Indian Standards on Piles

- IS 2911 : Part 1 : Sec 1 : 1979 Driven cast-in-situ concrete 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 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

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

- **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 up to 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 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.

\[ q_{zu} = \frac{\Delta Q_z}{S \Delta z} \]

- The frictional resistance per unit area at any depth

\[ Q_{pu} \]

- Ultimate skin friction resistance of pile

\[ Q_{pu} = q_{pu} A_p \]

- Ultimate point load

\[ q_{pu} = \text{bearing capacity of soil} \]

\[ A_p = \text{bearing area of pile} \]

\[ Q_u = Q_{pu} + Q_{s} \]

- Ultimate load capacity in compression

\[ Q_u = Q_{pu} \]

- Ultimate load capacity in tension

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

\[ q_{pu} = cN_c + q'N_q + 0.5yDN_d \]

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

\[ q_{pu} = cN_c + q'N_q \]

- Hence Pile load capacity

\[ Q_{pu} = q_{pu} A_p = (cN_c + q'N_q)A_p \]
**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
    \[
    Q_{pa} = A_p q_p' N'_{cq} = A_p q_p' \frac{3}{2} N'_{cq}
    \]
  - \( q_p' = 0.5 P_{atm} N'_{cq} \tan \phi' \)
  - \( P_{atm} \) = Atmospheric pressure
  - \( (L/D)_{cr} \) value typically ranges from 150 for loose to medium sand to 200 for dense sands.
  - Correlation of limiting point resistance with SPT value
    \[
    q_p' = 0.4 (N^* L/D) \leq 4 P_{atm} (N^*)
    \]
  - \( N^* \) value shall be taken as an average for a zone ranging from 10D above to 4D below the pile point.

- **Saturated Clays:**
  \[
  Q_{pa} = N'_c c_q A_p = 9 c_q A_p
  \]

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

- Pile point bearing capacity based on the theory of expansion of cavities
  \[
  Q_{pa} = A_p q_p' = A_p \left( c N'_c + \sigma'_N ight)
  \]
- Mean effective normal stress at pile end
  \[
  \sigma'_N = \frac{1}{3} (1 + 2 K_r) q'
  \]
- \( N'_c = f (L_c) \)
- \( L_c = \frac{1}{1 + I_{II}} \)
- \( I_{II} \) = Reduced rigidity index of soil
- \( I_c \) = rigidity index
- \( E_c = \frac{G}{(c' + q' + \tan \phi') + \frac{1}{2} (1 + K_r) \tan \phi'}\]
- \( N'_c = \frac{4}{3} (\ln L_c + 1) + \frac{2}{1 + 1}
  \]

**Type of soil**

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>65-150</td>
</tr>
<tr>
<td>Silt</td>
<td>50-75</td>
</tr>
<tr>
<td>Clay</td>
<td>150-250</td>
</tr>
</tbody>
</table>

**Baldi et al. (1981):**

- For mechanical cone resistance
  - \( I_c = \frac{1}{1 + 1/4} \)
- For electric cone resistance
  - \( I_c = \frac{1}{1 + 1/4} \)

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

\[
Q_{pa} = A_p \left( c N'_c + q_p' N'_c \right)
\]

\[
N'_c = \left( \tan \phi' + \sqrt{1 + \tan^2 \phi'} \right) \left( e^{\frac{1}{2} \pi \cot \phi'} \right)
\]

- Sand
- Clay
  - \( 60^\circ \leq \eta' \leq 90^\circ \)
  - \( \eta' = N'_c (1 - N'_c) \cot \phi' \)
Point Load capacity of Pile: Coyle and Costello’s (1981)
Method for Granular Soils

\[ Q_{pc} = A_p q' N_q' \]

\( N_q' \) 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):

\[ Q_{pc} = A_p q' \left( N_q + 1 \right) \]

\( N_q = \tan^2\left( 45 + \phi'/2 \right) \)
\( q' = \) unconfined compression strength of rock
\( \phi' = \) effective friction angle of rock

Frictional Resistance of Pile: In Sand

The frictional resistance of pile may be computed as

\[ Q_{fr} = \sum S \Delta L f_s \]

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.

\[ f_s = K \sigma_{y}' \tan \delta \leq f_{ul} \]

Soil-Pile interface friction angle \( \delta \) varies from 0.5\( \phi' \) to 0.8\( \phi' \). Earth pressure coefficient \( K \) depends on both soil type and pile installation.

