s_\gamma$) | Surcharge Term ($s_q$) |
|---|---|---|---|---|
| Shape Factors, $s_c$, $s_\gamma$, $s_q$ | $\phi = 0$ | $1+\left(\frac{B_f}{5L_f}\right)$ | 1.0 | 1.0 |
| | $\phi > 0$ | $1+\left(\frac{B_f}{L_f}\right)\left(\frac{N_q}{N_c}\right)$ | $1-0.4\left(\frac{B_f}{L_f}\right)$ | $1+\left(\frac{B_f}{L_f}\tan\phi\right)$ |

*Note*: Shape factors, s, should not be applied simultaneously with inclined loading factors, i. See Section 8.4.3.5.

# Inclined Base Factors

### 8.4.3.4 Inclined Base

In general, inclined footings for bridges should be avoided or limited to inclination angles, α, less than about 8 to 10 degrees from the horizontal. Steeper inclinations may require keys, dowels or anchors to provide sufficient resistance to sliding. For footings inclined to the horizontal, Table 8-7 provides equations for the correction factors to be used in Equation 8-6.

**Table 8-7**

**Inclined base correction factors (Hansen and Inan, 1970; AASHTO, 2004 with 2006 Interims)**

| Factor | Friction Angle | Cohesion Term (c) $b_c$ | Unit Weight Term (γ) $b_\gamma$ | Surcharge Term (q) $b_q$ |
|---|---|---|---|---|
| Base Inclination Factors, $b_c$, $b_\gamma$, $b_q$ | $\phi = 0$ | $1-\left(\frac{\alpha}{147.3}\right)$ | 1.0 | 1.0 |
| | $\phi > 0$ | $b_q-\left(\frac{1-b_q}{N_c \tan\phi}\right)$ | $(1\text{-}0.017\alpha \tan\phi)^2$ | $(1\text{-}0.017\alpha \tan\phi)^2$ |
| ϕ = friction angle, degrees; α = footing inclination from horizontal, upward +, degrees | | | | |

# Groundwater Table Correction Factors

**8.4.3.2 Location of the Ground Water Table**

If the ground water table is located within the potential failure zone above or below the base of a footing, buoyant (effective) unit weight should be used to compute the overburden pressure. A simplified method for accounting for the reduction in shearing resistance is to apply factors to the two terms in the bearing capacity equation that include a unit weight term. Recall that the cohesion term is neither a function of soil unit weight nor effective stress. The ground water factors may be computed by interpolating values between those provided in Table 8-5 ($D_W$ = depth to water from ground surface).

**Table 8-5**
**Correction factor for location of ground water table**
**(AASHTO, 2004 with 2006 Interims)**

| $D_W$ | $C_{W\gamma}$ | $C_{Wq}$ |
|---|---|---|
| 0 | 0.5 | 0.5 |
| $D_f$ | 0.5 | 1.0 |
| $> 1.5B_f + D_f$ | 1.0 | 1.0 |
| *Note:* For intermediate positions of the ground water table, interpolate between the values shown above. | | |

Embedment Depth Factors

### 8.4.3.3 Embedment Depth

Because the effect on bearing capacity of the depth of embedment was accounted for by considering it as an equivalent surcharge applied at the footing bearing elevation, the effect of the shearing resistance due to the failure surface actually passing through the footing embedment cover was neglected in the theory. If the backfill or cover over the footing is known to be a high-quality, compacted granular material that can be assumed to remain in place over the life of the footing, additional shearing resistance due to the backfill can be accounted for by including in the surcharge term the embedment depth correction factor, $d_q$, shown in Table 8-6. Otherwise, the depth correction factor can be conservatively omitted.

**Table 8-6**
**Depth correction factors**
**(Hansen and Inan, 1970; AASHTO, 2004 with 2006 Interims)**

| Friction Angle, $\phi$ (degrees) | $D_f/B_f$ | $d_q$ |
|---|---|---|
| 32 | 1 | 1.20 |
| | 2 | 1.30 |
| | 4 | 1.35 |
| | 8 | 1.40 |
| 37 | 1 | 1.20 |
| | 2 | 1.25 |
| | 4 | 1.30 |
| | 8 | 1.35 |
| 42 | 1 | 1.15 |
| | 2 | 1.20 |
| | 4 | 1.25 |
| | 8 | 1.30 |

*Note:* The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0.

# Load Inclination Factors

- A convenient way to account for the effects of an inclined load applied to the footing by the column or wall stem is to consider the effects of the axial and shear components of the inclined load individually. If the vertical component is checked against the available bearing capacity and the shear component is checked against the available sliding resistance, the inclusion of load inclination factors in the bearing capacity equation can generally be omitted. The bearing capacity should, however, be evaluated by using effective footing dimensions, as discussed in Section 8.4.3.1 and in the footnote to Table 8-4, since large moments can frequently be transmitted to bridge foundations by the columns or pier walls. **The simultaneous application of shape and load inclination factors can result in an overly conservative design.**
- Unusual column geometry or loading configurations should be evaluated on a case-by-case basis relative to the foregoing recommendation before the load inclination factors are omitted. An example might be a column that is not aligned normal to the footing bearing surface. In this case, an inclined footing may be considered to offset the effects of the inclined load by providing improved bearing efficiency (see Section 8.4.3.4). Keep in mind that bearing surfaces that are not level may be difficult to construct and inspect. (FHWA NHI-06-089)

# Allowable Bearing Capacity

### 8.4.6 Bearing Capacity Factors of Safety

The minimum factor of safety applied to the calculated ultimate bearing capacity will be a function of:

- The confidence in the design soil strength parameters c and $\phi$,
- The importance of the structure, and
- The consequence of failure.

Typical minimum factors of safety for shallow foundations are in the range of 2.5 to 3.5. A minimum factor of safety against bearing capacity failure of 3.0 is recommended for most bridge foundations. This recommended factor of safety was selected through a combination of applied theory and experience. **Uncertainty in the magnitudes of the loads and the available soil bearing strength are combined into this single factor of safety.** The general equation to compute the allowable bearing capacity as a function of safety factor is:

$$q_{all} = \frac{q_{ult}}{FS} \qquad \text{8-13}$$

where: $q_{all}$ = allowable bearing capacity (ksf) (kPa)

$q_{ult}$ = ultimate bearing capacity (ksf) (kPa)

FS = the applied factor of safety

# Bearing Capacity Example

- Given
  - Square shallow foundation, 5' x 5'
  - Foundation depth = 2'
  - Cohesionless
  - Unit weight 121 pcf
  - Internal friction angle 31 degrees
  - Load on foundation = 76 kips
  - Groundwater table very deep
- Find
  - Factor of safety against bearing capacity failure
- Solution
  - Governing equation:

$$q_{ult} = cN_c s_c b_c + qN_q C_{wq} s_q b_q d_q + 0.5\gamma B_f N_\gamma C_{w\gamma} s_\gamma b_\gamma$$

  - We can neglect factors due to groundwater table ($C_w$), load inclination (b) and depth (d)

# Bearing Capacity Example

- Bearing capacity "N" factors for 31 degree friction angle
  - $N_c = 32.7$
  - $N_q = 20.6$
  - $N_\gamma = 26.0$

Table 8-2

Bearing Capacity Factors (AASHTO, 2004 with 2006 Interims)

| $\phi$ | $N_c$ | $N_q$ | $N_\gamma$ | $\phi$ | $N_c$ | $N_q$ | $N_\gamma$ |
|---|---|---|---|---|---|---|---|
| 0 | 5.14 | 1.0 | 0.0 | 23 | 18.1 | 8.7 | 8.2 |
| 1 | 5.4 | 1.1 | 0.1 | 24 | 19.3 | 9.6 | 9.4 |
| 2 | 5.6 | 1.2 | 0.2 | 25 | 20.7 | 10.7 | 10.9 |
| 3 | 5.9 | 1.3 | 0.2 | 26 | 22.3 | 11.9 | 12.5 |
| 4 | 6.2 | 1.4 | 0.3 | 27 | 23.9 | 13.2 | 14.5 |
| 5 | 6.5 | 1.6 | 0.5 | 28 | 25.8 | 14.7 | 16.7 |
| 6 | 6.8 | 1.7 | 0.6 | 29 | 27.9 | 16.4 | 19.3 |
| 7 | 7.2 | 1.9 | 0.7 | 30 | 30.1 | 18.4 | 22.4 |
| 8 | 7.5 | 2.1 | 0.9 | 31 | 32.7 | 20.6 | 26.0 |
| 9 | 7.9 | 2.3 | 1.0 | 32 | 35.5 | 23.2 | 30.2 |
| 10 | 8.4 | 2.5 | 1.2 | 33 | 38.6 | 26.1 | 35.2 |
| 11 | 8.8 | 2.7 | 1.4 | 34 | 42.2 | 29.4 | 41.1 |
| 12 | 9.3 | 3.0 | 1.7 | 35 | 46.1 | 33.3 | 48.0 |
| 13 | 9.8 | 3.3 | 2.0 | 36 | 50.6 | 37.8 | 56.3 |
| 14 | 10.4 | 3.6 | 2.3 | 37 | 55.6 | 42.9 | 66.2 |
| 15 | 11.0 | 3.9 | 2.7 | 38 | 61.4 | 48.9 | 78.0 |
| 16 | 11.6 | 4.3 | 3.1 | 39 | 67.9 | 56.0 | 92.3 |
| 17 | 12.3 | 4.8 | 3.5 | 40 | 75.3 | 64.2 | 109.4 |
| 18 | 13.1 | 5.3 | 4.1 | 41 | 83.9 | 73.9 | 130.2 |
| 19 | 13.9 | 5.8 | 4.7 | 42 | 93.7 | 85.4 | 155.6 |
| 20 | 14.8 | 6.4 | 5.4 | 43 | 105.1 | 99.0 | 186.5 |
| 21 | 15.8 | 7.1 | 6.2 | 44 | 118.4 | 115.3 | 224.6 |
| 22 | 16.9 | 7.8 | 7.1 | 45 | 133.9 | 134.9 | 271.8 |

