ANALYSIS AND DESIGN OF PILE FOUNDATIONS

H.G. POULOS

Coffey Geosciences Pty. Ltd.

COURSE OUTLINE

- Aspects of pile foundation design & construction; axial load capacity
- Analysis of settlement of piles & pile groups
- Innovative Aspects of Pile Design
  - Piled raft foundations
  - Compensated piled rafts
  - Controlled stiffness inserts
- Piles subjected to lateral & vertical ground movements

COURSE OBJECTIVES

- Assist in understanding behaviour of pile foundations and factors affecting behaviour.
- Present results of modern analyses which may be useful in design.
- Emphasize the importance of ground movements in pile design.

LECTURE 1

ASPECTS OF DEEP FOUNDATION DESIGN AND CONSTRUCTION & AXIAL LOAD CAPACITY

OUTLINE

- Pile types
- Design considerations
- Installation issues
- Design for safety – approaches
- Ultimate axial capacity of piles
- Structural design issues
- Durability issues

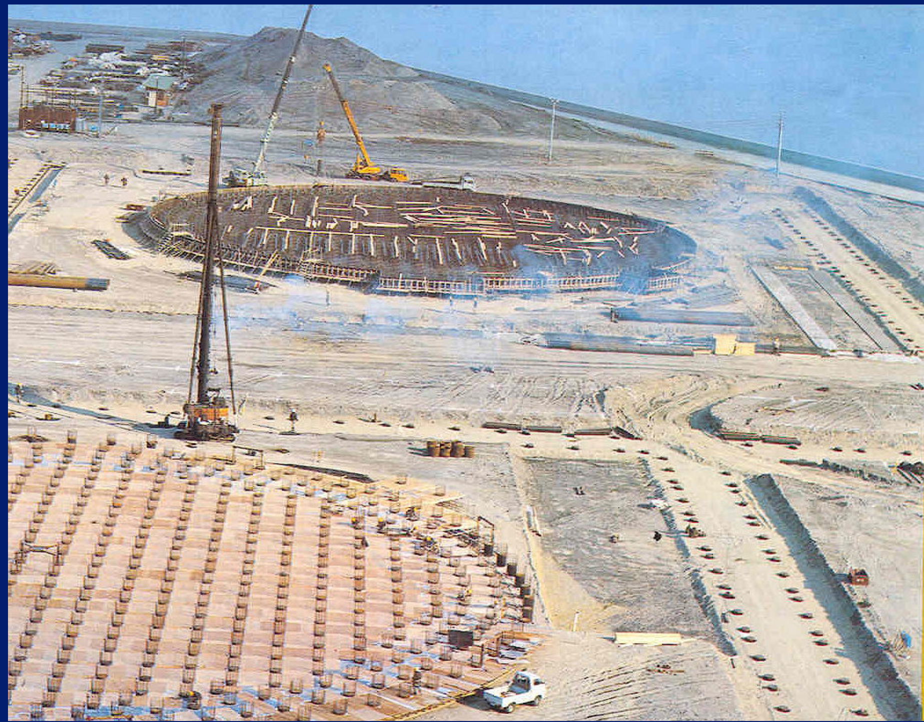
TYPICAL APPLICATIONS OF PILES

Marine and harbour works

TYPICAL APPLICATIONS OF PILES

Roads & bridges

TYPICAL APPLICATIONS OF PILES



Buildings

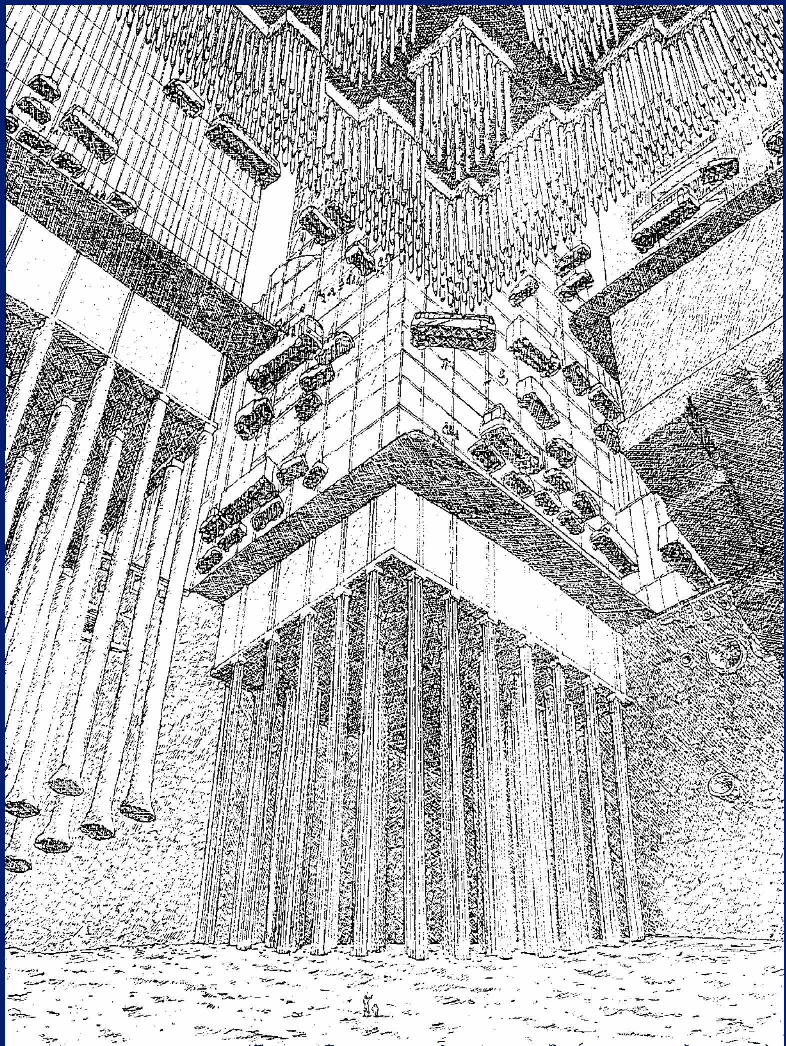
Storage Tanks

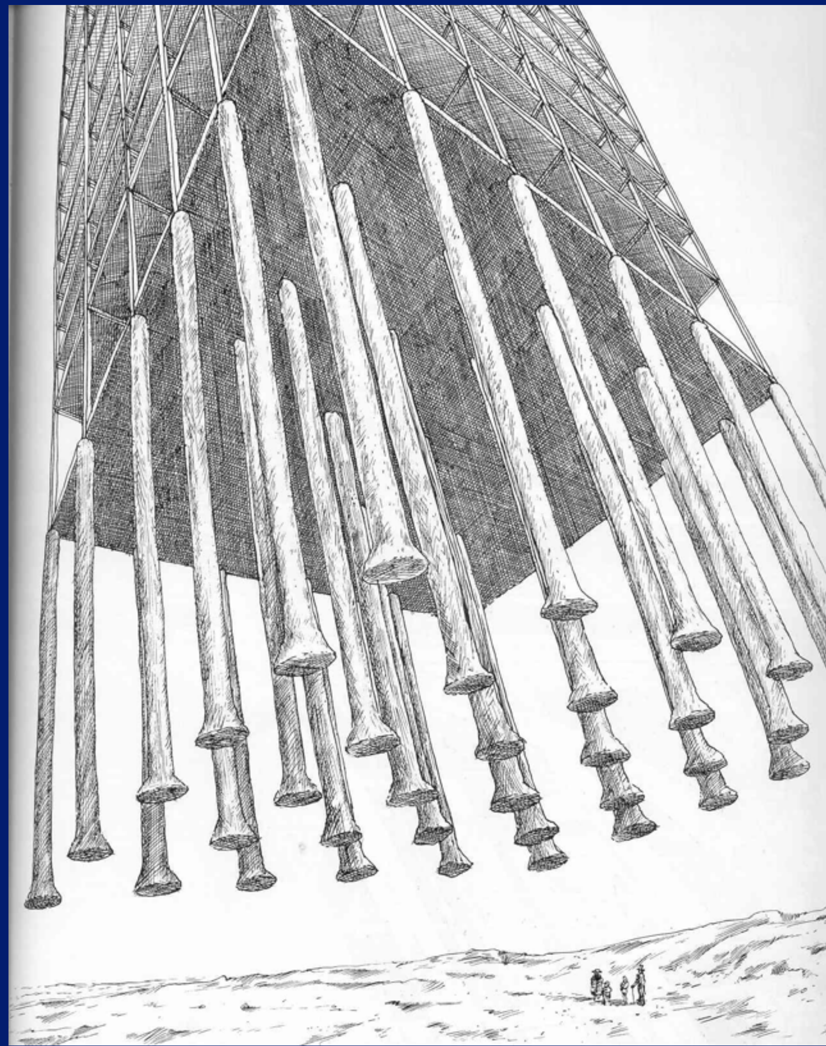
TYPICAL APPLICATIONS OF PILES

Retaining Wall, Sydney

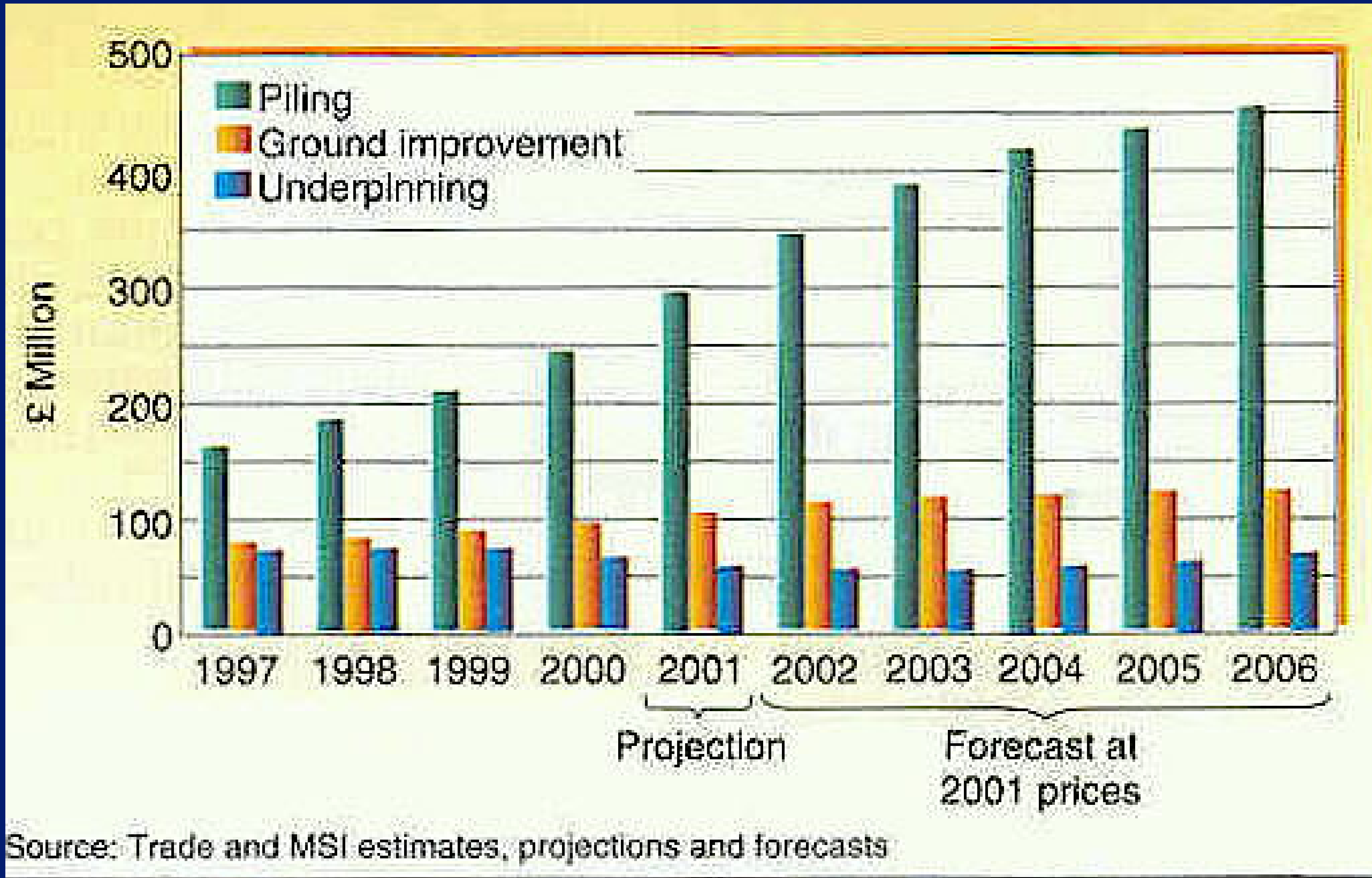
Retaining Wall, Hong Kong

PERSPECTIVE ON PILES





INDUSTRY OUTLOOK



TYPES OF PILE

- CLASSIFICATION BY MATERIAL
  - Steel
  - Concrete
  - Timber
- BY EFFECT OF INSTALLATION
  - Displacement
  - Low displacement
  - Non-displacement
- BY METHOD OF INSTALLATION
  - Driven
  - Driven & cast-in-place
  - Bored (drilled)
  - Composite
  - Screwed

EFFECTS OF PILE INSTALLATION

Bored Pile | Driven Pile in Clay | Driven Pile in Sand

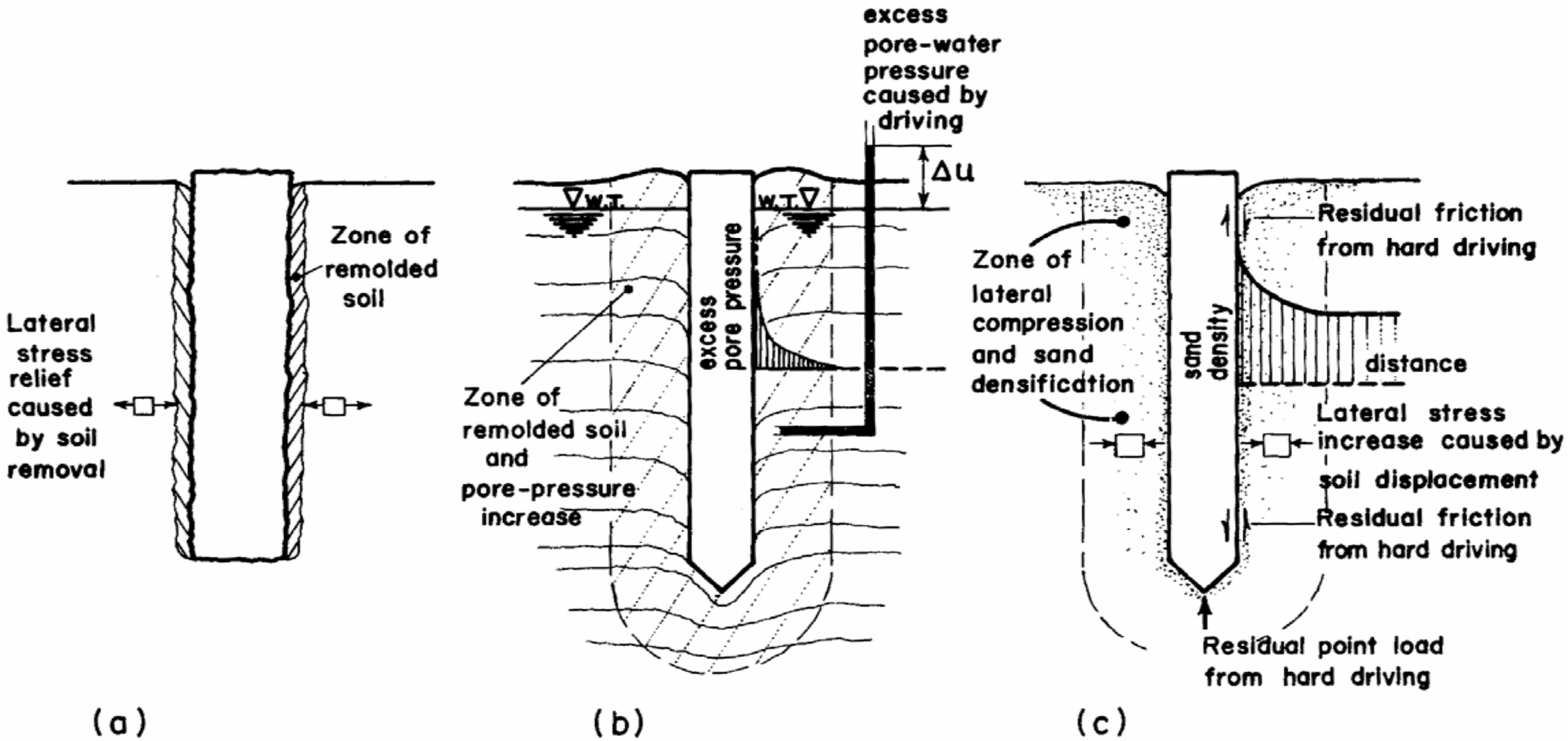


Figure 6. Effects of placement of pile into a soil mass.