<table>
<thead>
<tr>
<th>Pile type</th>
<th>( K )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bored or jetted</td>
<td>( K_0 = 1 - \sin \phi' )</td>
</tr>
<tr>
<td>Low-displacement driven</td>
<td>( K_0 = 1 - \sin \phi' ) to 1.4( K_0 = 1.4 \left( 1 - \sin \phi' \right) )</td>
</tr>
<tr>
<td>High-displacement driven</td>
<td>( K_0 = 1 - \sin \phi' ) to 1.8( K_0 = 1.8 \left( 1 - \sin \phi' \right) )</td>
</tr>
</tbody>
</table>
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:

$$K = 0.50 + 0.008D_j$$
$$K \tan \delta = 0.18 + 0.0065D_j$$

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

$$Q_s = (f_s)_{u.s.} S_L$$
$$= \left( K \sigma'_{v}, \tan \delta \right) S_L$$

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 $\phi'$ with effective confining pressure which may be used to compute $K$ and $\delta$.

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

Effective vertical stress at the depth of interest

Effective confining stress during triaxial test

Friction angle obtained through triaxial testing at some confining pressure $\sigma''$.

Typical values of $K$ from a number of pile tests:

<table>
<thead>
<tr>
<th>Source</th>
<th>H piles</th>
<th>Pipe</th>
<th>Present concrete</th>
<th>Timber</th>
<th>Tapered Timber</th>
</tr>
</thead>
<tbody>
<tr>
<td>Master and Hunter (1976)</td>
<td>1.4-1.9</td>
<td>1.2-1.3</td>
<td>1.45-1.6</td>
<td>1.25</td>
<td></td>
</tr>
<tr>
<td>Terres (1971)</td>
<td>0.5</td>
<td>0.7</td>
<td>1.25</td>
<td>1.25*</td>
<td></td>
</tr>
<tr>
<td>Ireland (1977)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>API (1984)</td>
<td>1.0</td>
<td>0.81</td>
<td>1.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Frictional Resistance of Pile In Clays: $\alpha$-method

Proposed by Tomlinson (1971):

$$f_s = \alpha c_u$$

Empirical adhesion factor
Frictional Resistance of Pile In Clays: \( \alpha \)-method

Randolph and Murphy (1985):

\[
Q_{\alpha} = \sum \alpha \sigma' \Delta L
\]

Sladen (1992):

\[f_s = \alpha \sigma' = \bar{\tau} \tan \delta\]

and \(\bar{\tau} = k K_{\text{side}} \sigma'\)

Correction factor for soil disturbance on sides

With the above relationships, \( \alpha \) can be determined as a function of effective overburden and undrained shear strength

\[\alpha = C_1 \left( \frac{\sigma'_u}{c_u} \right)\]

\(C_1\) and \(n\) are constants depending on soil properties and type of pile installation.

Frictional Resistance of Pile In Clays: \( \lambda \)-method

Proposed by Vijayvergiya and Focht (1972):

\[
(f_s)_{\lambda} = \lambda \left( \bar{\tau} + 2c_u \right)
\]

- Mean unstrained shear strength
- \(\lambda\) varies with the length of embedded pile

Ultimate skin friction resistance of pile

\[Q_{\alpha} = (f_s)_{\lambda} \cdot S \cdot L\]

Value of \(\bar{\tau}\) 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: \( \beta \)-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.

\[f_s = \beta \sigma'_u = K \tan \phi'_u \sigma'_u\]

Effective friction angle of remolded clay at certain depth

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

\[K = (1 - \sin \phi'_u)\]

For Normally Consolidated Clay

\[K = (1 - \sin \phi'_u) \sqrt{OCR}\]

For Over Consolidated Clay

Total frictional resistance of pile:

\[Q_{\beta} = \sum f_s \cdot S \Delta L\]
Pile Load Capacity in Cohesionless Soils

\[ Q_s = A_p (0.5\gamma f N_q) + A_p (P_D N_{qD}) + \sum_{i=1}^{n} K_i P_{D1} \tan \delta A_{sl} \]

- \( A_p \) = cross-sectional area of the pile
- \( D \) = stem diameter of pile
- \( \gamma \) = unit weight of soil
- \( N_q, N_f \) = bearing capacity factor taken for general shear
- \( P_D \) = effective overburden pressure (critical depth taken as 15\( D \) for \( \phi \leq 30^\circ \) and 20\( D \) for \( \phi > 40^\circ \) —Indian Railways recommend only 6\( D \) for \( \phi = 26^\circ \))
- \( K_f \) = coefficient of earth pressure
- \( P_{D1} \) = effective overburden pressure of corresponding layer
- \( \delta \) = angle of wall friction usually taken as \( 3/4\phi \) of soil.
- \( A_s \) = surface area of pile.

For Bored Piles

For Driven Piles

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

\[ Q_{dr} = A_p P_D N_q + \sum_{i=1}^{n} K_i P_{D1} \tan \delta \]

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 \( \text{kN/m}^2 \) for sand.

