Deep Foundation

Deign Methods

Pile Selection Guide

Table L-11

GUIDELINES FOR DRIVEN PILES

Type of Pile	Normal Size Range	Typical Pile Load, kN	Structural Considerations	Installation Considerations	Notes
(a) Timber	180 to 250 mm tip	180 to 450	Must be checked in accordance with NBC Section 4.3	Cannot be inspected. Susceptible to damage during hard driving. Tip reinforcement recommended where driven to end bearing	Preservative treatment normally required. (CSA O80-1970)
(b) Steel sections (H, WF)	200 to 350 mm	350 to 1 800	Must be checked in accordance with NBC Sections 4.5 and 4.6 End bearing: allowable working stresses usually $\ngtr 0.3\, f_y$ when driven to end bearing refusal on rock or dense strata, but higher stresses possible under specific controlled conditions Friction: usually working stresses are governed by geotechnical considerations and rarely exceed about 80 MPa In pipe piles, concrete strength does not normally contribute to pile capacity unless the pile is driven to end bearing	May be damaged during driving but load capacity not necessarily reduced	Tip points often required for hard driving. Average thickness of flange or web, $t \geq 1$ cm. Projection of flange $\ngtr 14\, t$
(c) Pipe sections	200 to 600 mm diam.	350 to 1 800		Suitable for inspection after driving. Concrete quality highly dependent on placement method	Normally driven closed-end. Tip reinforcement or drive shoe required when driven open-end. Pipe thickness $>$ 5 mm, but 10 mm recommended
(d) Precast concrete sections	200 to 300 mm 300 to 900 mm	350 to 1 000 900 to 2 500	End bearing: capacity must be checked in accordance with NBC Section 4.5. Normally $f'_c > 27.5$ MPa Friction: the capacity of friction piles is normally governed by both installation method and geotechnical considerations; the average compressive stress under load rarely exceeds 10 MPa	Cannot be inspected. Careful selection and driving method required to prevent damage	Refer to ACI 70-50. Possible tensile stresses in concrete during "soft" driving. High compressive stresses in concrete during "hard" driving. Tip reinforcement usually essential
Column 1	2	3	4	5	6

Ultimate Pile Load Capacity

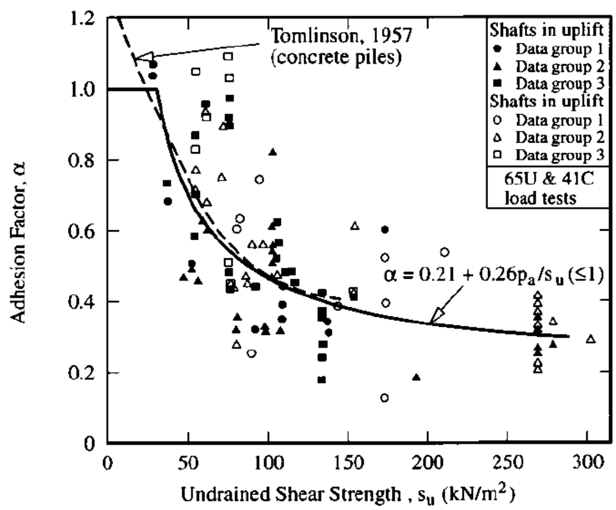
- **Shaft capacity computed via:**
  - **Total stress (α) method**
  - **Effective stress (β) method**
  - **Hybrid (λ) method**
  - **Using SPT data**
  - **Using CPT data**
  - **Using PMT data**

Shaft Resistance

- $f_s = \alpha.s_u$ **(alpha method)**
- $f_s = \beta.s_v'$ **(beta method)**
- $f_s = \lambda.(s_{vm}' + 2\ s_{um})$ **(lambda method)**
- $f_s = a + bN$ **(SPT data)**
- $f_s = (q_c/A)^n$ **(CPT method)**
- $fs = fn(p_{lim})$ **(PMT method)**

For design, an upper limit usually placed on f_s

Shaft Capacity in Clay (Alpha Method)



Shaft Capacity in Clay (Alpha Method)

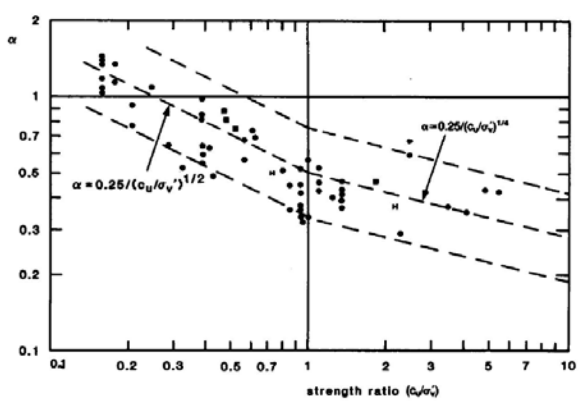
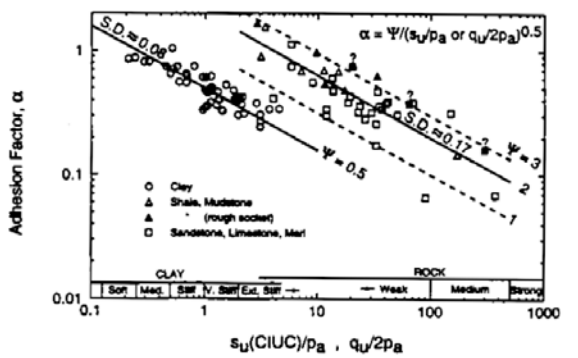


Figure 4.9 Variation of α with strength ratio

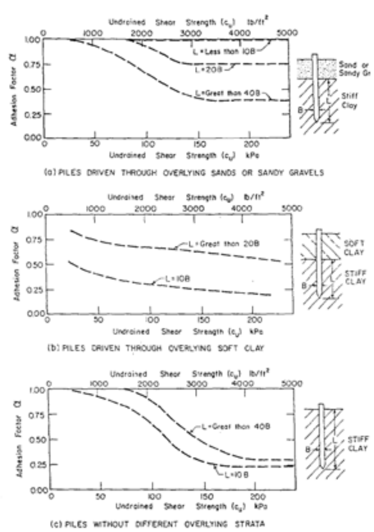
Correlation between α and normalized shear strength ratio (Fleming et al, 1985)

Shaft Capacity in Clay (Alpha Method)



