CE-632
Foundation Analysis and Design

Pile Foundations

1

Indian Standards on Piles

- IS 2911 : Part 1 : Sec 1 : 1979 Driven cast in-situ concrete piles
- IS 2911 : Part 1 : Sec 2 : 1979 Bored cast-in-situ piles
- IS 2911 : Part 1 : Sec 3 : 1979 Driven precast concrete piles
- IS 2911 : Part 1 : Sec 4 : 1984 Bored precast concrete piles
- IS 2911 : Part 2 : 1980 Timber piles
- IS 2911 : Part 3 : 1980 Under reamed piles
- IS 2911 : Part 4 : 1985 Load test on piles
- IS 5121 : 1969 Safety code for piling and other deep foundations
- IS 6426 : 1972 Specification for pile driving hammer
- IS 6427 : 1972 Glossary of Terms Relating to Pile Driving Equipment
- IS 6428 : 1972 Specification for pile frame
- IS 9716 : 1981 Guide for lateral dynamic load test on piles
- IS 14362 : 1996 Pile boring equipment - General requirements
- IS 14593 : 1998 Bored cast-in-situ piles founded on rocks - Guidelines
- IS 14893 : 2001 Non-Destructive Integrity Testing of Piles (NDT) - Guidelines

2

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.

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Types of Piles

- Steel Piles
  - Pipe piles
  - Rolled steel H-section piles
- Concrete Piles
  - Pre-cast Piles
  - Cast-in-situ Piles
  - Bored-in-situ piles
- Timber Piles
- Composite Piles

Steel Piles: Facts

- Usual length: 15 m – 60 m
- Usual Load: 300 kN – 1200 kN
- Advantage:
  - Relatively less hassle during installation and easy to achieve cutoff level.
  - High driving force may be used for fast installation
  - Good to penetrate hard strata
  - Load carrying capacity is high
- Disadvantage:
  - Relatively expensive
  - Noise pollution during installation
  - Corrosion
  - Bend in piles while driving

Concrete Piles: Facts

- Pre-cast Piles:
  - Usual length: 10 m – 45 m
  - Usual Load: 7500 kN – 8500 kN
- Cast-in-situ Piles:
  - Usual length: 5 m – 15 m
  - Usual Load: 200 kN – 500 kN
- Advantage:
  - Relatively cheap
  - It can be easily combined with concrete superstructure
  - Corrosion resistant
  - It can bear hard driving
- Disadvantage:
  - Difficult to transport
  - Difficult to achieve desired cutoff

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

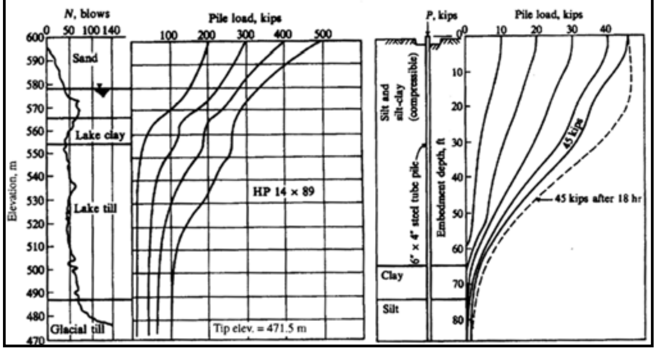
- In loose cohesionless soils
  - Densifies the soil upto a distance of 3.5 times the pile diameter (3.5D) which increases the soil's resistance to shearing
  - The friction angle varies from the pile surface to the limit of compacted soil
- In dense cohesionless soils
  - The dilatancy effect decreases 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

- With the increasing load on a pile initially the resistance is offered by side friction and when the side resistance is fully mobilized to the shear strength of soil, the rest of load is supported by pile end. At certain load the soil at the pile end fails, usually in punching shear, which is defined as the ultimate load capacity of pile.



