Introduction to Bearing Capacity Analysis

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An Introduction To Bearing Capacity Analysis

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CONTENTS

(The figures, tables and formulas in this publication may at times be a little difficult to read, but they are the best available. **DO NOT PURCHASE THIS PUBLICATION IF THIS LIMITATION IS NOT ACCEPTABLE TO YOU**.)

1. BEARING CAPACITY OF SOILS. Stresses transmitted by a foundation to underlying soils must not cause bearing-capacity failure or excessive foundation settlement. The design bearing pressure equals the ultimate bearing capacity divided by a suitable factor of safety. The ultimate bearing capacity is the loading intensity that causes failure and lateral displacement of foundation materials and rapid settlement. The ultimate bearing capacity depends on the size and shape of the loaded area, the depth of the loaded area below the ground surface, groundwater conditions, the type and strength of foundation materials, and the manner in which the load is applied. Allowable bearing pressures may be estimated from Table 1 on the basis of a description of foundation materials. Bearing-capacity analyses are summarized below.

2. SHEAR STRENGTH PARAMETERS.

2.1 APPROPRIATE ANALYSES. Bearing-capacity calculations assume that strength parameters for foundation soils are accurately known within the depth of influence of the footing. The depth is generally about 2 to 4 times the footing width but is deeper if subsoils are highly compressible.

2.1.1 COHESIONLESS SOILS. Estimate Φ' from the Standard Penetration Test or the cone penetration resistance. For conservative values, use Φ' = 30 degrees.

2.1.2 COHESIVE SOILS. For a short-term analysis, estimate s_{u1}, from the Standard Penetration Test or the vane shear resistance. For long-term loadings, estimate Φ' from correlations with index properties for normally consolidated soils.

2.2 DETAILED ANALYSES.

2.2.1 COHESIONLESS SOILS. Determine Φ' from drained (S) triaxial tests on undisturbed samples from test pits or borings.

2.2.2 COHESIVE SOILS. For a short-term analysis, determine s, from Q triaxial tests on undisturbed samples. For a long-term analysis, obtain Φ' from drained direct shear (S) tests on undisturbed samples. For transient loadings after consolidation, obtain f and c parameters from consolidated-undrained (R) triaxial tests with pore pressure measurements on undisturbed samples. If the soil is dilative, the strength should be determined from drained S tests.

3. METHODS OF ANALYSIS.

3.1 SHALLOW FOUNDATIONS.

3.1.1 GROUNDWATER LEVEL (GWL). The ultimate bearing capacity of shallow foundations subjected to vertical, eccentric loads can be computed by means of the formulas shown in Figure 1. For a groundwater level well below the bottom of the footing, use a moist unit soil weight in the equations given in Figure 1. If the groundwater level is at ground surface, use a submerged unit soil weight in the equations.

3.1.2 INTERMEDIATE GROUNDWATER LEVELS. Where the groundwater level is neither at the surface nor so deep, so as not to influence the ultimate bearing capacity, use graphs and equations given in Figure 2.

3.1.3 ECCENTRIC OR INCLINED FOOTING LOADS. In practice, many structure foundations are subjected to horizontal thrust and bending moment in addition to vertical loading. The effect of these loadings is accounted for by substituting equivalent eccentric and/or inclined loads. Bearing capacity formulas for this condition are shown in Figure 3. An example of the method for computing the ultimate bearing capacity for an eccentric inclined load on a footing is shown in Figure 4.

3.1.4 LOADING COMBINATIONS AND SAFETY FACTORS. The ultimate bearing capacity should be determined for all combinations of simultaneous loadings. A distinction is made between normal and maximum live load in bearing capacity computations. The normal live load is that part of the total live load that acts on the foundation at least once a year; the maximum live load acts only during the simultaneous occurrence of several exceptional events during the design life of the structure. A minimum factor of safety of 2.0 to 3.0 is required for dead load plus normal live load, and 1.5 for dead load plus maximum live load. Safety factors selected should be based on the extent of loadings and consequences of failure. Also, high safety factors should be selected if settlement estimates are not made. In general, separate settlement analysis should be made.