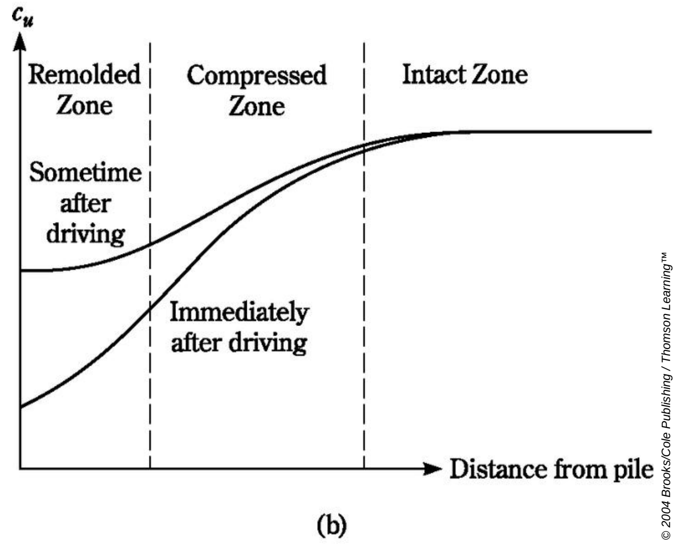
Kulhawy & Phoon, 1993

Soft-stiff clay Adhesion factors

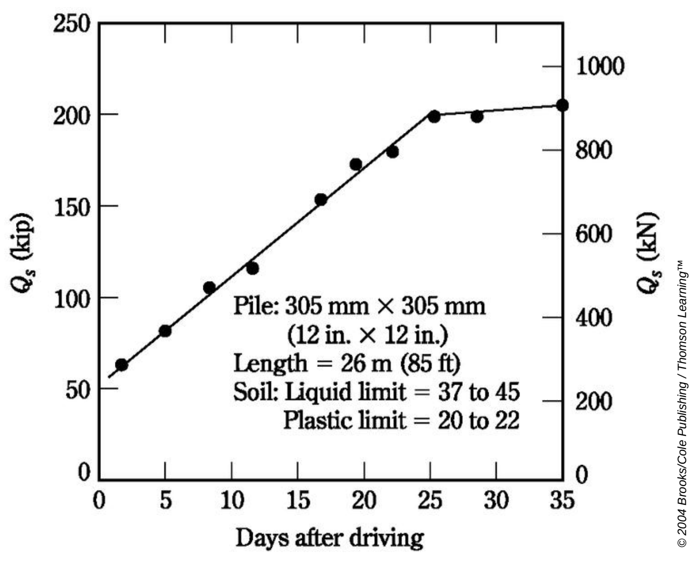


NOTE : (1) Curves not applicable to H or cruciform section or to bored or driven and cast-in place piles.
(2) Safety factor should not be less than 2.5 except for design based on adequate loading test data.

FIG 2.1 (After Tomlinson, 1970)
ADHESION FACTOR vs. SHEAR STRENGTH FOR DIFFERENT PENETRATIONS INTO STIFF CLAY

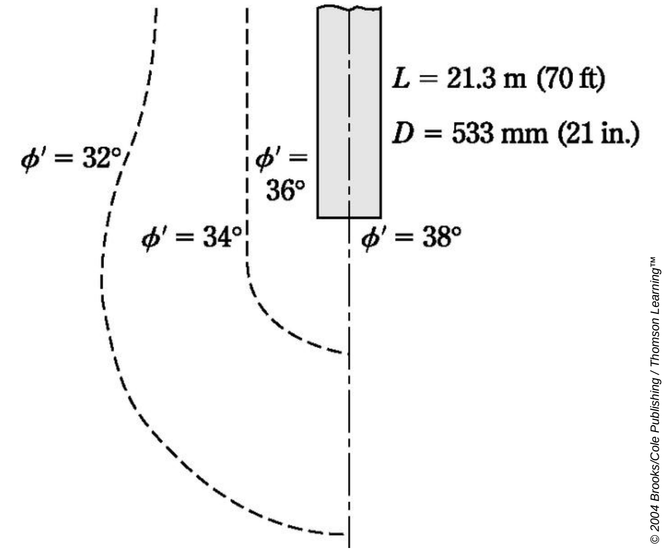


Nature of variation of undrained shear strength (c_u) with time around a pile driven into soft clay



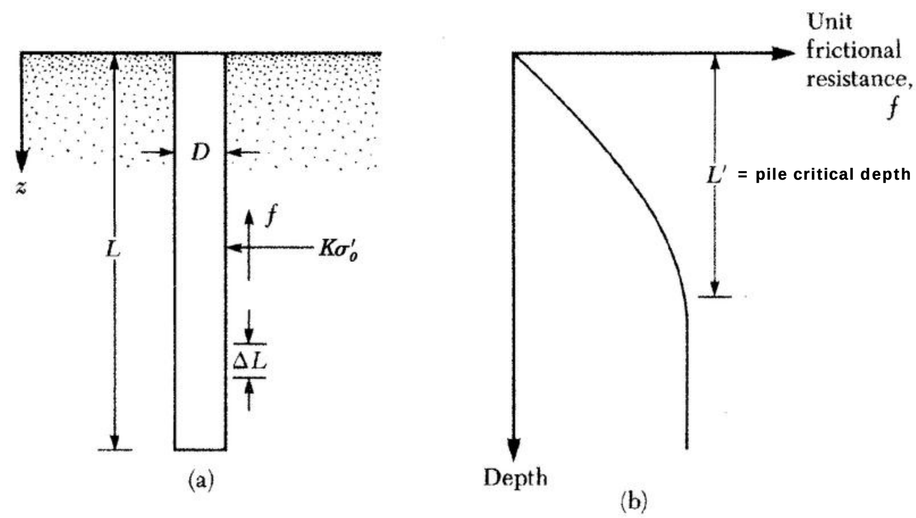
Variation of Q_s with time for a pile driven into soft clay

(based on load test results of Terzaghi and Peck, 1967)



Compaction of sand near driven piles

(after Meyerhof, 1961)



Unit frictional resistance for piles in sand

- For $z = 0$ to L'

$$f_s = K\sigma_o'\tan\delta = \beta\tan\delta$$

Where $\beta = K\sigma_o'$

- For $z = L'$ to L

$$fs = f_{z=L}$$

$$Q_s = f_s \Sigma p \, \Delta L$$

Where

p = perimeter of pile

ΔL = incremental pile length which p and fs are taken constant

Shaft Capacity in Sand (Beta Method)

$$\beta = K_s \tan\delta$$

- $K_s = fn(K_o$, installation method) sands

  or $K_s = (1\text{-}\sin\phi')\tan\phi'\,(OCR)^{0.5}$ clays

- $\delta = fn(\phi'$, interface materials) sands

δ is the shaft soil friction angle

Shaft Capacity in Sand (Beta Method)