Load Transfer Mechanism of Piles

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

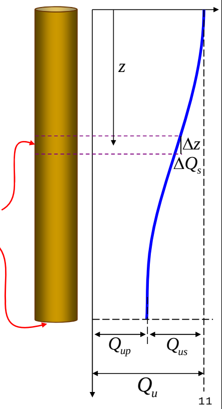
  $S =$ perimeter of pile

- Ultimate skin friction resistance of pile $\longrightarrow$ Q_{su}
- Ultimate point load $\longrightarrow$ $Q_{pu} = q_{pu}.A_p$

  $q_{pu} =$ bearing capacity of soil

  $A_p =$ bearing area of pile

- Ultimate load capacity in compression $\longrightarrow$ $Q_u = Q_{pu} + Q_{su}$
- Ultimate load capacity in tension $\longrightarrow$ $Q_u = Q_{su}$



Point Load capacity of Pile: General Bearing Capacity approach

- Ultimate bearing capacity of soil considering general bearing capacity equation. Shape, inclination, and depth factors are included in bearing capacity factors

$$q_{pu} = cN_c^* + q'N_q^* + 0.5\gamma DN_\gamma^*$$

- Since pile diameter is relatively small, third term may be dropped out

$$q_{pu} = cN_c^* + q'N_q^*$$

- Hence Pile load capacity

$$Q_{pu} = q_{pu}.A_p = \left(cN_c^* + qN_q^*\right).A_p$$

Point Load capacity of Pile: Meyerhof's (1976) Method

- Granular soils:
  Point bearing capacity of pile increases with depth in sands and reaches its maximum at an embedment ratio L/D = $(L/D)_{cr}$.
  Therefore, the point load capacity of pile is

$$Q_{pu} = A_p.q'.N_q^* < A_p.q_{ul}$$

$$q_{ul} = 0.5 P_a N_q^* \tan\phi' \qquad P_a = \text{Atmospheric pressure}$$

  - $(L/D)_{cr}$ value typically ranges from 15D for loose to medium sand to 20D for dense sands.
  - Correlation of limiting point resistance with SPT value

$$q_{ul} = 0.4\left(N''\right)\frac{L}{D} \le 4P_a\left(N''\right)$$

  N" value shall be taken as an average for a zone ranging from 10D above to 4D below the pile point.

- Saturated Clays:

$$Q_{pu} = N_c^*.c_u.A_p = 9.c_u.A_p$$

Point Load capacity of Pile: Vesic's (1977) Method

- Pile point bearing capacity based on the theory of expansion of cavities

$$Q_{pu} = A_p.q_{up} = A_p.\left(c.N_c^* + \sigma_o' N_\sigma^*\right)$$

Mean effective normal stress at pile end $\longrightarrow \quad \sigma_o' = \left(\frac{1+2K_o}{3}\right)q'$

$$N_\sigma^* = f\left(I_{rr}\right) \qquad I_{rr} = \frac{I_r}{1+I_r\Delta}$$

avg vol strain at pile end

Reduced rigidity index of soil

$$I_r = \text{rigidity index} = \frac{G_s}{\left(c' + q'\tan\phi'\right)} = \frac{E_s}{2\left(1+\mu_s\right)\left(c'+q'\tan\phi'\right)}$$

$$N_c^* = \frac{4}{3}\left(\ln I_{rr} + 1\right) + \frac{\pi}{2} + 1$$

Type of soil	I_r
Sand	75-150
Silt	50-75
Clay	150-250

Baldi et al. (1981):

For mechanical cone resistance	For electric cone resistance
$I_r = \frac{3}{q_f/q_c}$	$I_r = \frac{1.7}{q_f/q_c}$

Point Load capacity of Pile: Janbu's (1976) Method

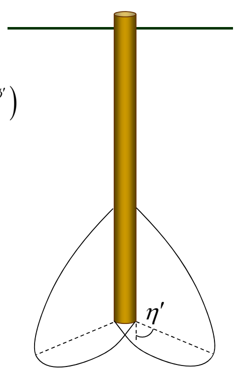
$$Q_{pu} = A_p\left(c.N_c^* + q'.N_q^*\right)$$

$$N_q^* = \left(\tan\phi' + \sqrt{1+\tan^2\phi'}\right)^2\left(e^{2\eta'\tan\phi'}\right)$$

$$60^o \le \eta' \le 90^o$$

Clay — Sand

$$N_c^* = \left(N_q^* - 1\right)\cot\phi'$$

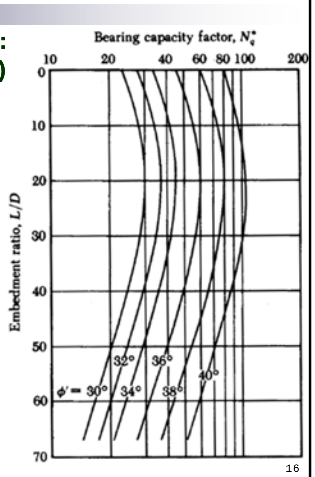


Point Load capacity of Pile: Coyle and Costello's (1981) Method for Granular Soils

$$Q_{pu} = A_p . q' . N_q^*$$

N_q^* is a function of $\frac{L}{D}$ ratio

L is length of pile below G.L.