3.2 DEEP FOUNDATIONS. Methods for computing the ultimate bearing capacity of deep foundations are summarized in Figure 5. These analyses are applicable to the design of deep piers and pile foundations, as subsequently described. When the base of the foundation is located below the ground surface at a depth greater than the width of the foundation, the factor of safety should be applied to the net load (total weight of structure minus weight of displaced soil).

3.3 STRATIFIED SUBSOILS. Where subsoils are variable with depth, the average shear strength within a depth below the base equal to the width of the loaded area controls the bearing capacity, provided the strength at a depth equal to the width of the loaded area or lower is not less than one-third the average shear strength of the upper layer; otherwise, the bearing capacity is governed by the weaker lower layer. For stratified cohesive soils, calculate the ultimate bearing capacity from the chart in Figure 6. The bearing pressure on the weaker lower layer can be calculated by distributing the surface load to the lower layer at an angle of 30 degrees to the vertical.

(These presumed values of the allowable bearing pressure are estimates and may need alteration upwards or downwards. No addition has been made for the depth of embedment of the foundation. Reference should be made to other parts of the Manual when using this table.)

Group	Types and conditions of rocks and soils	Strength of Rock Material	Presumed Allowable Bearing Pressure Ton /sq ft	Remarks
Rocks	Massive igneous and metamorphic rocks (granite, diorite, basalt, gneiss) in sound condition[a]	High to very high	100	These values are based on the assumption that the foundations are carried down to unweathered rock.
	Foliated metamorphic rocks (slate, schist) in sound condition[a,b]	Medium to high	30	
	Sedimentary rocks: cemented shale, siltstone, sandstone, limestone without cavities, thoroughly cemented conglomerates, all in sound condition[a,b]	Medium to high	10–40	
	Compaction shale and other argillaceous rocks in sound condition[a,d]	Low to medium	5	
	Broken rocks of any kind with moderately close spacing of discontinuities (1 ft or greater), except argillaceous rocks (shale)		10	
	Thinly bedded limestone, sandstones, shale		See note c	
	Heavily shattered or weathered rocks		See note c	
Non-cohesive soils	Dense gravel or dense sand and gravel		>6	Width of foundation (B) not less than 3 ft. Groundwater level assumed to be at a depth not less than B below the base of the foundation.
	Compact gravel or compact sand and gravel		2–6	
	Loose gravel or loose sand and gravel		<2	
	Dense sand		>3	
	Compact sand		1–3	
	Loose sand		<1	
Cohesive soils	Very stiff to hard clays or heterogeneous mixtures such as till		3–6	Cohesive soils are susceptible to long-term consolidation settlement
	Stiff clays		1.5–3	
	Firm clays		0.75–1.5	
	Soft clays and silts		<0.75	
	Very soft clays and silts		not applicable	
Organic soils	Peat and organic soils		not applicable	
Fill	Fill		not applicable	

[a] Sound rock conditions allow minor cracks at spacing not less than 3 ft.

[b] The above values for sedimentary or foliated rocks apply where the strata or foliation are level or nearly so, and, then only if the area has ample lateral support. Tilted strata and their relation to nearby slopes or excavations shall be assessed by a person knowledgeable in this field of work.

[c] To be assessed by examination in situ, including loading tests if necessary, by a person knowledgeable in this field of work.

[d] These rocks are apt to swell on release of stress, and on exposure to water they are apt to soften and swell appreciably.