DISPLACEMENT PILES

Installed by driving or jacking

- TIMBER
  - Marine & temporary structures, domestic buildings
  - Durability concerns
- STEEL TUBES
  - Readily extended
  - Corrosion concerns
  - Usually more expensive
- PRECAST CONCRETE
  - Common lengths 12-15 m
  - Cost & appearance advantages
  - Not suited to hard driving
  - Not easily spliced
- PROPRIETARY TYPES
  - Many use temporary steel casing
  - Casing withdrawn as concrete placed
  - Limitations on length

INSTALLATION OF DISPLACEMENT PILES

- Usually installed via pile driving hammer.
- Hammer types:
  - Drop hammer (typically 1-5 t mass)
  - Steam hammer
    - Single acting
    - Double acting
  - Diesel hammer – less used in recent years
  - Hydraulic hammer – ram raised by fluid
  - Vibratory hammer – sheet piles, piles in sand.

EQUIPMENT FOR PILE JACKING

Gold Coast Convention Centre, Australia

EQUIPMENT FOR PILE JACKING

The pile
being jacked in

CONCERNS WITH DISPLACEMENT PILES

- Vibrations during installation
- Excess pore pressures generated
- Soil movements (vertical & lateral)
- Access for driving rigs
- Headroom in confined spaces

PROBLEMS FROM VERTICAL SOIL DISPLACEMENTS

- Uplift, causing squeezing, necking or cracking
- Uplift. Resulting in shaft lifting off base
- Uplift, resulting in loss of stiffness & bearing capacity
- Ground heave lifting adjacent piles
- Ground heave, separating pile segments, inducing extra tensile forces in joints, possible tensile cracking of adjacent piles

PROBLEMS FROM LATERAL SOIL DISPLACEMENTS

- Squeezing or waisting of piles
- Inclusions of soil forced into pile
- Shearing of piles, or bends in joints
- Collapse of casing prior to concreting
- Movement & damage to adjacent structures

SMALL DISPLACEMENT PILES

H-Sections & Rolled Steel Sections

- Useful for punching through hard layers
- BUT, problems with bending about weak axis

Steel Tube Piles

- Lees resistance to water & waves
- Plugs can be removed
- Can fill with concrete
- Better lateral resistance

Pre-Drilled Piles

- Usually pre-bored over part depth, then driven
- Useful if have hard layers near surface

INSTALLATION ISSUES FOR DISPLACEMENT PILES

- Pile driving analysis
  - Assess driveability
  - Stresses during driving
- Instability during driving
- Time effects

PILE DRIVING FORMULAE

- Many available.
- Most uncertain is Engineering News formula. Usually need to use high safety factor (e.g. 6). Actual safety factor can be between 1.1 & 30!
- Formulae involving least uncertainty are Janbu, Danish, Hiley.

NOTE WARNINGS OF:

1. **Newton** – warned against application of his theory to problems involving impact via "the stroke of a hammer".
2. **Terzaghi** (1943) – "all existing pile formulae are utterly misleading..it is necessary to take into consideration the vibrations which are produced by the impact".

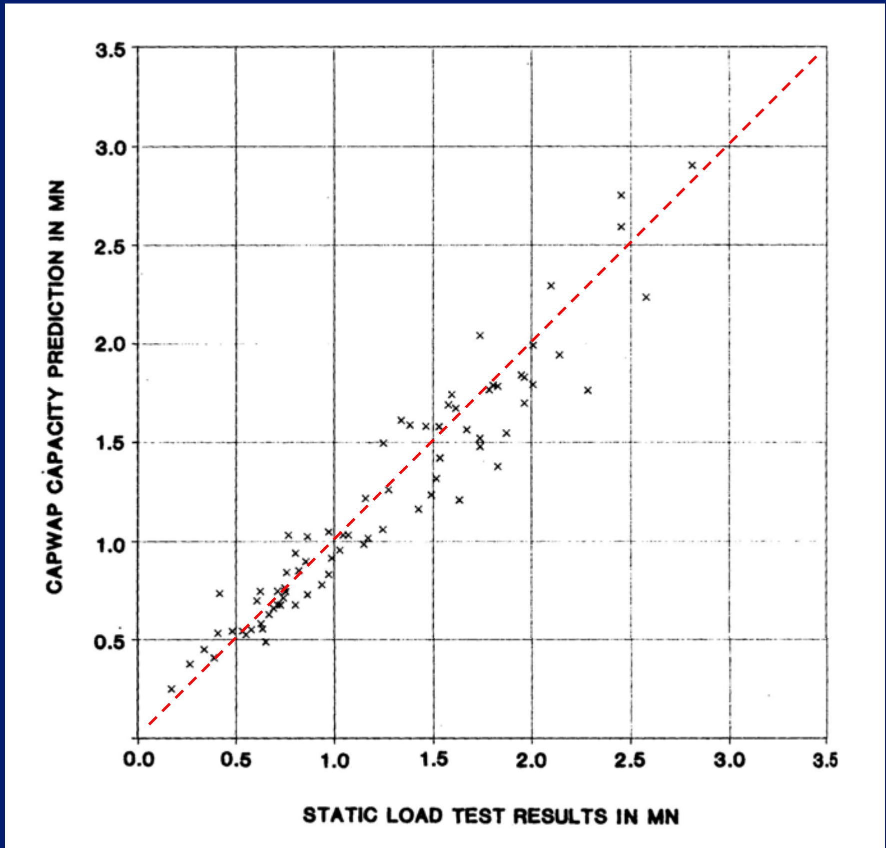
WAVE EQUATION ANALYSIS

- Takes account of fact that each hammer blow produces a stress wave that moves down the pile length at the speed of sound, so that the entire length of the pile is not stressed simultaneously, as assumed in Newtonian impact theory.
- Basic equation is of following form:

$$\partial^2 D / \partial t^2 = (E/\rho)\, \partial^2 D / \partial z^2 + R$$

- Can solve analytically or numerically.
- Most convenient numerical solution is that by Smith (1960).

RELIABILITY OF WAVE EQUATION



- Wave equation is at least as good as best of pile driving formulae.
- Typically, pile capacity can be predicted to 25% for piles in sand, 40% for piles in clay.
- Problem with “set-up” for piles in clay.
- Restrike blowcount is a better indicator of static capacity

STANDARD DEVIATION of Q_{msd}/Q_{calc} FOR VARIOUS DYNAMIC METHODS

Method	*Av. SD*	*No. of Tests*
Danish	0.31	314
Engineering News	0.87	285
Eytelwein	0.66	78
Gates	0.41	114
Hiley	0.38	164
Janbu	0.30	192
Pacific Coast	0.35	114
Weisbach	0.41	123
Wave Equation	0.26	78
Case Method	0.12	130
CAPWAP	0.13	67

Denver & Skov, (1988)

DEVELOPMENTS IN DYNAMIC ANALYSIS

- Use of dynamic F/E analysis
  - 2.5 mm quake is generally adequate
- Improved 1-D models (Randolph & Simons, 1986; Lee et al, 1988)
  - Use of solutions of Novak for dynamic pile response to evaluate damping values. Radiation damping associated with spring.
  - Gives a greater degree of damping than conventional Smith analysis.
  - Results agree well with 3-D F/E analyses.

INSTABILITY DURING DRIVING

- Flutter mechanism (Burgess, 1975,1976)
- Directional instability during driving
  - Slender piles deviate from assumed lines of action
- Likelihood related to:
  - Pile stiffness
  - Length
  - Driving resistance

CRITICAL LENGTH Lc FOR FLUTTER

a) End Bearing:

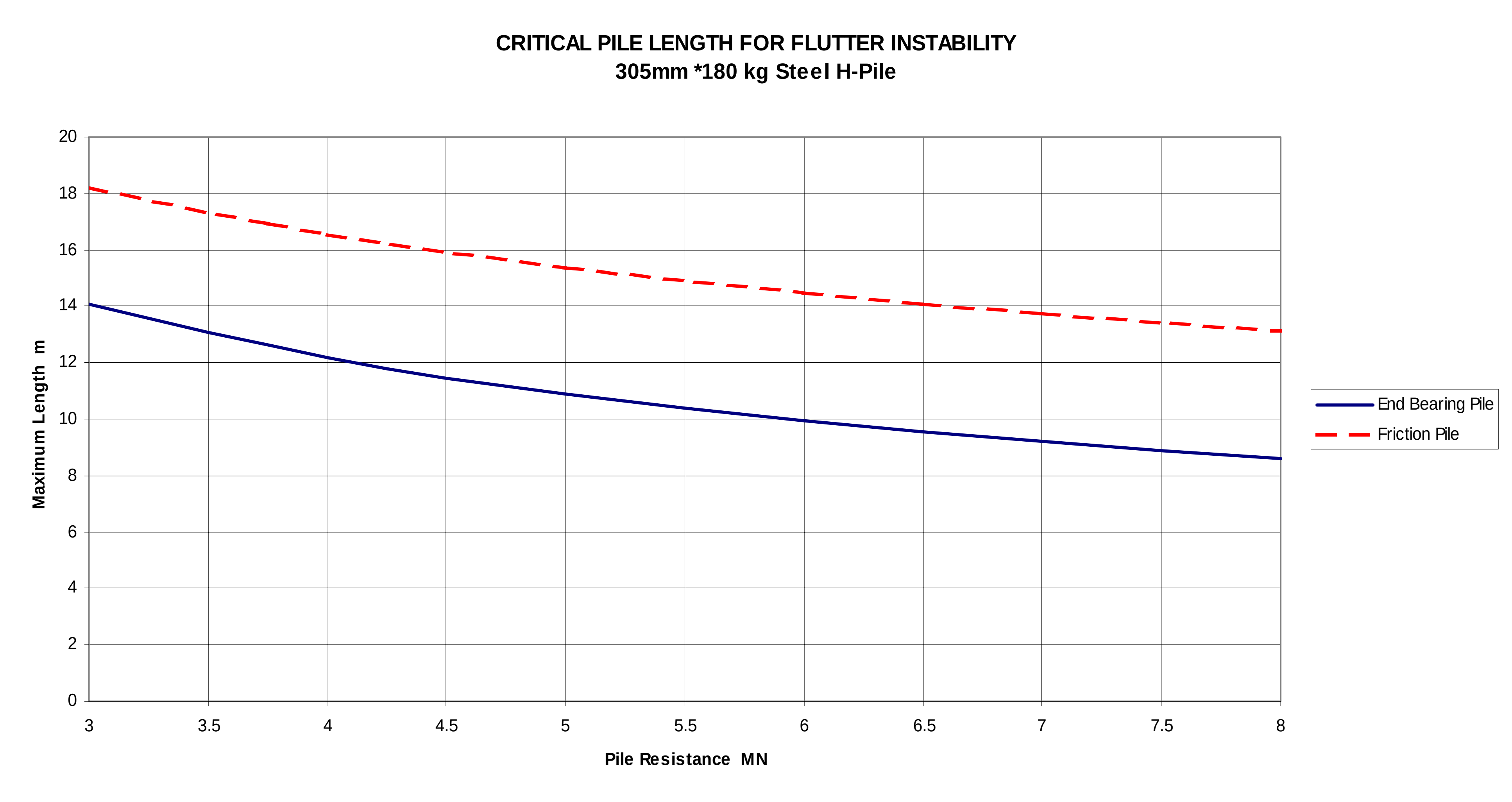
$$L_c = (20.05\ EI / P)^{0.5}$$

b) Friction Pile:

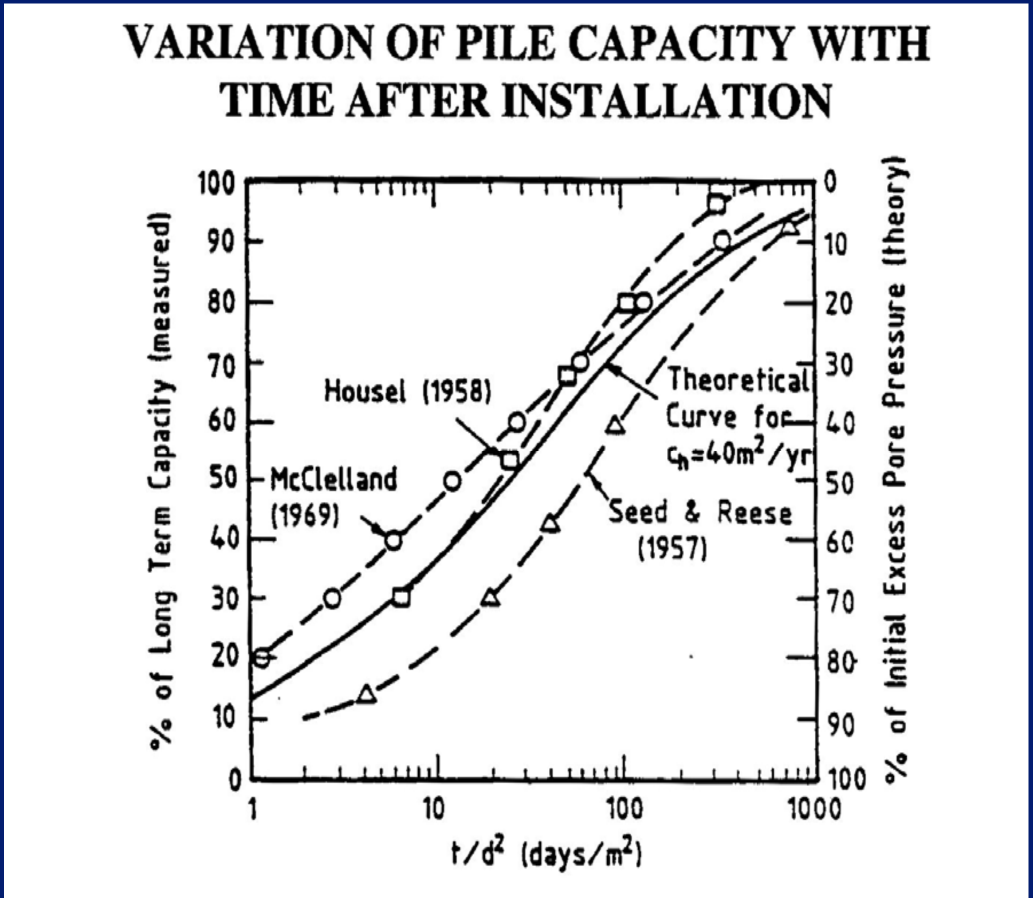
$$L_c = (40.7\ EI / f_s)^{0.333}$$

(EI = bending stiffness, P=load, f_s = average shaft friction)