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 \( \text{kg/cm}^2 \) (1,500 \( \text{psi} \)) for percent driven piles and 100 to 110 \( \text{kg/cm}^2 \) (1000 to 1100 \( \text{psi} \)) 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.
It seems logical that K value shall be close to the coefficient of earth pressure at rest $K_0$, as described in earlier methods. However, type of installation has a major impact on how the earth pressure may vary from $K_0$, as shown in the figure below.

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 $K_0$ are listed below.

<table>
<thead>
<tr>
<th>Method of installation</th>
<th>$K/K_0$*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driven large displacement piles (Concrete piles)</td>
<td>1 to 2</td>
</tr>
<tr>
<td>Driven small displacement piles (Steel $H$ piles)</td>
<td>0.75 to 1.75</td>
</tr>
<tr>
<td>Bored cast in-situ piles</td>
<td>0.7 to 1</td>
</tr>
<tr>
<td>Jetted piles</td>
<td>0.5 to 0.7</td>
</tr>
</tbody>
</table>

* $K_0 = (1 - \sin \phi) = \text{coefficient of earth pressure at rest.}
\[ Q_u = A_p N_c c_p + \sum_{i=1}^{n} a_i c_i A_i \]

- \( N_c \): bearing capacity factor in clays which is taken as 9
- \( c_p \): average cohesion at pile toe
- \( a_i \): adhesion factor
- \( c_i \): average cohesion of the ith layer on the side of the pile
- \( A_i \): surface area of pile stem in the ith layer.
- \( a \): adhesion between shaft of pile and clay.

Tolinsen's recommendations:

1. For \( \sigma' / c_i \geq 1 \), \( \alpha = 0.5 \left( \frac{\sigma'}{c_i} \right)^{0.5} \), but \( \geq 1 \)
2. For \( \sigma' / c_i < 1 \), \( \alpha = 0.5 \left( \frac{\sigma'}{c_i} \right)^{0.25} \), but \( < 0.5 \) and \( \geq 1 \)

For bored piles, the value of \( \alpha \) as obtained above is to be multiplied by 0.8.

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**Meyerhof's Formula for Driven Piles based on SPT value**

**For Sand:**
\[ Q_u = 4(L/D)N_c A_p + (N/5)A_i \text{ tons (where } L/D > 10) \]
\[ Q_u = 40A_p + (N/5)A_i \text{ tons (where } A_p \text{ and } A_i \text{ are in } m^2) \]

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

**For Non-plastic silt and fine sand:**
\[ Q_u = 3(L/D)N_c A_p + (N/6)A_i \text{ tons (when } A_p \text{ and } A_i \text{ are in } m^2) \]
\[ Q_u = 30A_p + (N/6)A_i \text{ tons for } L/D > 10 \]

**For Clays:**
\[ Q_u = 9cA_p + \alpha cA_i = 9 \left( \frac{N}{20} \right) (10)A_p + \alpha \left( \frac{N}{20} \right) (10)A_i \text{ tons} \]
\[ = 4.5N A_p + \left( \frac{N}{2} \right) A_i \text{ metric tons (assuming } \alpha = 1) \]

---

**Table:**

<table>
<thead>
<tr>
<th>SPT values</th>
<th>Consistency</th>
<th>Range of cohesion (kN/m²)</th>
<th>Adhesion factor α</th>
<th>Driven</th>
<th>Bored</th>
</tr>
</thead>
<tbody>
<tr>
<td>N &lt; 4</td>
<td>Soft to very soft</td>
<td>1 to 25</td>
<td>0.01-0.25</td>
<td>&gt; 1.0</td>
<td>Reduce the value of α</td>
</tr>
<tr>
<td>4 to 8</td>
<td>Medium stiff</td>
<td>25 to 50</td>
<td>0.25-0.5</td>
<td>0.7-0.8</td>
<td>driven values by factor 2</td>
</tr>
<tr>
<td>8 to 15</td>
<td>stiff</td>
<td>50 to 100</td>
<td>0.5-1.0</td>
<td>0.4-0.3</td>
<td></td>
</tr>
<tr>
<td>&gt; 12</td>
<td>stiff to hard</td>
<td>&gt; 100</td>
<td>&gt; 1.0</td>
<td>0.3-0.2</td>
<td></td>
</tr>
</tbody>
</table>

The value of \( c \) for clays is \( N/16 \) to \( N/20 \) kg/cm² (approximately) as derived from \( N \) values.

The value of \( \alpha \) shall be limited to 0.5 for sensitive clays.

The value of \( \alpha \) may be more than 0.7 in clays overlain by sand.
The ultimate point bearing capacity:

\[
q_u = \frac{q_{u0} + q_{u1} + q_{u2}}{2}
\]

- \(q_{u0}\) = average SCPT value for 2D below pile toe
- \(q_{u1}\) = minimum SCPT value for 2D below pile toe
- \(q_{u2}\) = average of the envelope of minimum SCPT value over 2D above the toe of the pile.