# Bearing Capacity Example

- Shape Factors
- $s_c$ = 1+(5/5)(20.6/32.7) = 1.63
- $s_\gamma$=1-0.4(5/5) = 0.6
- $s_q$ = 1+(5/5)(tan(31)) = 1.6

**Table 8-4**

**Shape correction factors (AASHTO, 2004 with 2006 Interims)**

| Factor | Friction Angle | Cohesion Term ($s_c$) | Unit Weight Term ($s_\gamma$) | Surcharge Term ($s_q$) |
|---|---|---|---|---|
| Shape Factors, $s_c$, $s_\gamma$, $s_q$ | $\phi = 0$ | $1+\left(\frac{B_f}{5L_f}\right)$ | 1.0 | 1.0 |
| | $\phi > 0$ | $1+\left(\frac{B_f}{L_f}\right)\left(\frac{N_q}{N_c}\right)$ | $1-0.4\left(\frac{B_f}{L_f}\right)$ | $1+\left(\frac{B_f}{L_f}\tan\phi\right)$ |

*Note*: Shape factors, s, should not be applied simultaneously with inclined loading factors, i. See Section 8.4.3.5.

# Bearing Capacity Example

- Other variables
  - c=0 (problem statement)
  - q=(121)(2) = 242 psf
  - γ= 121 pcf (problem statement)
- Substitute and solve
  - $q_{ult}$ = (0) + (242)(20.6)(1.6) + (0.5)(121)(5)(26.0)(0.6) = 0 + 7976.32 + 4719 = 12,695 psf
- Compute ultimate load
  - $Q_{ult} = q_{ult} * A$
  - (12695)(5)(5) = 317,383 lbs. = 317.3 kips
- Compute Factor of Safety
  - FS = 317.3/76 = 4.17
- It's also possible to do this using the pressures
  - $q_a$ = 76,000/(25) = 3040 psf
  - FS = 12,695/3040 = 4.17

# Effect of Groundwater Table and Layered Soils on Bearing Capacity

- Layered Soils are virtually unavoidable in real geotechnical situations
- Softer layers below the surface can and do significantly affect both the bearing capacity and settlement of foundations
- Pore water pressure increases; reduces both effective stress and shear strength in the soil (same problem as is experienced with unsupported slopes)

- Three ways to analyze layered soil profiles:
  - **Use the lowest of values of shear strength, friction angle and unit weight below the foundation. Simplest but most conservative. Use groundwater factors in conjunction with this.**
  - Use weighted average of these parameters based on relative thicknesses below the foundation. Best balance of conservatism and computational effort. Use width of foundation B as depth for weighted average
  - Consider series of trial surfaces beneath the footing and evaluate the stresses on each surface (similar to slope failure analysis.) Most accurate but calculations are tedious; use only when quality of soil data justify the effort
- Groundwater considered using the groundwater correction factor

# Groundwater Example

- Given
  - Previous example
  - Groundwater table is 3' below base of foundation
- Find
  - Bearing Capacity
- Solution
  - Note that groundwater factor $C_{w\gamma}$ is based on $D_w$, which is distance from surface of *soil* to groundwater table
  - $D_w = 2' + 3' = 5'$
    - Foundation depth plus distance below the base of the foundation

$$q_{ult} = cN_c s_c b_c + qN_q C_{wq} s_q b_q d_q + 0.5\gamma B_f N_\gamma C_{w\gamma} s_\gamma b_\gamma$$

- Solution
  - Values for interpolation (from Table 8-5)
    - $D_w = D_f = 2'$ (base of foundation):
      - $C_{wq} = 1.0$
      - $C_{w\gamma} = 0.5$
    - $D_w = 1.5 * B_f + D_f = (1.5)(5') + 2' = 9.5'$ (bottom of influence zone): = 1.0
      - $C_{wq} = 1.0$
      - $C_{w\gamma} = 1.0$
    - Interpolating:
      - $C_{wq} = 1.0$
      - $C_{w\gamma} = 0.7$
- Substitute and solve
  - $q_{ult}$ = (0) + (242)(20.6)(1.6)(1.0) + (0.5)(121)(5)(26.0)(0.6)(0.7) = 0 + 7976.32 + 3303.3 = 11,280 psf (11% decrease)

# The Other "Workarounds" for Bearing Capacity Theory

- We have already discussed workarounds to the following:
  - Strip footing (shape factors)
  - Level base (base inclination factors)
  - Vertical load (load inclination factors)
  - Homogeneous soil (groundwater factors and related theory)
  - General Shear (modified c and $\varphi$ values)
- Other workarounds we will discuss are as follows:
  - Load eccentricity (load is off centre or there is a moment that accompanies the load
    - Load eccentricity is unavoidable in some circumstances because of the geometry of the structure and the site
    - Eccentricity not only impacts bearing capacity but also the basic stability of the foundation base (foundation liftoff)
  - Footings on top of or on slopes

# Eccentric Loading of Foundations

- Eccentric loading occurs when a footing is subjected to eccentric vertical loads, a combination of vertical loads and moments, or moments induced by shear loads transferred to the footing.
  - Abutments and retaining wall footings are examples of footings subjected to this type of loading condition.
  - Moments can also be applied to interior column footings due to skewed superstructures, impact loads from vessels or ice, seismic loads, or loading in any sort of continuous frame.
- Eccentricity is the distance from the effective point of loading to the centroid of the foundation.
  - This distance can be one-way (strip and circular footings) or two-way (square or rectangular footings)
  - Eccentricity can occur either because the loading is not at the centroid or there is a moment on the foundation.
  - In the case of a moment, the eccentricity is computed by dividing the moment on the foundation by the applied load, i.e., $e = M/P$

# Ways of Accounting for Eccentricity

The concept of an effective area loaded by an equivalent uniform pressure is an approximation made to account for eccentric loading and was first proposed by Meyerhof (1953). Therefore, the equivalent uniform pressure is often referred to as the "**Meyerhof pressure**." The concept of equivalent footing and Meyerhof pressure is used for geotechnical analysis during sizing of the footing, i.e., bearing capacity and settlement analyses. However, the structural design of a footing should be performed using the actual trapezoidal or triangular pressure distributions that model the pressure distribution under an eccentrically loaded footing more conservatively. A comparison of the two loading distributions is shown in Figure 8-17.

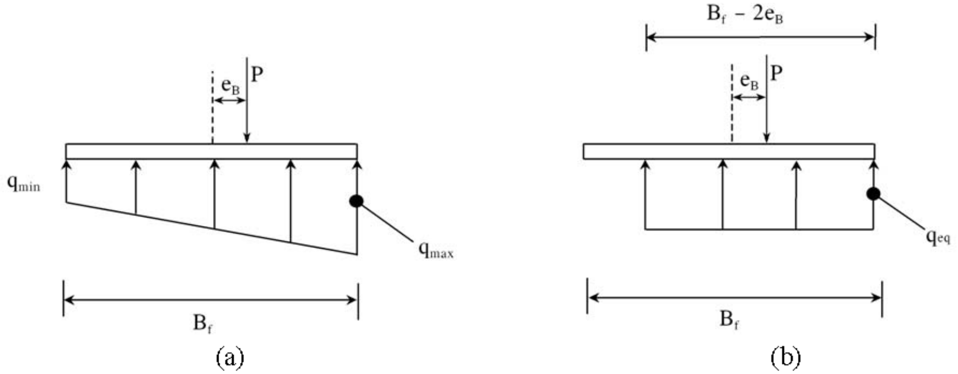



**Figure 8-17. Eccentrically loaded footing with (a) Linearly varying pressure distribution (structural design), (b) Equivalent uniform pressure distribution (sizing the footing).**

Limiting eccentricities are defined to ensure that zero contact pressure does not occur at any point beneath the footing. These limiting eccentricities vary for soil and rock. Footings founded on soil should be designed such that the eccentricity in any direction ($e_B$ or $e_L$) is less than one-sixth (1/6) of the actual footing dimension in the same direction. For footings founded on rock, the eccentricity should be less than one-fourth (1/4) of the actual footing dimension. If the eccentricity does not exceed these limits, a separate calculation for stability with respect to overturning need not be performed. If eccentricity does exceed these limits, the footing should be resized.

# Expressing Load Eccentricity and Inclination

- Load Divided by Inclination Angle
  - Total Load *P*
  - Total Vertical Load $P_v$
  - Total Horizontal Load $P_h$
  - Angle of Inclination $\alpha = \arctan(P_h/P_v)$
  - Load can be concentric or eccentric
- Load and Eccentricity
  - Total Vertical Load *P*
  - Eccentricity from centroid of foundation *e*
  - Horizontal Load (if any) not included
- Moment and Eccentric Load
  - Total eccentric vertical Load *P* with eccentricity *e*
  - Replace with concentric vertical load *P* and eccentric moment $M=Pe$
- Continuous Foundations
  - Moments, loads expressed as per unit length of foundation, thus $P/b$ or $M/b$

# Eccentricity and Equivalent Footing Procedure

#### 8.4.3.1 Footing Shape (Eccentricity and Effective Dimensions)

The following two issues are related to footing shape:

- Distinguishing a strip footing from a rectangular footing. The general bearing capacity equation is applicable to strip footings, i.e., footings with $L_f/B_f \geq 10$. Therefore, footing shape factors should be included in the equation for the ultimate bearing capacity for rectangular footings with $L_f/B_f$ ratios less than 10.

