Overview of Foundation Concepts and Foundation Alternatives

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Short Course on

Geotechnical Investigations for Structural Engineering

13– 15 November, 2014

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Bearing Capacity

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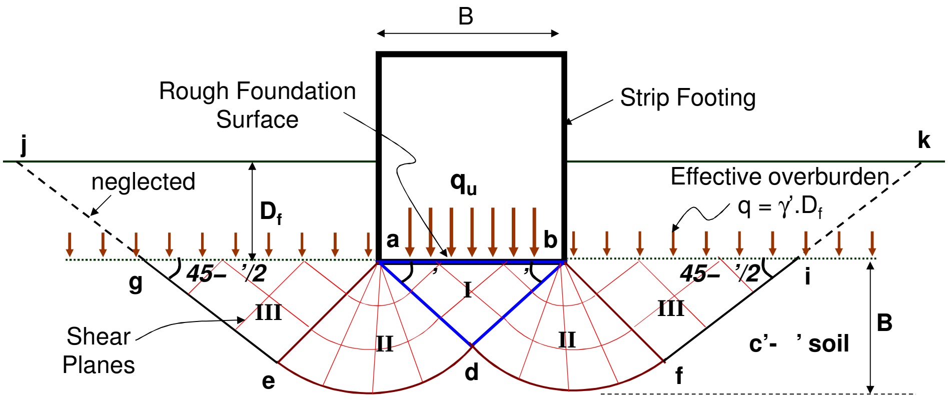
Terzaghi’s Bearing Capacity Theory



Assumption

- L/B ratio is large → plain strain problem
- $D_f \leq B$
- Shear resistance of soil for D_f depth is neglected
- General shear failure
- Shear strength is governed by Mohr-Coulomb Criterion

Terzaghi’s Bearing Capacity Theory

$$\boxed{q_u = c.N_c + q.N_q + 0.5\gamma' B.N_\gamma}$$

Terzaghi’s bearing capacity equation

Terzaghi’s bearing capacity factors

Local Shear Failure:

Modify the strength parameters such as: $c'_m = \frac{2}{3}c'$ $\phi'_m = \tan^{-1}\left(\frac{2}{3}\tan\phi'\right)$

Square and circular footing:

$$q_u = 1.3c'.N_c + q.N_q + 0.4\gamma' B.N'_\gamma$$ For square

$$q_u = 1.3c'.N_c + q.N_q + 0.3\gamma' B.N'_\gamma$$ For circular

Total and Effective Stress Analysis

- Total stress parameters
  - c and ϕ
  - From UC or UU test
  - In bearing capacity equation, use total overburden and bulk/saturated density.

- Effective stress parameters
  - c' and ϕ'
  - From direct shear test, CU or CD test
  - In bearing capacity equation, use effective overburden and bulk/submerged density.

Terzaghi's Bearing Capacity Theory

Effect of water table:

Case I: $D_w \leq D_f$

Surcharge, $q = \gamma . D_w + \gamma' \left(D_f - D_w \right)$

Case II: $D_f \leq D_w \leq (D_f + B)$

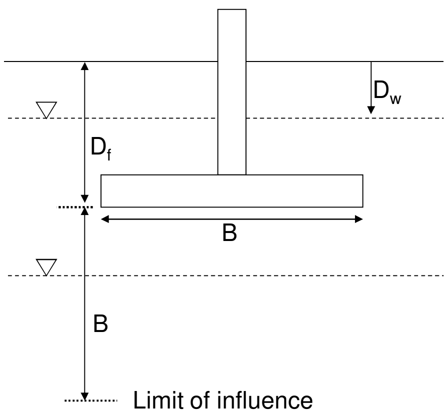
Surcharge, $q = \gamma . D_F$

In bearing capacity equation replace γ by-

$$\bar{\gamma} = \gamma' + \left(\frac{D_w - D_f}{B} \right) (\gamma - \gamma')$$

Case III: $D_w > (D_f + B)$

No influence of water table.



IS:6403-1981 Recommendations

Net Ultimate Bearing capacity: $q_{nu} = c.N_c.s_c.d_c.i_c + q.(N_q - 1).s_q.d_q.i_q + 0.5\gamma.B.N_\gamma.s_\gamma.d_\gamma.i_\gamma$

For cohesive soils → $q_{nu} = c_u.N_c.s_c.d_c.i_c$ where, $N_c = 5.14$

N_c, N_q, N_γ *as per Vesic(1973) recommendations*

Shape Factors → For rectangle, $s_c = 1 + 0.2\frac{B}{L}$ $s_q = 1 + 0.2\frac{B}{L}$ $s_\gamma = 1 - 0.4\frac{B}{L}$

For square and circle, $s_c = 1.3$ $s_q = 1.2$

$s_\gamma = 0.8$ for square, $s_\gamma = 0.6$ for circle

Depth Factors → $d_c = 1 + 0.2\frac{D_f}{L}\tan\left(45 + \frac{\phi'}{2}\right)$

$d_q = d_\gamma = 1 + 0.1\frac{D_f}{L}\tan\left(45 + \frac{\phi'}{2}\right)$ for $\phi' \geq 10^o$

$d_q = d_\gamma = 1$ for $\phi' < 10^o$

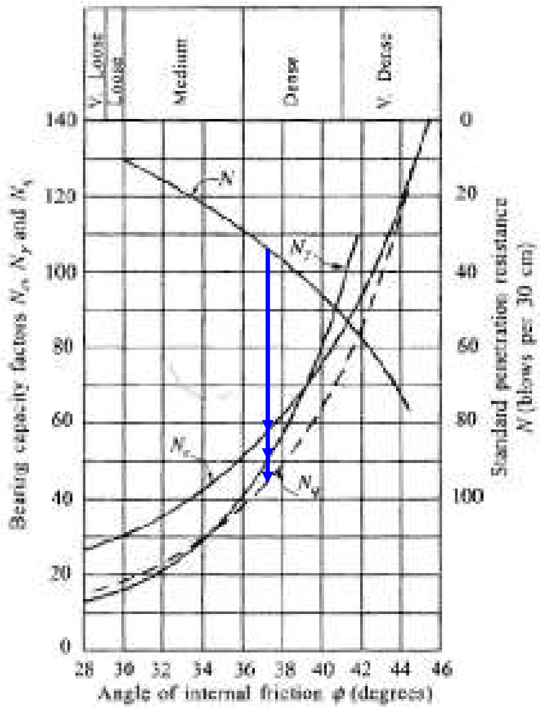
Inclination Factors → $i_c = i_q = \left(1 - \frac{\beta^o}{90}\right)^2$ $i_\gamma = \left(1 - \frac{\beta}{\phi'}\right)^2$

Bearing Capacity Correlations with SPT-value

Peck, Hansen, and Thornburn (1974)

&

IS:6403-1981 Recommendation



Bearing Capacity Correlations with SPT-value

Teng (1962):

For Strip Footing: $$q_{nu} = \frac{1}{6}\left[3N''^2.B.R'_w + 5\left(100 + N''^2\right).D_f.R_w\right]$$

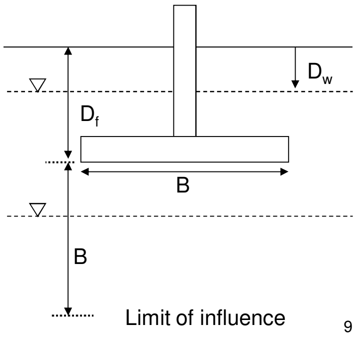
For Square and Circular Footing: $$q_{nu} = \frac{1}{3}\left[N''^2.B.R'_w + 3\left(100 + N''^2\right).D_f.R_w\right]$$

For $D_f > B$, take $D_f = B$

Water Table Corrections:

$$R_w = 0.5\left(1 + \frac{D_w}{D_f}\right) \qquad [R_w \le 1$$

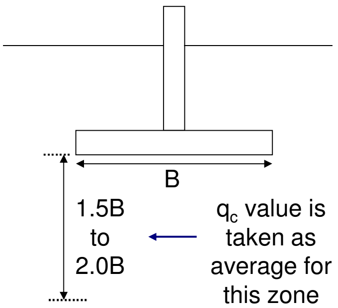
$$R'_w = 0.5\left(1 + \frac{D_w - D_f}{D_f}\right) \qquad [R'_w \le 1$$

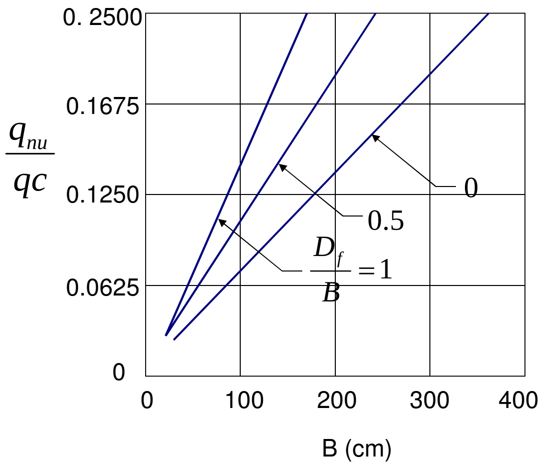


Bearing Capacity Correlations with CPT-value

IS:6403-1981 Recommendation:

Cohesionless Soil





Schmertmann (1975):

$$N_\gamma \cong N_q \cong \frac{q_c}{0.8} \quad \leftarrow \quad \text{in } \frac{\text{kg}}{\text{cm}^2}$$

