Foundations

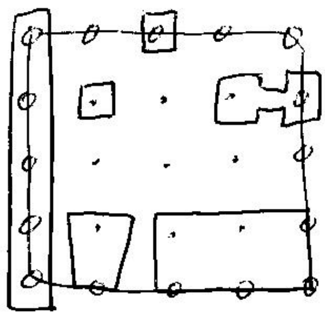
1. Purpose

: To prevent strength failure of soil & large deformation

2. Types: Shallow/Deep

1) **Shallow foundations** - include all those types which are designed to spread the building loads over a sufficient area of soil near ground surface to secure adequate bearing capacity & small deformation

① Spread footing
- single(independent) footing
- strip(or wall or continuous) footing
- combined footing



② Raft(mat) foundation

2) **Deep foundations** - used in case where the soil near the ground surface are incapable of supporting mat or single footings.

① Piles : installed by driving, vibration as well as by excavation or drilling

② Piers(drilled) : always installed by excavation or drilling

③ Caissons : may be defined as a large water-tight box that is used to exclude water and semi-fluid material during the excavation of foundations & ultimately becomes an integral part of the substructure (types - open, pneumatic, box caissons)

Homework #2 : Study the Caissons in depth (Submit 6pages report)

3. Requirements of satisfactory foundations

1) Must be properly located with regard to any future influence which could adversely affect its performance

- frost action : 3/4 of the max. frost penetration or thaw line
- adjacent structures
- scour and undermining by river currents & wave action
- ground water level
- underground defects such as faults, caves, mines

-soil volume change(h/o)

TABLE 7.14 RELATION OF SOIL INDEX PROPERTIES AND PROBABLE VOLUME CHANGES FOR HIGHLY PLASTIC SOILS.

Data from Index Tests[a]			*Estimation of Probable Expansion*[b]	
Colloid Content (Percent minus 0.001 mm)	*Plasticity Index*	*Shrinkage Limit, Percent*	*Percent Total Volume Change (Dry to Saturated Condition)*	*Degree of Expansion*
>28	>35	<11	>30	Very high
20–31	25–41	7–12	20–30	High
13–23	15–28	10–16	10–20	Medium
<15	<10	>15	<10	Low

[a] All three index tests should be considered in estimating expansive properties.
[b] Based on a vertical loading of 1.0 psi as for concrete canal lining. For higher loadings the amount of expansion is reduced, depending on the load and on the clay characteristics.

Homework #3 : Treating Expansive Soils, ASCE magazine, Aug.1987
Summary Report (3-4 pages)

2) Safe against bearing capacity failure

3) Not settle enough to damage structure

4) Factor of safety

① Total Safety Factor (FS)

$$q_{all} = \frac{q_{ult}}{FS}$$

TABLE 4.6 MINIMUM SAFETY FACTORS FOR DESIGN OF SHALLOW FOUNDATIONS*.
Preliminary note. The selection of safety factors for design cannot be made properly without assessing the degree of reliability of all other parameters that enter into design, such as design loads, strength and deformation characteristics of the soil mass, etc. In view of this, each case is to be considered separately by the designer. The following table may be used as a guide for permanent structures in reasonably homogeneous soil conditions.

			Soil Exploration	
Category	*Typical Structures*	*Characteristics of the Category*	*Complete*	*Limited*
A	Railway bridges Warehouses Blast furnaces Hydraulic Retaining walls Silos	Maximum design load likely to occur often; consequences of failure disastrous	3.0	4.0
B	Highway bridges Light industrial and public buildings	Maximum design load may occur occasionally; consequences of failure serious	2.5	3.5
C	Apartment and office buildings	Maximum design load unlikely to occur	2.0	3.0

*Vesić (1975).