CRITICAL LENGTH FOR FLUTTER - EXAMPLE



DISPLACEMENT PILES – TIME EFFECTS

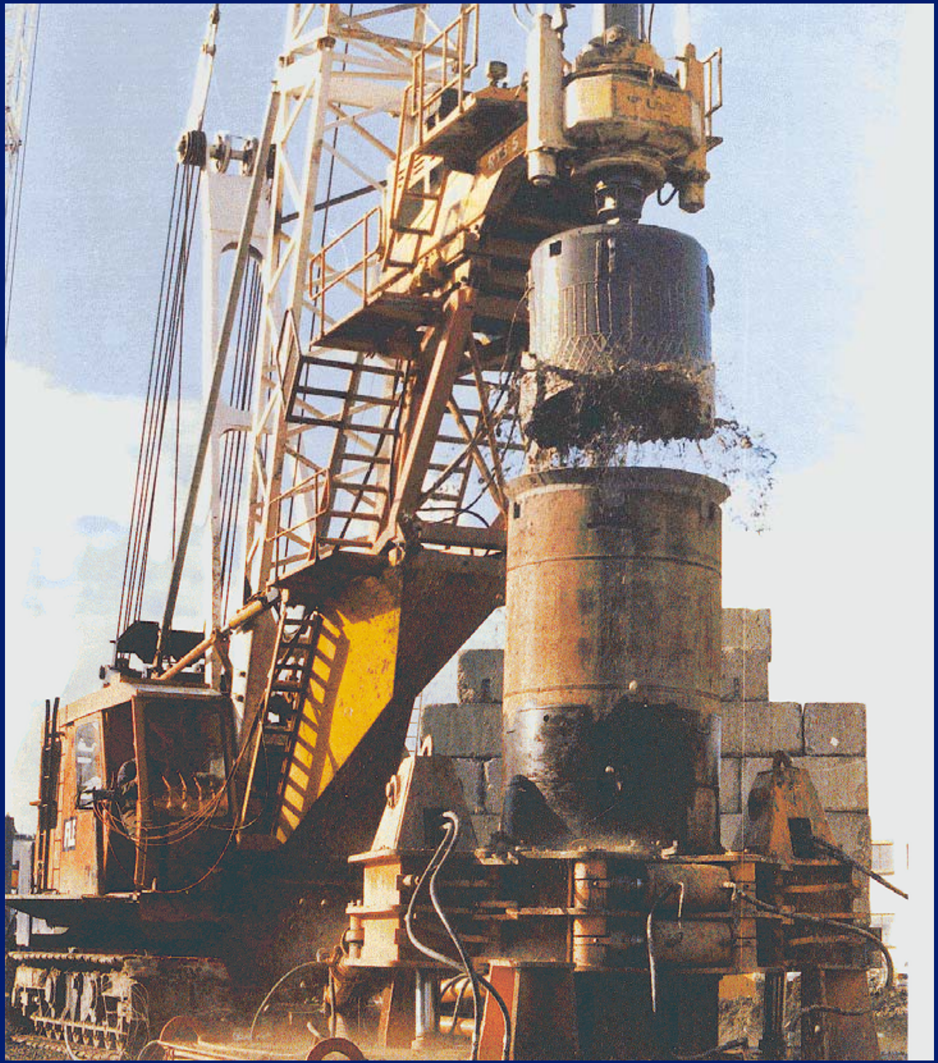


- Dissipation of excess pore pressures developed during driving
- Usually leads to increased load capacity with time – “SET - UP”
- Theoretical solutions and filed data shown
- Can also have set-up effects for driven piles in sand, especially carbonate sands
- May be due to chemical processes at pile-soil interface

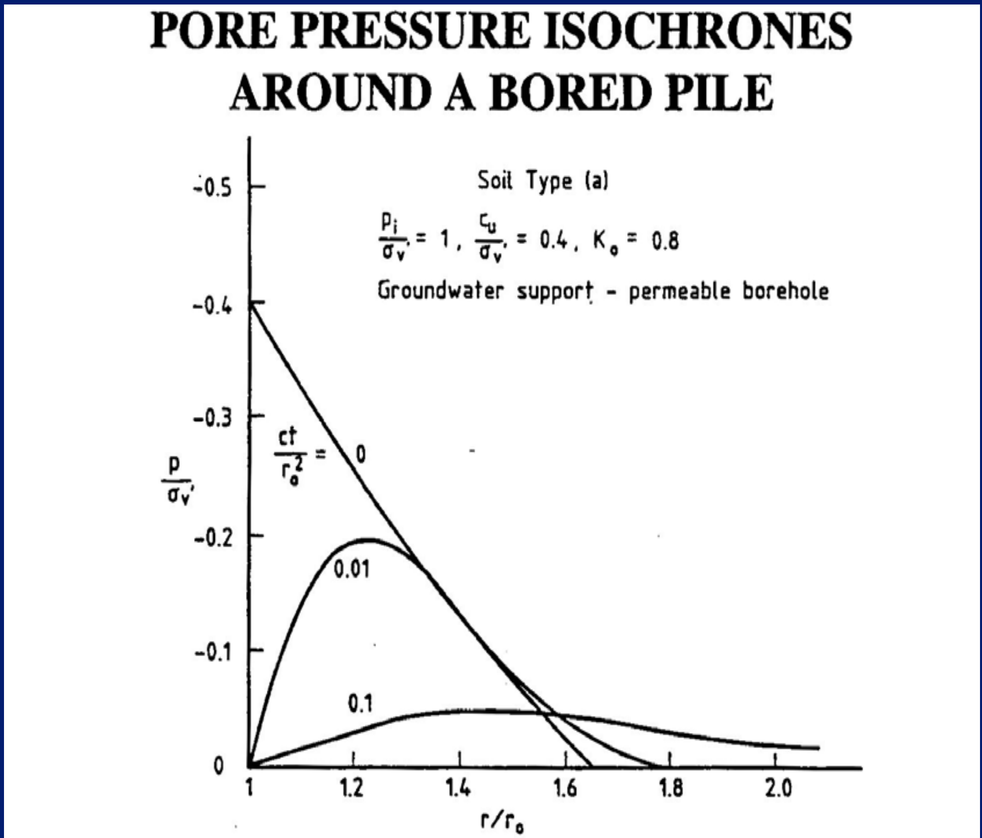
NON - DISPLACEMENT PILES

- **INSTALLATION METHODS**
  - Dry
  - Slurry
  - Casing
- **ADVANTAGES**
  - Absence of ground heave
  - No excessive noise or vibration
  - Can install with limited headroom
  - Length & diameter can be easily varied
  - Base can be enlarged
  - Can inspect prior to concreting
  - Can obtain very high capacity
- **PROBLEMS**
  - Loosening of sandy soils
  - Softening of clays
  - Possible waisting or necking of shaft
  - Water inflow can damage shaft
  - Belled bases difficult, especially in sandy soils
  - Need to place concrete as soon as possible after drilling!

BORED PILE CONSTRUCTION



BORED PILES- TIME EFFECTS

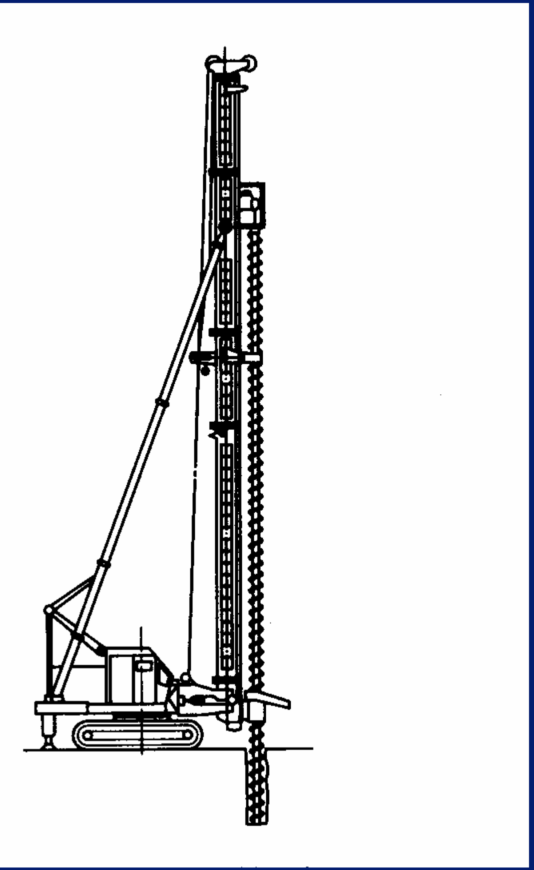


- Reduction is suction after drilling
- Consequent softening & possible caving of hole
- Solutions for pore pressures vs time shown
- Typically, for a bored pile 0.8 m diameter, with soil $c_v = 10^{-5}$ m/s, T=0.1 corresponds to less than ½ hour !

BORED PILE CONSTRUCTION-PRECAUTIONS

- Pile should be supported by casing through soft or loose soils to prevent collapse
- Casing provided to seal off water-bearing layers
- Strict control of density of drilling fluid, if used
- Compare soil & rock cuttings from pile & descriptions form site investigation
- Shear strength tests from bottom of selected piles to check against design assumptions
- Plumb deep holes immediately after concreting; compare plumbed depth with that at end of drilling
- Proper measures for base cleaning – video or visual inspection where possible
- Safety procedures followed strictly
- **Time interval between end of boring & concreting kept as short as possible, no longer than 6 hours.**

CONTINUOUS FLIGHT AUGER (CFA) PILES



- Flight auger has hollow stem
- Borehole walls supported by soil rising within flights
- Concrete (fluid grout) injected down hollow stem
- Reinforcing cage installed after auger removed
- Strict construction control ESSENTIAL, especially if require end bearing
- Checks via recording of volume of concrete and torque on drill stem
- Size Limits: Diameter 1.5m; Length 35m

DESIGN REQUIREMENTS

- **Ultimate limit state**
  - Adequate capacity (geotechnical & structural) to resist ultimate load combinations
- **Serviceability limit state**
  - Deflections and differential at normal “working” loads are within tolerable limits
- **Durability**
  - Piles must remain durable during design life, or else be designed for acceptable deterioration

DESIGN CONSIDERATIONS

- Selection of pile type and installation method
- Size & number of piles for adequate factor of safety
- Settlement & differential settlement checks
- Effects of lateral loading
- Effect of ground movements (if any)
- Evaluation of pile performance – load testing

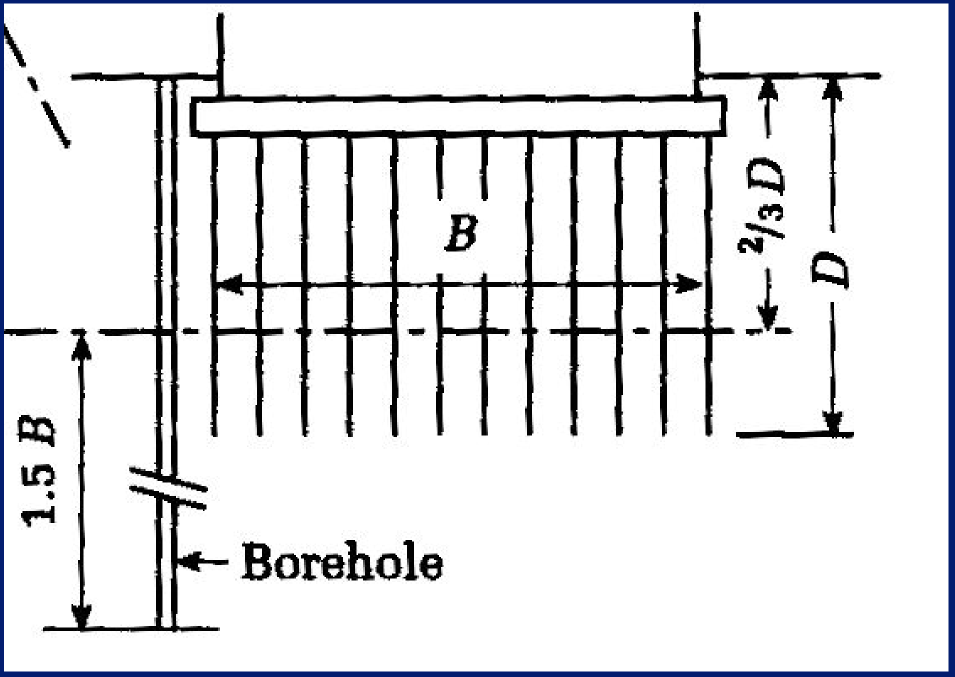
SELECTION OF PILE TYPE

Depends on:

- Location & type of structure
- Ground conditions
- Access for piling equipment
- Durability requirements
- Effects of installation on adjacent piles, structures, people
- Relative costs