Table 1

Estimates of allowable bearing pressure

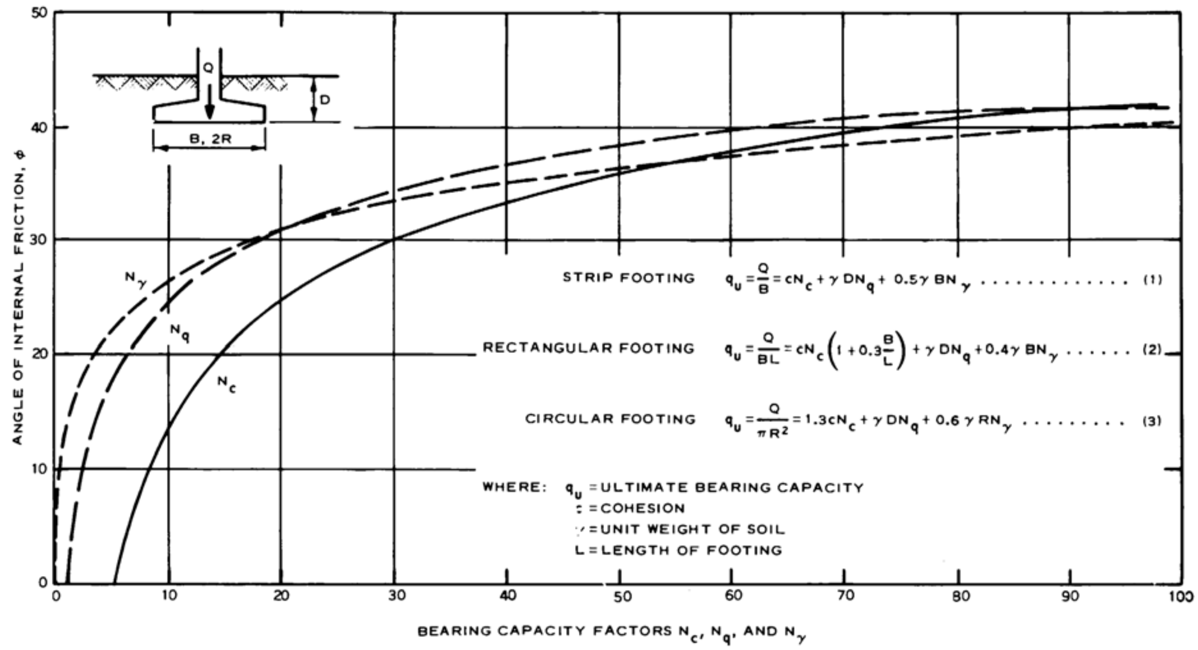
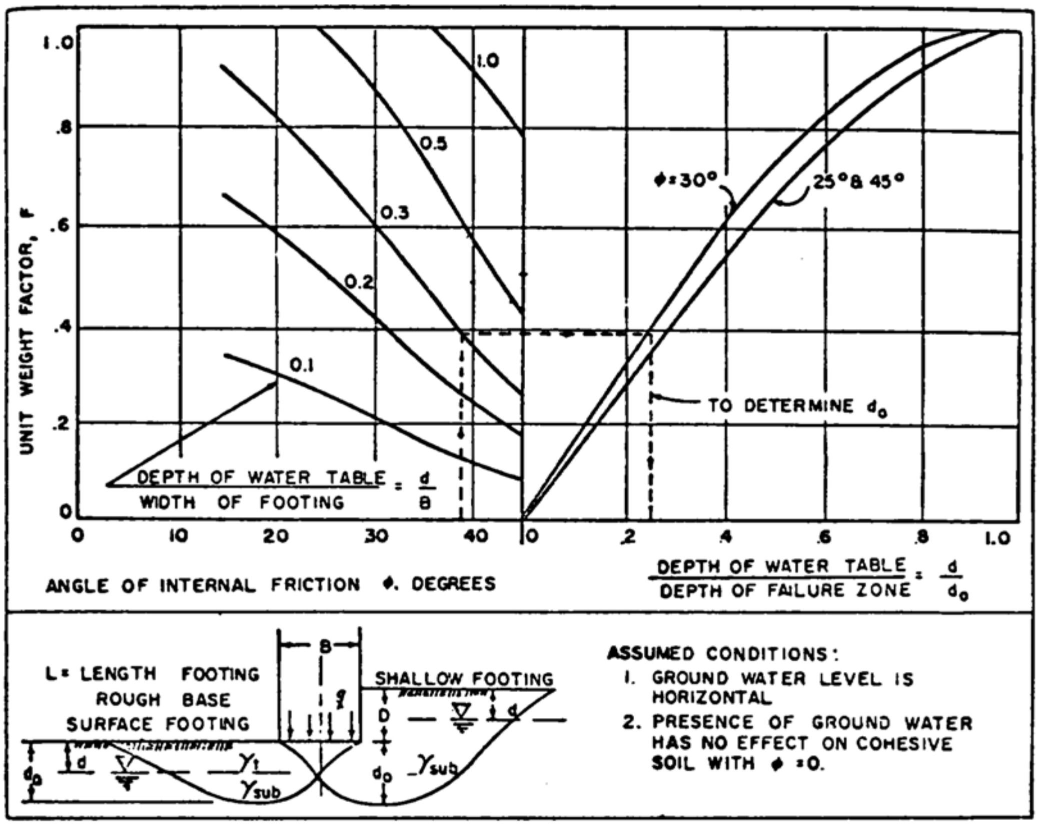