Bearing Capacity Correlations with CPT-value

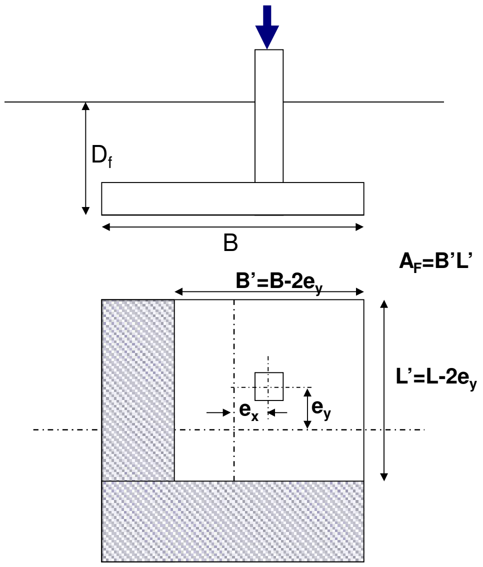
IS:6403-1981 Recommendation:

Cohesive Soil

$$q_{nu} = c_u.N_c.s_c.d_c.i_c$$

Soil Type	Point Resistance Values (q_c) kgf/cm^2	Range of Undrained Cohesion (kgf/cm^2)
Normally consolidated clays	$q_c < 20$	$q_c/18$ to $q_c/15$
Over consolidated clays	$q_c > 20$	$q_c/26$ to $q_c/22$

Effective Area Method for Eccentric Loading



In case of Moment loading

$$e_x = \frac{M_y}{F_V}$$

$$e_y = \frac{M_x}{F_V}$$

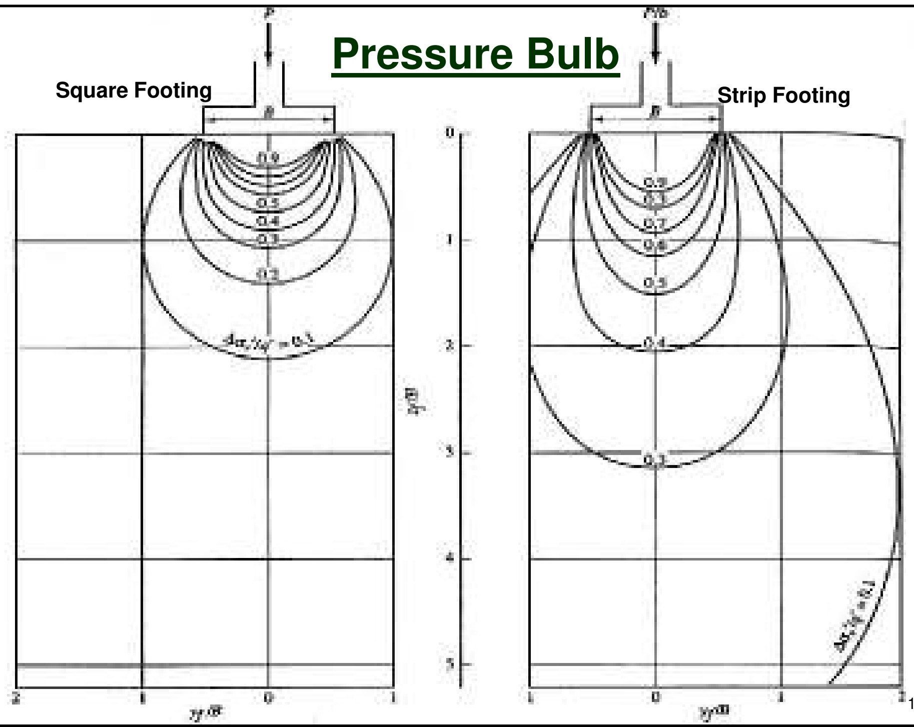
In case of Horizontal Force at some height but the column is centered on the foundation

$$M_y = F_{Hx}.d_{FH}$$

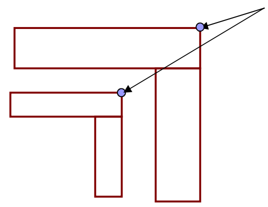
$$M_x = F_{Hy}.d_{FH}$$

Settlement

Pressure Bulb

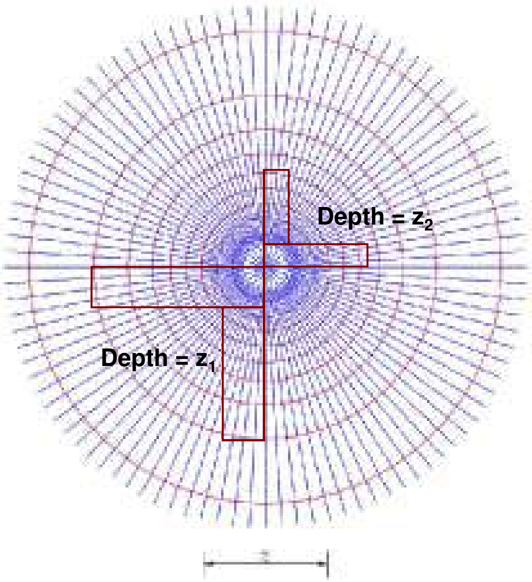


Newmark's Chart

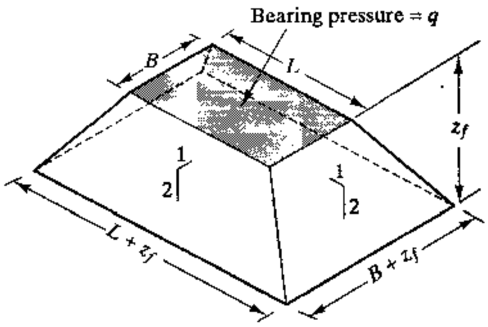


- Determine the depth, z, where you wish to calculate the stress increase
- Adopt a scale as shown in the figure
- Draw the footing to scale and place the point of interest over the center of the chart
- Count the number of elements that fall inside the footing, N
- Calculate the stress increase as:

$$\sigma_{zz} = n \times 0.001\ p$$



Approximate Methods



Rectangular Foundation:

$$\Delta\sigma_z = q\frac{B.L}{(B+z).(L+z)}$$

Square/Circular Foundation:

$$\Delta\sigma_z = q\frac{B^2}{(B+z)^2}$$

Strip Foundation:

$$\Delta\sigma_z = q\frac{B}{(B+z)}$$

Settlement

Settlement
S = $S_e + S_c + S_s$

Immediate Settlement S_e	Primary Consolidation S_c	Secondary Consolidation S_s

- Immediate Settlement: Occurs immediately after the construction. This is computed using elasticity theory (Important for Granular soil)
- Primary Consolidation: Due to gradual dissipation of pore pressure induced by external loading and consequently expulsion of water from the soil mass, hence volume change. (Important for Inorganic clays)
- Secondary Consolidation: Occurs at constant effective stress with volume change due to rearrangement of particles. (Important for Organic soils)

For any of the above mentioned settlement calculations, we first need vertical stress increase in soil mass due to net load applied on the foundation

Elastic settlement of Foundation

Elastic settlement:

$$S_e = \int_0^H \varepsilon_z dz = \frac{1}{E_s}\int_0^H \left(\Delta\sigma_z - \mu_s \Delta\sigma_x - \mu_s \Delta\sigma_y\right) dz$$

$E_s =$ Modulus of elasticity

$H =$ Thickness of soil layer

$\mu_s =$ Poisson's ratio of soil

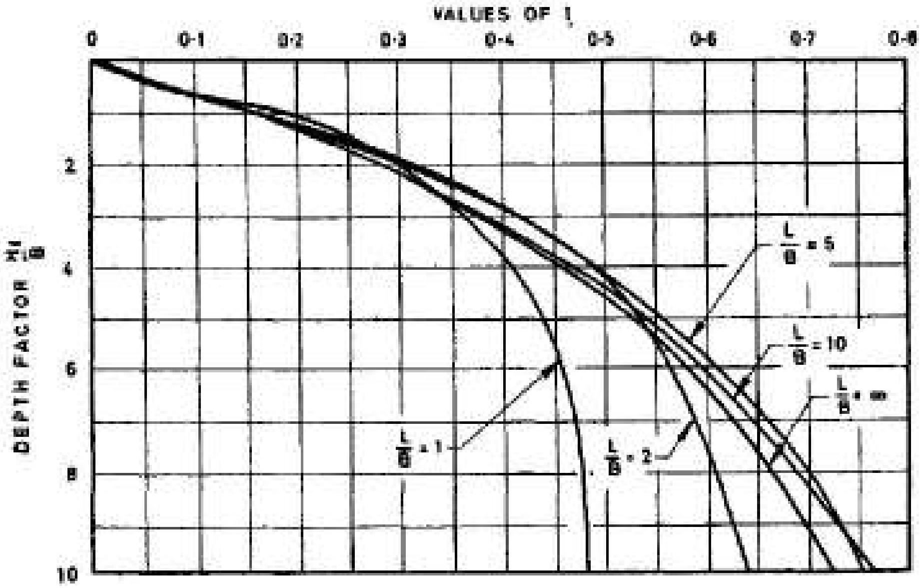
Elastic settlement for Flexible Foundation:

$$S_e = \frac{qB}{E_s}\left(1-\mu_s^2\right) I_f$$

I_f **= influence factor: depends on the rigidity and shape of the foundation**

E_s **= Avg elasticity modulus of the soil for (4B) depth below found^n level**

Steinbrenner's Influence Factors for Settlement of the Corners of loaded Area *LxB* on Compressible Stratus of *= 0.5*, and Thickness H_t