- Use of the effective dimensions of footings subjected to eccentric loads. Eccentric loading occurs when a footing is subjected to eccentric vertical loads, a combination of vertical loads and moments, or moments induced by shear loads transferred to the footing. Abutments and retaining wall footings are examples of footings subjected to this type of loading condition. Moments can also be applied to interior column footings due to skewed superstructures, impact loads from vessels or ice, seismic loads, or loading in any sort of continuous frame. Eccentricity is accounted for by distributing the non-uniform pressure distribution due to the eccentric load as an equivalent uniform pressure over an "effective area" that is smaller than the actual area of the original footing such that the point of application of the eccentric load passes through the centroid of the "effective area." The eccentricity correction is usually applied by reducing the width ($B_f$) and length ($L_f$) such that:

$$B'_f = B_f - 2e_B \qquad 8\text{-}7$$

$$L'_f = L_f - 2e_L \qquad 8\text{-}8$$

where, as shown in Figure 8-16, $e_B$ and $e_L$ are the eccentricities in the $B_f$ and $L_f$ directions, respectively. These eccentricities are computed by dividing the applied moment in each direction by the applied vertical load. It is important to maintain consistent sign conventions and coordinate directions when this conversion is done. The reduced footing dimensions $B'_f$ and $L'_f$ are termed the <u>effective</u> footing dimensions. When eccentric load occurs in both directions, the equivalent uniform bearing pressure is assumed to act over an effective fictitious area, A', where (AASHTO, 2004 with 2006 Interims):

$$A' = B'_f L'_f \qquad 8\text{-}9$$

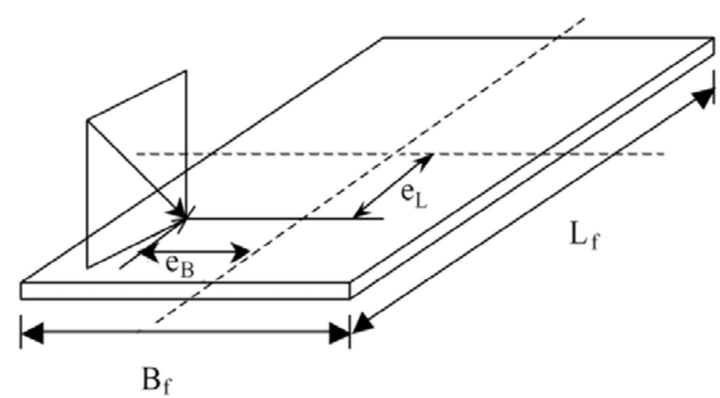
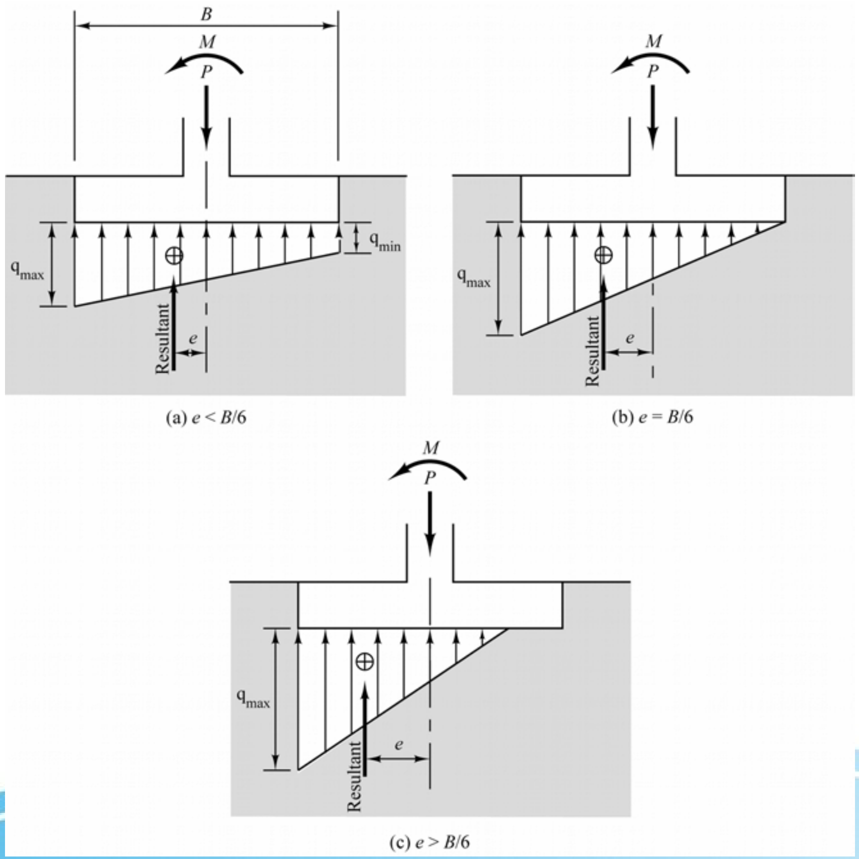



**Figure 8-16. Notations for footings subjected to eccentric, inclined loads (after Kulhawy, 1983).**

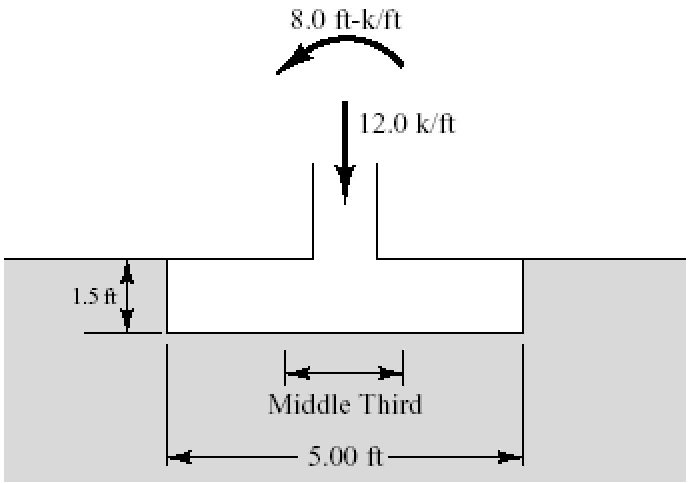
# One Way Loading

- One-way loading is loading along one of the centre axes of the foundation
- Three cases to consider (see right)
- Resultant loads outside the "middle third" result in foundation lift-off and are thus not permitted at all
- After this reduced footing size can be computed



# Example of One-Way Eccentricity

- Given
  - Continuous Foundation as shown
  - Groundwater table at great depth
  - Weight of foundation (concrete) not included in load shown
- Find
  - Whether resultant force acts in middle third
  - Minimum and maximum bearing pressures
  - Reduced bearing area



# Example of One Way Eccentricity

- Compute Weight of Foundation
  - $W_f/b = (5)(1.5)(150) = 1125$ lb/ft
- Compute eccentricity
  - $$e = \frac{(M/b)}{Q/b} = \frac{8000}{12000+1125} = 0.61\,ft.$$

$$\frac{B}{6} = \frac{5}{6} = 0.833\,ft. > 0.61\,ft.$$

  - Thus, eccentricity is within the "middle third" of the foundation and foundation can be analysed further without enlargement at this point

# Equations for One-Way Pressures with Eccentric/Moment Loads

Finite (Square/Rectangular) Footings:

$W_f$ is foundation weight

$$q_{max,min} = \left(\frac{P + W_f}{A}\right)\left(1 \pm \frac{6e}{B}\right)$$

"Infinite" (Continuous) Footings:

If q at any point is less than zero, resultant is outside the middle third

$$q_{max,min} = \left(\frac{\frac{P}{b} + \frac{W_f}{b}}{B}\right)\left(1 \pm \frac{6e}{B}\right)$$

# Example of One Way Eccentricity

- Compute minimum and maximum bearing pressures

$$q_{min} = \left( \frac{\frac{Q}{b} + \frac{W_f}{b}}{B} \right) \left( 1 - \frac{6e}{B} \right)$$

$$q_{min} = \left( \frac{12000 + 1125}{5} \right) \left( 1 - \frac{6 \times 0.61}{5} \right) = 703\,psf$$

$$q_{max} = \left( \frac{\frac{Q}{b} + \frac{W_f}{b}}{B} \right) \left( 1 + \frac{6e}{B} \right)$$

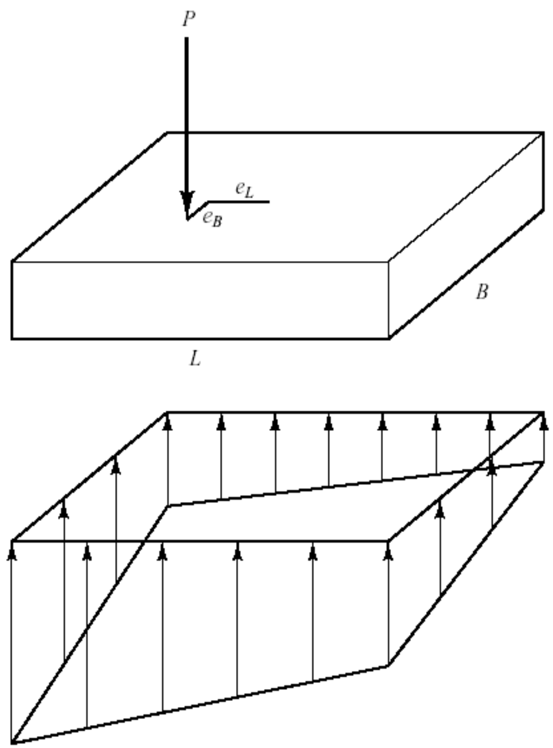
$$q_{max} = \left( \frac{12000 + 1125}{5} \right) \left( 1 + \frac{6 \times 0.61}{5} \right) = 4546\,psf$$