Figure 1

Ultimate bearing capacity of shallow foundations under vertical eccentric loads



CONTINUOUS FOOTING:

SURFACE FOOTING: D = O

$q_{ult} = cN_c + \gamma_{SUB} + F(\gamma_T - \gamma_{SUB}) \; \frac{B}{2} N_\gamma$

SHALLOW FOOTING: $D < B$

IF $d < D$

$q_{ult} = cN_c + \gamma_{SUB} D + (\gamma_T - \gamma_{SUB}) d \;\; N_q + 0.5 \gamma_{SUB} B N_\gamma$

IF $D < d < (D + d_o)$

$q_{ult} = cN_c + \gamma_T D N_q + \gamma_{SUB} + F(\gamma_T - \gamma_{SUB}) \; \frac{B}{2} N_\gamma$

RECTANGULAR FOOTING:

SURFACE FOOTING: D = O

$q_{ult} = cN_c (1 + 0.3\frac{B}{L}) + \gamma_{SUB} + F(\gamma - \gamma_{SUB}) \;\; 0.4 B N_\gamma$

SHALLOW FOOTING: $D < B$. IF $d < D$

$q_{ult} = cN_c (1 + 0.3\frac{B}{L}) + \gamma_{SUB} D + (\gamma_T - \gamma_{SUB}) d \;\; N_q + 0.4 \gamma_{SUB} B N_\gamma$

IF $D < d < (D + d_o)$

$q_{ult} = cN_c (1 + 0.3\frac{B}{L}) + \gamma_T D N_q + \gamma_{SUB} + F(\gamma_T - \gamma_{SUB}) \;\; 0.4 B N_\gamma$

CIRCULAR FOOTING: RADIUS = R = B/2

SURFACE FOOTING: D = O

$q_{ult} = 1.3 cN_c + \gamma_{SUB} + F(\gamma_T - \gamma_{SUB}) \;\; 0.6 R N_\gamma$

SHALLOW FOOTING: $D < 2R$, IF $d < D$

$q_{ult} = 1.3\, cN_c + \gamma_T - \gamma_{SUB} D + (\gamma_T - \gamma_{SUB}) d \;\; N_q + 0.6 \gamma_{SUB} R N_\gamma$

IF $D < d < (D + d_o)$

$q_{ult} = 1.3 cN_c + \gamma_T D N_q + \gamma_{SUB} + F(\gamma_T - \gamma_{SUB}) \;\; 0.6 R N_\gamma$