Elastic settlement of Foundation

Soil Strata with Semi-infinite depth

Shape	I_f *Flexible Foundation* Centre	Corner	Average	I_f *Rigid Foundation*
Circle*	1.00	0.64	0.85	0.86
Square	1.12	0.56	0.95	0.82
Rectangle				
$L/B = 1.5$	1.36	0.68	1.20	1.06
$L/B = 2$	1.52	0.76	1.30	1.20
$L/B = 5$	2.10	1.05	1.83	1.70
$L/B = 10$	2.52	1.26	2.25	2.10
$L/B = 100$	3.38	1.69	2.96	3.40

*Use diameter for B

Elastic settlement of Foundation

E in kPa

Type of soil	*SPT*	*CPT*
Sand (normally consolidated)	$E = 500 (N + 15)$	$E = 2 \text{ to } 4\, q_c$
		$E = 2(1 + D_r^2)\, q_c$
Sand (saturated)	$E = 250 (N + 15)$	
Sand (overconsolidated)		$E = 6 \text{ to } 30\, q_c$
Gravelly sand	$E = 1200 (N + 6)$	
Clayey sand	$E = 320 (N + 15)$	$E = 3 \text{ to } 6\, q_c$
Silty sand	$E = 300 (N + 6)$	$E = 1 \text{ to } 2\, q_c$
Soft clay		$E = 5 \text{ to } 8\, q_c$

Several other sets of correlations available

Elastic settlement of Foundation

Type of soil	μ
Clay, saturated	0.4 - 0.5
Clay, unsaturated	0.1 - 0.3
Sandy clay	0.2 - 0.3
Silt	0.3 - 0.35
Sand (dense)	
Coarse (void ratio = 0.4 - 0.7.)	0.15
Fine grained (void ratio = 0.4 - 0.7)	0.25
Rock	0.1 - 0.4 (depends somewhat on type of rock)
Loess	0.1 - 0.3
Ice	0.36
Concrete	0.15

Strain Influence Factor Method for Sandy Soil: Schmertmann and Hartman (1978)

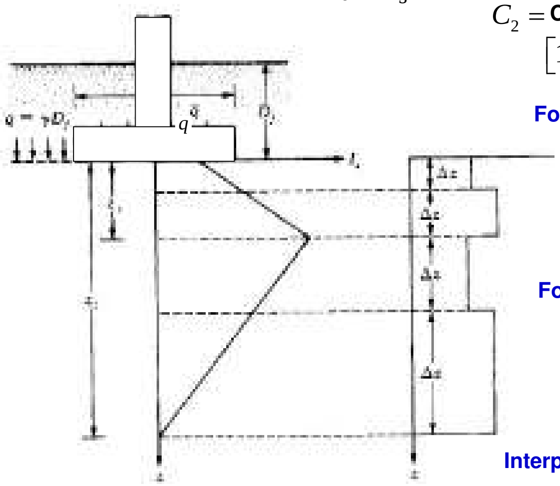
$$S_e = C_1 C_2 \left(q - \gamma D_f\right) \sum_0^{z_2} \frac{I_z}{E_s} \Delta z$$

C_1 = **Correction factor for foundation depth**

$$\left[1 - 0.5\left\{\gamma D_f / \left(q - \gamma D_f\right)\right\}\right]$$

C_2 = **Correction factor for creep effects**

$$\left[1 + 0.2 \log\left(\text{time in years}/0.1\right)\right]$$



For square and circular foundation:

$I_z = 0.1$ at $z = 0$

$I_z = 0.5$ at $z = z_1 = 0.5B$

$I_z = 0$ at $z = z_2 = 2B$

For foundation with L/B >10:

$I_z = 0.2$ at $z = 0$

$I_z = 0.5$ at $z = z_1 = B$

$I_z = 0$ at $z = z_2 = 4B$

Interpolate the values for 1 < L/B < 10

Settlement due to Primary Consolidation

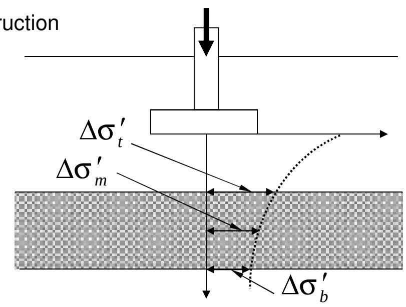
For NC clay $\left(\sigma_c' < \sigma_o' < \sigma_o' + \Delta\sigma_{av}'\right)$ $\quad S_c = \dfrac{C_c H_c}{1+e_o} \log\left(\dfrac{\sigma_o' + \Delta\sigma_{av}'}{\sigma_o'}\right)$

For OC clay $\left(\sigma_o' + \Delta\sigma_{av}' < \sigma_c'\right)$ $\quad S_c = \dfrac{C_s H_c}{1+e_o} \log\left(\dfrac{\sigma_o' + \Delta\sigma_{av}'}{\sigma_o'}\right)$

For OC clay $\left(\sigma_o' < \sigma_c' < \sigma_o' + \Delta\sigma_{av}'\right)$ $\quad S_c = \dfrac{C_s H_c}{1+e_o} \log\left(\dfrac{\sigma_c'}{\sigma_o'}\right) + \dfrac{C_c H_c}{1+e_o} \log\left(\dfrac{\sigma_o' + \Delta\sigma_{av}'}{\sigma_c'}\right)$

σ_o' = Average effective vertical stress before construction
$\Delta\sigma_{av}'$ = Average increase in effective vertical stress
σ_c' = Effective pre-consolidation pressure
e_o = Initial void ratio of the clay layer
C_c = Compression Index
C_s = Swelling Index
H_c = Thickness of the clay layer

$$\Delta\sigma_{av}' = \frac{1}{6}\left(\Delta\sigma_t' + 4\Delta\sigma_m' + \Delta\sigma_b'\right)$$



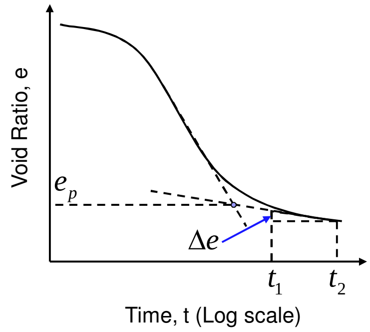
Settlement Due to Secondary Consolidation

$$S_s = \frac{C_\alpha H_c}{1+e_p}\log\left(\frac{t_2}{t_1}\right)$$

C_α = Secondary Compression Index $= \dfrac{\Delta e}{\log(t_2/t_1)}$

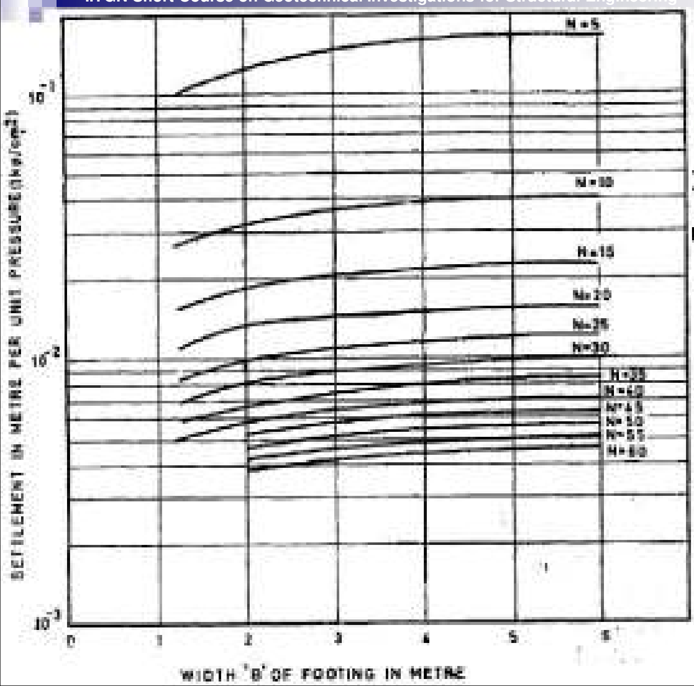
e_p = Void ratio at the end of primary consolidation

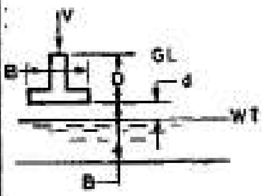
H_c = Thickness of Clay Layer

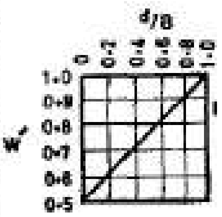


Secondary consolidation settlement is more important in the case of organic and highly-compressible inorganic clays

Total Settlement from SPT Data for Cohesionless soil







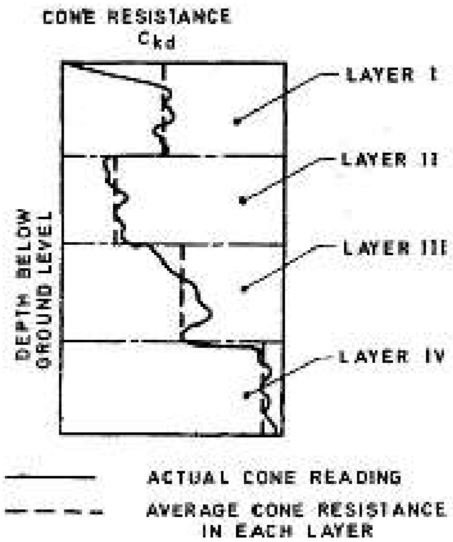
Multiply the settlement by factor W'