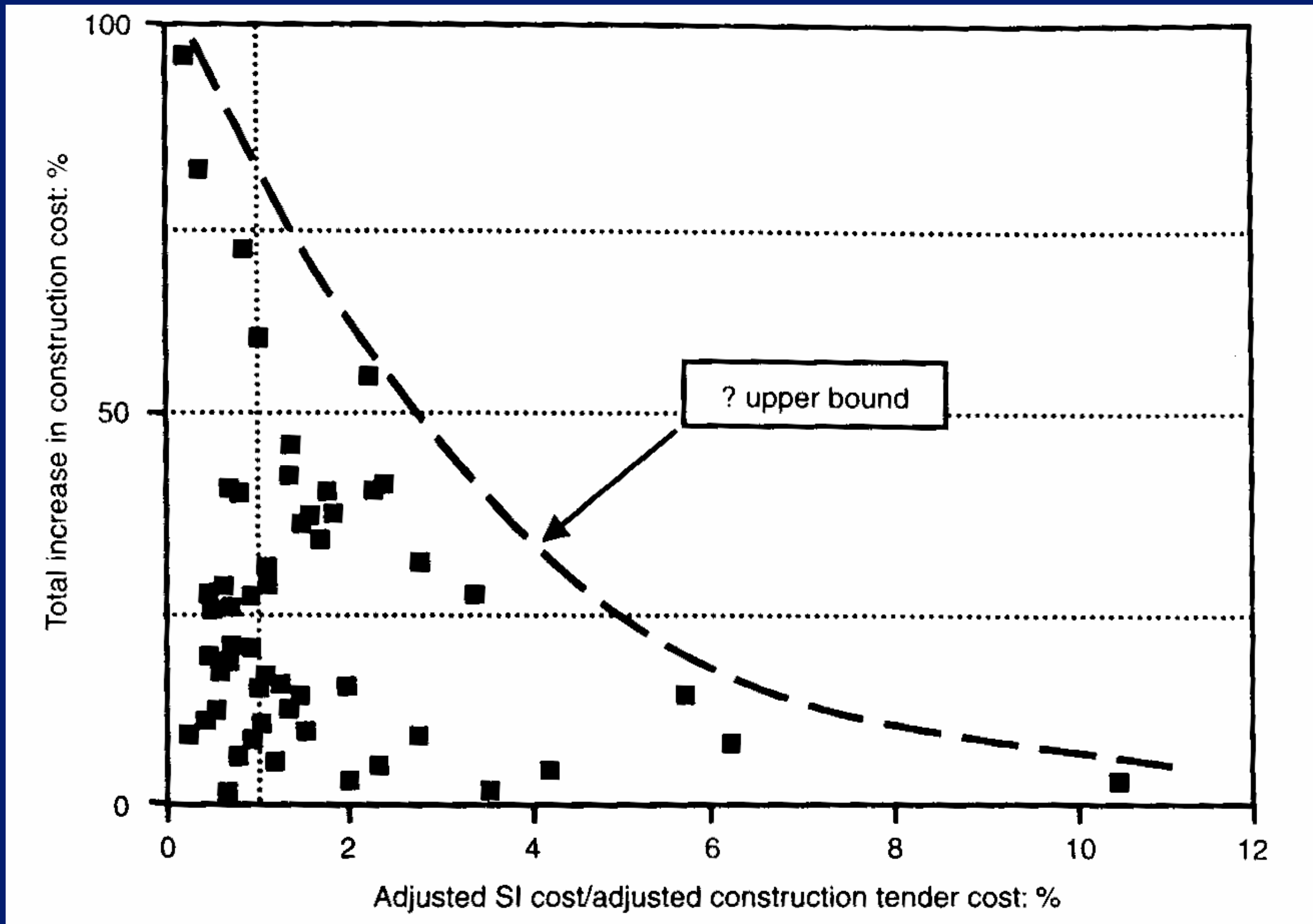
GROUND INVESTIGATIONS

- Is as important for pile foundations as for shallow foundations.
- Need to extend exploration to depth of influence of pile or pile group.
- Need to prove rock or founding material – usually drill min. 3m into rock.
- Beware of compressible layers below the pile tips.



Tomlinson’s suggestion for depth of investigation

GROUND INVESTIGATIONS – THE REAL COSTS



Clayton (2001)

DESIGN FOR SAFETY – ALTERNATIVE APPROACHES

OVERALL SAFETY FACTOR

$$P_w = P_u / FS$$

PARTIAL SAFETY FACTORS (European)

- Factor up loads and use factored-down soil parameters to compute design resistance

LOAD & RESISTANCE FACTORED DESIGN (LRFD)

- Resistance computed using factored down ultimate shaft & base capacities = R_d
- Load factored up by load factors $=S_d$
- $\mathbf{R_d > S_d}$

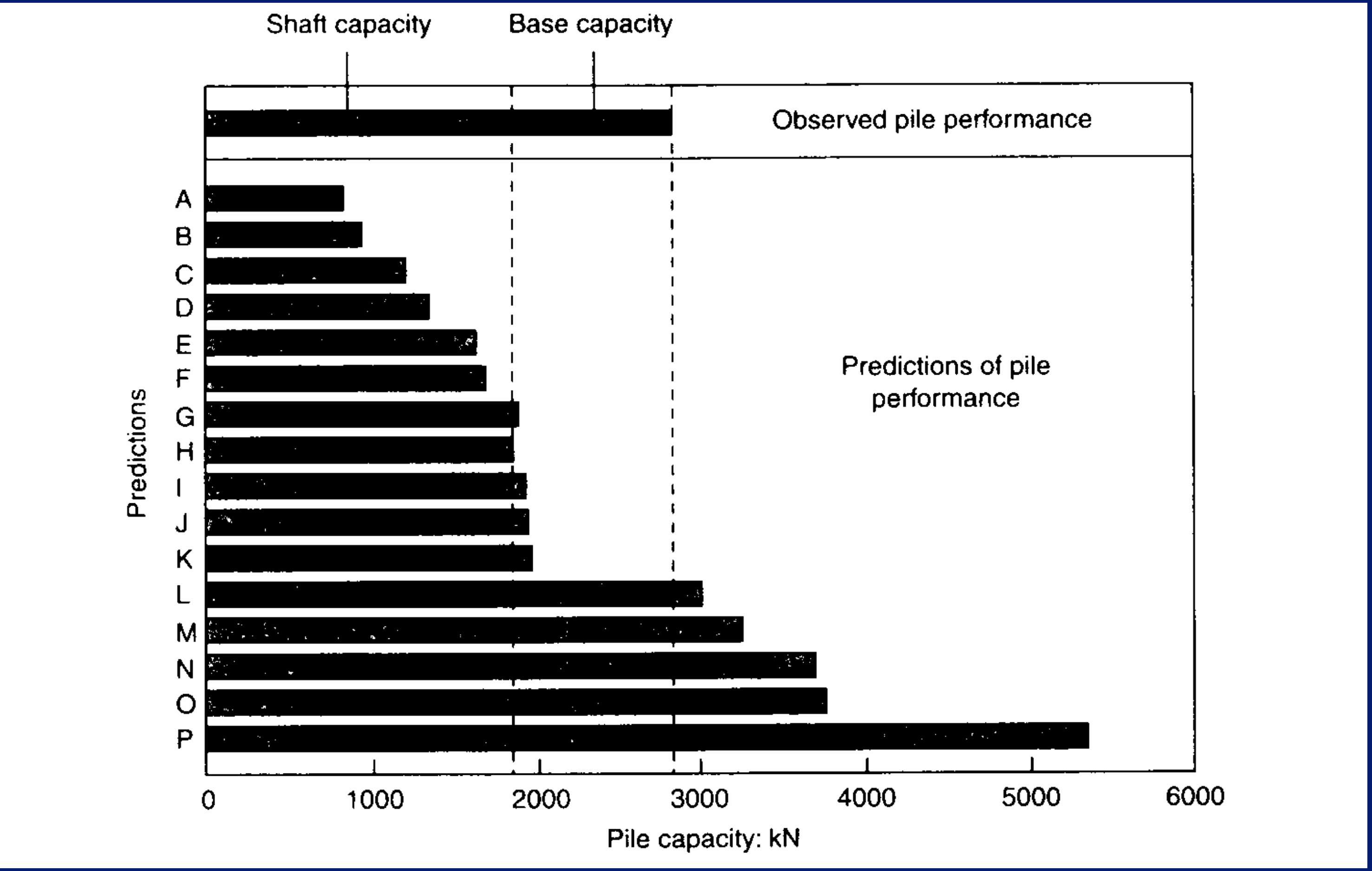
PROBABILISTIC APPROACH

- P(failure) < Allowable value (e.g. 10^{-4})

CATEGORIES OF ANALYSIS & DESIGN METHODS

- ***Category 1***
  - Empirical
- ***Category 2***
  - Soundly-based, simplified theory and/or charts
- ***Category 3***
  - 3A - Site-specific theory - simple soil
  - 3B - Site-specific - simple nonlinear soil
  - 3C - Site-specific - proper soil model

ULTIMATE LOAD CAPACITY



How good are Modern Predictions?

NOT VERY GOOD AT ALL!

Clayton (2001)

ULTIMATE LOAD CAPACITY

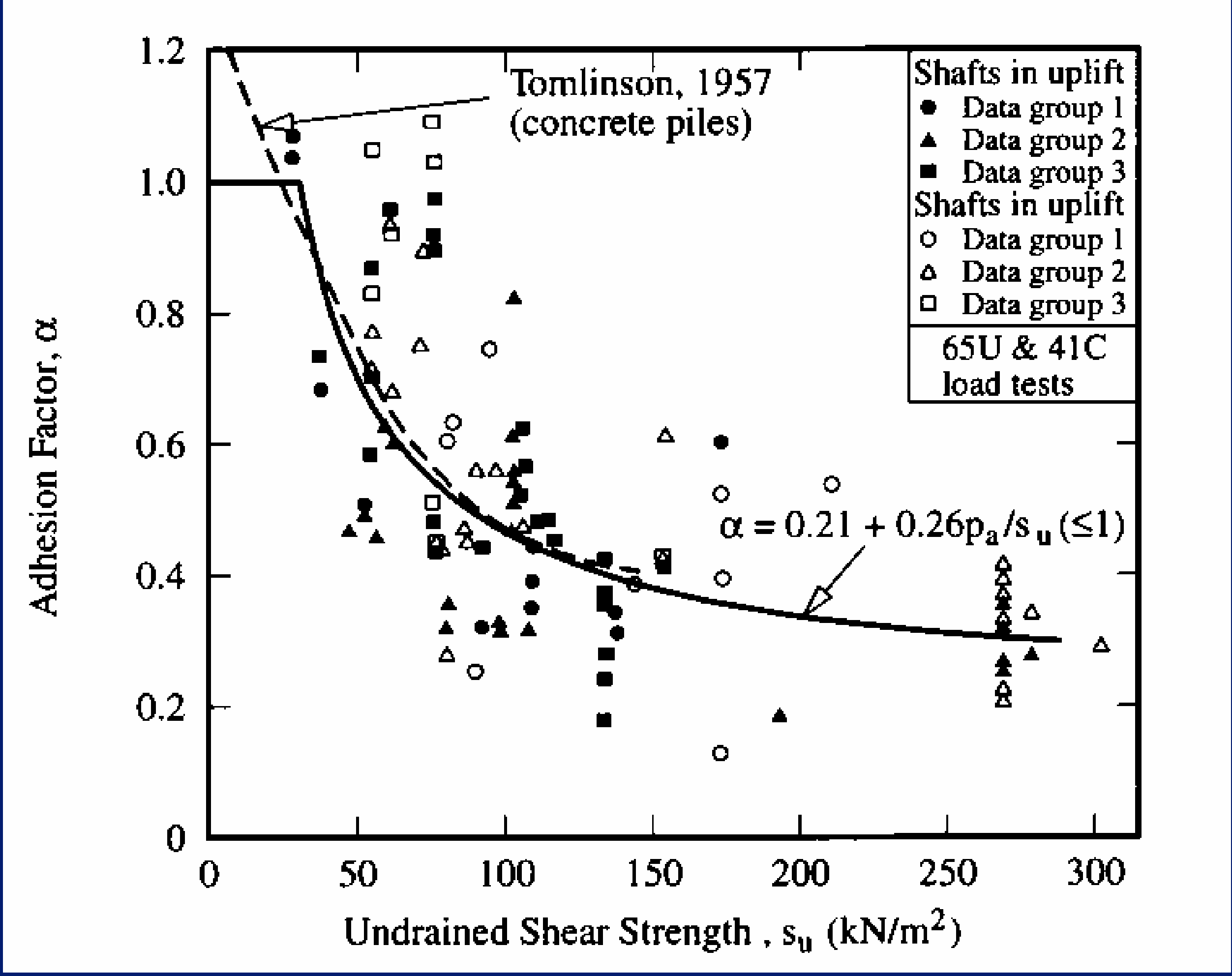
- Usually add resistances of shaft and base
- Shaft capacity computed via:
  - Total stress (α) method
  - Effective stress (β) method
  - Hybrid (λ) method
  - Using SPT data
  - Using CPT data
  - Using PMT data

SHAFT CAPACITY OF PILES IN CLAY

- $f_s = \alpha.s_u$ (alpha method)
- $f_s = \beta.\sigma_v'$ (beta method)
- $f_s = \lambda.(\sigma_{vm}' + 2\, s_{um})$ (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 OF PILES IN CLAY – ALPHA METHOD