Figure 2

Ultimate bearing capacity with groundwater effect

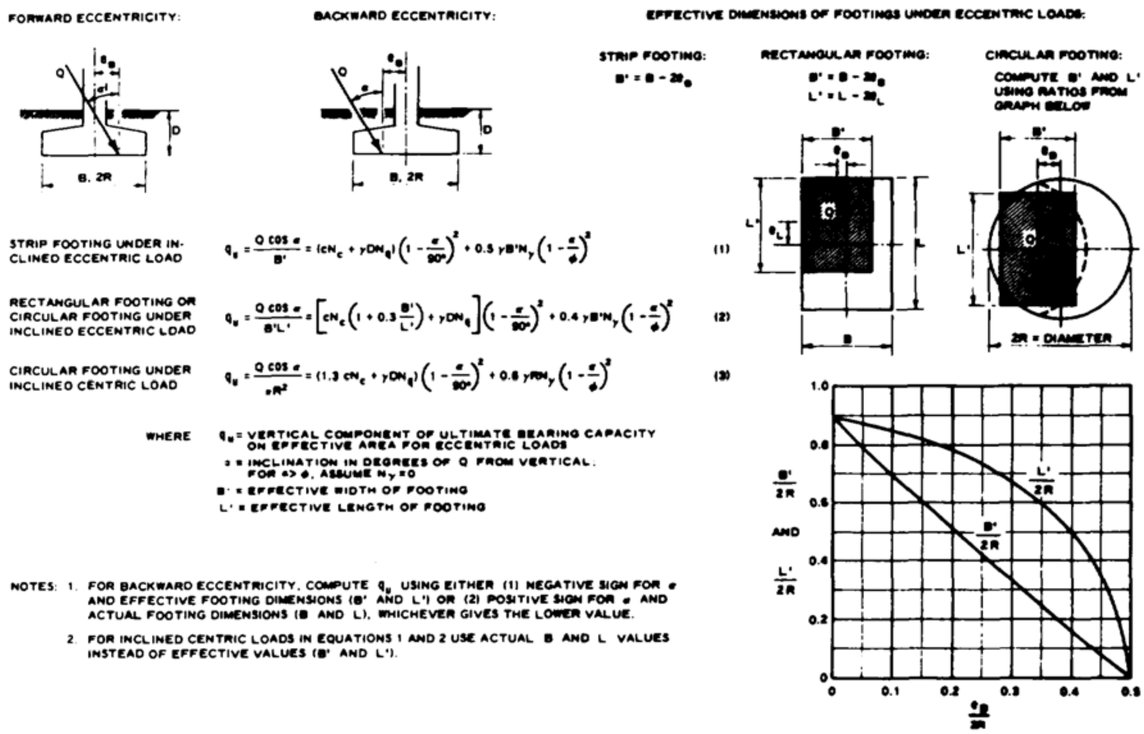


Figure 3

Ultimate bearing capacity of shallow foundations under eccentric inclined loads

4. TENSION FORCES. Footings subjected to a sustained uplift force, Tu, should be designed with a minimum factor of safety of 1.5 with respect to weight forces resisting pullout expressed as:

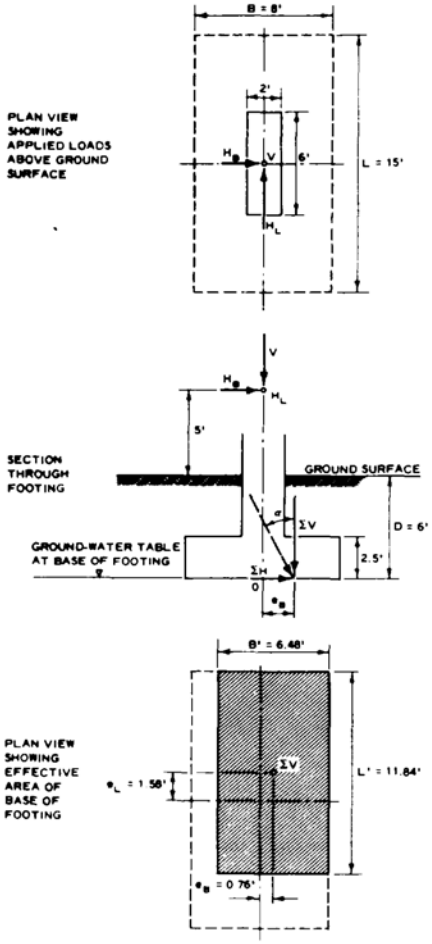
$$W/T_U \geq 1.5 \qquad \text{(Eq. 1)}$$

where W is the total effective weight of soil and concrete located within the prism bounded by vertical lines at the base of the footing. Use total unit weights above the water table and the buoyant unit weight below. If the shear resistance on the vertical sides of the prism defined above is considered, a minimum safety factor of 2 should be used. The lateral earth pressure on the vertical sides of the prism should not exceed earth pressure at rest and should be considered as active earth pressure if the soil is not well compacted.

5. BEARING CAPACITY OF ROCK. For a structure founded on rock, adequate exploration is necessary to determine the number and extent of defects, such as joints, shear zones, and solution features. Estimates of the allowable bearing pressure can be obtained from Table 1. Conservative estimates of the allowable bearing pressure can be obtained from the following expression:

$$Q_a = 0.2\ q_u \qquad \text{(Eq. 2)}$$

Allowable bearing pressures for jointed rock can also be estimated from RQD values using Table 2. Local experience should always be ascertained.