Total Settlement from CPT Data for Cohesionless soil

$$S_t = \frac{H_t}{C}\ln\left[\frac{\sigma_o + \Delta\sigma}{\sigma_o}\right]$$

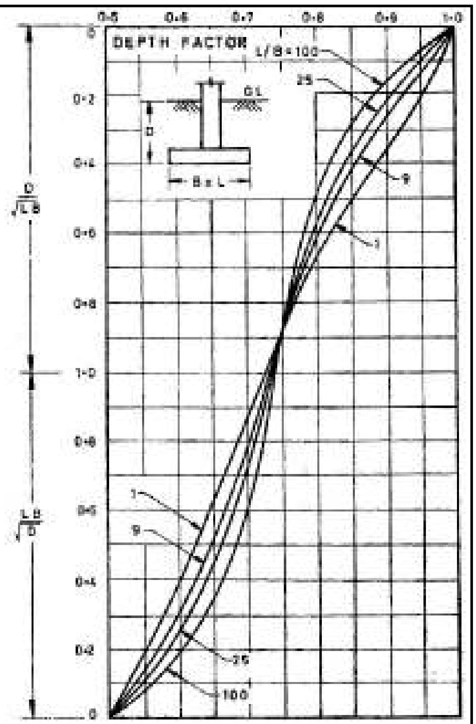
$$C = \frac{3}{2}\left(\frac{q_c}{\sigma_o}\right)$$

- Depth profile of cone resistance can be divided in several segments of average cone resistance
- Average cone resistance can be used to calculate constant of compressibility.
- Settlement of each layer is calculated separately due to foundation loading and then added together



Fox's Depth Correction Factor for Rectangular Footings of (L)x(B) at Depth (D)

$$\frac{(S_c)_{Embedded}}{(S_c)_{Surface}} = \text{Depth factor}$$



Rigidity Factor as per IS:8009-1976

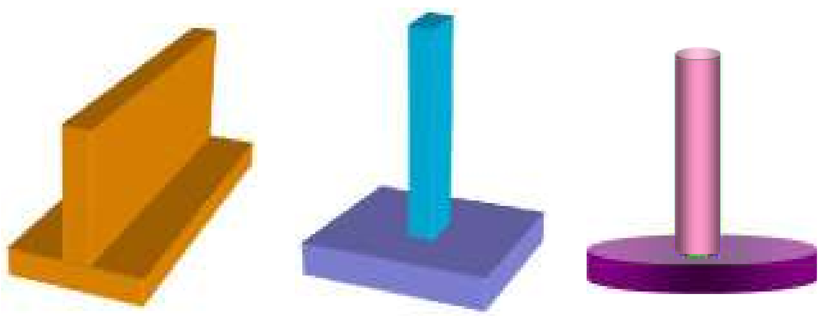
$$\frac{\text{Total settlement of rigid foundation}}{\text{Total settlement at the center of flexible foundation}}$$

$$\text{Rigidity factor} = 0.8$$

Shallow Foundation Design

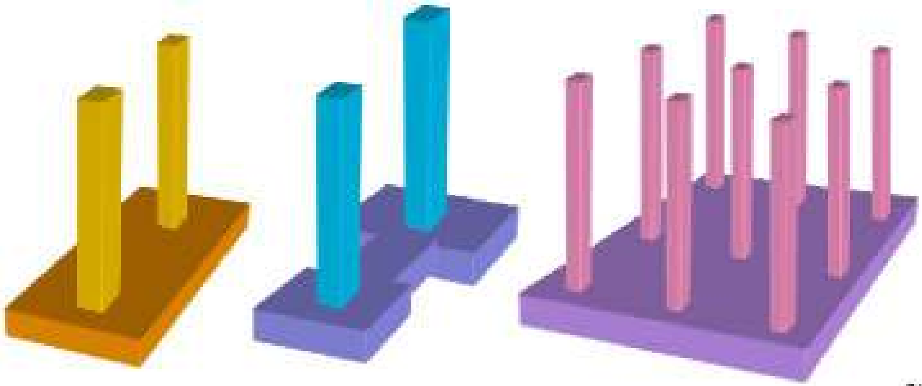
Common Types of Footing

- Strip footing
- Spread Footing



Common Types of Footing

- Combined Footing
- Raft or Mat footing



Location and depth of Foundation

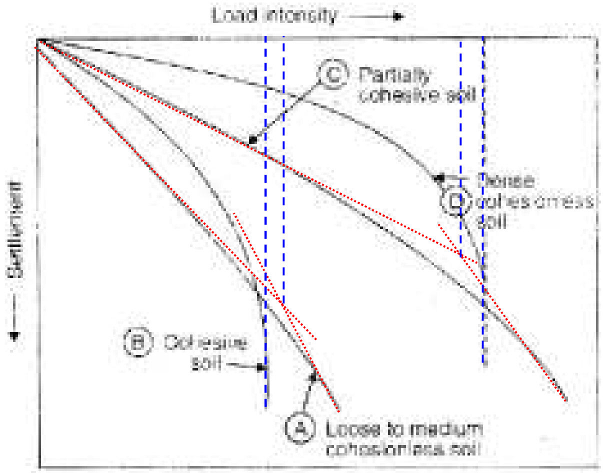
- IS:1904-1986: Minimum depth of foundation = 0.50 m.
- Foundation shall be placed below the zone of
  - Excessive volume change due to moisture variation (usually exists within 1.5 to 3.5 m depth)
  - Topsoil or organic material
  - Unconsolidated material such as waste dump
- Foundations adjacent to flowing water (flood water, rivers, etc.) shall be protected against scouring.
- A raised water table may cause damage to the foundation by
  - Floating the structure
  - Reducing the effective stress beneath the foundation
  - Water logging around the building: proper drainage system around the foundation may be required so that water does not accumulate.

Location and depth of Foundation

- Footings on surface rock or sloping rock faces
  - Shallow rock beds: foundation on the rock surface after chipping
  - Rock bed with slope: provide dowel bars of minimum 16 mm diameter and 225 mm embedment into the rock at 1 m spacing.
- Footings adjacent to existing structures
  - Minimum horizontal distance between the foundations shall not be less than the width of larger footing. Otherwise, the principal of 2H:1V distribution be used to minimize influence to old structure
  - Proper care is needed during excavation phase of foundation construction beyond merely depending on the 2H:1V criteria. Excavation may cause settlement to old foundation due to lateral bulging in the excavation and/or shear failure due to reduction in overburden stress in the surrounding of old foundation

Plate Load Test – IS:1888-1982

Plate Load Test: Bearing Capacity



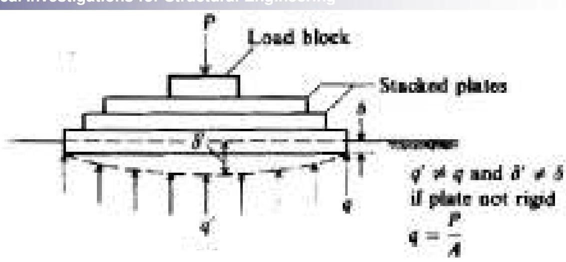
For cohesionless soil

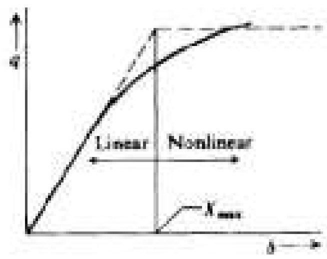
$$\boxed{\frac{q_{uf}}{q_{up}} = \frac{B_f}{B_p}}$$

For cohesive soil

$$\boxed{q_{uf} = q_{up}}$$

Modulus of Sub-grade Reaction





For footings on clay

$$k_s = k_1 \frac{B_1}{B}$$

For footings on sand

$$k_s = k_1 \left(\frac{B + B_1}{2B} \right)^2$$

TABLE 1 PERMISSIBLE DIFFERENTIAL SETTLEMENTS AND TILT (ANGULAR DISTORTION) FOR SHALLOW FOUNDATION IN SOILS

(Clause 16.3.4)

Sl No.	Type of Structure	Isolated Foundations: Sand and Hard Clay: Maximum settlement	Differential settlement	Angular distortion	Isolated Foundations: Plastic Clay: Maximum settlement	Differential settlement	Angular distortion	Raft Foundations: Sand and Hard Clay: Maximum settlement	Differential settlement	Angular distortion	Raft Foundations: Plastic Clay: Maximum settlement	Differential settlement	Angular distortion
		mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
i)	For steel structure	50	·003 3L	1/300	50	·003 3L	1/300	75	·003 3L	1/300	100	·003 3L	1/300
ii)	For reinforced concrete structures	50	·001 5L	1/666	75	·001 5L	1/666	75	·002 1L	1/500	100	·002L	1/500
iii)	For multistoreyed buildings												
	a) RC or steel framed buildings with panel walls	60	·002L	1/500	75	·002L	1/500	75	0·002 5L	1/400	125	0·003 3L	1/300
	b) For load bearing walls												
	1) L/H = 2*	60	·000 2L	1/5 000	60	·000 2L	1/5 000	Not likely to be encountered					
	2) L/H = 7*	60	·000 4L	1/2 500	60	·000 4L	1/2 500						
iv)	For water towers and silos	50	·001 5L	1/666	75	·001 5L	1/666	100	·002 5L	1/400	125	·002 5L	1/400

NOTE — The values given in the table may be taken only as a guide and the permissible total settlement/different settlement and tilt (angular distortion) in each case should be decided as per requirements of the designer.