Point Load Capacity of Pile resting on Rock

Goodman (1980): $Q_{pu} = A_p . q_u \left(N_\phi + 1 \right)$

$N_\phi = \tan^2 \left(45 + \phi'/2 \right)$

q_u = unconfined compression strength of rock

ϕ' = effective friction angle of rock

To consider the influence of distributed fractures in rock which are not reflected by the compression tests on small samples, the compression strength for design is taken as $\longrightarrow \left(q_u \right)_{design} = \frac{\left(q_u \right)_{lab}}{5}$

Type of rock	q_u MN/m²	q_u lb/in²	Angle of friction, ϕ' (deg)
Sandstone	70–140	10,000–20,000	27–45
Limestone	105–210	15,000–30,000	30–40
Shale	35–70	5000–10,000	10–20
Granite	140–210	20,000–30,000	40–50
Marble	60–70	8500–10,000	25–30

Frictional Resistance of Pile: In Sand

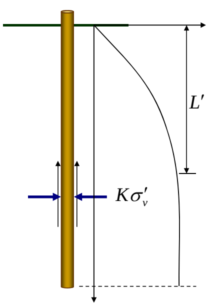
- The frictional resistance of pile may be computed as

$$Q_{su} = \sum S . \Delta L . f_{sz}$$

- The unit frictional resistance increases with the depth and reaches its maximum at the depth of approximately 15D to 20D, as shown in the adjacent figure.

$$f_{sz} = K . \sigma'_v . \tan\delta \leq f_{sL'}$$

- Soil-Pile interface friction angle δ varies from 0.5ϕ' to 0.8ϕ'.Earth pressure coefficient depends on both soil type and pile installation.



Pile type	K
Bored or jetted	$\approx K_o = 1 - \sin\phi'$
Low-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.4K_o = 1.4(1 - \sin\phi')$
High-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.8K_o = 1.8(1 - \sin\phi')$

Frictional Resistance of Pile: In Sand

Bhushan (1982) suggested that the value of K and $K.\tan\delta$ for large displacement piles can be computed as

$$K = 0.50 + 0.008D_r$$

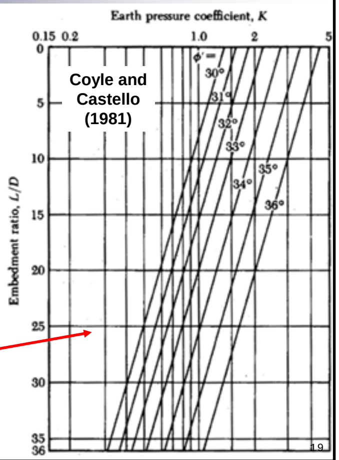
$$K.\tan\delta = 0.18 + 0.0065D_r$$

Coyle and Castello (1981) proposed that ultimate skin frictional resistance of pile can be computed as

$$Q_{su} = (f_s)_{av}.S.L$$

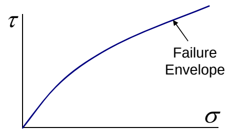
$$= \left(K\overline{\sigma'}_v.\tan\delta\right).S.L$$

Avg effective overburden



Frictional Resistance of Pile: In Sand

Zeitlen and Paikowski (1982) suggested that limiting f_s is automatically accounted for by the decrease in ϕ' with effective confining pressure which may be used to compute K and δ.



$$\phi' = \phi'_o - 5.5\log\frac{\sigma'_v}{\sigma'_o}$$

Effective vertical stress at the depth of interest

Effective confining stress during triaxial test

Friction angle obtained through triaxial testing at some confining pressure σ'_o.