Remarks:

1. For temporary structures, these factors can be reduced to 75 percent of the above values. However, in no case should safety factors lower than 2.0 be used.
2. For exceptionally tall buildings, such as chimneys and towers, or generally whenever progressive bearing capacity failure may be feared, these factors should be increased by 20 to 50 percent.
3. The possibility of flooding of foundation soil and/or removal of existing overburden by scour or excavation should be given adequate consideration.
4. It is advisable to check both the short-term (end-of-construction) and long-term stability, unless one of the two conditions is clearly less favorable.
5. It is understood that all foundations will be analyzed also with respect to maximum tolerable total and differential settlement. If settlement governs the design, higher safety factors must be used.

② Partial Safety Factor

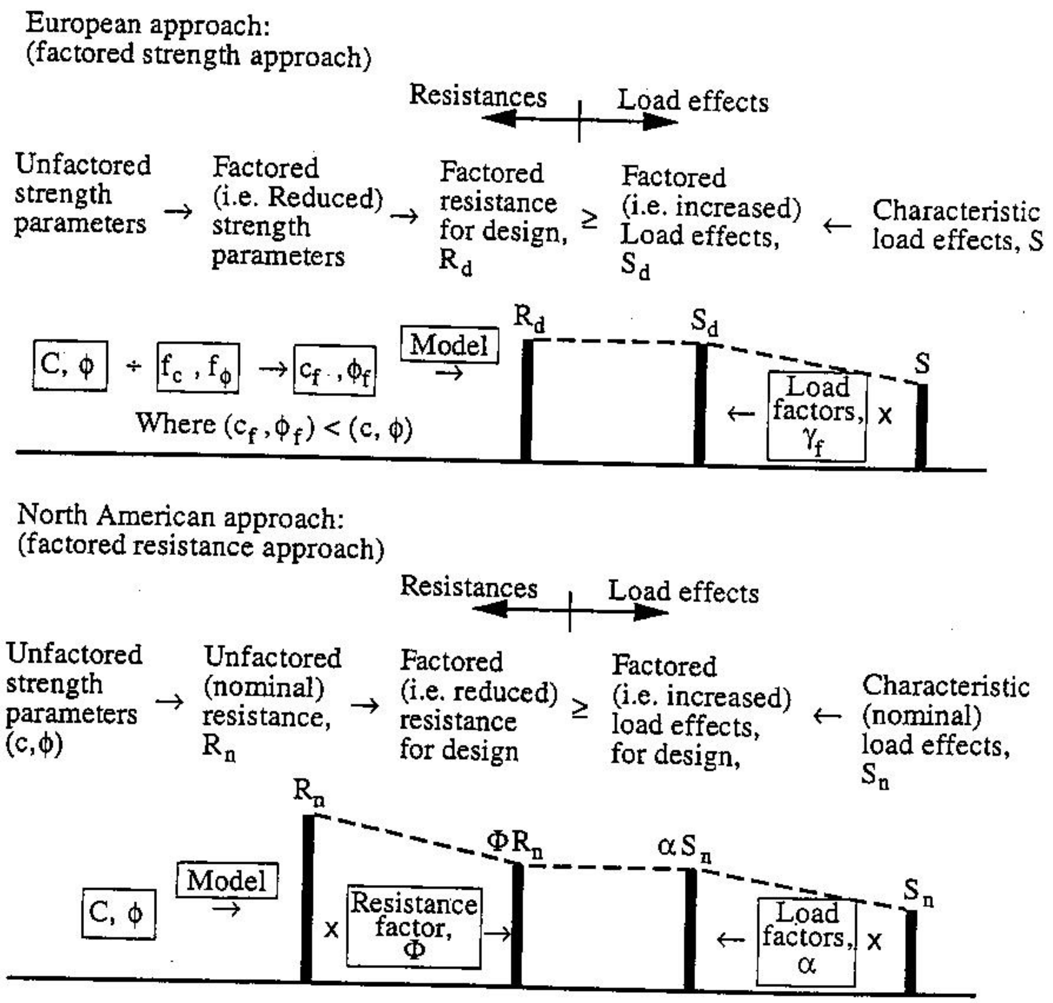


Figure 2.3. LRFD (North American) approach vs partial factor (European) approach (Becker, 1996).