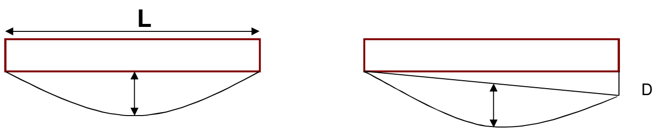
L denotes the length of deflected part of wall/raft or centre-to-centre distance between columns.

H denotes the height of wall from foundation footing.

*For intermediate ratios of L/H, the values can be interpolated.

Differential Settlement

- Caused by variations in soil profile and structural loads
- Consider construction tolerances
- Consider best case/worst case scenarios
- Use δ_D/δ ratios

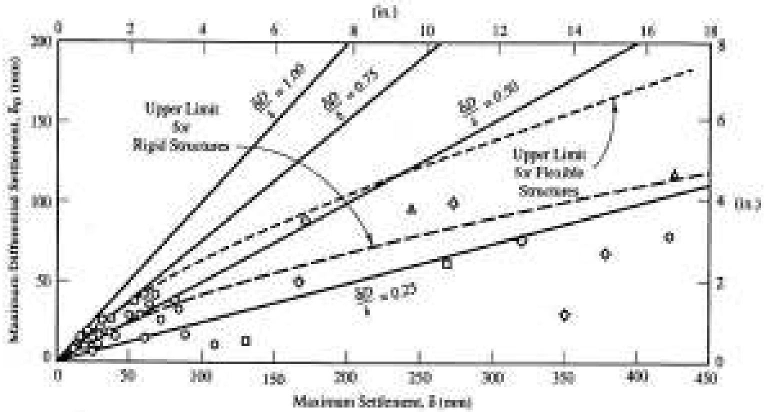


δ = maximum settlement

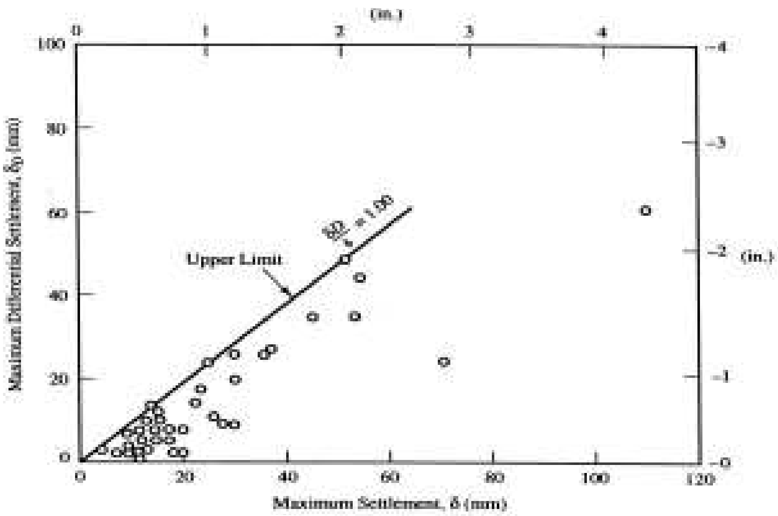
δ_D= differential settlement

δ_D/ = angular distortion

Total and Differential Settlement for Clays



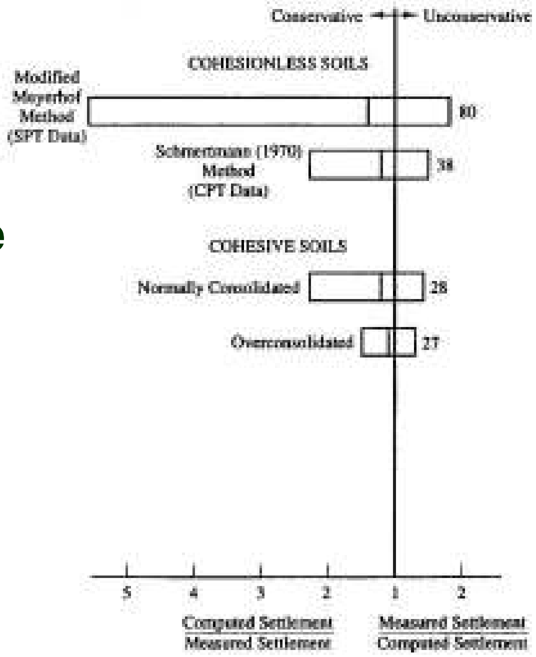
Total and Differential Settlement for Sands



Design values of δ_D/δ Ratios

Predominant Soil Type Below Footings	Design Value of δ_D/δ: Flexible Structures	Design Value of δ_D/δ: Rigid Structures
Sandy		
Natural soils	0.9	0.7
Compacted fills of uniform thickness underlain by stiff natural soils	0.5	0.4
Clayey		
Natural soils	0.8	0.5
Compacted fills of uniform thickness underlain by stiff natural soils	0.4	0.3

How Accurate are our Settlement Predictions?



Allowable Bearing Pressure

- Maximum bearing pressure that can be applied on the soil satisfying two fundamental requirements
  - Bearing capacity with adequate factor of safety – net safe bearing capacity
  - Settlement within permissible limits (critical in most cases) – net safe bearing pressure

Allowable Bearing Pressure

Teng's (1962) Correlation:

Net safe bearing pressure

$$q_{np} = 1.4\left(N_{cor} - 3\right)\left(\frac{B+0.3}{2B}\right)^2 R'_w C_D S_a \quad kN/m^2$$

S_a in mm and all other dimensions in meter.

$$C_D = \text{ depth correction factor } = 1 + \frac{D_f}{B} \le 2$$

$$N_{cor} = C_N . N$$

$$C_N = \left(\frac{1.75}{\sigma'_o/P_a + 0.7}\right) \quad \text{for } 0 < \left(\sigma'_o/P_a\right) \le 1.05$$

$$C_N = \left(\frac{3.5}{\sigma'_o/P_a + 0.7}\right) \quad \text{for } 1.05 < \left(\sigma'_o/P_a\right) \le 2.8$$

$\sigma'_o =$ Effective Overburden stress

Allowable Bearing Pressure

Meyerhof's (1974) Correlation:

Net safe bearing pressure

$$q_{np} = 0.49N''R_{D1}S_a \quad kN/m^2 \quad \text{for } B \le 1.2 \text{ m}$$

$$q_{np} = 0.32N''R_{D2}\left(\frac{B+0.3}{B}\right)^2 S_a \quad kN/m^2 \quad \text{for } B > 1.2 \text{ m}$$

$R_{D1} =$ depth correction factor $= 1 + 0.2\frac{D_f}{B} \le 1.2$

$R_{D2} =$ depth correction factor $= 1 + 0.33\frac{D_f}{B} \le 1.33$

Bowel's (1982) Correlation:

$$q_{np} = 0.73N''R_{D1}S_a \quad kN/m^2 \quad \text{for } B \le 1.2 \text{ m}$$

$$q_{np} = 0.48N''R_{D2}\left(\frac{B+0.3}{B}\right)^2 S_a \quad kN/m^2 \quad \text{for } B > 1.2 \text{ m}$$

N-value corrected for overburden using bazaraa's equation, but the N-value must not exceed field value

Allowable Bearing Pressure

IS Code recommendation: Use total settlement correlations with SPT data to determine safe bearing pressure.

Correlations for raft foundations:

Rafts are mostly safe in bearing capacity and they do not show much differential settlements as compared to isolated foundations.

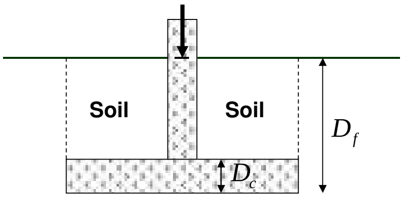
Teng's Correlation: $q_{np} = 0.7\left(N''-3\right)R_w' C_D S_a \quad kN/m^2$

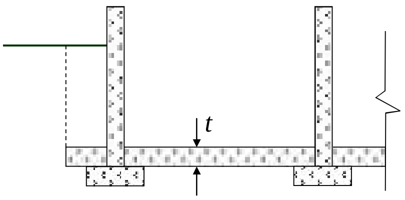
Peck, Hanson, and Thornburn (1974): $q_{a-net} = 0.88C_w N''S_a \quad kN/m^2$

Correlations using CPT data:

Meyerhof's correlations may be used by substituting $q_c/2$ for N, where q_c is in kg/cm^2.