SHAFT CAPACITY OF PILES IN CLAY – ALPHA METHOD

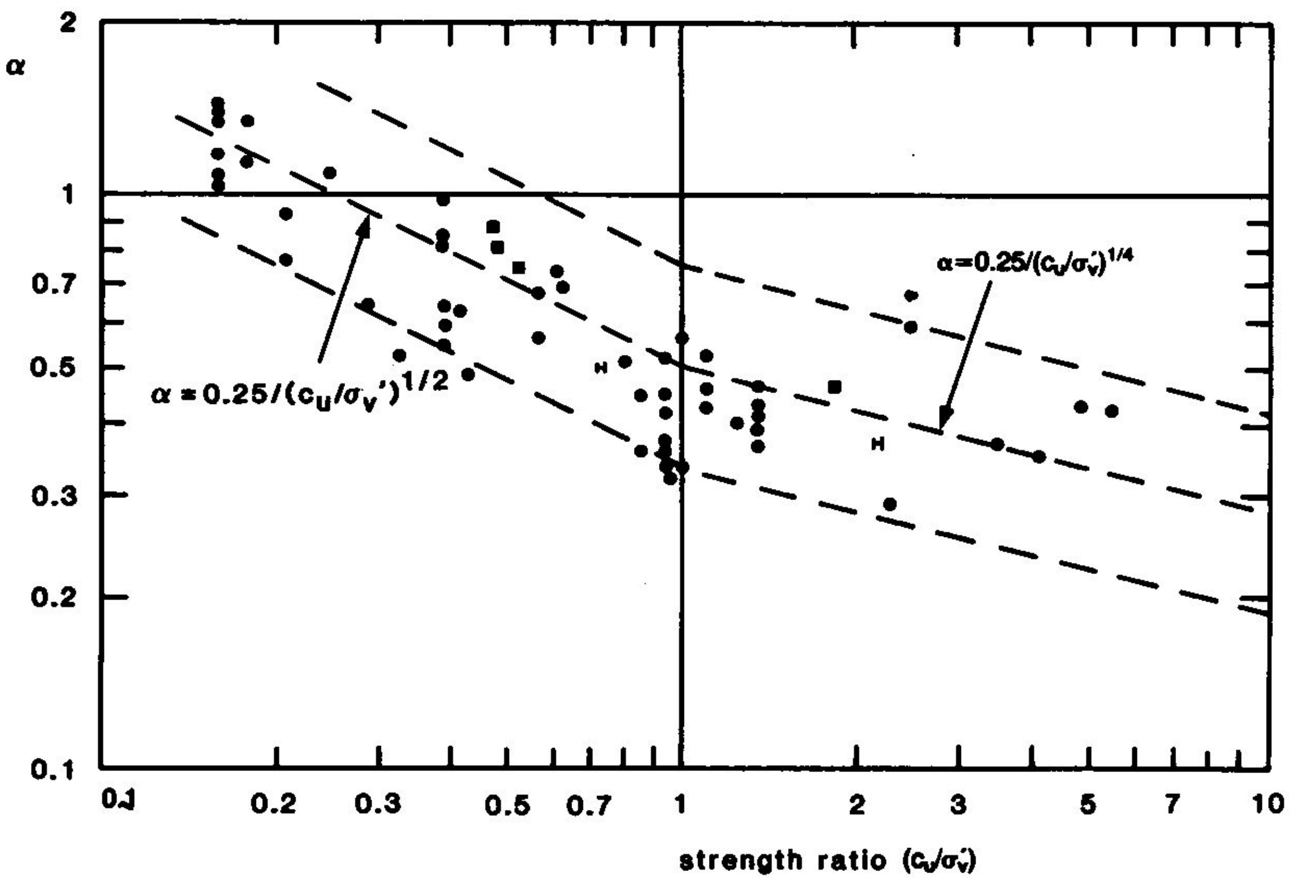
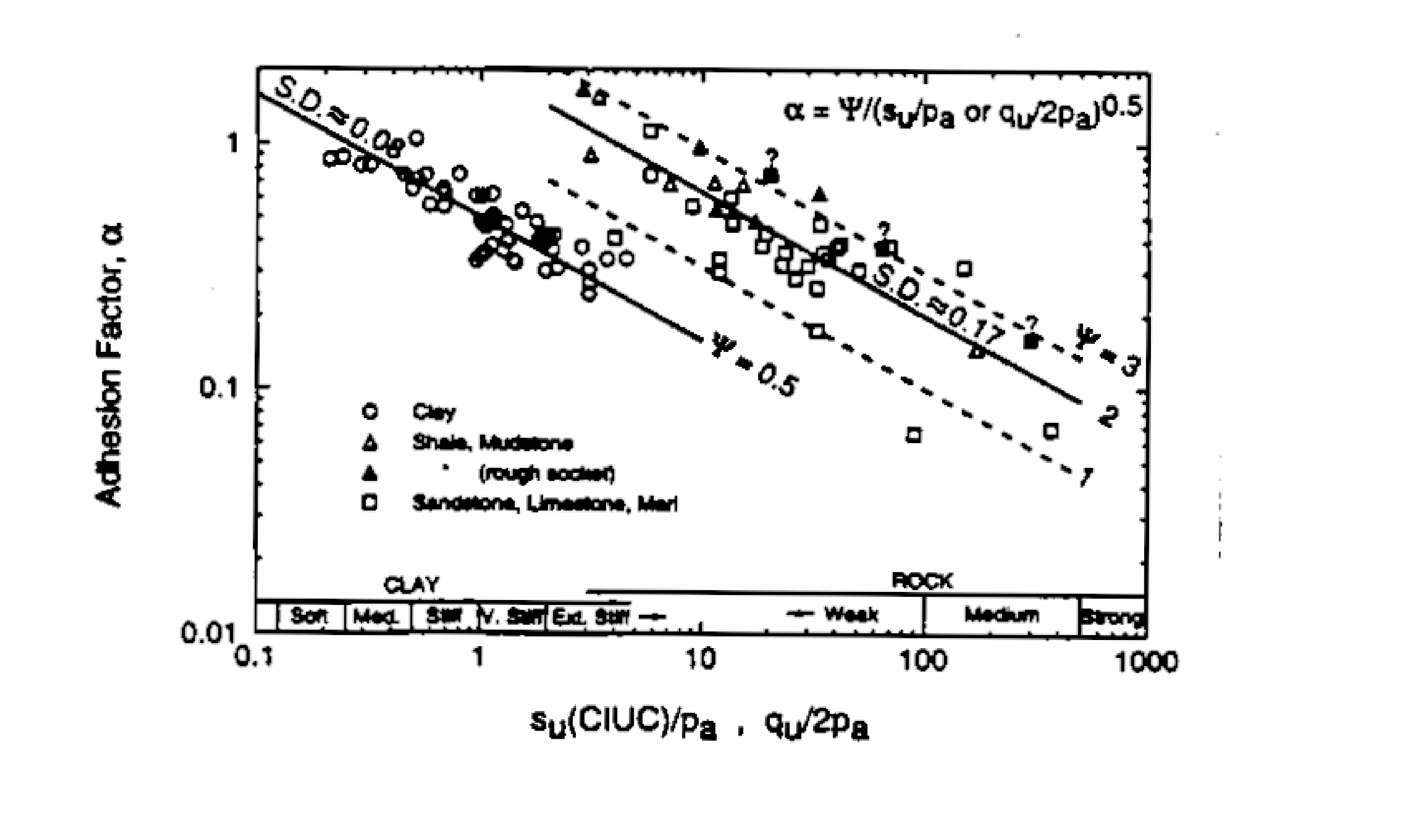


Figure 4.9 Variation of α with strength ratio

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

SHAFT CAPACITY OF PILES IN CLAY – ALPHA METHOD



Kulhawy & Phoon, 1993.

SHAFT CAPACITY OF PILES IN CLAY – BETA METHOD

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

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

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

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

SHAFT CAPACITY OF PILES 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 OF PILES IN SAND – BETA METHOD

COFFEY

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)

SHAFT CAPACITY OF PILES IN SAND – RANDOLPH METHOD

- Randolph (2003):

$$K = K_{min} + (K_{max} - K_{min})\, e^{-\mu h/d}$$

where $K_{min} = 0.2\text{-}0.4$

$K_{max} = (0.01\text{-}0.02)\, q_c/\sigma_{vo}'$

h=distance above pile tip

d= pile diameter

μ=0.05 for typical pile diameters

SHAFT CAPACITY OF PILES IN SAND – MTD METHOD

Jardine & Chow (1996):

$$f_s = [q_c/45(\sigma_{vo}'/p_a)^{0.13} + (d/h)^{0.38} + \Delta\sigma_{rd}'].\tan\delta_{cv}$$

where
- pa=atmospheric pressure
- $\Delta\sigma_{rd}'$= stress increase due to dilation (small)
- h=distance above pile tip
- d= pile diameter
- δ_{cv} = interface friction angle (critical state)

SHAFT CAPACITY OF PIPE PILES IN SAND – GAVIN & LEHANE (2003)

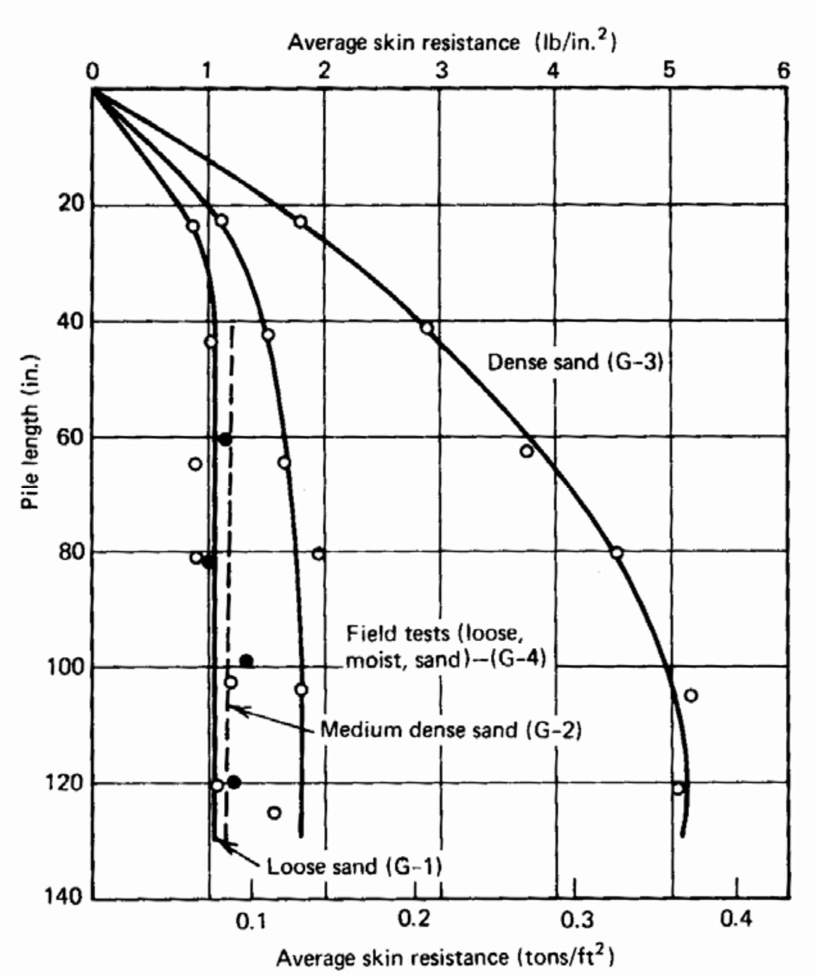
$$f_s = [0.029 q_b (h/R)^{-0.38} (\sigma_v'/p_a)^{0.12}] . \mathbf{\tan\delta_{cv}}$$

where

p_a=atmospheric pressure
h=distance above pile tip
R=pile radius
σ_v' = vertical effective stress
q_b= base bearing capacity
δ_{cv} = interface friction angle (critical state)

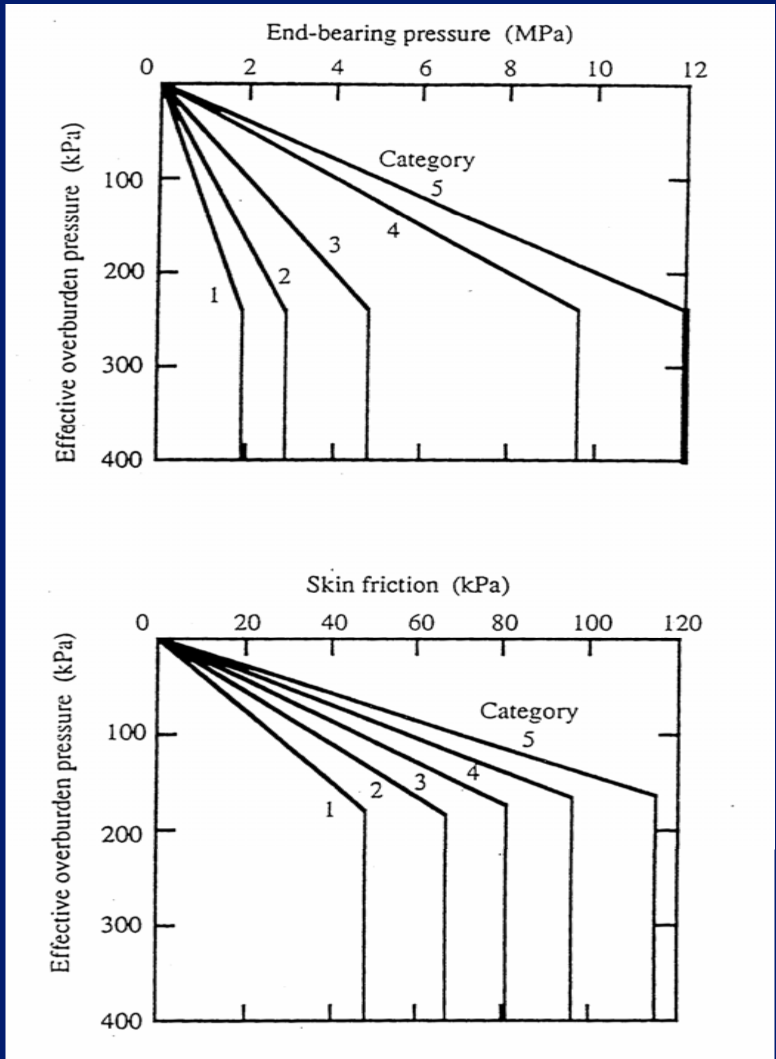
(Dilatancy during shear ignored).

SHAFT CAPACITY OF PILES IN SAND – VESIC'S 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 OF PILES 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

ISSUES RELATED TO SHAFT CAPACITY

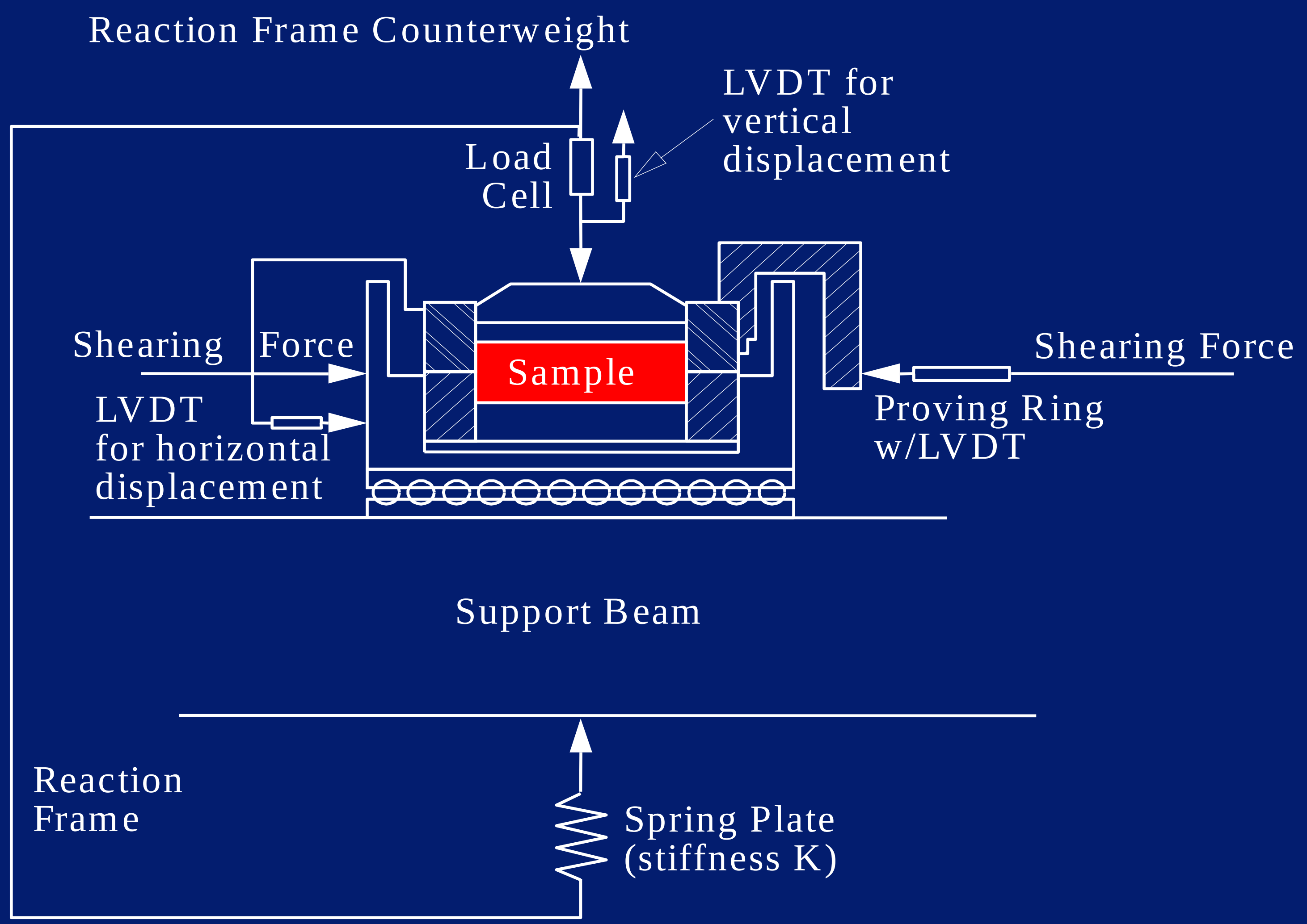
- Is there an upper limit to friction in sand?
  NO; apparent limit due to effects of :
- *residual stresses in pile,*
- *effects of decreasing friction angle with effective stress*
- *near-surface overconsolidation effects.*

BUT, we generally adopt an upper limit in design.