Interface Materials	*Typical Field Analogy*	δ / φ’
Sand/rough concrete	Cast-in-place	1.0
Sand/smooth concrete	Precast	0.8 to 1.0
Sand/rough steel	Corrugated	0.7 to 0.9
Sand smooth steel	Coated	0.5 to 0.7
Sand/timber	Pressure-treated	0.8 to 0.9

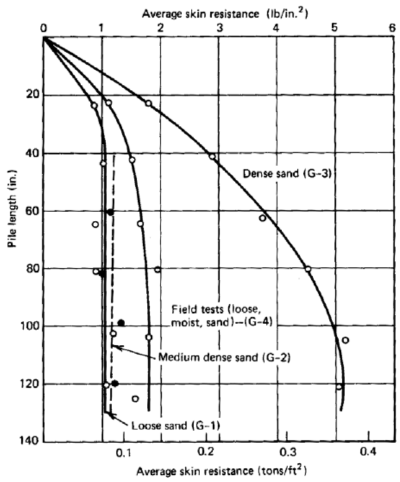
(Stas & Kulhawy, 1984)

Shaft Capacity in Sand (Beta Method)

Foundation type & installation method	K_s / K_o
Jetted pile	0.5 – 0.67
Drilled shaft, cast-in-place	0.67 – 1.0
Driven pile, small displacement	0.75 – 1.25
Driven pile, large displacement	1 - 2

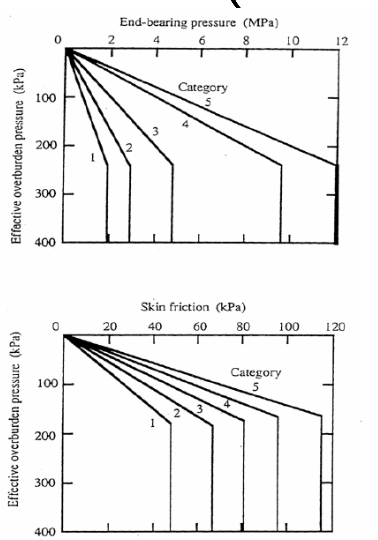
Stas & Kulhawy, 1984)

Vesic Tests



- These tests indicated the existence of a "critical depth", beyond which the shaft friction becomes constant.
- Much controversy about this issue.
- Results may be related to:
  - Dependence of ϕ' on stress level
  - Effects of over-consolidation near surface
  - Volume changes near pile
  - Residual stresses in test piles.

Shaft Capacity in Sand (Practical Design)



- Use beta method.
- Impose upper limit on skin & base resistances.
- Example of API design:
  - 1 = v. loose sand
  - 2 = loose sand
  - 3 = med. Dense sand
  - 4 = dense sand
  - 5 = v. dense sand

Shaft Resistance

Developments in effective stress analysis

- **Jardine, Chow et al (1996-1998) - Ks related to CPT values; allowances for open-ended piles**
- **Yasufuku et al (1997) - Ks related to depth and lateral pressures**
- **Miller & Lutenegger (1997) - Ks related to at rest and maximum stress ratios**

End Bearing

In clays:

$f_b = N_c . s_b$

$N_c \sim 6 + L/d <= 9$

s_b = average undrained shear strength within depth of influence of base

In sands:

$f_b = N_q . \sigma_{vb}'$

N_q = function of ϕ', σ_{vb}' = vertical effective overburden stress at level of pile base.

Usually impose upper limit, depending on relative density,

End Bearing Failure Assumptions

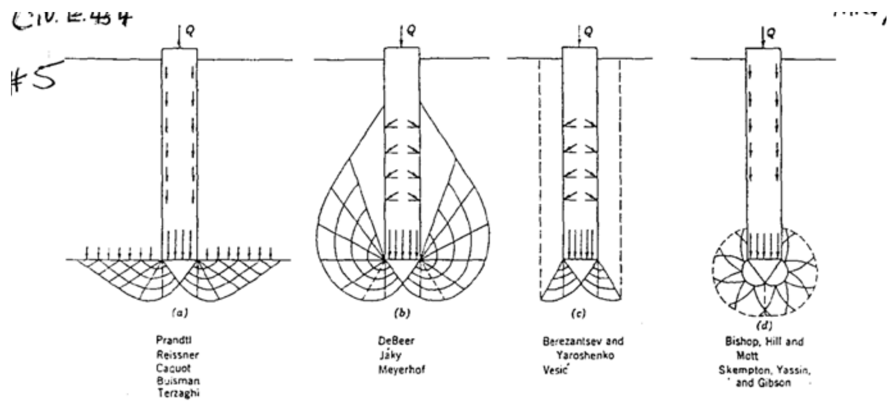


Fig. 33.5 Assumed failure patterns under deep foundations. (From Vesic, 1967).

End Bearing Failure Assumptions

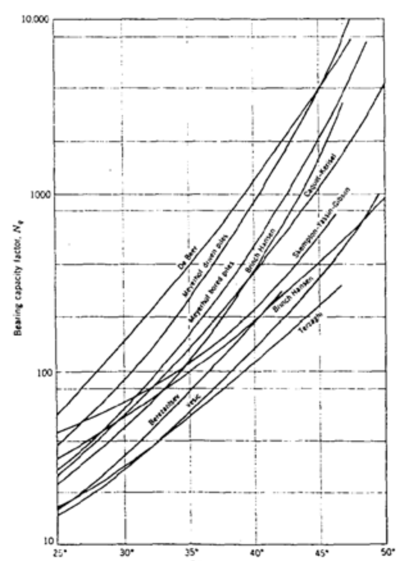


Fig. 33.4 Bearing capacity factors for deep circular foundations. (From Vesic, 1967).