# Example of One-Way Eccentricity

- Since the resultant is within the middle third, we can compute the reduced foundation size B'
- As this is a continuous footing experiencing one-way eccentricity, we do not need to consider an L'
- From previous computations:
  - B = 5'
  - e = 0.61'
- Reduced Foundation Width
  - B' = 5 – (2)(0.61) = 3.78'

# Two-Way Eccentricity

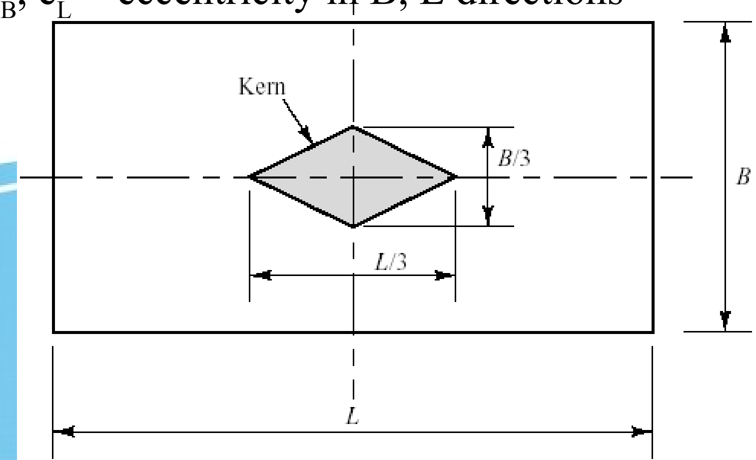
- Eccentricity in both "B" and "L" directions produces a planar distribution of stress



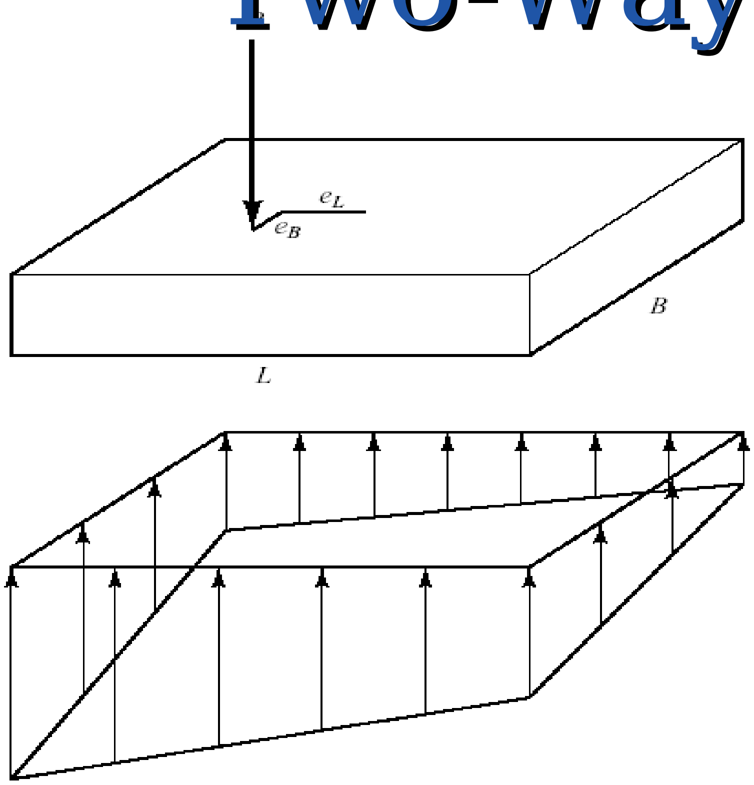

- Kern of Stability
  - Foundation stable against overturn only if resultant falls in the kern in the centre of the foundation
  - Resultant in the kern if

$$\frac{6e_B}{B} + \frac{6e_L}{L} \leq 1$$

$e_B$, $e_L$ = eccentricity in B, L directions



# Bearing Pressure at Corners Two-Way Eccentricity

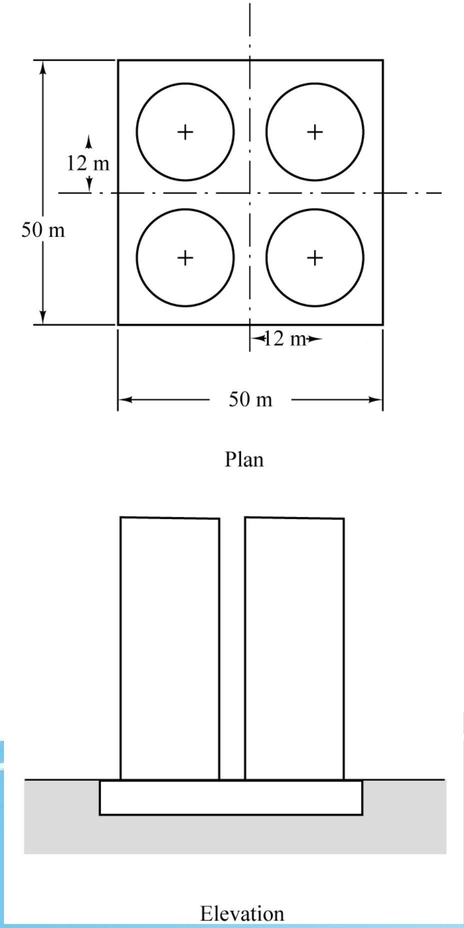



- Helpful hint to prevent confusion of eccentricity of finite vs. infinite (continuous) foundations
  - Always use one-way eccentricity equations for continuous foundations
  - Always use two-way eccentricity equations for finite foundations
  - Two-way equations will reduce to one-way equations if one of the eccentricities ($e_B$, $e_L$) is zero

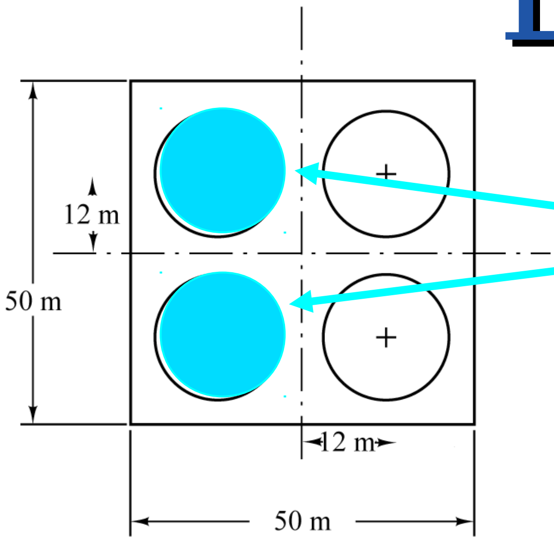
$$q_{1,2,3,4} = \left(\frac{Q+W_f}{BL}\right)\left(1 \pm \frac{6e_B}{B} \pm \frac{6e_L}{L}\right)$$

# Two-Way Eccentricity Example

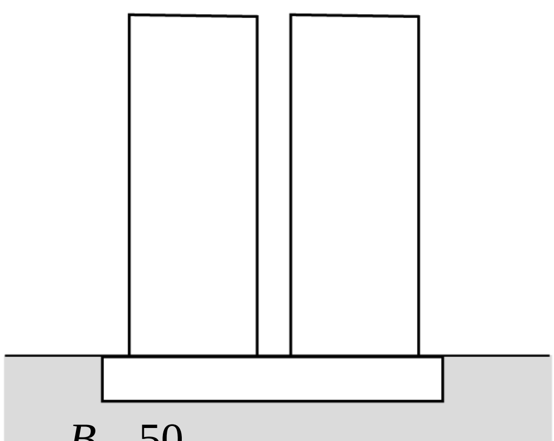
- Given
  - Grain silo design as shown
  - Each silo has an empty weight of 29 MN; can hold up to 110 MN of grain
  - Weight of mat = 60 MN
  - Silos can be loaded independently of each other
- Find
  - Whether or not eccentricity will be met with the various loading conditions possible
  - Eccentricity can be one-way or two-way



# Two-Way Eccentricity Example




Plan



$$\frac{B}{6} = \frac{50}{6} = 8.33\,m > 6.67\,m$$

Elevation

- One-Way Eccentricity
  - Largest Loading: two adjacent silos full and the rest empty
  - Q = (4)(29) + 2(110) + 60 = 396 MN
  - M = (2)(110)(12) = 2640 MN-m