(A) SOIL AND LOADING CONDITIONS

UNIFORM SANDY SILT:

γ = 112.5 LB/CU FT = 0.056 TON/CU FT

γ_{SUB} = 50 LB/CU FT = 0.025 TON/CU FT

c = 0.06 TON/SQ FT

ϕ = 25°

DL + NORMAL LL:

V = 113 TONS VERTICAL

H_B = 11 TONS LATERAL

H_L = 23 TONS LONGITUDINAL

(B) COMPUTATION OF NET VERTICAL LOAD

VERTICAL LOAD ABOVE GROUND SURFACE = V = 113.0 TONS

EFFECTIVE WEIGHT OF SOIL ABOVE BASE OF FOOTING = $\frac{(8 \times 15 \times 6)(112.5)}{2000}$ = 40.5 TONS

WEIGHT OF CONCRETE (150 LB/CU FT) IN FOOTING IN EXCESS OF DISPLACED SOIL

$$= \frac{[(8 \times 15 \times 2.5) + (2 \times 6 \times 3.5)](150 - 112.5)}{2000} = 6.5 \text{ TONS}$$

NET VERTICAL LOAD = ΣV = 160.0 TONS

(C) COMPUTATION OF ECCENTRICITY (e) AND INCLINATION (α)

TAKING MOMENTS ABOUT O:

$$e_B = \frac{\Sigma M_B}{\Sigma V} = \frac{11(5 + 6)}{160} = 0.76 \text{ FT} \qquad B' = B - 2e_B = 8 - 1.52 = 6.48 \text{ FT}$$

$$e_L = \frac{\Sigma M_L}{\Sigma V} = \frac{23(5 + 6)}{160} = 1.58 \text{ FT} \qquad L' = L - 2e_L = 15 - 3.16 = 11.84 \text{ FT}$$

$$\Sigma H = \sqrt{(11)^2 + (23)^2} = 25.5 \text{ TONS} \qquad \alpha = \text{ARCTAN} \frac{\Sigma H}{\Sigma V} = \text{ARCTAN} \frac{25.5}{160} = \text{ARCTAN } 0.159 = 9°$$

(D) COMPUTATION OF VERTICAL COMPONENT OF ULTIMATE BEARING CAPACITY

$$q_u = \left[cN_c\left(1 + 0.3\frac{B'}{L'}\right) + \gamma D N_q\right]\left(1 - \frac{\alpha}{90°}\right)^2 + 0.4\gamma B' N_\gamma \left(1 - \frac{\alpha}{\phi}\right)^2$$

$$= \left[0.06 \times 24\left(1 + 0.3\frac{6.48}{11.84}\right) + (0.056 \times 6 \times 13)\right]\left(1 - \frac{9}{90}\right)^2 + (0.4 \times 0.025 \times 6.48 \times 10)\left(1 - \frac{9}{25}\right)^2$$

$$= (1.67 + 4.36)(0.81) + (0.65)(0.41) = 4.88 + 0.27 = 5.15 \text{ TONS/SQ FT}$$

(E) COMPUTATION OF FACTOR OF SAFETY WITH RESPECT TO BEARING CAPACITY

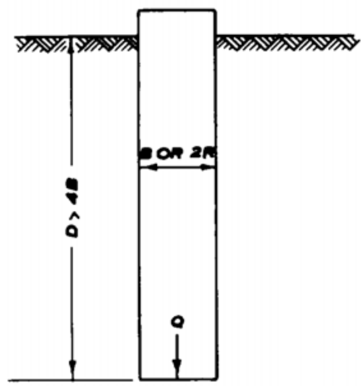
ACTUAL BEARING PRESSURE = $q_o = \frac{\Sigma V}{B'L'} = \frac{160}{6.48 \times 11.84} = 2.09$ TONS/SQ FT

FACTOR OF SAFETY = F.S. = $\frac{q_u}{q_o} = \frac{5.15}{2.09} = 2.5 > 2.0$ REQUIRED F.S. FOR DL + NORMAL LL

NOTE: COMPUTATION SHOULD BE REPEATED FOR DL + MAXIMUM LL. F.S. SHOULD BE GREATER THAN 1.5 FOR THIS CONDITION.