Net vs. Gross Allowable Bearing Pressure





Gross load

$$Q_g = Q_c + B^2 D_c \gamma_c + B^2 \left(D_f - D_c\right)\gamma$$

$$q_g = \frac{Q_g}{B^2} = \frac{Q_c}{B^2} + \gamma D_f + D_c\left(\gamma_c - \gamma\right)$$

$$q_n = q_g - \gamma D_f = \frac{Q_c}{B^2} + D_c\left(\gamma_c - \gamma\right)$$

$\left(\gamma_c - \gamma\right)$ is small, so it may be neglected

$$q_n = \frac{Q_c}{B^2}$$

$$\frac{Q_c}{B^2} \le q_{a-net}$$

$$q_g = \frac{Q_c}{B^2} + D_c \gamma_c + t\gamma_c$$

$$q_n = q_g - \gamma D_f = \frac{Q_c}{B^2} + \gamma_c\left(D_c + t\right) - \gamma D_f$$

Usually D_c+t is much smaller than D_f

$$q_n = \frac{Q_c}{B^2} - \gamma D_f$$

$$\frac{Q_c}{B^2} \le q_{a-net} + \gamma D_f$$

$$\frac{Q_c}{B^2} \le q_{a-gross}$$

SUMMARY of Terminology

Gross Loading Intensity

Total pressure at the level of foundation including the weight of superstructure, foundation, and the soil above foundation.

$$q_g = \frac{Q_{\text{superstructure}} + Q_{\text{Foundation}} + Q_{\text{soil}}}{A_{\text{Foundation}}}$$

Net Loading Intensity

Pressure at the level of foundation causing actual settlement due to stress increase. This includes the weight of superstructure and foundation only.

$$q_n = q_g - \gamma D_f$$

Ultimate Bearing capacity:

Maximum gross intensity of loading that the soil can support against shear failure is called ultimate bearing capacity.

q_u from

Bearing capacity calculation

Net Ultimate Bearing Capacity:

Maximum net intensity of loading that the soil can support at the level of foundation.

$$q_{nu} = q_u - \gamma D_f$$

SUMMARY of Terminology

Net Safe Bearing capacity:

Maximum net intensity of loading that the soil can safely support without the risk of shear failure.

$$q_{ns} = \frac{q_{nu}}{FOS}$$

Gross Safe Bearing capacity:

Maximum gross intensity of loading that the soil can safely support without the risk of shear failure.

$$q_{gs} = q_{ns} + \gamma D_f$$

Safe Bearing Pressure:

Maximum net intensity of loading that can be allowed on the soil without settlement exceeding the permissible limit.

q_{ps} from settlement analysis

Allowable Bearing Pressure:

Maximum net intensity of loading that can be allowed on the soil with no possibility of shear failure or settlement exceeding the permissible limit.

q_{a-net} Minimum of bearing capacity and settlement analysis

Loads on Foundation

- **Permanent Load:** This is actual service load/sustained loads of a structure which give rise stresses and deformations in the soil below the foundation causing its settlement.
- **Transient Load:** This momentary or sudden load imparted to a structure due to wind or seismic vibrations. Due to its transitory nature, the stresses in the soil below the foundation carried by such loads are allowed certain percentage increase over the allowable safe values.
- **Dead Load:** It includes the weight of the column/wall, footings, foundations, the overlaying fill but excludes the weight of the displaced soil
- **Live Load:** This is taken as per the specifications of IS:875 (pt-2) – 1987.

Loads for Proportioning and Design of Foundation IS:1904 - 1986

- Following combinations shall be used
  - Dead load + Live load
  - Dead Load + Live load + Wind/Seismic load
- For cohesive soils only 50% of actual live load is considered for design (Due to settlement being time dependent)
- For wind/seismic load < 25% of Dead + Live load
  - Wind/seismic load is neglected and first combination is used to compare with safe bearing load to satisfy allowable bearing pressure
- For wind/seismic load ≥ 25% of Dead + Live load
  - It becomes necessary to ensure that pressure due to second combination of load does not exceed the safe bearing capacity by more than 25%. When seismic forces are considered, the safe bearing capacity shall be increased as specified in IS: 1893 (Part-1)-2002 (see next slide). In non-cohesive soils, analysis for liquefaction and settlement under earthquake shall also be made.

Table 1 Percentage of Permissible Increase in Allowable Bearing Pressure or Resistance of Soils

(*Clause* 6.3.5.2)

Sl No.	Foundation	Type of Soil Mainly Constituting the Foundation		
		Type I Rock or Hard Soils: Well graded gravel and sand gravel mixtures with or without clay binder, and clayey sands poorly graded or sand clay mixtures (GB, CW, SB, SW, and SC)[1] having N[2] above 30, where N is the standard penetration value	*Type II Medium Soils*: All soils with N between 10 and 30, and poorly graded sands or gravelly sands with little or no fines (SP[1]) with $N > 15$	*Type III Soft Soils*: All soils other than SP[1] with $N < 10$
(1)	(2)	(3)	(4)	(5)
i)	Piles passing through any soil but resting on soil type I	50	50	50
ii)	Piles not covered under item i	—	25	25
iii)	Raft foundations	50	50	50
iv)	Combined isolated RCC footing with tie beams	50	25	25
v)	Isolated RCC footing without tie beams, or unreinforced strip foundations	50	25	—
vi)	Well foundations	50	25	25

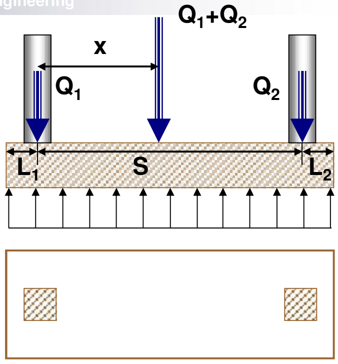
Other considerations for Shallow Foundation Design

- For economical design, it is preferred to have square footing for vertical loads and rectangular footing for the columns carrying moment
- Allowable bearing pressure should not be very high in comparison to the net loading intensity leading to an uneconomical design.
- It is preferred to use SPT or Plate load test for cohesionless soils and undrained shear strength test for cohesive soils.
- In case of lateral loads or moments, the foundation should also be checked to be safe against sliding and overturning. The FOS shall not be less than 1.75 against sliding and 2.0 against overturning. When wind/seismic loads are considered the FOS is taken as 1.5 for both the cases.

Combined Footings

- Combined footing is preferred when
  - The columns are spaced too closely that if isolated footing is provided the soil beneath may have a part of common influence zone.
  - The bearing capacity of soil is such that isolated footing design will require extent of the column foundation to go beyond the property line.
- Types of combined footings
  - Rectangular combined footing
  - Trapezoidal combined footing
  - Strap beam combined footing

Rectangular Combined Footing



- If two or more columns are carrying almost equal loads, rectangular combined footing is provided
- Proportioning of foundation will involve the following steps
  - Area of foundation $A = \dfrac{Q_1 + Q_2}{q_{a-net}}$
  - Location of the resultant force $x = \dfrac{Q_2 S}{Q_1 + Q_2}$
  - For uniform distribution of pressure under the foundation, the resultant load should pass through the center of foundation base.
    Length of foundation, $L = 2\left(L_1 + S\right)$
    Offset on the other side, $L_2 = L - S - L_1 > 0$
  - The width of foundation, $B = A/L$

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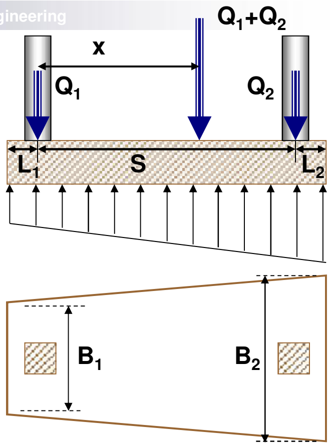
Trapezoidal Combined Footing



- If one of the columns is carrying much larger load than the other one, trapezoidal combined footing is provided
- Proportioning of foundation will involve the following steps if L, and L$_1$ are known
  - Area of foundation $A = \dfrac{Q_1 + Q_2}{q_{a-net}}$
  - Location of the resultant force $x = \dfrac{Q_2 S}{Q_1 + Q_2}$
  - For uniform distribution of pressure under the foundation, the resultant load should pass through the center of foundation base. This gives the relationship,

$$x + L_1 = \left(\frac{B_1 + 2B_2}{B_1 + B_2}\right)\frac{L}{3}$$

  - Area of the footing, $\dfrac{B_1 + B_2}{2} L = A$

Solution of these two equations gives B$_1$ and B$_2$

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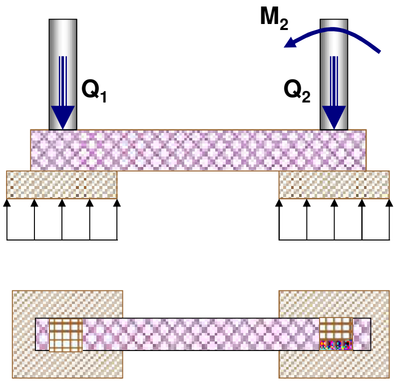
Strap Combined Footing



- Strap footing is used to connect an eccentrically loaded column footing to an interior column so that the moment can be transferred through the beam and have uniform stress distribution beneath both the foundations.
- This type of footing is preferred over the rectangular or trapezoidal footing if distance between the columns is relatively large.
- Some design considerations:
  - Strap must be rigid: $I_{strap}/I_{footing} > 2$.
  - Footings should be proportioned to have approximately equal soil pressure in order to avoid differential settlement
  - Strap beam should not have contact with soil to avoid soil reaction to it.

Pile Foundation Design

When is it needed

- Top layers of soil are highly compressible for it to support structural loads through shallow foundations.
- Rock level is shallow enough for end bearing pile foundations provide a more economical design.
- Lateral forces are relatively prominent.
- In presence of expansive and collapsible soils at the site.
- Offshore structures
- Strong uplift forces on shallow foundations due to shallow water table can be partly transmitted to Piles.
- For structures near flowing water (Bridge abutments, etc.) to avoid the problems due to erosion.