- Laboratory testing?

  CNS test is useful

CNS LABORATORY TESTING



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END BEARING CAPACITY

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 CAPACITY FACTOR N_q

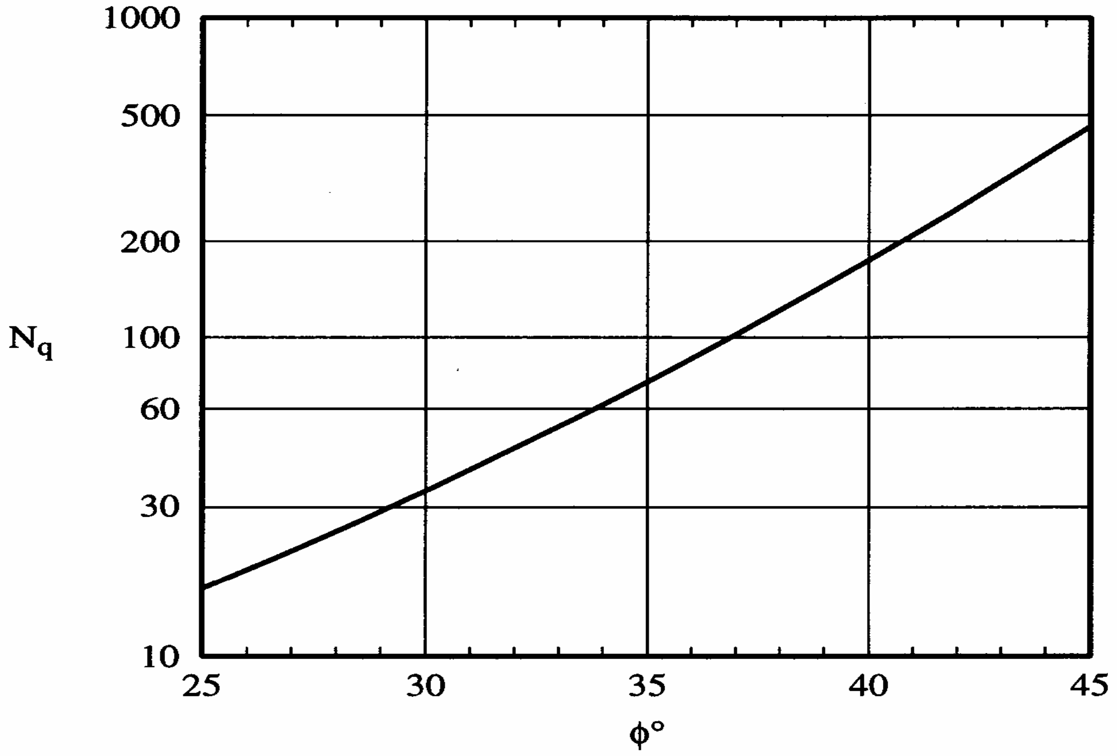


Fig.2 Variation of bearing capacity factor with friction angle
(after Berezantzev et al 1961)

END BEARING CAPACITY OF TUBE PILES (Randolph, 2003)

Available end bearing resistance of a soil plug:

$$q_{bplug}/\sigma_{v'base} = e\ 4\beta h_p/d_i$$

where $\beta = f_s/\sigma_v'$

h_p=height of soil plug

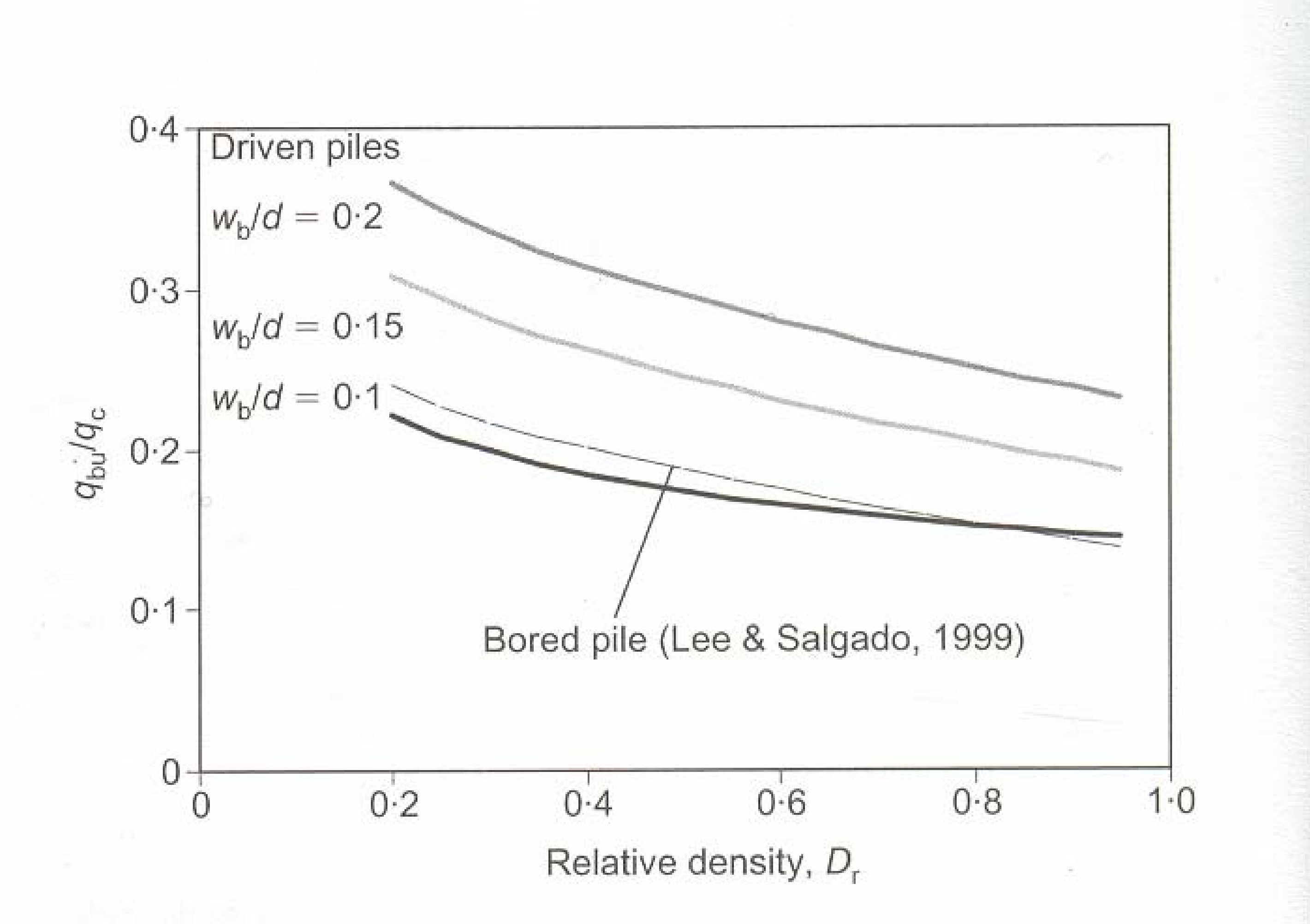
d_i = internal diameter of tube

$\sigma_{v'base}$ = ambient vertical effective stress at base of plug

END BEARING CAPACITY OF TUBE PILES (Randolph, 2003)

- Available end bearing pressure increases rapidly with plug length
- Lengths of only a few diameters can provide sufficient internal resistance to ensure plugged failure mode during static loading (regardless of diameter)

END BEARING CAPACITY OF TUBE PILES (Randolph, 2003)



END BEARING CAPACITY OF TUBE PILES (Gavin & Lehane, 2003)

$$f_b = (q_{plug} R_i^2 + q_{ann} 2Rt) / R^2$$

- $q_{plug} = q_c(1 - IFR) + IFR\, q_{plugmin}$ (IFR< 1)
- $q_{plug} = q_{plugmin}$ (IFR ≥ 1)
- $q_{ann} = q_c$
- $q_{plugmin} = 0.1 q_c$
- IFR = incr. filling ratio = 0 (fully plugged)
  = 1 (fully cored)
- R=pile outer radius, R_i=pile inner radius, t=wall thickness

END BEARING CAPACITY – CORRELATIONS WITH SPT (Decourt, 1995)

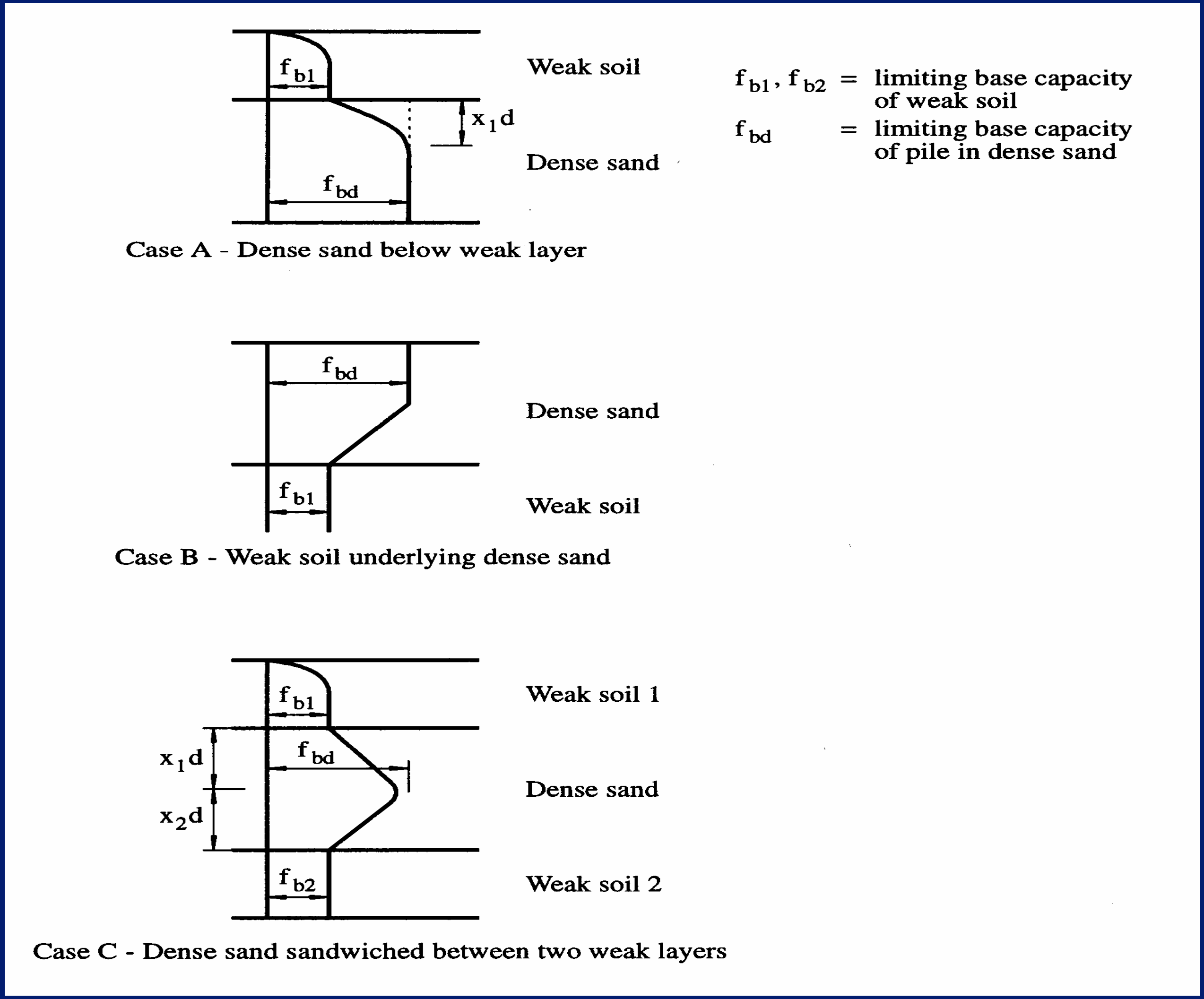
$$f_b = K.\ N_p \leq f_{blim}$$

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 CAPACITY OF LAYERED SOIL PROFILES



ISSUES RELATED TO END BEARING

- 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

USE OF STATIC CONE PENETRATION TEST (CPT) DATA

Three approaches:

- Use of measured sleeve resistance for f_s
  (Nottingham & Schmertmann, 1995)
- Use of measured cone resistance q_c for f_s (&f_b)
  (Bustamante & Gianeselli, 1982)
- Use of q_c to obtain soil properties for conventional analysis

USE OF STATIC CONE PENETRATION TEST (CPT) DATA

SHAFT FRICTION IN CLAYS

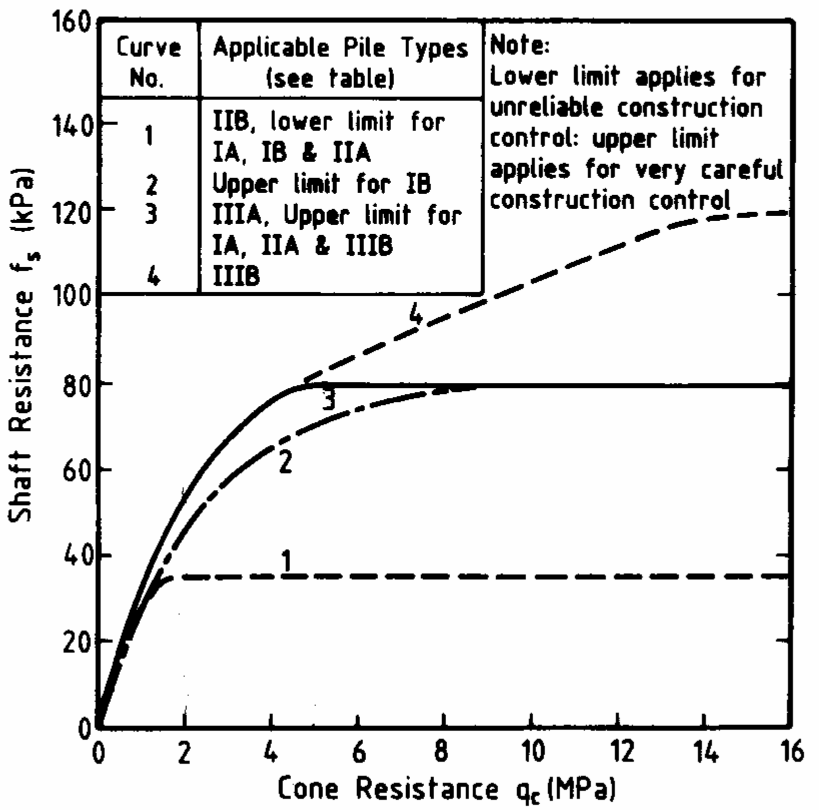


Fig. 25. Design values of shaft resistance for piles in clay (based on Bustamante & Gianeselli, 1982)