Correlation of SPT and CPT:

<table>
<thead>
<tr>
<th>Soil type</th>
<th>(q_s (\text{kN/m}^2))</th>
<th>(q_s (\text{kN/m}^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>150-200</td>
<td>1.5-2.0</td>
</tr>
<tr>
<td>Silts, sandy silts and slightly cohesive silt-sand</td>
<td>200-250</td>
<td>2.0-2.5</td>
</tr>
<tr>
<td>Clean fine to medium sand and slightly silty sands</td>
<td>300-400</td>
<td>3.0-4.0</td>
</tr>
<tr>
<td>Coarse sand and sand with little gravel</td>
<td>500-600</td>
<td>5.0-6.0</td>
</tr>
<tr>
<td>Sandy gravel and gravel</td>
<td>800-100</td>
<td>8.0-10.0</td>
</tr>
</tbody>
</table>

Note: \(q_s\) in kN/m² = 1.5 to 10 times \(N\) value depending on soil type.

Pile Load Capacity: Other Correlations with SPT value

<table>
<thead>
<tr>
<th>Reference</th>
<th>Relationship</th>
<th>Applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Briauz et al. (1985)</td>
<td>(q_f = 19.7p_s(N_{80})^{0.36})</td>
<td>Sand</td>
</tr>
<tr>
<td></td>
<td>(q_f = 3p_s)</td>
<td>Cast in place, sand</td>
</tr>
<tr>
<td></td>
<td>(q_p = 0.1p_sN_{80})</td>
<td>Bored pile, sand</td>
</tr>
<tr>
<td></td>
<td>(q_p = 0.15p_sN_{80})</td>
<td>Bored pile, gravelly sand</td>
</tr>
<tr>
<td></td>
<td>(q_p = 0.3p_sN_{80})</td>
<td>Driven piles, all soils</td>
</tr>
</tbody>
</table>

Briauz et al. (1985)

unit frictional resistance \(f = 0.224p_s(N_{80})^{0.28}\)

\(p_s\) = atmospheric pressure

average unit frictional resistance

\(f_{av} = 0.224p_s(N_{80})^{0.28}\)
**Point Load Capacity of Pile: Correlation with CPT data by LCPC Method**

- 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(\( q_{c\text{avg}} \)) or lower than 0.7(\( q_{c\text{avg}} \)).
- Compute the \( (q_c)_{eq} \) as an average of the remaining \( q_c \) values.

Briaud and Miran (1991):
- \( k_s = 0.6 \) for clay and silt
- \( k_s = 0.375 \) for sand and gravel

\[ q_{pu} = (q_c)_{eq} k_p \]

\( q_{pu} \): Equivalent avg. cone resistance
\( k_p \): Empirical bearing capacity factor

**Pile Load Capacity: Correlation with CPT by Dutch Method**

- Compute the average \( q_c \) value for a zone of D below the pile tip for \( y \) varying from 0.7 to 4. Define \( q_{c1} \) as the minimum value of above \( (q_{c\text{avg}}) \).
- 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

\[ q_p = \frac{(q_{c1} + q_{c2})}{2} k_s' \leq 150. p_a \]

DeRuitter and Beringen (1979)
- \( k_s' = 1.0 \) for OCR = 1
- \( k_s' = 0.67 \) for OCR = 2 to 4

**Pile Load Capacity: Correlation with CPT by Dutch Method**

Nottingham and Schmertmann (1975) and Schmertmann (1978)

\[ q_p = R_1 R_2 \left( q_{c1} + q_{c2} \right) k_s' \leq 150. p_a \]

- \( R_1 \): Reduction factor as function of \( c_s \)
- \( R_2 = 1 \) for electrical cone penetrometer
- \( R_2 = 0.6 \) for mechanical cone penetrometer

Schmertmann (1978)

<table>
<thead>
<tr>
<th>( c_s )</th>
<th>( R_1 )</th>
<th>( R_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;0.5</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>0.75</td>
<td>0.64</td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>0.53</td>
<td></td>
</tr>
<tr>
<td>1.25</td>
<td>0.42</td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>0.36</td>
<td></td>
</tr>
<tr>
<td>1.75</td>
<td>0.33</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>0.30</td>
<td></td>
</tr>
</tbody>
</table>
Pile Load Capacity: Correlation with CPT data in Sand by Dutch Method

- Frictional cone resistance

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

- Frictional cone resistance

**Allowable Pile Capacity**

- 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:**

<table>
<thead>
<tr>
<th>Case</th>
<th>Factor of safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. On total capacity</td>
<td>2.5</td>
</tr>
<tr>
<td>2. On shaft resistance</td>
<td>1.5</td>
</tr>
<tr>
<td>3. On base resistance</td>
<td>3.0</td>
</tr>
</tbody>
</table>

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