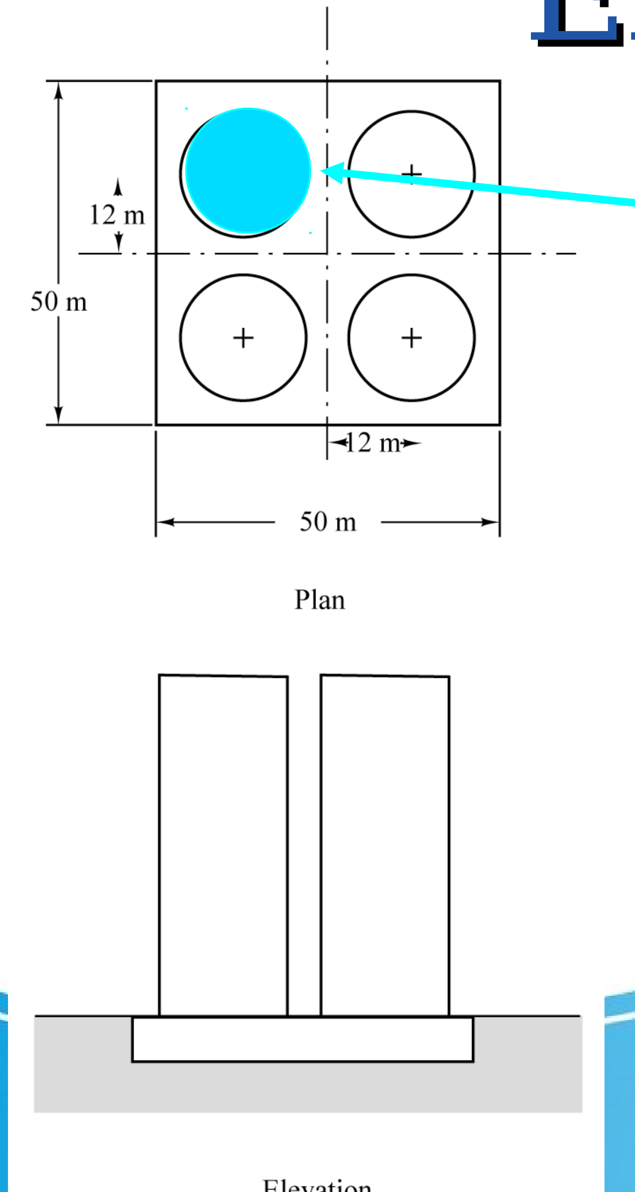
$$e = \frac{M}{Q}$$

$$e = \frac{2640}{396}$$

$$e = 6.67\,m$$

Eccentricity OK for one-way eccentricity

# Two-Way Eccentricity Example



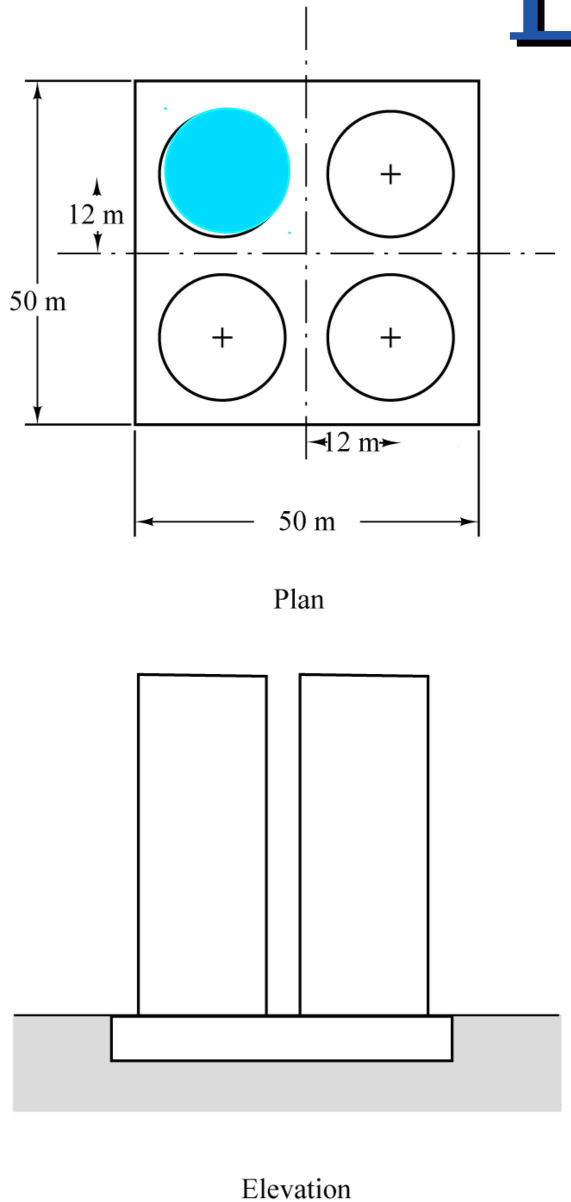

- Two-Way Eccentricity
  - Largest Loading: one silo full and the rest empty
  - P = (4)(29) + 110 + 60 = 286 MN
  - $M_B = M_L = (110)(12) = 1320$ MN-m

$$e_B = e_L = \frac{M}{Q} = \frac{1320}{286} = 4.62\text{m}$$

$$\frac{6e_B}{B} + \frac{6e_L}{L} = 2\left(\frac{(6)(4.62)}{50}\right) = 1.11 > 1$$

Not acceptable

# Two-Way Eccentricity Example



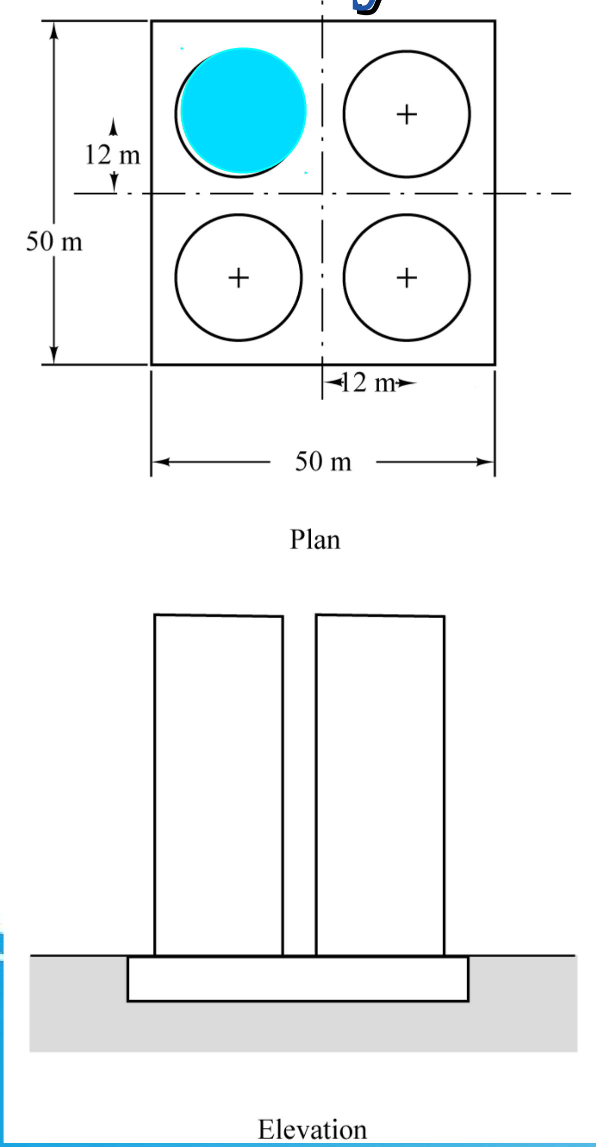

- Two-Way Eccentricity
  - Solution to Eccentricity Problem: increase the size of the mat

$$\frac{6e_B}{B}+\frac{6e_L}{L}=2\left(\frac{(6)(4.62)}{B}\right)=1$$

$$B=L=55.4\,m$$

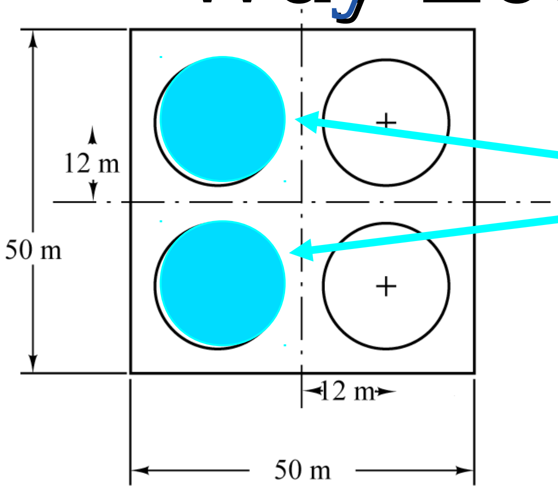
  - Necessary to also take other considerations into account (bearing failure, settlement, etc.)

# Equivalent Footing Using Two-Way Eccentricity Example

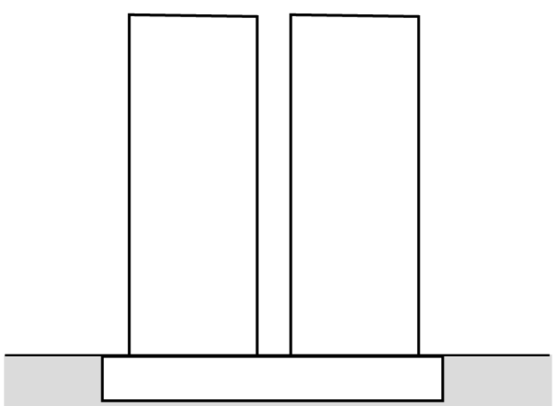



- Largest Loading: one silo full and the rest empty
- Result of Two-Way Eccentricity Analysis
- $e_B = e_L = 4.62$ m
- B = L = 55.4 m (expanded foundation)
- Equivalent Footing Dimensions
- $B' = B - 2e_B = 55.4 - (2)(4.62)$
- $B' = 45.8$ m $= L'$ (as B = L and $e_B = e_L$)

# Equivalent Footing Using Two-Way Eccentricity Example




Plan



$$\frac{B}{6}=\frac{50}{6}=8.33m>6.67m$$

Elevation

- One-Way Eccentricity
  - Largest Loading: two adjacent silos full and the rest empty
  - B = L = 55.4 m (expanded foundation)
  - $e_B$ = 6.67m
  - $e_L$ = 0 m
  - B' = B'-2$e_B$ = 55.4 – (2)(6.67) = 42.1 m
  - L = L' = 55.4 m

# Other Notes on Bearing Capacity Factors

- Two ways to handle B' and L' values when computing shape factors (which are a function of B/L):
  - AASHTO (2002) guidelines recommend calculating the shape factors, s, by using the effective footing dimensions, B′ and L'.
  - However, the original references (e.g., Vesić, 1975) do not specifically recommend using the effective dimensions to calculate the shape factors. Since the geotechnical engineer typically does not have knowledge of the loads causing eccentricity, use full footing dimensions be used to calculate the shape factors for use in computation of ultimate bearing capacity.
  - Either is acceptable for problems in this class. In practice, which one you would use would depend upon a) the project (a highway project would tend to use AASHTO recommendations) and how well the location of the loads was known.
- Bowles (1996) also recommends that the shape and load inclination factors (s and i) should not be combined.
- In certain loading configurations, the designer should be careful in using inclination factors together with shape factors that have been adjusted for eccentricity (Perloff and Baron, 1976). The effect of the inclined loads may already be reflected in the computation of the eccentricity. Thus an overly conservative design may result.