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

USE OF STATIC CONE PENETRATION TEST (CPT) DATA

SHAFT FRICTION IN SANDS

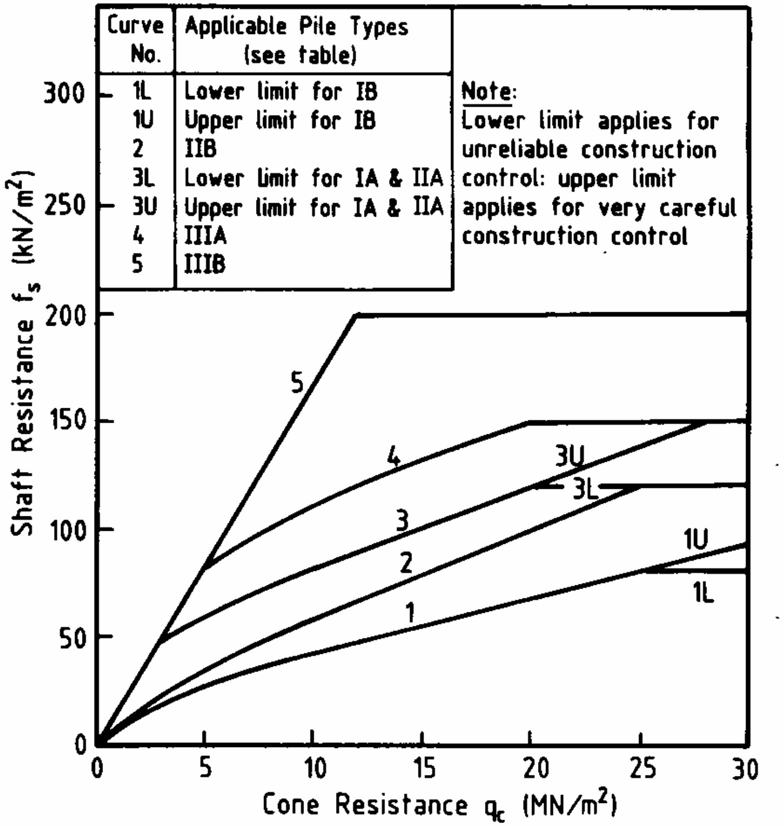
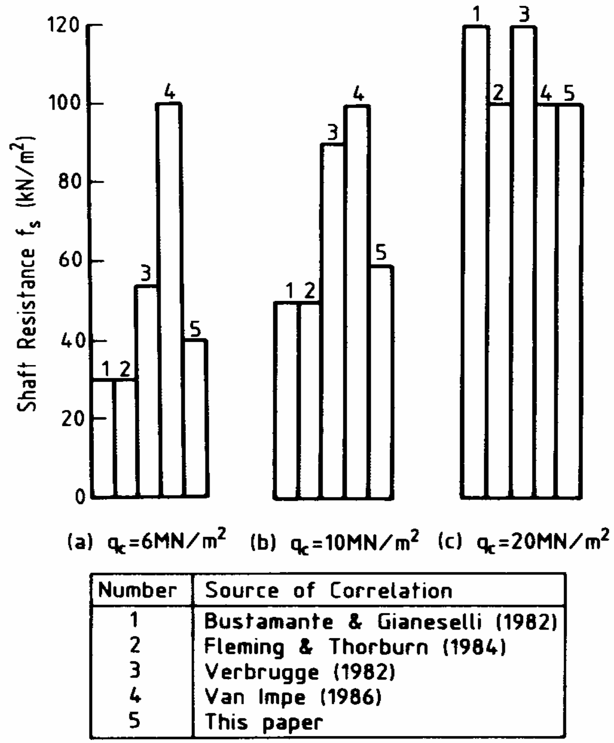
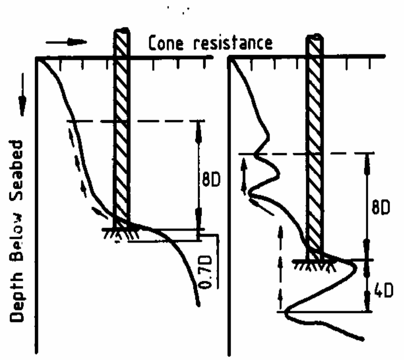


Fig. 26. Design values of shaft resistance for piles in sand (based on Bustamante & Gianeselli, 1982)



Beware of the potential variability of different methods for sands!

USE OF STATIC CONE PENETRATION TEST (CPT) DATA
END BEARING



$$q_p = \frac{(A+B)/2+C}{2}$$

Key:

D. : Diameter of the pile.

A. : Average cone resistance below the tip of the pile over a depth which may vary between 0.7D and 4D

B. : Minimum cone resistance recorded below the pile tip over the same depth of 0.7D to 4D

C. : Average of the envelope of minumim cone resistances recorded above the pile tip over a height which may vary between 6D and 8D. In determining this envelope, values above the minimum value selected under B are to be disregarded

q_p : Ultimate unit point resistance of the pile

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 q_c below the tip (typically 0.2 – 0.5 times the average)
- 0.2 recommended by Randolph (2003)

PILES TO ROCK

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

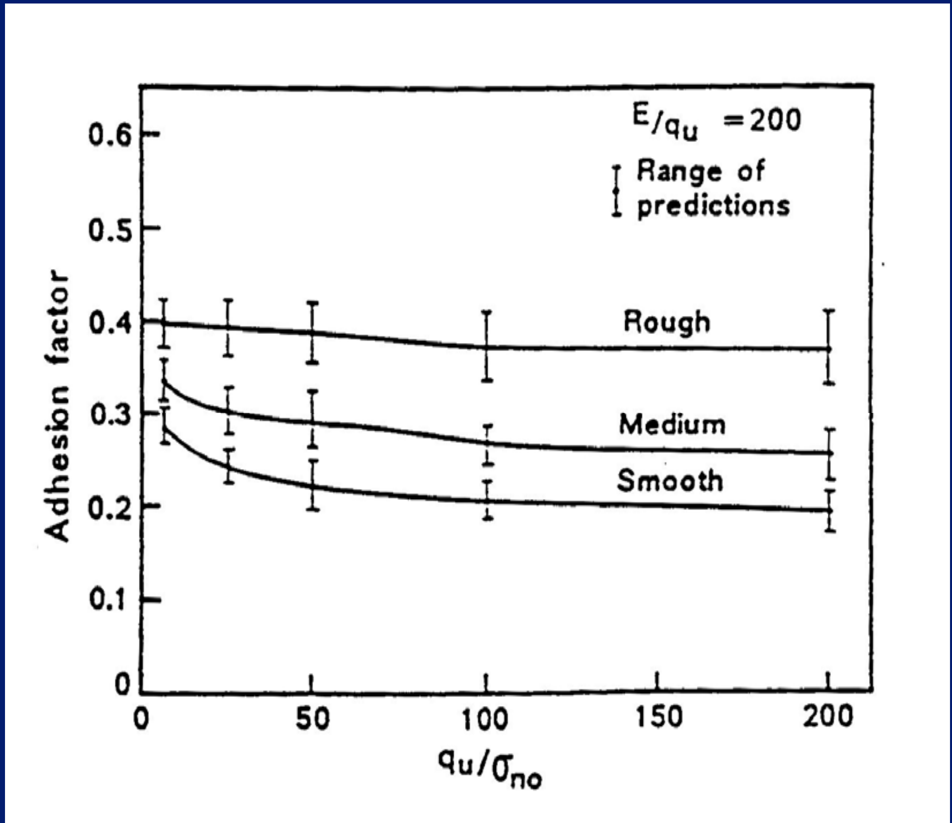
$$\mathbf{f_s = a.\ (q_u)^{b} \quad MPa}$$

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

PILES TO ROCK - Shaft Friction Parameters

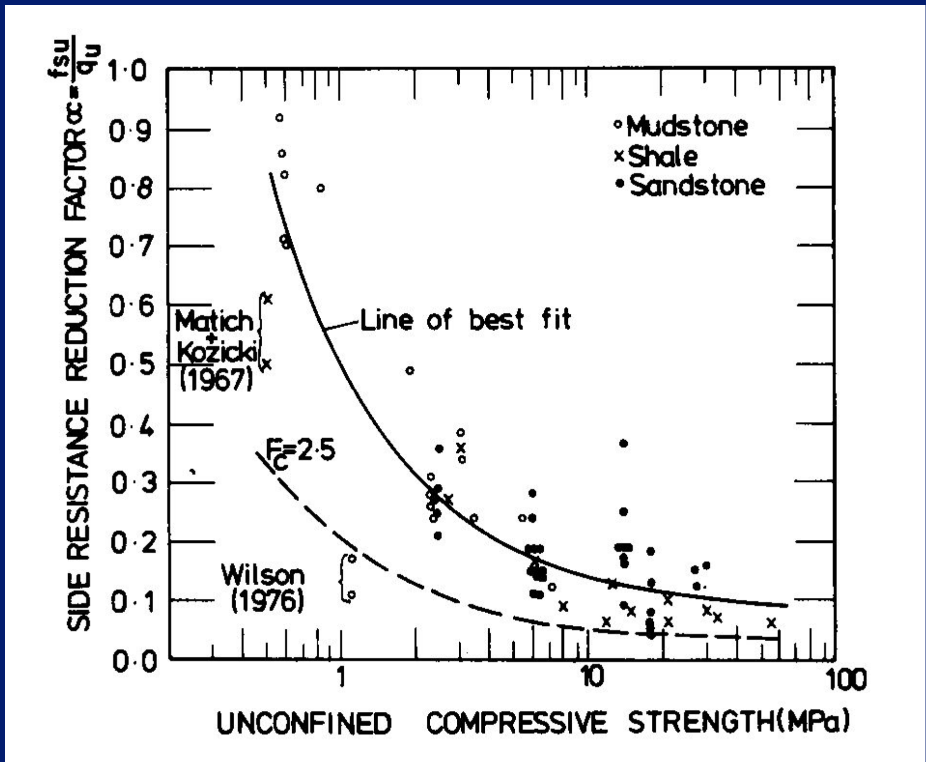
Method	*a*	*b*
Rosenberg & Journeaux (1976)	0.375	0.515
Horvath (1976)	0.33	0.5
Horvath & Kenney (1979)	0.20-0.25	0.5
Meigh & Wolski (1979)	0.22	0.6
Williams & Pells (1981)	$\alpha.\beta$	1.0
Rowe & Armitage (1987)	0.45	0.5
Zhang & Einstein (1998)	0.4 (smooth) 0.8 (rough)	0.5

Shaft Friction – Importance of Surface Roughness



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

PILES TO ROCK -
Reduction Factors a & b (Williams & Pells, 1981)



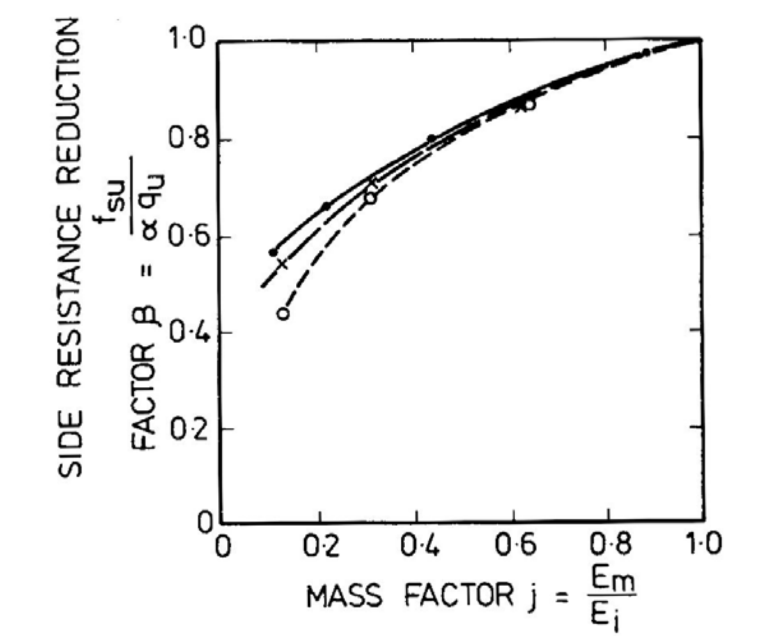


FIG. 7. Mass modulus factor β for Melbourne mudstone. NOTES: ●——● MW mudstone, normal roughness; ×— —× HW mudstone roughened; ○- - - ○ HW mudstone, normal roughness.