Figure 4

Example of bearing capacity computation for inclined eccentric load on rectangular footing



BEARING CAPACITY FACTORS, N_c FOR FOUNDATIONS IN CLAY ($\phi = 0°$)

B/L	N_c
1 (SQUARE OR CIRCLE)	9.0
0.5	8.2
0 (STRIP FOOTING)	7.5

NOTE: BEARING CAPACITY FACTORS BASED ON A SMOOTH BASE AND D/B GREATER THAN 4

B = WIDTH OF FOOTING
L = LENGTH OF FOOTING
D = DEPTH OF FOOTING

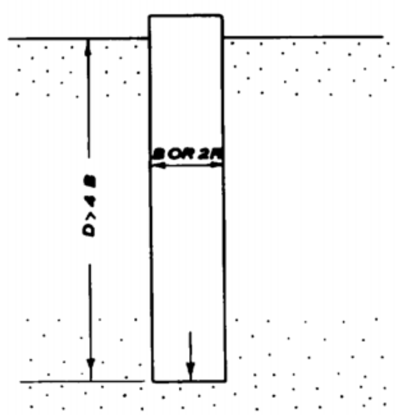
$Q/L = cBN_c + \gamma DB + 2Df_s$ (STRIP LOADING)

$Q = cB^2N_c + \gamma DB^2 + 4BDf_s$ (SQUARE LOADING)

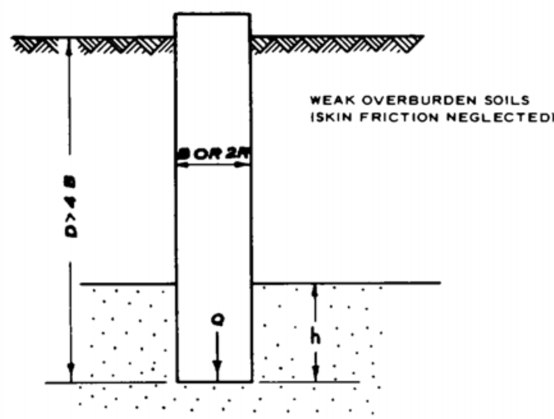
$Q = c\pi R^2N_c + \gamma D\pi R^2 + 2\pi RDf_s$ (CIRCULAR LOADING)

NOTE: THE SKIN FRICTION, f_s, IS USUALLY TAKEN AS ONE-HALF THE UNCONFINED COMPRESSIVE STRENGTH OF THE CLAY FOUNDATION. FOR PILES THE VALUE OF f_s SHOULD NOT EXCEED THE MINIMUM ADHESION VALUE GIVEN IN PARAGRAPH 46b. BECAUSE SKIN FRICTION, f_s, IS NOT ALWAYS RELIABLE, IT IS OFTEN IGNORED.

(a) DEEP FOUNDATION IN HOMOGENEOUS CLAY



COMPLETE EMBEDMENT IN SAND



PARTIAL EMBEDMENT IN SAND

Complete embedment in sand:

$Q/L = \gamma DBN_q + 0.5\gamma B^2 N_\gamma + 2Df_s$ (STRIP LOADING)

$Q = \gamma DB^2N_q + 0.4\gamma B^3N_\gamma + 4BDf_s$ (SQUARE LOADING)

$Q = \gamma D\pi R^2N_q + 0.6\gamma\pi R^3N_\gamma + 2\pi RDf_s$ (CIRCULAR LOADING)

$f_s = 1/2\,K\gamma D \tan\delta$

Partial embedment in sand:

$Q/L = \gamma DBN_q + 0.5\gamma B^2N_\gamma + 2hf_s$ (STRIP LOADING)

$Q = \gamma DB^2N_q + 0.4\gamma B^3N_\gamma + 4Bhf_s$ (SQUARE LOADING)

$Q = \gamma D\pi R^2N_q + 0.6\gamma\pi R^3N_\gamma + 2\pi Rhf_s$ (CIRCULAR LOADING)

$f_s = K\gamma(D - h/2) \tan\delta$

WHERE K = COEFFICIENT OF EARTH PRESSURE DEPENDENT ON DENSITY OF SAND AND METHOD OF FOUNDATION PLACEMENT (SEE CHAPTER 12)

N_q AND N_γ = BEARING CAPACITY FACTORS FOR SHALLOW FOUNDATIONS (SEE FIGURE 6.1)

δ = ANGLE OF FRICTION BETWEEN SAND AND FOUNDATION ($\delta < \phi$)