# Bearing Capacity for Foundation at Top of a Slope

**8.4.3.6 Sloping Ground Surface**

Placement of footings on or adjacent to slopes requires that the designer perform calculations to ensure that both the bearing capacity and the overall slope stability are acceptable. The bearing capacity equation should include corrections recommended by AASHTO as adapted from NAVFAC (1986b) to design the footings. Calculation of overall (global) stability is discussed in Chapter 6.

For sloping ground surface, Equation 8-6 is modified to include terms $N_{cq}$ and $N_{\gamma q}$ that replace the $N_c$ and $N_\gamma$ terms. The modified version is given by Equation 8-10. There is no surcharge term in Equation 8-10 because the surcharge effect on the slope side of the footing is ignored.

$$q_{ult} = c(N_{cq}) s_c b_c + 0.5\gamma B_f (N_{\gamma q}) C_{w\gamma} s_\gamma b_\gamma \qquad \text{8-10}$$

Charts are provided in Figure 8-18 to determine $N_{cq}$ and $N_{\gamma q}$ for footings on (Figure 8-18a) or close to (Figure 8-18d) slopes for cohesive ($\phi = 0^o$) and cohesionless ($c = 0$) soils. As indicated in Figure 8-18d, the bearing capacity is independent of the slope angle if the footing is located beyond a distance, 'b,' of two to six times the foundation width, i.e., the situation is identical to the case of horizontal ground surface.

Other forms of Equation 8-10 are available for cohesive soils ($\phi = 0^o$). However, because footings located on or near slopes consisting of cohesive soils, they are likely to have design limitations due to either settlement or slope stability, or both, the presentation of these equations is omitted here. The reader is referred to NAVFAC (1986a, 1986b) for discussions of these equations and their applications and limitations.

Equation 8-10, which includes the width term for cohesionless soils, is useful in designing footings constructed within bridge approach fills. In this case, obtain $N_{\gamma q}$ from Figure 8-18(c) or 8-18(f) and then compute the ultimate bearing capacity by using Equation 8-10.

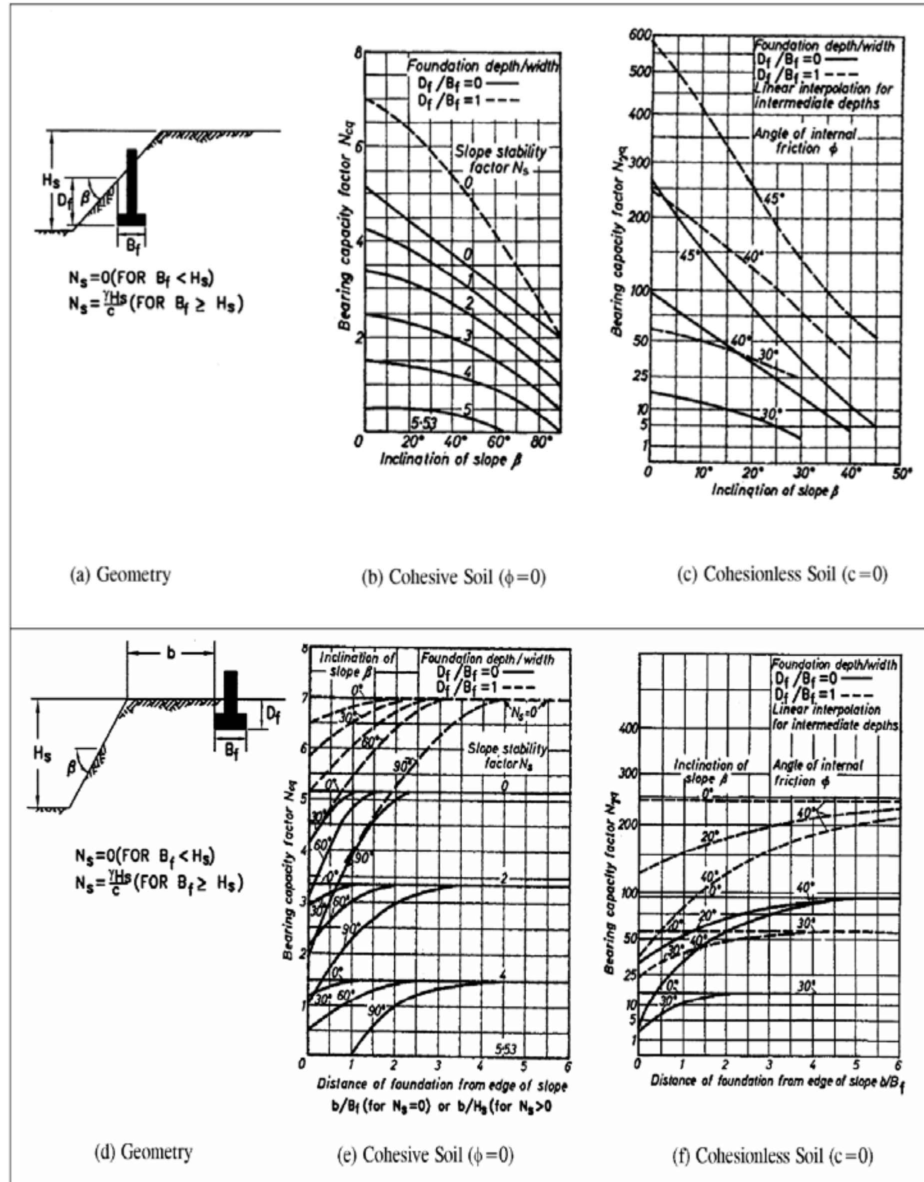



**Figure 8-18. Modified bearing capacity factors for continuous footing on sloping ground (after Meyerhof, 1957, from AASHTO, 2004 with 2006 Interims)**

# Example of Footings on Slopes

- Given
  - Strip footing to be constructed on top of the slope
  - Soil properties: c = 75 kPa, $\gamma = 18.5$ kN/m$^3$, water table very deep
  - H = 8 m, B = 3 m, D = 1.5 m, b = 2 m, Slope Angle = 30 deg.
- Find
  - Ultimate Bearing Capacity of Footing, using solution method of previous slide
- Solution
  - B < H since 3 m < 8 m
  - Obtain $N_{cq}$ from Figure 8-18(e) for Case I with $N_o = 0$
  - D/B = 1.5/3 = 0.5
  - b/B = 2/3 = 0.667
- Solution
  - $N_{cq}$ for D/B = 0 and Slope Angle of 30 deg. = 4.9
  - $N_{cq}$ for D/B = 1 and Slope Angle of 30 deg. = 6.4
  - Linearly interpolating, $N_{cq}$ = (6.4+4.9)/2 = 5.7
  - $N_{\gamma q}$ = 1 since the soil is purely cohesive
  - B/2 = D
  - Shape factors are unity because it is a continuous footing; water table and embedment factors are like wise not considered
    - $q_{ult}$ = (75)(5.7)+(1.5)(18.5) = 455.25 kPa