Types of Piles Based on Their Function and Effect of Installation

- Piles based on their function
  - End Bearing Piles
  - Friction Piles
  - Compaction Piles
  - Anchor Piles
  - Uplift Piles
- Effect of Installation
  - Displacement Piles
  - Non-displacement Piles

Displacement Piles

- In loose cohesionless soils
  - Densifies the soil upto a distance of 3.5 times the pile diameter (3.5D) which increases the soil's resistance to shearing
  - The friction angle varies from the pile surface to the limit of compacted soil
- In dense cohesionless soils
  - The dilatancy effect decreases the friction angle within the zone of influence of displacement pile (3.5D approx.).
  - Displacement piles are not effective in dense sands due to above reason.
- In cohesive soils
  - Soil is remolded near the displacement piles (2.0 D approx.) leading to a decreased value of shearing resistance.
  - Pore-pressure is generated during installation causing lower effective stress and consequently lower shearing resistance.
  - Excess pore-pressure dissipates over the time and soil regains its strength.
- Example: Driven concrete piles, Timber or Steel piles

Non-displacement Piles

- Due to no displacement during installation, there is no heave in the ground.
- Cast in-situ piles may be cased or uncased (by removing casing as concreting progresses). They may be provided with reinforcement if economical with their reduced diameter.
- Enlarged bottom ends (three times pile diameter) may be provided in cohesive soils leading to much larger point bearing capacity.
- Soil on the sides may soften due to contact with wet concrete or during boring itself. This may lead to loss of its shear strength.
- Concreting under water may be challenging and may resulting in waisting or necking of concrete in squeezing ground.
- Example: Bored cast in-situ or pre-cast piles

Load Transfer Mechanism of Piles

- The frictional resistance per unit area at any depth → $q_{sz} = \dfrac{\Delta Q_z}{S.\Delta z}$

  $S =$ perimeter of pile

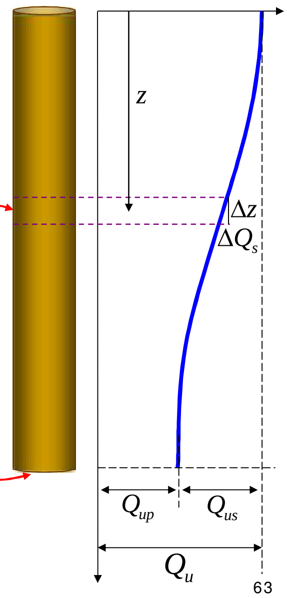
- Ultimate skin friction resistance of pile → Q_{su}

- Ultimate point load → $Q_{pu} = q_{pu}.A_p$

  $q_{pu} =$ bearing capacity of soil
  $A_p =$ bearing area of pile

- Ultimate load capacity in compression → $Q_u = Q_{pu} + Q_{su}$

- Ultimate load capacity in tension → $Q_u = Q_{su}$



IS:2911 → Pile Load Capacity in Cohesionless Soils

$$Q_u = A_P(0.5D\gamma N_\gamma) + A_p(P_D N_q) + \sum_{1}^{n} K_1 P_{D1} \tan\delta A_{s1}$$

A_P = cross-sectional area of the pile
D = stem diameter of pile
γ = unit weight of soil
$N_q\ N_\gamma$ = bearing capacity factor taken for general shear
P_D = effective overburden pressure (critical depth taken as 15D for $\phi \le 30°$ and 20D for $\phi > 40°$—Indian Railways recommend only 6D for $\phi = 26°$
K_1 = coefficient of earth pressure
P_{D1} = effective overburden pressure of corresponding layer (This effect is controlled by prescribing limiting friction).
δ = angle of wall friction usually taken as 3/4ϕ of soil.
A_s = surface area of pile.

IS:2911 → Pile Load Capacity in Cohesionless Soils

End bearing resistance = small (N_γ effect) + very large (N_q effect) + friction

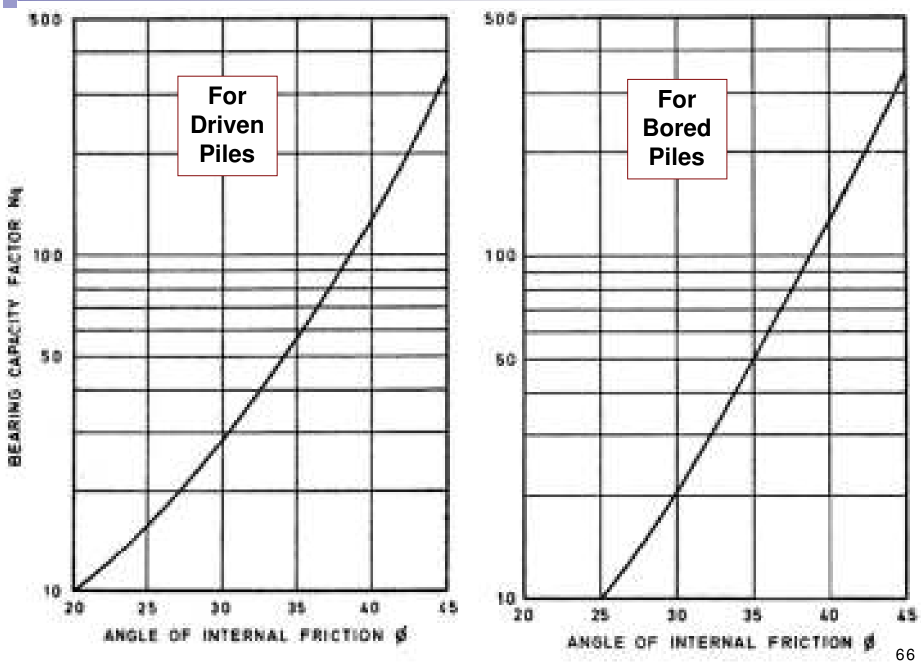
$$Q_{uk} = A_p P_D N_q + \sum_{1}^{n} K_i P_{Di} \tan\delta$$

A conservative value of $K = 1$ can be assumed for all piles except for piles with steel liners, where $K = 0.7$ can be assumed. *The ultimate frictional resistance should preferably be restricted to 6 t/m² in sands.*

IS 2911—Part 1 Sec. 2 states that for bored piles in loose to medium sands, K value of 1 to 1.5 can be used.

IS 2911 also states that the ultimate base resistance in sand should be restricted to a maximum value of 150 kg/cm² (1500 *t*/m²) for precast driven piles and 100 to 110 kg/cm² (1000 to 1100 *t*/m²) for cast in-situ piles.

In working out pile capacities using static formula, for piles longer than 15 to 20 pile diameter, maximum effective overburden at the pile tip should correspond to pile length equal to 15 to 20 diameters.



IS:2911 → Pile Load Capacity in Cohesionless Soils

For driven cast in-situ piles, value of ϕ is kept unchanged.

For driven precast piles the value ϕ is changed to $(\phi + 40)/2$ to take care of compaction due to pile driving. (Thus, if for $\phi = 30°$, ϕ is taken as 35° for driven piles).

For bored cast in-situ piles where the bottom of the hole is cleaned thoroughly by continuous mud circulation, value ϕ is assumed as unchanged.

For bored cast in-situ piles where continuous mud circulation is not used for cleaning the base, the value of is reduced by 3 to 5 degrees.

IS:2911 → Pile Load Capacity in Cohesionless Soils

IS code recommends K-value to be chosen between 1 and 2 for driven piles and 1 and 1.5 for bored piles. However, it is advisable to estimate this value based on the type of construction and fair estimation of the disturbance to soil around pile. Typical values of ratio between K and K_o are listed below.

Method of installation	K/K_0*
Driven large displacement piles (Concrete piles)	1 to 2
Driven small displacement piles (Steel *H* piles)	0.75 to 1.75
Bored cast in-situ piles	0.7 to 1
Jetted piles	0.5 to 0.7

*$K_0 = (1 - \sin\phi)$ = coefficient of earth pressure at rest.

IS:2911 → Pile Load Capacity in Cohesive Soils

$$Q_u = A_p N_c c_p + \sum_{i=1}^{n} \alpha_i c_i A_{si}$$

N_c = bearing capacity factor in clays which is taken as 9
c_p = average cohesion at pile toe
α_i = adhesion factor
c_i = average cohesion of the ith layer on the side of the pile
A_{si} = surface area of pile stem in the ith layer.
$\alpha_i c_i$ = adhesion between shaft of pile and clay.

Tomlinson's recommendations

$$\text{For } \overline{\sigma'}_v / c_u \geq 1 \quad \rightarrow \quad \alpha = 0.5\left(\overline{\sigma'}_v / c_u\right)^{0.5}, \text{ but } \not> 1$$

$$\text{For } \overline{\sigma'}_v / c_u < 1 \quad \rightarrow \quad \alpha = 0.5\left(\overline{\sigma'}_v / c_u\right)^{0.25}, \text{ but } \not< 0.5 \text{ and } \not> 1$$

For bored piles the value of α as obtained above is to be multiplied by 0.8.

IS:2911 → Pile Load Capacity in Cohesive Soils

SPT values of clays N	Consistency	Range of cohesion (kN/m^2)	Range of cohesion (kg/cm^2)	Adhesion factor α Driven	Adhesion factor α Bored
< 4	Soft to very soft	1 to 25	0.01–0.25	> 1.0	Reduce the driven values by factor 0.8
4 to 8	Medium stiff	25 to 50	0.25–0.5	0.7–0.4	
8 to 15	Stiff	50 to 100	0.5–1.0	0.4–0.3	
≥ 15	Stiff to hard	≥ 100	> 1.0	0.3–0.25	

The value of c for clays is $N/16$ to $N/20$ kg/cm^2 (approximately) as derived from N values.