Typical values of K from a number of pile tests:

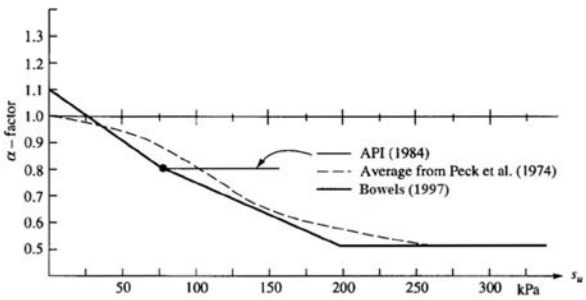
Source	H piles	Pipe	Precast concrete	Timber	Tapered Timber
Mansur and Hunter (1970)	1.4–1.9	1.2–1.3	1.45–1.6	1.25	
Tavenas (1971)	0.5		0.7		1.25*
Ireland (1957)					
API (1984)		1.0 or 0.8‡	1.0		

Frictional Resistance of Pile In Clays: α-method

Proposed by Tomlinson (1971):

$$f_s = \alpha.c_u$$

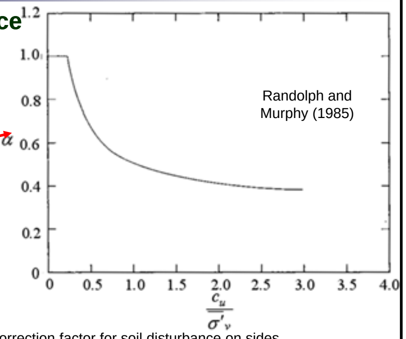
Empirical adhesion factor



Frictional Resistance of Pile In Clays: α-method

Randolph and Murphy (1985):

$$Q_{su} = \sum \alpha . c_u . S . \Delta L$$



Sladen (1992):

$$f_s = \alpha . c_u = \overline{\sigma'}_h . \tan\delta$$

and $\overline{\sigma'}_h = \kappa K_{o,NC} \overline{\sigma'}_v$

correction factor for soil disturbance on sides

With the above relationships, α can be determined as a function of effective overburden and undrained shear strength

$$\alpha = C_1 . \left(\overline{\sigma'}_v / c_u \right)^n$$

C_1 and n are constants depending on soil properties and type of pile installation

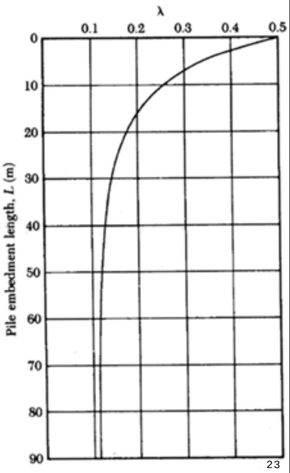
Frictional Resistance of Pile In Clays: λ-method

Proposed by Vijayvergiya and Focht (1972):

$$\left(f_s \right)_{av} = \lambda \left(\overline{\sigma'}_v + 2c_u \right)$$

Mean undrained shear strength

λ varies with the length of embedded pile



Ultimate skin friction resistance of pile

$$Q_{su} = \left(f_s \right)_{av} . S . L$$

Value of $\overline{\sigma'}_v$ and c_u are computed as weighted average over the embedded depth of pile

This method usually overpredicts the capacity of piles with embedded length less than 15 m.

Frictional Resistance of Pile In Clays: β-method

In saturated clays displacement piles induce excess pore pressure near pile surface during installation which eventually dissipates within a month or so. Hence, the frictional resistance of pile may be estimated on the basis of effective stress parameters of clay in a remolded state.

$$f_s = \beta . \sigma'_v = K \tan \phi'_R . \sigma'_v$$

Effective friction angle of remolded clay at certain depth

Earth pressure coefficient may be estimated as the earth pressure at rest:

$$K = \left(1 - \sin \phi'_R \right)$$ For Normally Consolidated Clay

$$K = \left(1 - \sin \phi'_R \right) \sqrt{\text{OCR}}$$ For Over Consolidated Clay

Total frictional resistance of pile:

$$Q_{su} = \sum f_s . S . \Delta L$$

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_γ = bearing capacity factor taken for general shear
P_D = effective overburden pressure (critical depth taken as 15D for $\phi \leq 30°$ 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

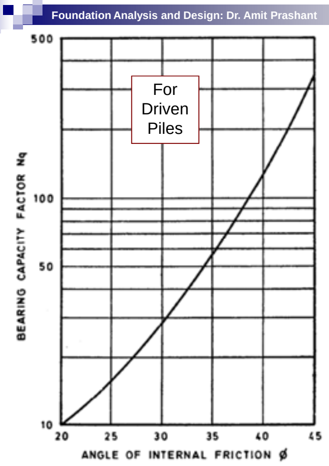
$$Q_{ult} = A_p P_D N_q + \sum_{1}^{n} K_1 P_{D1} \tan\delta$$

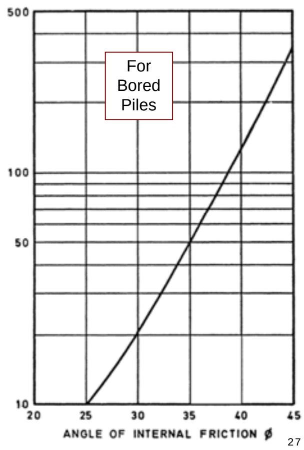
A conservative value of $K = 1$ can be assumed for all piles except for piles with steel liners, where $K = 0.7$ can be assumed. *The ultimate frictional resistance should preferably be restricted to 6 t/m² in sands.*

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.