TABLE 4.8 VALUES OF MINIMUM PARTIAL FACTORS[a].

Category	*Item*	*Load Factor*	*Resistance Factor*
Loads	Dead loads	(f_d) 1.25 (0.85)	
	Live loads, wind or earthquake	(f_l) 1.5	
	Water pressures	(f_u) 1.25 (0.85)	
Shear strength	Cohesion (c) (stability; earth pressures)		(f_c) 0.65
	Cohesion (c) (foundations)		(f_c) 0.5
	Friction ($\tan\phi$)		(f_ϕ) 0.8

Note: Load factors given in parentheses apply to dead loads and water pressures when their effects are beneficial, as for dead loads resisting instability by sliding, overturning or uplift.

[a] Meyerhof (1984).

Table 10.5.5.2.4-1 Resistance Factors for Geotechnical Resistance of Drilled Shafts.

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (*O'Neill and Reese, 1999*)	0.45
	Tip resistance in clay	Total Stress (*O'Neill and Reese, 1999*)	0.40
	Side resistance in sand	β-method (*O'Neill and Reese, 1999*)	0.55
	Tip resistance in sand	O'Neill and Reese (*1999*)	0.50
	Side resistance in IGMs	O'Neill and Reese (*1999*)	0.60
	Tip resistance in IGMs	O'Neill and Reese (*1999*)	0.55
	Side resistance in rock	Horvath and Kenney (*1979*) O'Neill and Reese (*1999*)	0.55
	Side resistance in rock	Carter and Kulhawy (*1988*)	0.50
	Tip resistance in rock	Canadian Geotechnical Society (*1985*) Pressuremeter Method (*Canadian Geotechnical Society, 1985*) O'Neill and Reese (*1999*)	0.50
Block Failure, φ_{bl}	Clay		0.55
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (*O'Neill and Reese, 1999*)	0.35
	Sand	β-method (*O'Neill and Reese, 1999*)	0.45
	Rock	Horvath and Kenney (*1979*) Carter and Kulhawy (*1988*)	0.40
Group Uplift Resistance, φ_{ug}	Sand and clay		0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		Values in Table 10.5.5.2.3-2, but no greater than 0.70
Static Load Test (uplift), φ_{upload}	All Materials		0.60

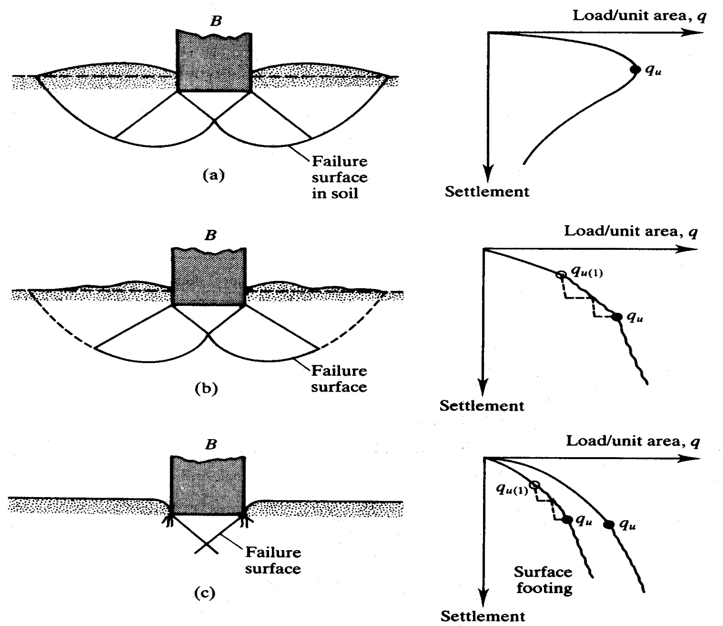
Shallow foundations

1. Modes of failure

① **General shear failure** - dense sand (Dr > 90%) or stiff clay(O.C. clay)