# Required Footing Setbacks

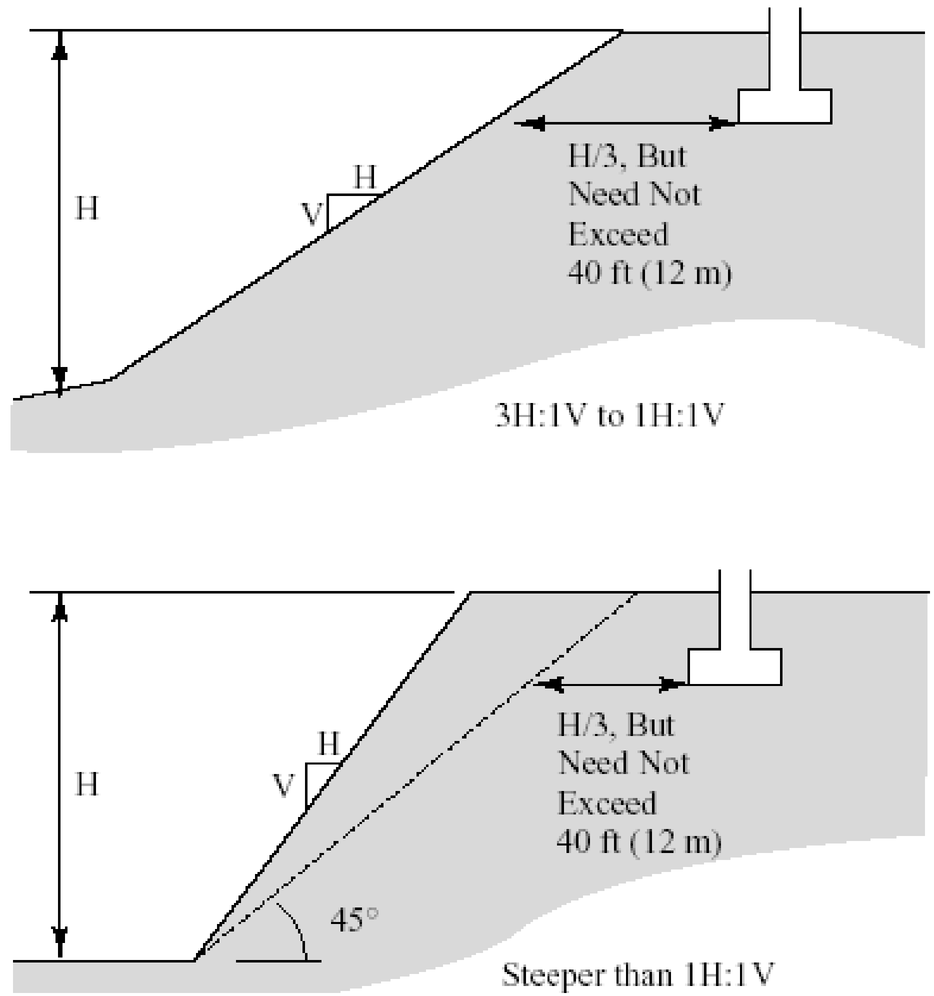



**Figure 8.14** Footing setback as required by the Uniform Building Code [1806.5] and the International Building Code [1805.3] for slopes steeper than 3 horizontal to 1 vertical. The horizontal distance from the footing to the face of the slope should be at least *H*/3, but need not exceed 40 ft (12 m). For slopes that are steeper than 1 horizontal to 1 vertical, this setback distance should be measured from a line that extends from the toe of the slope at an angle of 45°. (Adapted from the 1997 edition of the *Uniform Building Code*, © 1997, with the permission of the publisher, the International Conference of Building Officials and the 2000 edition of the International Building Code).

# Practical Aspects of Bearing Capacity Formulations

**8.4.7 Practical Aspects of Bearing Capacity Formulations**

This section presents some useful practical aspects of bearing capacity formulations. Several interesting observations are made here that provide practical guidance in terms of implementation and interpretation of the bearing capacity formulation and computed results.

**8.4.7.1 Bearing Capacity Computations**

The procedure to be used to compute bearing capacity is as follows:

1. Review the structural plans to determine the proposed footing widths. In the absence of data assume a pier footing width equal to 1/3 the pier column height and an abutment footing width equal to 1/2 the abutment height.

2. Review the soil profile to determine the position of the groundwater table and the interfaces between soil layer(s) that exist within the appropriate depth below the proposed footing level.

3. Review soil test data to determine the unit weight, friction angle and cohesion of all of the impacted soils. In the absence of test data, estimate these values for coarse-grained granular soils from SPT N-values (refer to Table 8-3). NOTE SPT N-values in cohesive soils should not be used to determine shear strengths for final design since the reliability of SPT N-values in such soils is poor.

4. Use Equation 8-6 with appropriate correction factors to compute the ultimate bearing capacity. The general case (continuous footing) may be used when the footing length is 10 or more times the footing width. Also the bearing capacity factor $N_\gamma$ will usually be determined for a rough base condition since most footings are poured concrete. However the smoothness of the contact material must be considered for temporary footings such as wood grillages (rough), or steel supports (smooth) or plastic sheets (smooth). The safety factor for the bearing capacity of a spread footing is selected both to limit the amount of soil strain and to account for variations in soil properties at footing locations.

5. The mechanism of the general bearing capacity failure is similar to the embankment slope failure mechanism. However, the footing analysis is a 3-dimensional analysis as opposed to the 2-dimensional slope stability analysis. The bearing capacity factors $N_c$, $N_q$ and $N_\gamma$ relate to the actual volume of soil involved in the failure zones. A cursory study of the failure cross sections in Figure 8-13, discloses that the depth and lateral extent of the failure zones and the values of $N_c$, $N_q$ and $N_\gamma$ are determined by the dimensions of the wedge-shaped zone directly below the footing. As the friction angle increases, the depth and width of the failure zones increase, i.e., more soil is impacted and more shear resistance is mobilized, thereby increasing the bearing capacity.

6. Substantial downward movement of the footing is required to mobilize the shearing resistance within the entire failure zone completely. Besides providing a margin of safety on shear strength properties, the relatively large safety factor of 3 commonly used in the design of footings controls the amount of strain necessary to mobilize the allowable bearing capacity fully. Settlement analysis (Section 8.5) is recommended to compute the allowable bearing capacity corresponding to a specified limiting settlement. That allowable bearing capacity may result in a factor of safety with respect to ultimate bearing capacity much larger than 3.

7. In reporting the results of bearing capacity analyses, the footing width that was used to compute the bearing capacity should always be included. Most often the geotechnical engineer must assume a footing width since bearing capacity analyses are completed before structural design begins. It is recommended that bearing capacity be computed for a range of possible footing widths and those values be included in the foundation report with a note stating that if other footing widths are used, the geotechnical engineer should be contacted. The state of the practice today is for the geotechnical engineer to develop location-specific bearing capacity charts on which allowable bearing capacity is plotted versus footing width for a family of curves representing specific values of settlement. Refer to Figure 8-10 for a schematic example of such a chart.

8. The **net** ultimate bearing pressure is the difference between the gross ultimate bearing pressure and the pressure that existed due to the ground surcharge at the bearing depth before the footing was constructed, q (= $\gamma_a D_f$). The net ultimate bearing pressure can thus be computed by subtracting the ground surcharge (q) from Equation 8-6:

$$q_{ult\,net} = q_{ult} - q \qquad \text{8-14}$$

$$q_{ult\,net} = cN_c s_c b_c + q(N_q - 1)C_{wq} s_q b_q d_q + 0.5\gamma B_f N_\gamma C_{w\gamma} s_\gamma b_\gamma \qquad \text{8-15}$$

The structural designer will typically include the self-weight of the concrete footing and the backfill over the footing (approximately equal to $\gamma_a D_f$) in the loads that contribute to the applied bearing stress. Therefore, if the geotechnical engineer computes and reports a net ultimate bearing pressure, the effect of the surcharge directly over the footing area is counted twice. Reporting an allowable bearing capacity computed from a net ultimate bearing pressure is conservative and generally not recommended provided that a suitable factor of safety is maintained against bearing capacity failure. If the geotechnical engineer chooses to report an allowable bearing capacity computed from a net ultimate bearing pressure, this fact should be clearly stated in the foundation report.

# Failure Zones for Bearing Capacity

**8.4.7.2 Failure Zones**

Certain practical information based on the geometry of the failure zone is as follows:

1. The bearing capacity of a footing is dependent on the strength of the soil within a depth of approximately 1.5 times footing width below the base of the footing unless much weaker soils exist just below this level, in which case a potential for punching shear failure may exist. Continuous soil samples and SPT N-values should be routinely specified within this depth. If the borings for a structure are done long before design, a good practice is to obtain continuous split spoon samples for the top 15 ft (4.5 m) of each boring where footings may be placed on natural soil. The cost of this sampling is minimal but the knowledge gained is great. At a minimum, continuous sampling to a depth of 15 ft (4.5 m) will generally provide the following information:

   a. thickness of existing topsoil.
   b. location of any thin zones of unsuitable material.
   c. accurate determination of depth of existing fill.
   d. improved ground water determination in the critical zone.
   e. representative samples in this critical zone to permit reliable determination of strength parameters in the laboratory and confident assessment of bearing capacity.

2. Often questions arise during excavation near existing footings as to the effect of soil removal adjacent to the footing on the bearing capacity of that footing. In general, for weaker soils the zone of lateral influence extends outside the footing edge less than twice the footing width. Reductions in bearing capacity can be estimated by considering the effects of surcharge removal within these zones. The theoretical lateral extent of this zone is shown in Figure 8-20. This figure is also useful in determining the effects of ground irregularities on bearing capacity or the effects of footing loads on adjacent facilities.

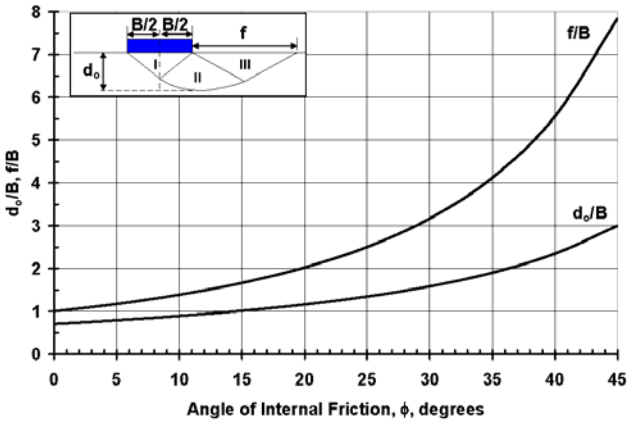



**Figure 8-20. Approximate variation of depth ($d_o$) and lateral extent (f) of influence of footing as a function of internal friction angle of foundation soil.**

As noted earlier, the general mechanism by which soils resist a footing load is similar to the foundation of an embankment resists shear failure. The load to cause failure must exceed the available soil strength within the failure zone. When failure occurs the footing plunges into the ground and causes an uplift of the soil adjacent to the sides of the footing. The resistance to failure is based on the soil strength and the amount of soil above the footing. Therefore, the bearing capacity of a footing can be increased by:

1. replacing or densifying the soil below the footing prior to construction.

2. increasing the embedment of the footing below ground, provided no weak soils exist within 1.5 times the footing width.

Common examples of improving bearing capacity are the support of temporary footings on pads of gravel or the embedment of mudsills a few feet below ground to support falsework. The design of these support systems is primarily done by bearing capacity analysis in which the results of subsurface explorations and testing are used. Structural engineers who review falsework designs should carefully check the soil bearing capacity at foundation locations.