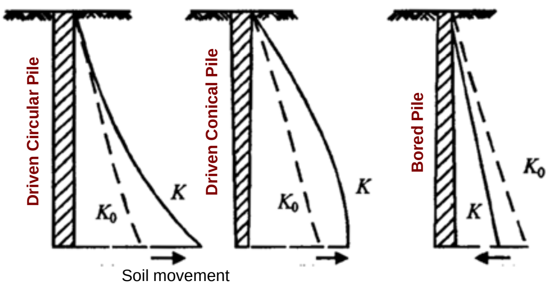
For driven precast piles the value ϕ is changed to $(\phi + 40)/2$ to take care of compaction due to pile driving. (Thus, if for $\phi = 30°$, ϕ is taken as 35° for driven piles).

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

It seems logical that K value shall be close to the coefficient of earth pressure at rest K_o as described in earlier methods. However, type of installation has a major impact on how the earth pressure may vary from K_o, as shown in the figure below.



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

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

For $\overline{\sigma'}_v / c_u < 1 \quad \rightarrow \quad \alpha = 0.5\left(\overline{\sigma'}_v / c_u\right)^{0.25}$, but $\not< 0.5$ 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
4 to 8	Medium stiff	25 to 50	0.25–0.5	0.7–0.4	driven
8 to 15	Stiff	50 to 100	0.5–1.0	0.4–0.3	values by
≥ 15	Stiff to hard	≥ 100	> 1.0	0.3–0.25	factor 0.8

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:

$Q_u = 4(L/D)NA_p + (\bar{N}/5)A_s$ in tons (where $L/D \not> 10$)

For L/D > 10

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

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

For Non-plastic silt and fine sand:

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

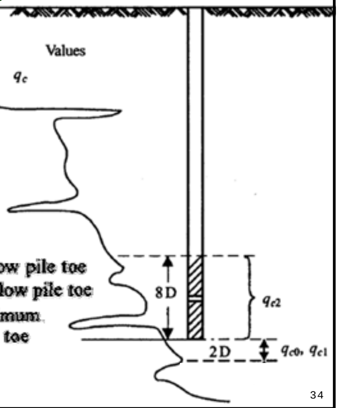
$Q_u = 30NA_p + (\bar{N}/6)A_s$ tons for $L/D > 10$

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:

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

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.

Pile Load Capacity: Other Correlations with SPT value

Reference	Relationship	Applicability
Briaud et al. (1985)	$q_p = 19.7 p_a (N_{60})^{0.36}$	Sand
Shioi and Fukui (1982)	$q_p = 3 p_a$	Cast in place, sand
	$q_p = 0.1 p_a N_{60}$	Bored pile, sand
	$q_p = 0.15 p_a N_{60}$	Bored pile, gravelly sand
	$q_p = 0.3 p_a N_{60}$	Driven piles, all soils

Briaud et al. (1985)

unit frictional resistance $f = 0.224 p_a (N_{60})^{0.29}$

p_a = atmospheric pressure

average unit frictional resistance

$$f_{av} = 0.224 p_a (N_{60})^{0.29}_{av}$$

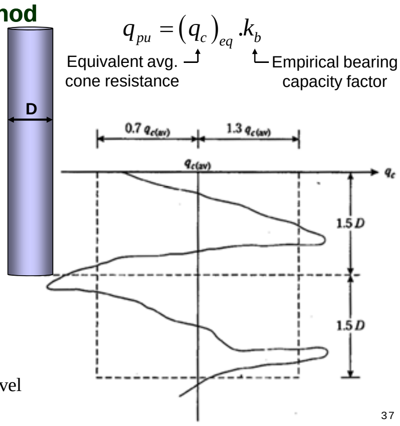
Point Load Capacity of Pile: Correlation with CPT data by LCPC Method

$$q_{pu} = (q_c)_{eq} . k_b$$

Equivalent avg. cone resistance

Empirical bearing capacity factor

- Get the average q_c value for a zone 1.5D above to 1.5D below the pile tip.
- Eliminate the q_c values that are higher than $1.3(q_c)_{avg}$ or lower than $0.7(q_c)_{avg}$.
- Compute the (q_c)eq as an average of the remaining q_c values.