The value of α shall be limited to 0.5 for sensitive clays.

The value of α may be more than 0.7 in clays overlain by sand.

Meyerhof's Formula for Driven Piles based on SPT value

For Sand:

$$Q_u = 4(L/D)NA_p + (\bar{N}/5)A_s \text{ in tons (where } L/D \ngtr 10)$$

For L/D > 10

$$Q_u = 40\,NA_p + (\bar{N}/5)A_s \text{ tons (where areas } A_p \text{ and } A_s \text{ are in m}^2)$$

A limiting value of 1000 t/m^2 for point bearing and 6 t/m2 is suggested

For Non-plastic silt and fine sand:

$$Q_u = 3(L/D)NA_p + (\bar{N}/6)A_s \text{ tons} \qquad (\text{when areas are in m}^2)$$

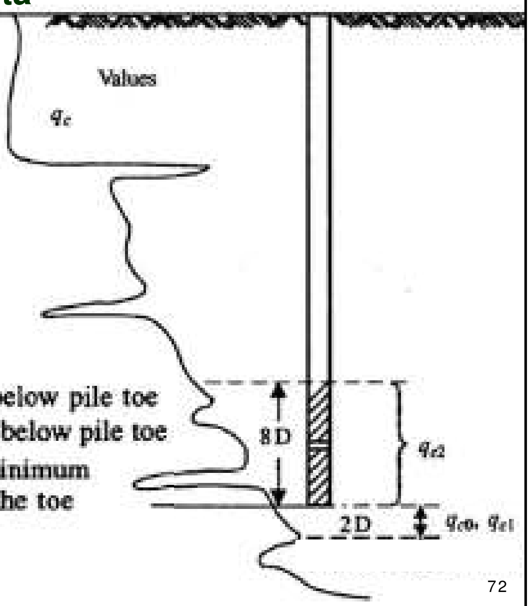
$$Q_u = 30\,NA_p + (\bar{N}/6)A_s \text{ tons} \qquad \text{for } L/D > 10$$

For Clays:

$$Q_u = 9cA_p + \alpha c A_s = 9\left(\frac{N}{20}\right)(10)A_p + \alpha\left(\frac{\bar{N}}{20}\right)(10)A_s \text{ (tons)}$$

$$= 4.5NA_p + (\bar{N}/2)A_s \text{ metric tons (assuming } \alpha = 1).$$

IS:2911 → Pile Load Capacity in Non-Cohesive Soils Based on CPT data



The ultimate point bearing capacity:

$$q_u = \frac{\dfrac{q_{c0} + q_{c1}}{2} + q_{c2}}{2}$$

q_{c0} = average SCPT value for $2D$ below pile toe
q_{c1} = minimum SCPT value for $2D$ below pile toe
q_{c2} = average of the envelope of minimum SCPT value over $8D$ above the toe of the pile.

IS:2911 → Pile Load Capacity in Non-Cohesive Soils Based on CPT data

The ultimate skin friction resistance:

Type of soil	Local side friction (q_c in kN/m^2)
q_c < 1000 kN/m^2	$q_c/30$ to $3q_c/30$
Clays	$q_c/25$ to $2q_c/25$
Silty sands, silty clays	$q_c/100$ to $4q_c/100$
Sands	$q_c/100$ to $2q_c/100$
Coarse sands and gravel	less than $q_c/150$

Correlation of SPT and CPT:

Soil type	$\frac{q_c\,(\text{kN/m}^2)}{N}$	$\frac{q_c\,(\text{kg/cm}^2)}{N}$
Clays	150–200	1.5–2.0
Silts, sandy silts and slightly cohesive silt-sand	200–250	2.0–2.5
Clean fine to medium sand and slightly silty sands	300–400	3.0–4.0
Coarse sand and sand with little gravel	500–600	5.0–6.0
Sandy gravel and gravel	800–100	8.0–10.0

Note: q_c in kg/cm^2 = 1.5 to 10 times N value depending on soil type.

Allowable Pile Capacity

$$\boxed{Q_{all} = \frac{Q_u}{FS}}$$

- Factor of Safety shall be used by giving due consideration to the following points
  - Reliability of soil parameters used for calculation
  - Mode of transfer of load to soil
  - Importance of structure
  - Allowable total and differential settlement tolerated by structure

Factor of Safety as per IS 2911:

Case	Factor of safety
1. On total capacity	2.5
2. On shaft resistance	1.5
3. On base resistance	3.0

Note: For dynamic formula, FS = 2.5 for soils and 1.5 for rocks is commonly used.

Load Tests on Piles

Initial Test — This test is required for one or more of the following purposes. This is done in case of important and/or major projects and number of tests may be one or more depending upon the number of piles required.

NOTE — In case specific information about strata and past guiding experience is not available, there should be a minimum of two tests.

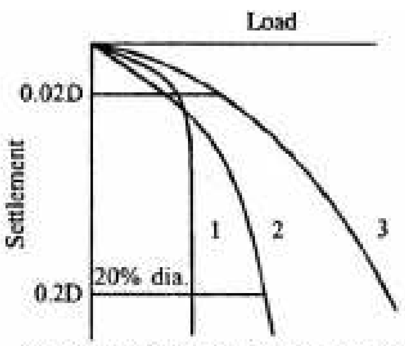
Note: Piles used for initial testing are loaded to failure or at least twice the design load. Such piles are generally not used in the final construction.

Routine Test This test is required for one or more of the following purposes. The number of tests may generally be one-half percent of the total number of piles required. The number of the test may be increased up to 2 percent in a particular case depending upon nature, type of structure and strata condition:

Note: During this test pile should be loaded upto 1.5 times the working (design) load and the maximum settlement of the test should not exceed 12 mm. These piles may be used in the final construction.

Vertical Load Test: Maintained Load Test

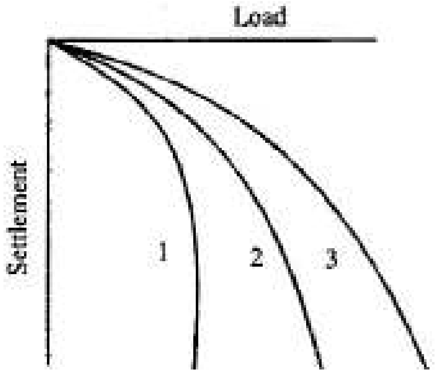
- The test can be initial or routine test
- The load is applied in increments of 20% of the estimated safe load. Hence the failure load is reached in 8-10 increments.
- Settlement is recorded for each increment until the rate of settlement is less than 0.1 mm/hr.
- The ultimate load is said to have reached when the final settlement is more than 10% of the diameter of pile or the settlement keeps on increasing at constant load.



(1) Shaft resistance; (2) Base resistance; (3) Total resistance

Vertical Load Test: Maintained Load Test

- After reaching ultimate load, the load is released in decrements of 1/6th of the total load and recovery is measured until full rebound is established and next unload is done.
- After final unload the settlement is measured for 24 hrs to estimate full elastic recovery.
- Load settlement curve depends on the type of pile



(1) Friction piles; (2) Bearing piles; (3) Friction and bearing piles.

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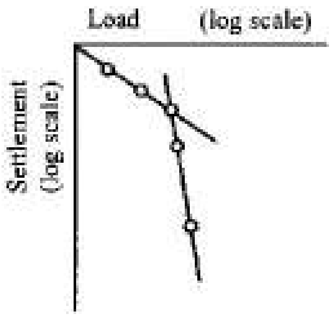
Vertical Load Test: Maintained Load Test → Ultimate Load

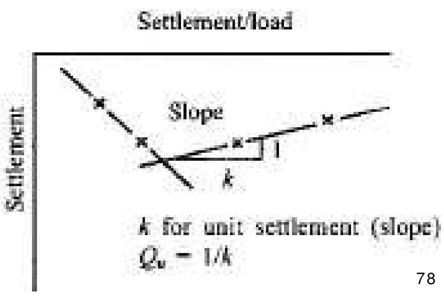
De Beer (1968):

Load settlement curve is plotted in a log-log plot and it is assumed to be a bilinear relationship with its intersection as failure load



Chin Fung Kee (1977):

Assumes hyperbolic curve. Relationship between settlement and its division with load is taken as to be bilinear with its intersection as failure load



Vertical Load Test: Maintained Load Test → Safe Load as per IS: 2911

Safe Load for Single Pile:

a) Two-thirds of the final load at which the total displacement attains a value of 12 mm unless otherwise required in a given case on the basis of nature and type of structure in which case, the safe load should be corresponding to the stated total displacement permissible.

b) 50 percent of the final load at which the total displacement equal 10 percent of the pile diameter in case of uniform diameter piles and 7.5 percent of bulb diameter in case of under-reamed piles.

Safe Load for Pile Group:

a) Final load at which the total displacement attains a value of 25 mm unless otherwise required in a given case on the basis of nature and type of structure, and

b) Two-thirds of the final load at which the total displacement attains a value of 40 mm.

Dynamic Pile Formula for Driven Piles: Modified Hiley Formula

$$\boxed{Q_u = \frac{\alpha . W . H \eta}{S + C/2}}$$

W = Weight of hammer

H = Height of fall

Q_u = Pile resistance or Pile capacity

S = Pile penetration for the last blow

α = Hammer fall efficiency

η = Efficiency of blow

C = Sum of temporary elastic compression of pile, dolly, packing, and ground

Note: Dynamic pile formula are not used for soft clays due to pore pressure evolution