End Bearing Factor (Nq)

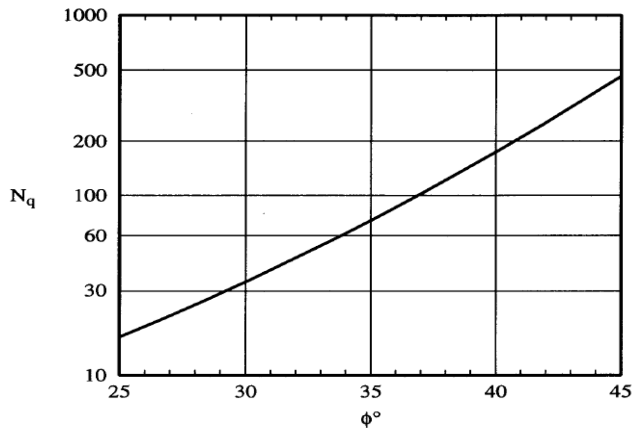


Fig.2 Variation of bearing capacity factor with friction angle
(after Berezantzev et al 1961)

End Bearing based on SPT

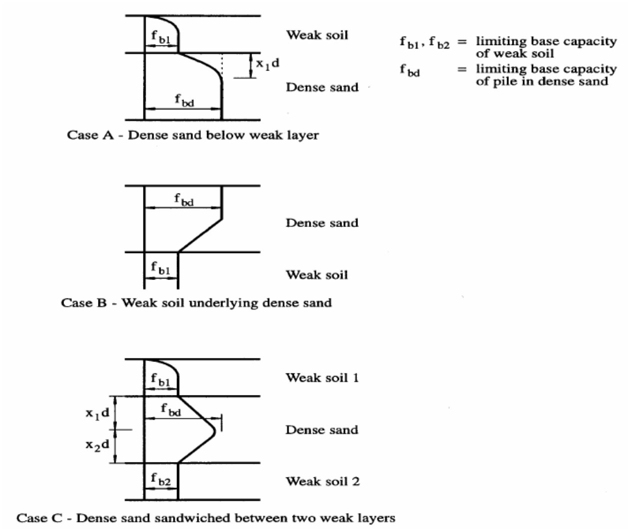
$$f_b = K.\ N_p \leq f_{blim}$$

where N_p = av. SPT in vicinity of base

f_{blim} = lim. Value of base resistance

Soil Type	*K (displ. Piles)*	*K (non-disp. piles*
Sand	0.325	0.165
Sandy silt	0.205	0.115
Clayey silt	0.165	0.100
Clay	0.100	0.080

End Bearing Layered Soils



End Bearing Issues

- Limiting base capacity with depth for sands?
  No, but limit value in design
- Layered soil profiles?
  Meyerhof conservative - effects may be limited to 3d below tip, BUT EFFECT CAN BE IMPORTANT
- Effects of Cyclic Loading?
  Small - can ignore

Cone Penetration Test (cpt)

Two approaches:

- Use of measured sleeve resistance for f_s
  (Nottingham & Schmertmann, 1995)
- Use of measured cone resistance for f_s (& f_b)
  (Bustamante & Gianeselli, 1982)

Shaft Resistance in Clays

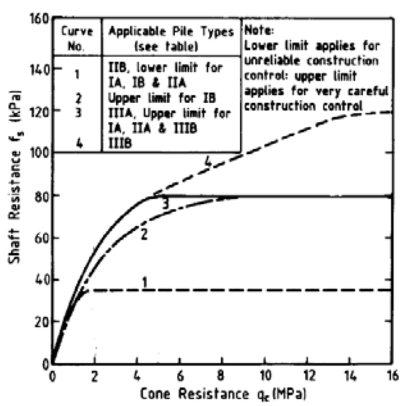


Fig. 25. Design values of shaft resistance for piles in clay (based on Bustamante & Gianeselli, 1982)

Table 8. Classification of pile types (Bustamante & Gianeselli, 1982)

Pile category	Type of pile
IA	Plain bored piles, mud bored piles, hollow auger bored piles, cast screwed piles Type I micropiles, piers, barrettes
IB	Cased bored piles Driven cast piles
IIA	Driven precast piles Prestressed tubular piles Jacked concrete piles
IIB	Driven steel piles Jacked steel piles
IIIA	Driven grouted piles Driven rammed piles
IIIB	High pressure grouted piles ($d > 0{\cdot}25$ m) Type II micropiles

Shaft Resistance in SAND

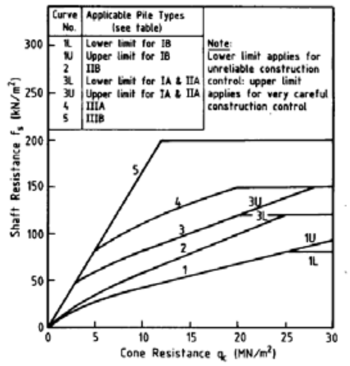
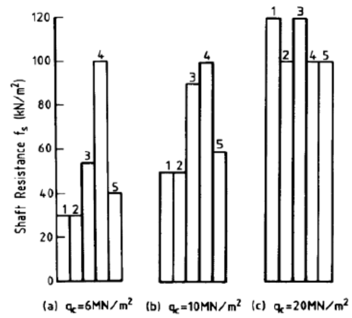


Fig. 26. Design values of shaft resistance for piles in sand (based on Bustamante & Gianeselli, 1982)



Number	Source of Correlation
1	Bustamante & Gianeselli (1982)
2	Fleming & Thorburn (1984)
3	Verbrugge (1982)
4	Van Impe (1986)
5	This paper

Beware of variability with different methods

End Bearing

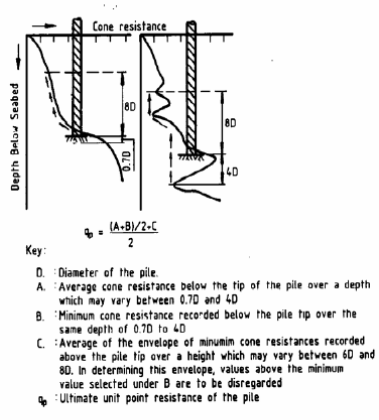


Figure 4.22 The use of CPT for pile-tip bearing capacity (De Ruiter & Beringen 1979).