# Presumptive Bearing Capacity

### 8.4.8 Presumptive Bearing Capacities

Many building codes include provisions that arbitrarily limit the amount of loading that may be applied on various classes of soils by structures subject to code regulations. These limiting loads are generally based on bearing pressures that have been observed to result in acceptable settlements. The implication is that on the basis of experience alone it may be presumed that each designated class of soil will safely support the loads indicated without the structure undergoing excessive settlements. Such values listed in codes or in the technical literature are termed presumptive bearing capacities.

#### 8.4.8.1 Presumptive Bearing Capacity in Soil

**The use of presumptive bearing capacities for shallow foundations bearing in soils is not recommended for final design of shallow foundations for transportation structures, especially bridges**. Guesses about the geology and nature of a site and the application of a presumptive value from generalizations in codes or in the technical literature are not a substitute for an adequate site-specific subsurface investigation and laboratory testing program. As an exception, presumptive bearing values are sometimes used for the preliminary evaluation of shallow foundation feasibility and estimation of footing dimensions for preliminary constructability or cost evaluations.

#### 8.4.8.2 Presumptive Bearing Capacity in Rock

Footings on intact sound rock that is stronger and less compressible than concrete are generally stable and do not require extensive study of the strength and compressibility characteristics of the rock. However, site investigations are still required to confirm the consistency and extent of rock formations beneath a shallow foundation.

Allowable bearing capacities for footings on relatively uniform and sound rock surfaces are documented in applicable building codes and engineering manuals. Many different definitions for sound rock are available. **In simple terms, however, "sound rock" can generally be defined as a rock mass that does not disintegrate after exposure to air or water and whose discontinuities are unweathered, closed or tight, i.e., less than about 1/8 in (3 mm) wide and spaced no closer than 3 ft (1 m) apart.** Table 8-8 presents allowable bearing pressures for intact rock recommended in selected local building codes (Goodman, 1989). These values were developed based on experience in sound rock formations, with the intention of satisfying both bearing capacity and settlement criteria in order to provide a satisfactory factor of safety. However, the use of presumptive values may lead to overly conservative and costly foundations. In such cases, most codes allow for a variance if the request is supported by an engineering report. Site-specific investigation and analysis is strongly encouraged.

In areas where building codes are not available or applicable, other recommended presumptive bearing values, such as those listed in Table 8-9, may be used to determine the allowable bearing pressure for sound rock. For footings designed by using these published values, the elastic settlements are generally less than 0.5 in (13 mm). Where the rock is reasonably sound, but fractured, the presumptive values listed in Tables 8-8 and 8-9 should be reduced by limiting the bearing pressures to tolerable settlements based on settlement analyses. Most building codes also provide reduced recommended bearing pressures to account for the degree of fracturing.

Peck, *et al.* (1974) presented an empirical correlation of presumptive allowable bearing pressure with Rock Quality Designation (RQD), as shown in Table 8-10. If the recommended value of allowable bearing pressure exceeds the unconfined compressive strength of the rock or allowable stress of concrete, the allowable bearing pressure should be taken as the lower of the two values. Although the suggested bearing values of Peck, *et al.* (1974) are substantially greater than most of the other published values and ignore the effects of rock type and conditions of discontinuities, they provide a useful guide for an upper-bound estimation as well as an empirical relationship between allowable bearing values and the intensity of fracturing and jointing (Table 8-10). Note that with a slight increase of the degree of fracturing of the rock mass, for example when the RQD value drops from 100 percent to 90 percent, the recommended bearing capacity value is reduced drastically from 600 ksf (29 MPa) to 400 ksf (19 MPa).

In no instance should the allowable bearing capacity exceed the allowable stress of the concrete used in the structural foundation. Furthermore, Peck, *et al.* (1974) also suggest that the average RQD for the bearing rock within a depth of the footing width ($B_f$) below the base of the footing should be used if the RQD values within the depth are relatively uniform. If rock within a depth of $0.5B_f$ is of poorer quality, the RQD of the poorer quality rock should be used to determine the allowable bearing capacity.

# Presumptive Bearing Capacity on Rock

**Table 8-8**

**Allowable bearing pressures for fresh rock of various types (Goodman, 1989)**

| Rock Type | Age | Location | Allowable Bearing Pressure tsf (MPa) |
|---|---|---|---|
| Massively bedded limestone[5] | | U.K.[6] | 80 (3.8) |
| Dolomite | L. Paleoz. | Chicago | 100 (4.8) |
| Dolomite | L. Paleoz. | Detroit | 20-200 (1.0 – 9.6) |
| Limestone | U. Paleoz. | Kansas City | 20-120 (0.5 – 5.8) |
| Limestone | U. Paleoz. | St. Louis | 50-100 (2.4 – 4.8) |
| Mica schist | Pre-Camb. | Washington | 20-40 (0.5 – 1.9) |
| Mica schist | Pre-Camb. | Philadelphia | 60-80 (2.9 – 3.8) |
| Manhattan schist | Pre-Camb. | New York | 120 (5.8) |
| Fordham gneiss | Pre-Camb. | New York | 120 (5.8) |
| Schist and slate | - | U.K.[6] | 10-25 (0.5 – 1.2) |
| Argillite | Pre-Camb. | Cambridge, MA | 10-25 (0.5 – 1.2) |
| Newark shale | Triassic | Philadelphia | 10-25 (0.5 – 1.2) |
| Hard, cemented shale | - | U.K.[6] | 40 (1.9) |
| Eagleford shale | Cretaceous | Dallas | 13-40 (0.6 – 1.9) |
| Clay shale | - | U.K.[6] | 20 (1.0) |
| Pierre shale | Cretaceous | Denver | 20-60 (1.0 – 2.9) |
| Fox Hills sandstone | Tertiary | Denver | 20-60 (1.0 – 2.9) |
| Solid chalk | Cretaceous | U.K.[6] | 13 (0.6) |
| Austin chalk | Cretaceous | Dallas | 30-100 (1.4 – 4.8) |
| Friable sandstone and claystone | Tertiary | Oakland | 8-20 (0.4 – 1.0) |
| Friable sandstone (Pico formation) | Quaternary | Los Angeles | 10-20 (0.5 – 1.0) |

*Notes:*

[1] According to typical building codes; reduce values accordingly to account for weathering or unrepresentative fracturing

[2] Values from Thorburn (1966) and Woodward, Gardner and Greer (1972).

[3] When a range is given, it relates to usual range in rock conditions.

[4] Sound rock that rings when struck and does not disintegrate. Cracks are unweathered and open less than 10 mm.

[5] Thickness of beds greater than 3 ft (1 m), joint spacing greater than 2 mm; unconfined compressive strength greater than 160 tsf (7.7 MPa) (for a 4 in (100 mm) cube).

[6] Institution of Civil Engineers Code of Practice 4.

**Table 8-9**

**Presumptive values of allowable bearing pressures for spread foundations on rock (modified after NAVFAC, 1986a, AASHTO 2004 with 2006 Interims)**

| Type of Bearing Material | Consistency In Place | Allowable Bearing Pressure tsf (MPa) | |
|---|---|---|---|
| | | Range | Recommended Value for Use |
| Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks) | Hard, sound rock | 120-200 (5.8 - 9.6) | 160 (7.7) |
| Foliated metamorphic rock: Slate, schist (sound condition allows minor cracks) | Medium-hard, sound rock | 60-80 (2.9-3.8) | 70 (3.4) |
| Sedimentary rock; hard cemented shales, siltstone, sandstone, limestone without cavities | Medium-hard, sound rock | 30-50 (1.4-2.4) | 40 (1.9) |
| Weathered or broken bedrock of any kind except highly argillaceous rock (shale). RQD less than 25 | Soft rock | 16-24 (0.8-1.2) | 20 (1) |
| Compacted shale or other highly argillaceous rock in sound condition | Soft rock | 16-24 (0.8-1.2) | 20 (1) |

*Notes:*

1. For preliminary analysis or in the absence of strength tests, design and proportion shallow foundations to distribute their loads by using presumptive values of allowable bearing pressure given in this table. Modify the nominal value of allowable bearing pressure for special conditions described in notes 2 through 8.
2. The maximum bearing pressure beneath the footing produced by eccentric loads that include dead plus normal live load plus permanent lateral loads shall not exceed the above nominal bearing pressure.
3. Bearing pressures up to one-third in excess of the nominal bearing values are permitted for transient live load from wind or earthquake. If overload from wind or earthquake exceeds one-third of nominal bearing pressures, increase allowable bearing pressures by one-third of nominal value.
4. Extend footings on soft rock to a minimum depth of 1.5 in (40 mm) below adjacent ground surface or surface of adjacent floor, whichever elevation is the lowest.
5. For footings on soft rock, increase allowable bearing pressures by 5 percent of the nominal values for each 1 ft (300 mm) of depth below the minimum depth specified in Note 4.
6. Apply the nominal bearing pressures of the three categories of hard or medium hard rock shown above where the base of the foundation lies on rock surface. Where the foundation extends below the rock surface, increase the allowable bearing pressure by 10 percent of the nominal values for each additional 1ft (300 mm) of depth extending below the surface.
7. For footings smaller than 3 ft (1 m) in the least lateral dimension, the allowable bearing pressure shall be the nominal bearing pressure multiplied by the least lateral dimension.
8. If the above-recommended nominal bearing pressure exceeds the unconfined compressive strength of intact specimen, the allowable pressure equals the unconfined compressive strength.

**Table 8-10**

**Suggested values of allowable bearing capacity (Peck, *et al.*, 1974)**

| RQD (%) | Rock Mass Quality | Allowable Pressure ksf (MPa) |
|---|---|---|
| 100 | Excellent | 600 (29) |
| 90 | Good | 400 (19) |
| 75 | Fair | 240 (12) |
| 50 | Poor | 130 (6) |
| 25 | Very Poor | 60 (3) |
| 0 | Soil-like | 20 (1) |

# Questions?