② **Local shear failure** - medium soil

③ **Punching shear failure** - loose sand (Dr: 20~30 %) or soft clay



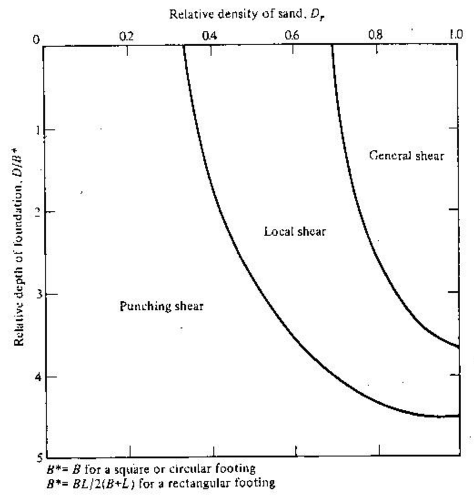


Fig. 3.10 Modes of failure of model footings in sand. (After Vesić, 1963a, as modified by De Beer, 1970.)

2. Bearing capacity estimation

① Field test

- Plate load test, CPT, SPT, DMT

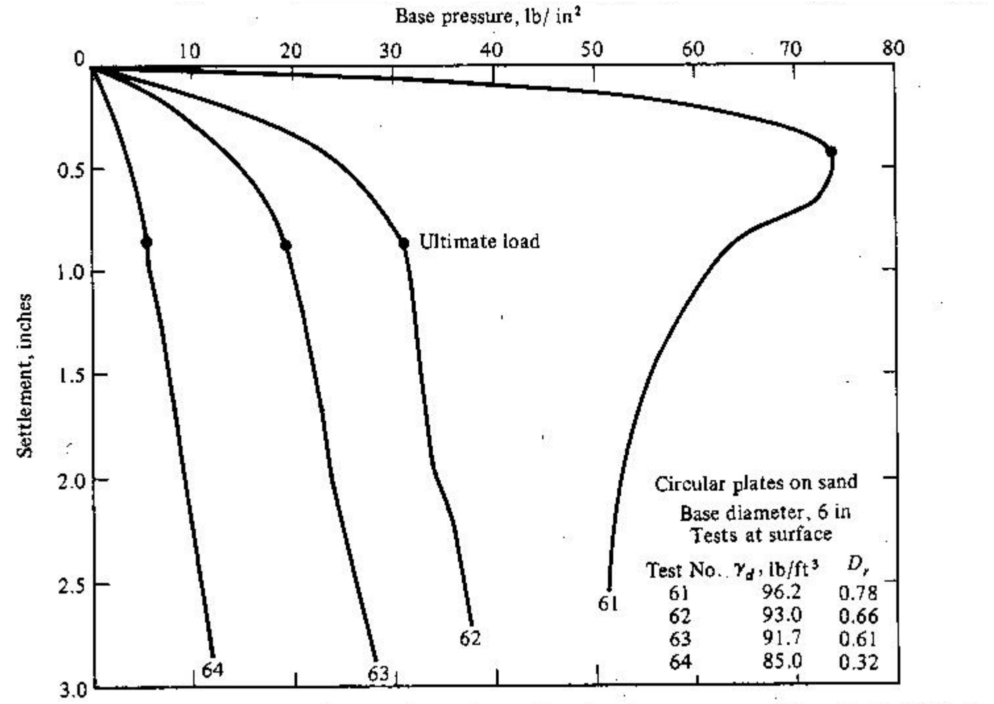


Fig. 3.11 Ultimate load criterion based on minimum slope of load-settlement curve. (After Vesić, 1963a.)

Homework #4 : Estimation of B.C. of Circular Footings on Sands Based on CPT.