- The Dutch approach uses the average of two average values:
  - q_c over a distance of y.d below the tip
  - q_c over a distance 8d above the tip
- Some other methods use a reduced average value of qc below the tip (typically 0.3 – 0.5 times the average)

Piles to Rock

- Ultimate shaft friction & end bearing usually related to rock strength q_u (unconfined compressive strength)

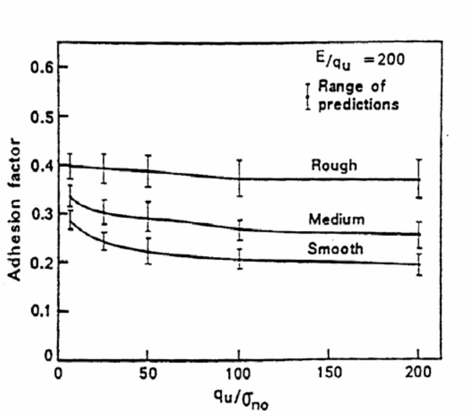
$$\mathbf{f_s = a.\,(q_u)^{\,b}} \quad \textbf{MPa}$$

$$f_b = a_1.\,(q_u)^{\,b_1} \quad \text{MPa}$$

Piles to Rock

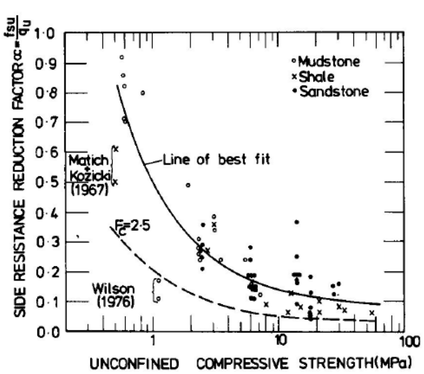
Method	*a*	*b*
Rosenberg & Journeaux (1976)	0.375	0.515
Horvath (1976)	0.33	0.5
Horvath & Kenney (1979)	0.20-0.25	0.5
Meigh & Wolski (1979)	0.22	0.6
Williams & Pells (1981)	$\alpha.\beta$	1.0
Rowe & Armitage (1987)	0.45	0.5
Zhang & Einstein (1998)	0.4 (smooth) 0.8 (rough)	0.5

Importance of Shaft Friction



- Kodikara et al (1992) showed that adhesion factor α depends on:
  - Surface roughness
  - Rock strength
  - Modulus ratio E/q_u

Piles to Rock
a, b reduction factors
(Williams & Pells 1981)



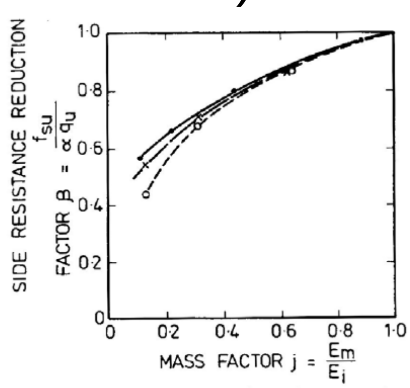


FIG. 7. Mass modulus factor β for Melbourne mudstone. NOTES: ●——● MW mudstone, normal roughness; ×– – –× HW mudstone roughened; ○– – –○ HW mudstone, normal roughness.

E_m = rock mass modulus
E_i = intact rock modulus

Piles to Rock
End Bearing Parameters

Method	a_1	b_1
Teng (1962)	5 – 8	1.0
Coates (1967)	3	1.0
ARGEMA (1992)	4.5 ($f_b \leq$ 10 MPa)	1.0
CGS (1985)	3Ksp.D	1.0
Zhang & Einstein (1998)	4.8 (mean) Range 3.0 – 6.6	0.5

Uplift Capacity

- In clays, shaft friction is similar to compression value
- For enlarged base piles, take lesser of values for two possible failure mechanisms:
  - Shaft + net base resistance + pile weight
  - Gross base resistance + pile weight
- Long-term capacity is often critical!

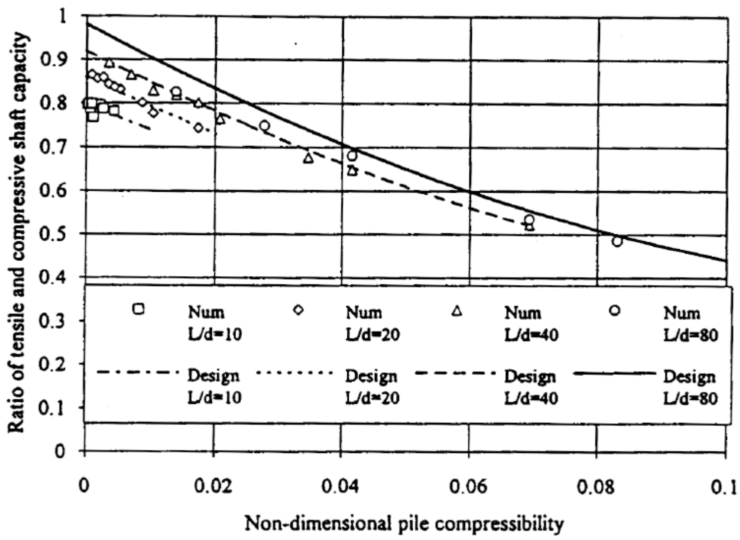
Uplift Capacity SAND

In sands, shaft resistance for uplift may be less than for compression, due to Poisson effect. Depends on relative pile compressibility factor x (De Nicola & Randolph, 1993) as follows:

$$Q_t/Q_c = \{1 - 0.2 \log_{10} [100 (L/d)]\} (1\text{-}8x+25x^2)$$

- Q_t = uplift shaft capacity
- Q_c = compressive shaft capacity
- L=pile length
- d= pile diameter
- $x = \nu_p \tan \delta (L/d) (G_{av}/E_p)$
- ν_p = pile Poisson's ratio
- G_{av} = average soil shear modulus along pile shaft
- E_p = pile Young's modulus
- δ = pile-soil interface friction angle

Uplift Capacity SAND
Single Pile



Cyclic Loading

- Main effect is DEGRADATION OF ULTIMATE SHAFT FRICTION
- Define degradation factor as:

$$D_\tau = \frac{f_s \text{ after cyclic ldg.}}{f_s \text{ for static ldg.}}$$

D_τ depends on:

- No. of cycles
- Amplitude of cyclic displacement
- Soil type
- Pile type

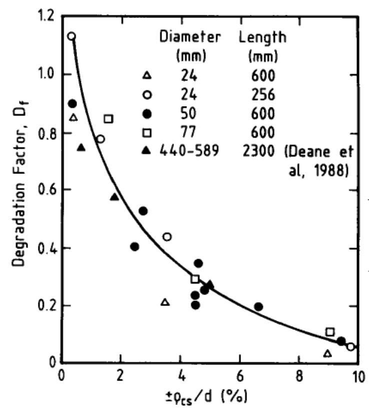


Fig. 21. Effect of normalized cyclic slip displacement on D_f with different pile diametes (after Lee, 1988)