(b) DEEP FOUNDATION IN SAND

Figure 5

Ultimate bearing capacity of deep foundations

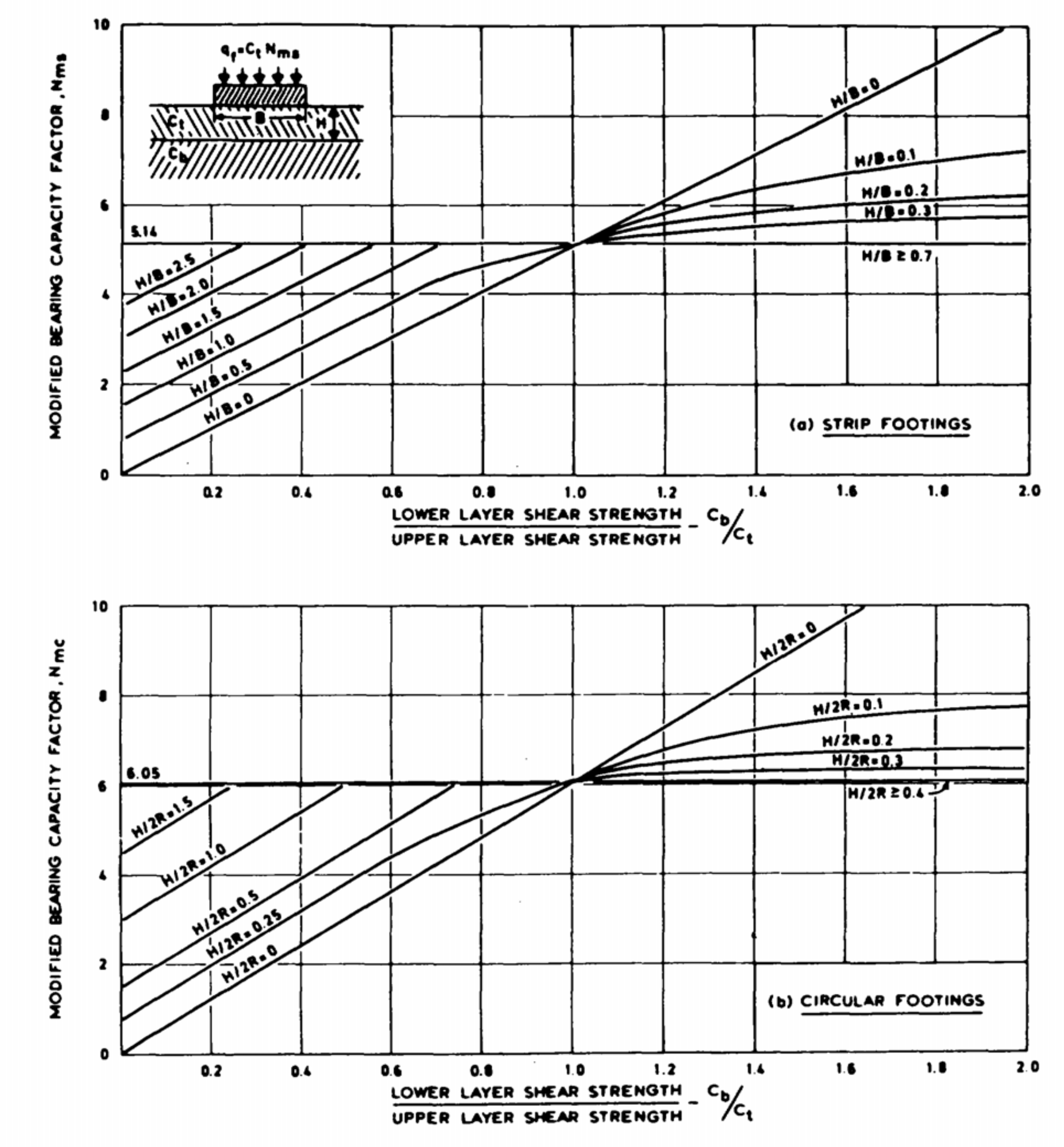


Figure 6

Bearing capacity factors for strip and circular footings on layered foundations in clay

RQD	Q_a (tsf)
100	300
90	200
75	120
50	65
25	30
0	10

If a tabulated value exceeds unconfined compressive strength of intact samples of the rock, allowable pressure equals unconfined compressive strength.

Table 2

Allowable Bearing Pressure for Jointed Rock