Briaud and Miran (1991):

$k_b = 0.6$ for clay and silt

$k_b = 0.375$ for sand and gravel

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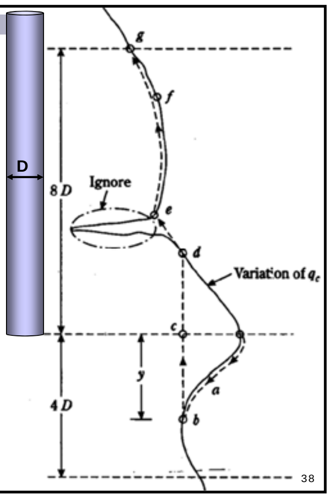
Pile Load Capacity: Correlation with CPT by Dutch Method

- Compute the average q_c value for a zone yD below the pile tip for y varying from 0.7 to 4. Define q_{c1} as the minimum value of above $(q_c)_{avg}$.
- Average the value of q_c for a zone of 8D above the pile tip, and get q_{c2}. Ignore sharp peaks during averaging.
- Calculate

Atmospheric Pressure

$$q_p = \frac{(q_{c1} + q_{c2})}{2} k_b' \le 150.p_a$$



DeRuiter and Beringen (1979)

$k_b' = 1.0$ for OCR = 1

$k_b' = 0.67$ for OCR = 2 to 4

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Pile Load Capacity: Correlation with CPT by Dutch Method

Nottingham and Schmertmann (1975) and Schmertmann (1978)

$$q_p = R_1 R_2 \frac{(q_{c1} + q_{c2})}{2} k_b' \le 150.p_a$$

$R_1 =$ Reduction factor as function of c_u

$R_2 = 1$ for electrical cone penetrometer

$R_2 = 0.6$ for mechanicsl cone penetrometer

Schmertmann (1978) →

$\frac{c_u}{p_a}$	R_1
≥0.5	1
0.75	0.64
1.0	0.53
1.25	0.42
1.5	0.36
1.75	0.33
2.0	0.30

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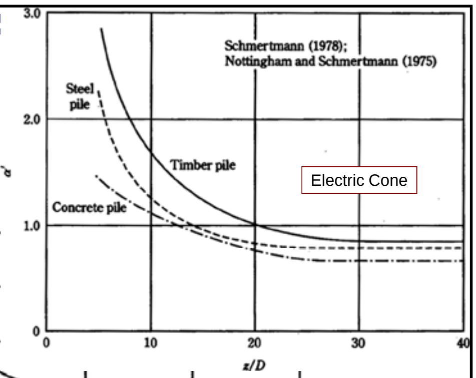
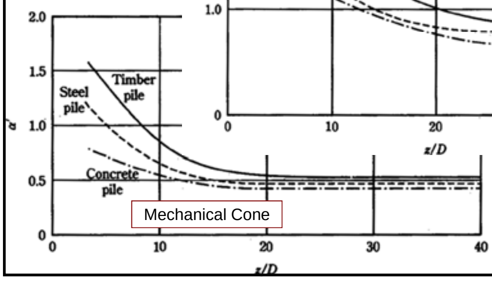
Pile Load Capacity: Correlation with CPT data in Sand by Dutch Method

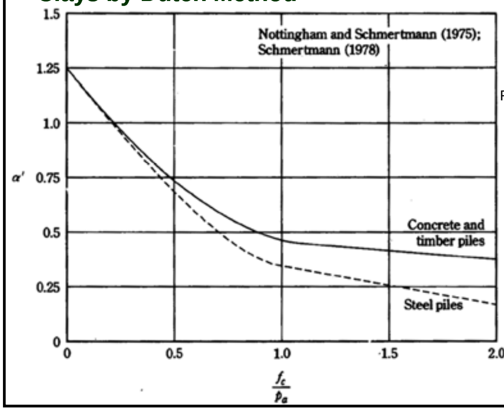




$f_s = \alpha'.f_c$

Frictional cone resistance

Pile Load Capacity: Correlation with CPT data in Clays by Dutch Method



$f_s = \alpha'.f_c$

Frictional cone resistance

Allowable Pile Capacity

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

- 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.