Cyclic Stability Diagram

- Can represent effect of cyclic loading on pile capacity via a CYCLIC STABILITY DIAGRAM
- Plots ***Mean*** axial load vs ***Cyclic*** axial load
- 3 zones:
  - Stable
  - Metastable
  - Unstable

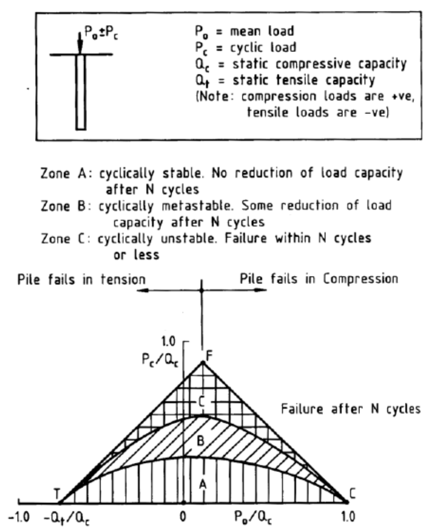
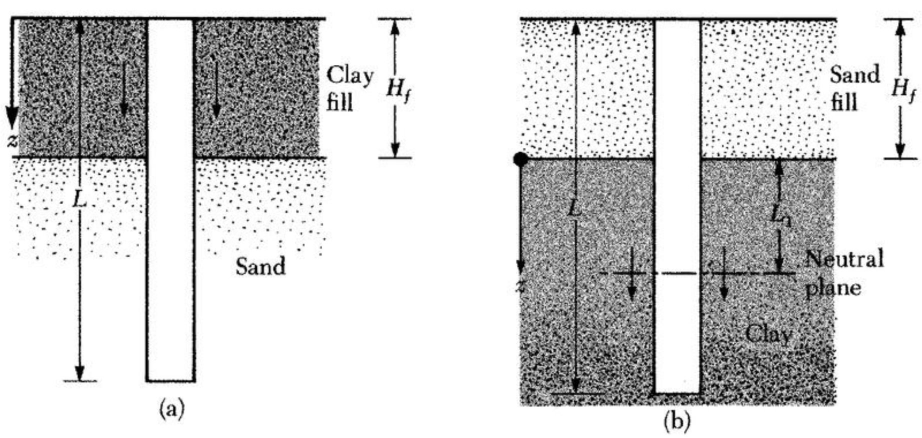
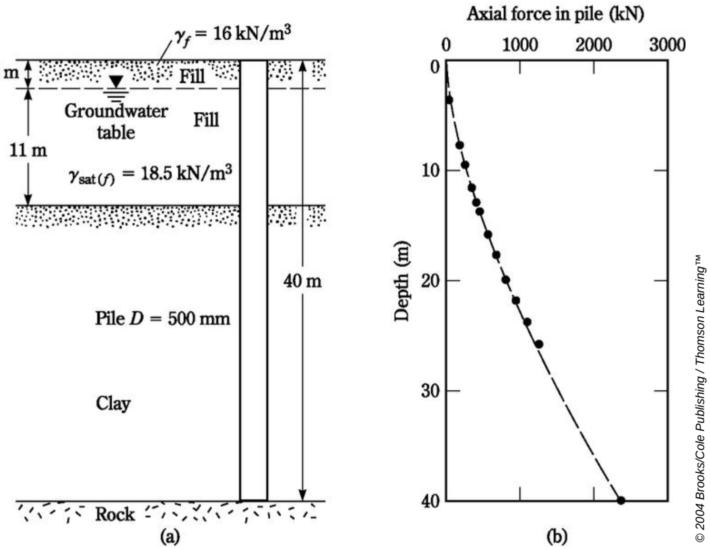


Figure 7.15 Main features of the cyclic stability diagram.

Negative Skin Friction

Down drag due to settlement

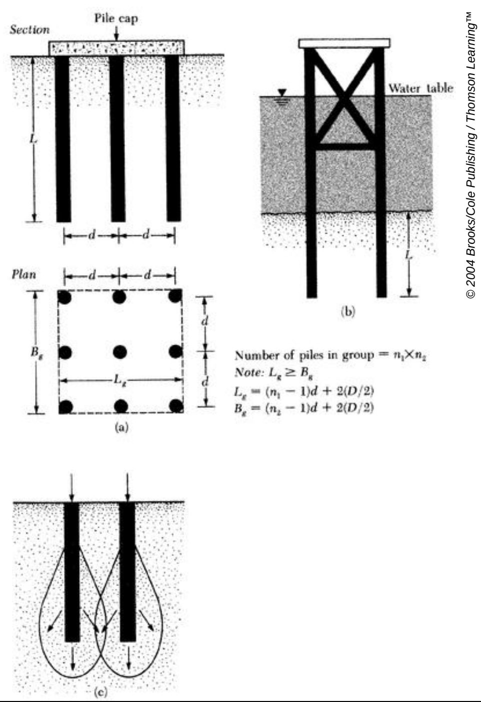




Negative skin friction on a pile in the harbor of Oslo, Norway

(based on Bjerrum et al. (1969) and Wong and Teh (1995))