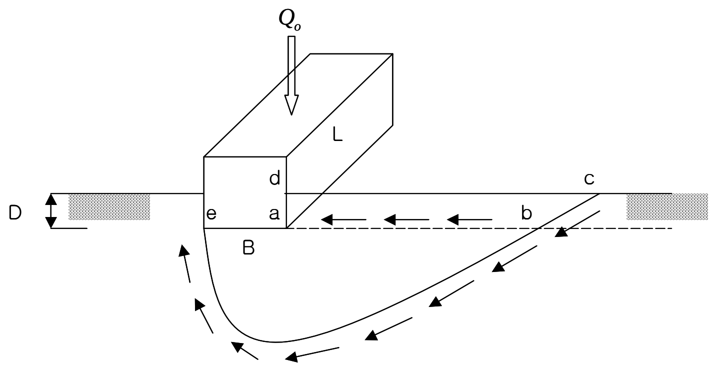
ASCE. J. of GT & GE Vol.131, No.4, pp. 442-452

Summary Report (3-4 pages)

② **Analytical method**(c, ϕ, γ known)

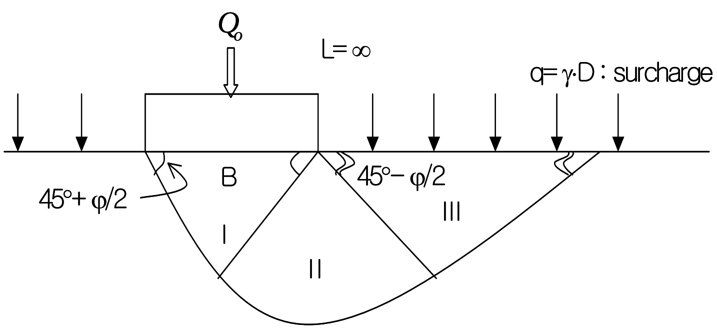
-computes the ultimate bearing capacity with field data

3. Computation of Ultimate Bearing Capacity (Terzaghi)



c, Φ, ɣ (known)

Real situation



Ideal situation

Ⅰ : Active Rankine Zone

Ⅱ : Radial Prandtl Zone

Ⅲ : Passive Rankine Zone

※ To be determined is the maximum unit load $q_o (= \frac{Q_o}{BL})$ which this foundation can support (Q_0 acting in a vertical direction at the centroid of the foundation)

Assumptions & simplifications

① The soil mass is of semi-infinite extent and homogeneous

② The load acts in a vertical direction at the center of foundation.

③ Shear strength properties are defined by a straight line Mohr envelop with strength parameters(c, Φ)

④ Rigid plastic stress-strain relationship → assume general shear failure

⑤ The footing length (L) is assumed to be large compared with the width (B) of the foundation → plane strain

※ if L/B>10, justified, L/B>5, practically accepted

⑥ The overburden soil is replaced by a uniformly distributed surcharge, q_s $(=\gamma D)$

→ the friction of $\overline{ab}, \overline{bc}$ is neglected

Solutions : Analytical solutions are found only for special cases.

case 1) $\gamma = 0$: $q_o = cN_c + q_s N_q$

c : cohesion

q_s : surcharge

N_q : $e^{\pi \tan \Phi} \tan^2(\frac{\pi}{4} + \frac{\Phi}{2})$

N_c : $(N_q - 1)\cot\Phi$

case 2) c=0, q_s**=0** : $q_o = \frac{1}{2}\gamma B N_\gamma$

N_γ : can be evaluated numerically only or approximately $N_\gamma \approx 2(N_q+1)\tan\Phi$

General case : c≠0, $q_s \neq 0$, $\gamma \neq 0$

$$q_o = cNc + q_s N_q + \frac{1}{2}\gamma B N_r$$ → Buisman - Terzaghi Equation

* This superposition is not strictly correct, however, it leads to errors which are on the safeside, not exceeding 20%. for, Φ = 30˚ ~40°, while if Φ=0, equal to 0.