Pile Groups

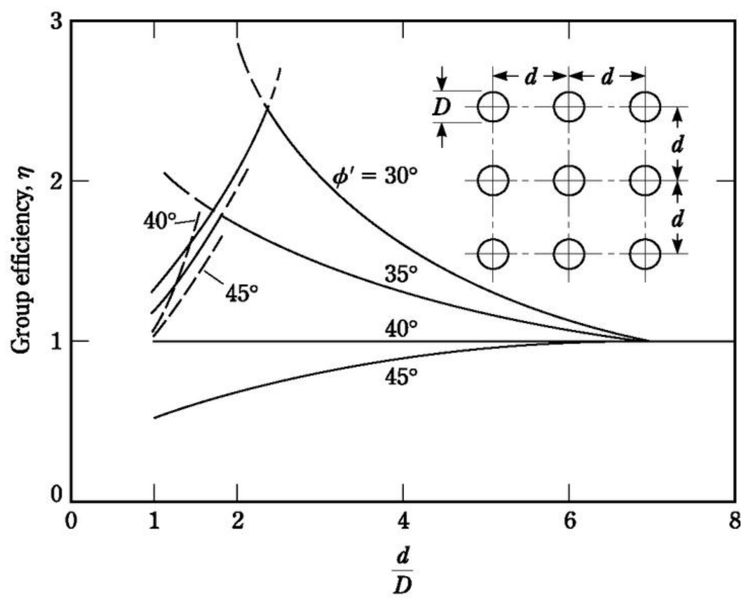


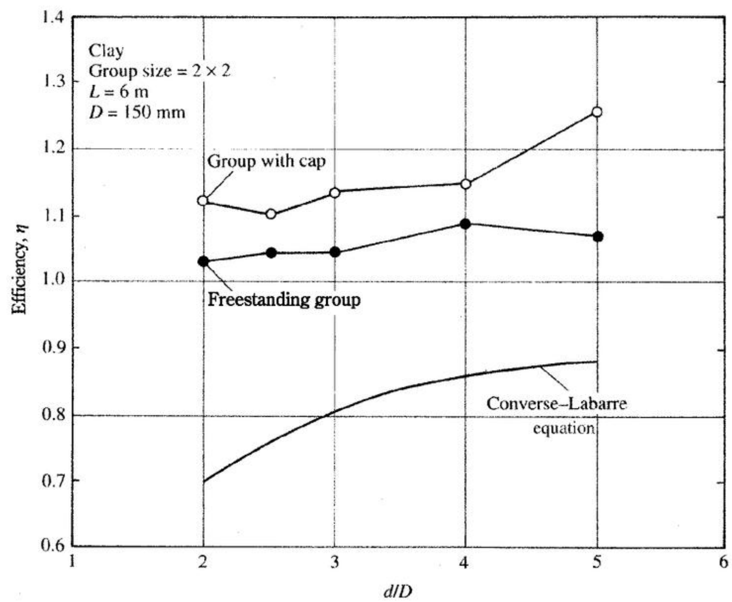
Pile Group Efficiency

Efficiency:

η = Group Capacity / Σ Individual Pile Capacities.

- For groups in clay, η usually < 1
- For groups driven in sand, η usually >1
- For groups (bored) in sand, η ~ 0.67
- For end bearing groups, η usually ~ 1





Friction Pile Groups in Clay

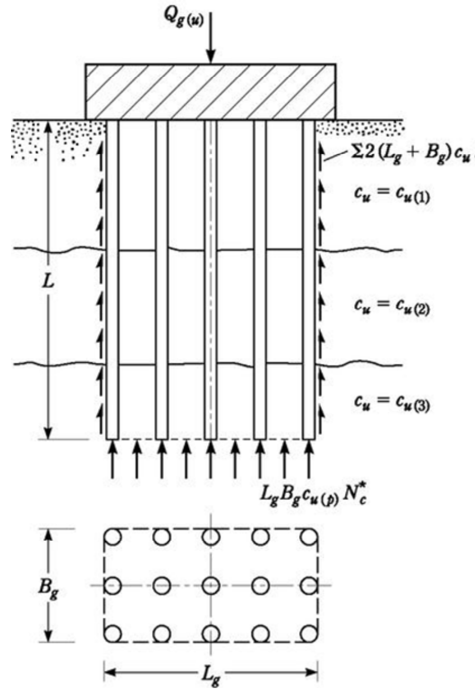
Group capacity (P_u) is lesser of:

- Sum of individual pile capacities (ΣP_1)
- Capacity of "block" containing piles + soil (P_B)

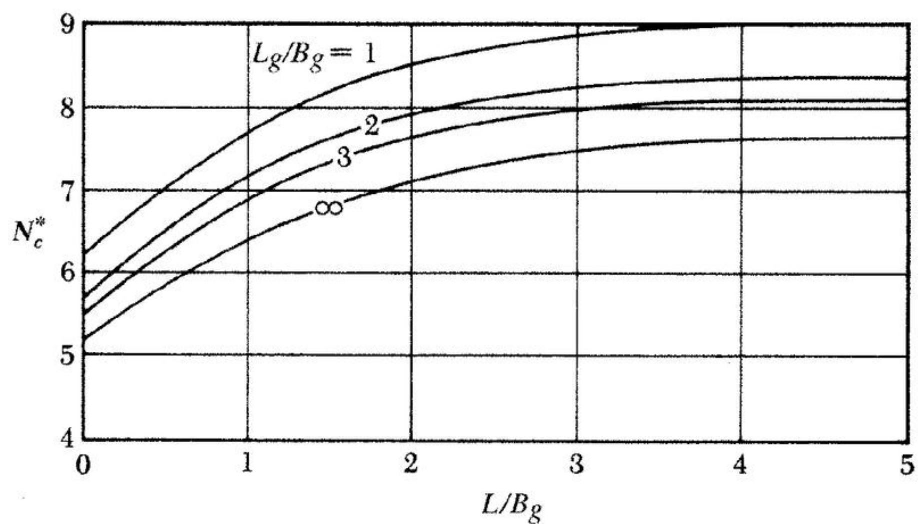
Empirical transition equation:

$$1 / P_u^2 = 1 / (\Sigma P_1)^2 + 1 / (P_B)^2$$

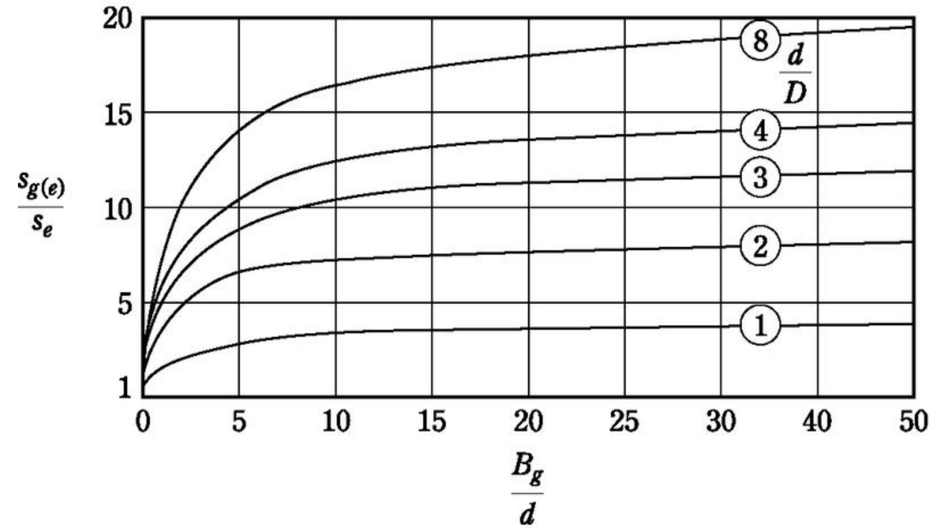
Block



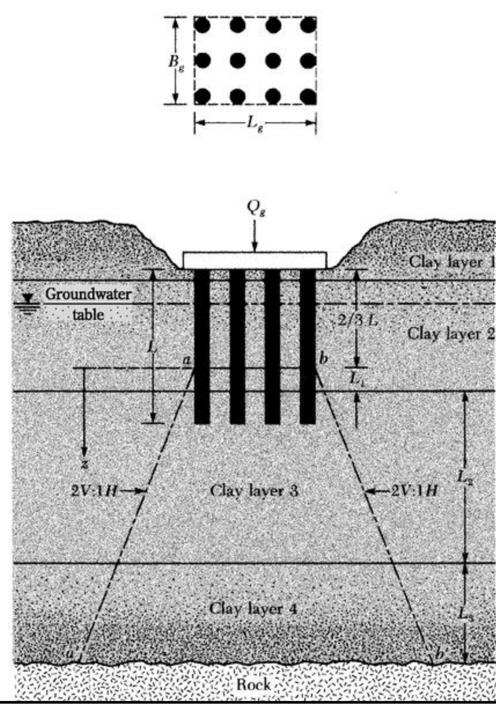
$Q_{g(u)}$
$\Sigma 2(L_g + B_g) c_u \Delta L$
$c_u = c_{u(1)}$
L
$c_u = c_{u(2)}$
$c_u = c_{u(3)}$
$L_g B_g c_{u(p)} N_c^*$
B_g
L_g



$L_g/B_g = 1$
2
3
∞
N_c^*
L/B_g
9
8
7
6
5
4
0
1
2
3
4
5



20
15
10
5
1
$\frac{s_{g(e)}}{s_e}$
0
5
10
15
20
25
30
40
50
$\frac{B_g}{d}$
8
4
3
2
1
$\frac{d}{D}$



B_g
L_g
Q_g
Clay layer 1
Groundwater table
2/3 L
Clay layer 2
L
a
b
L_1
z
2V:1H
Clay layer 3
L_2
Clay layer 4
L_3
a'
b'
Rock

Other Pile Group Cases

GROUP WITH CAP ON SURFACE

Group capacity (P_u) is lesser of:

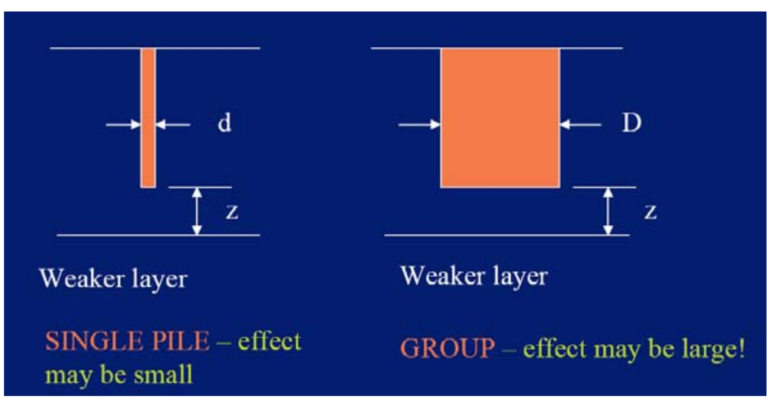
- Sum of individual pile capacities + net area of cap
- Capacity of "block" containing piles & soil, + capacity of portion of cap outside block perimeter.

GROUP ON PROFILE WITH UNDERLYING WEAK LAYER

- Take capacity as lesser of individual pile capacities, or capacity of block.

EFFECT OF WEAKER UNDERLYING LAYERS CAN BE VERY IMPORTANT!!

Effect of Weak Under Layer



Pile Structural Design

- Design for structural strength to resist
  - Axial force
  - Lateral shear force
  - Bending moment
- Make allowances for corrosion/ durability
- Consider possibility of buckling
  - Only likely to be of concern for slender piles in very soft clay with unsupported length.

Buckling

- Replace pile by equivalent cantilever
- CRITICAL LOAD is

$$P_{cr} = \frac{\pi^2 . E_p I_p}{4(S_R + J_R)^2 R^2}$$

(constant k)

$$P_{cr} = \frac{\pi^2 . E_p I_p}{4(S_R + J_R)^2 R^2}$$

(linearly increasing k)

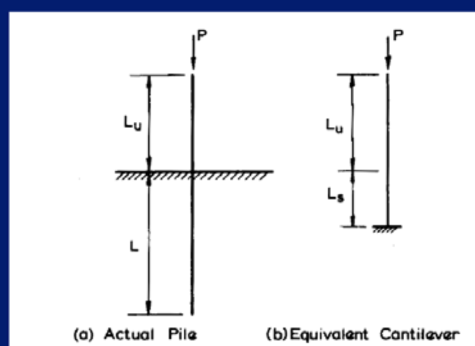


FIGURE 14.5 Partially embedded piles.

Buckling

Constant k_h

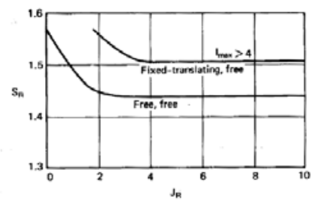


FIGURE 14.6 Dimensionless depth of fixity for buckling. Constant k_h (after Davisson and Robinson, 1965). (© Canada, 1965, by University of Toronto Press.)

$S_R = L_s/R$
$J_R = L_u/R$
$R = (E_p I_p / k_h d)^{0.25}$

$k_h = n_h z/d$

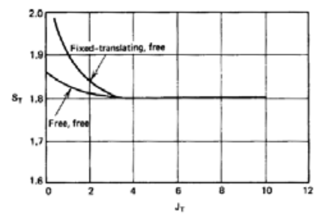


FIGURE 14.7 Dimensionless depth of fixity for buckling. Linearly varying k_h (after Davisson and Robinson, 1965). (© Canada, 1965, by University of Toronto Press.)

$S_T = L_s/T$
$J_T = L_u/T$
$T = (E_p I_p / n_h)^{0.20}$

Corrosion Rates for Steel

Corrosion penetration µm / year

Conditions	*Salt Water*	*Fresh Water*
Water at surface	100	50
Water in splash zone	300	200
Below water level	100	100
Bottom sediment	50	20

Corrosion Protection Methods

- Corrosion protection paint
- Polyethylene cover (steel pipes)
- Zinc coating
- Electro-chemical (cathodic) protection
- Cement or concrete cover