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 RESISTANCE OF SINGLE PILES – CLAY SOILS

- 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 RESISTANCE OF SINGLE PILES – SANDY SOILS

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

$x = \nu_p \tan \delta\ (L/d)\ (G_{av}/E_p)$

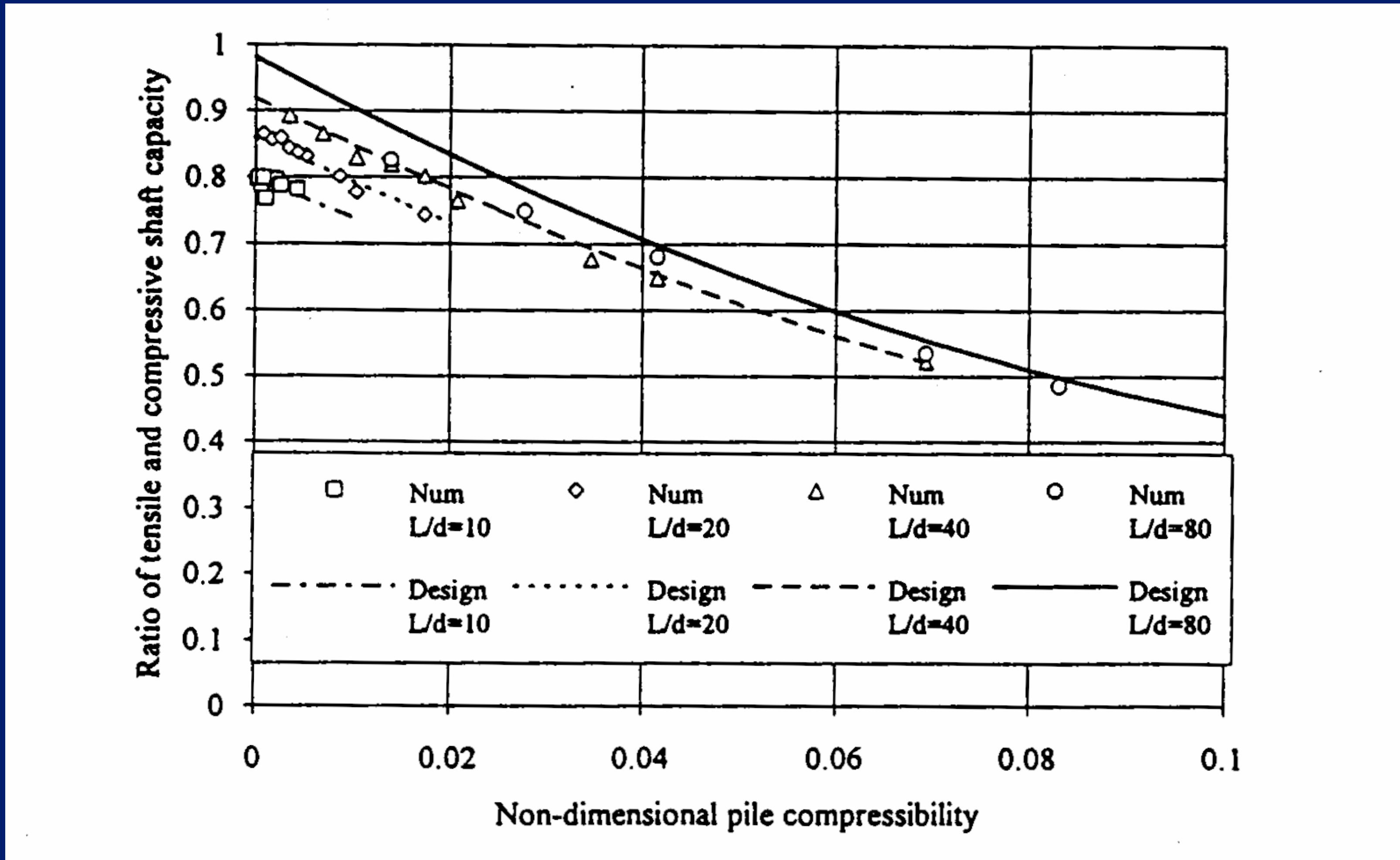
ν_p = pile Poisson's ratio

G_{av} = average soil shear modulus along pile shaft

E_p = pile Young's modulus

δ = pile-soil interface friction angle

UPLIFT RESISTANCE OF SINGLE PILES – SANDY SOILS



EFFECTS OF 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

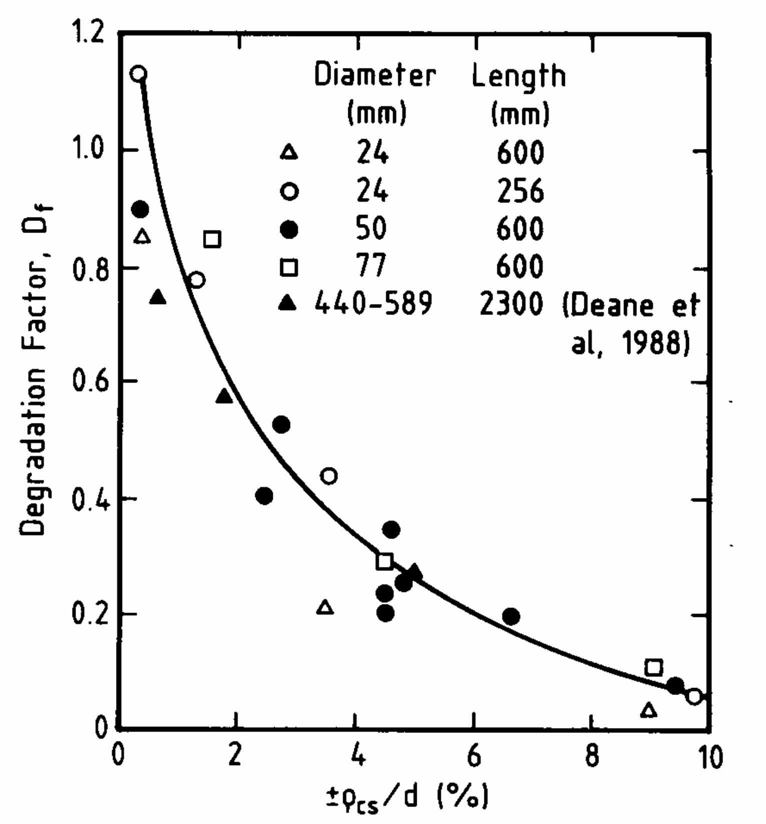


Fig. 21. Effect of normalized cyclic slip displacement on D_f with different pile diametes (after Lee, 1988)

Grouted pile in calcareous sand

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

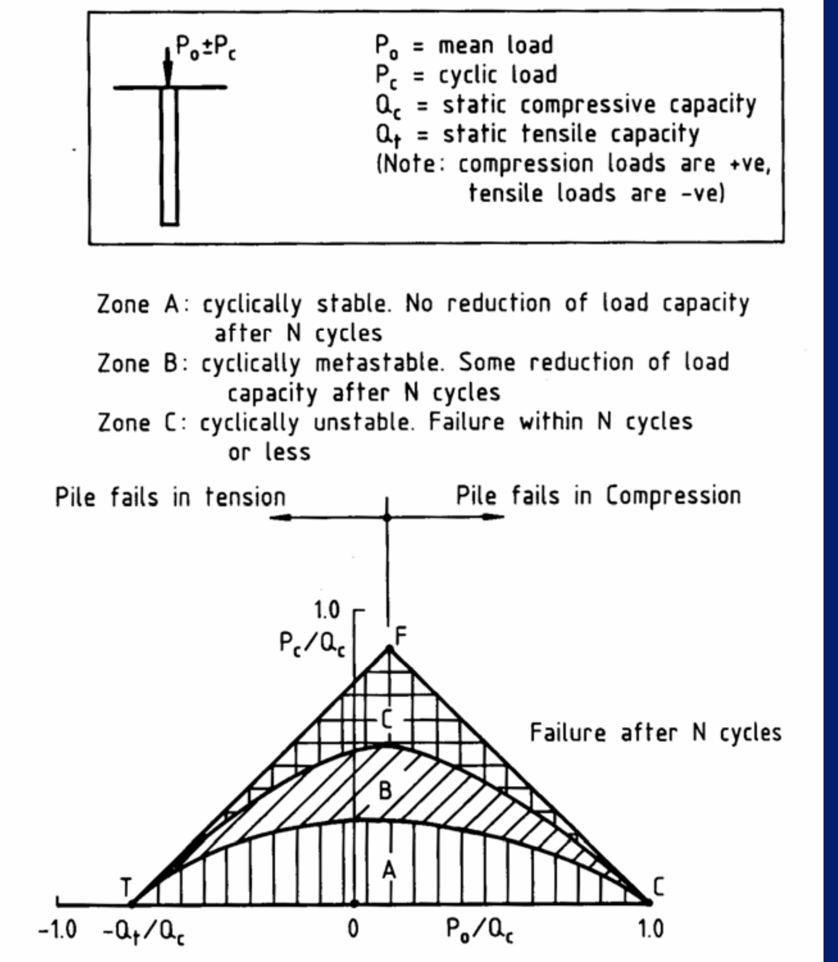


Figure 7.15 Main features of the cyclic stability diagram.

CYCLIC STABILITY DIAGRAMS

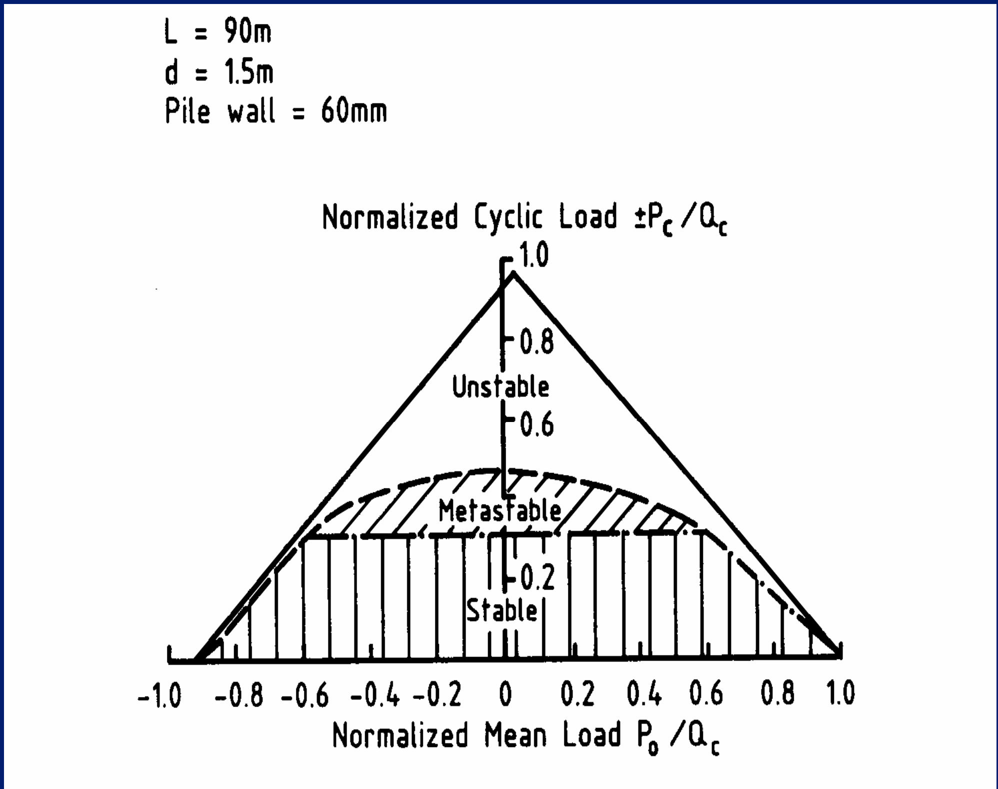


Figure 7.16 A cyclic stability diagram for a driven pile in clay; N = 100 cycles.

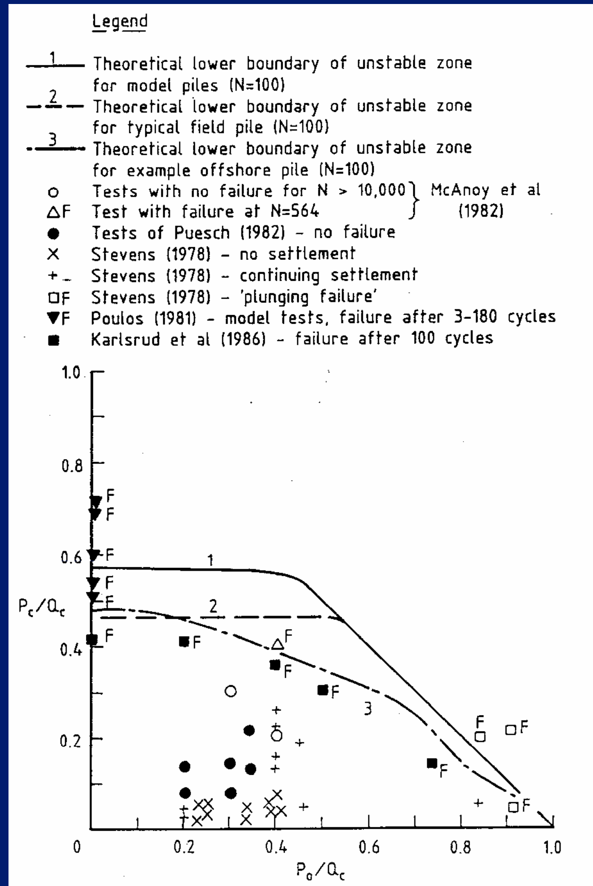


FIG.21 COMPARISON BETWEEN FIELD TEST DATA AND THEORY

Example (theoretical)

Theory vs Measurements

PILE GROUP EFFECTS ON AXIAL CAPACITY

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, $\eta \sim 0.67$
- 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:

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

OTHER CASES FOR GROUPS

GROUP WITH CAP ON SURFACE

Group capacity (P_u) is lesser of:

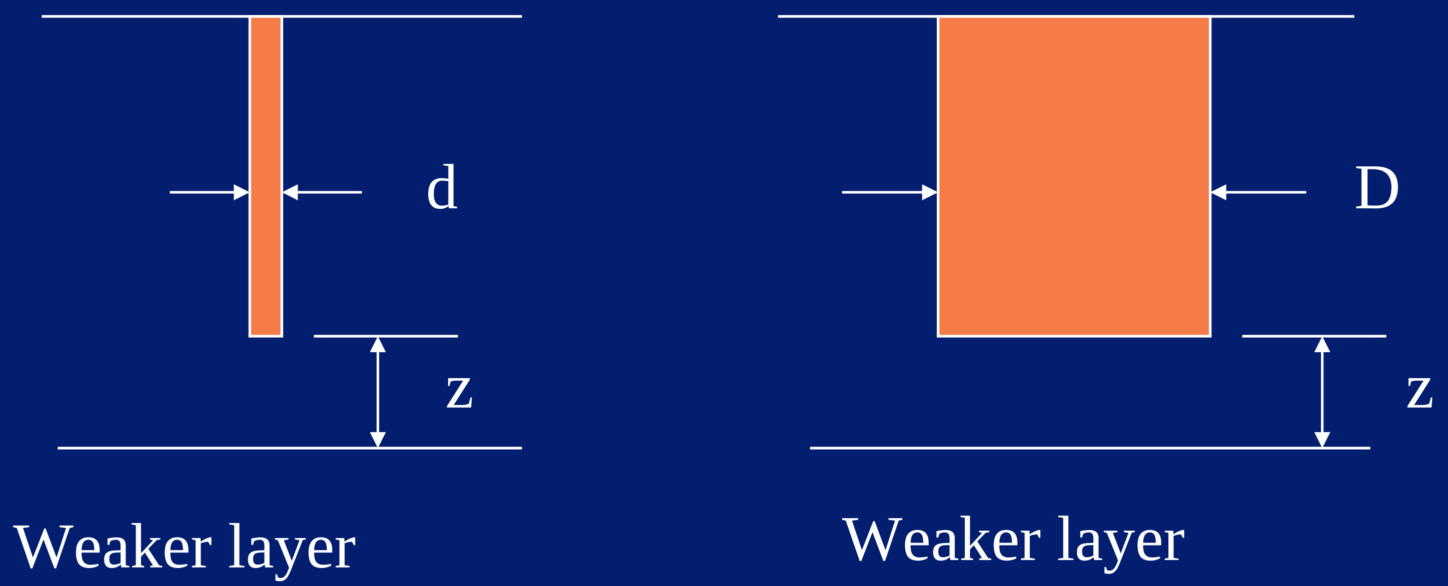
- Sum of individual pile capacities + net area of cap
- Capacity of “block” containing piles & soil, + capacity of portion of cap outside block perimeter.

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 WEAKER UNDERLYING LAYER



SINGLE PILE – effect may be small

GROUP – effect may be large!

STRUCTURAL DESIGN ISSUES

- 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.

PILE 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)

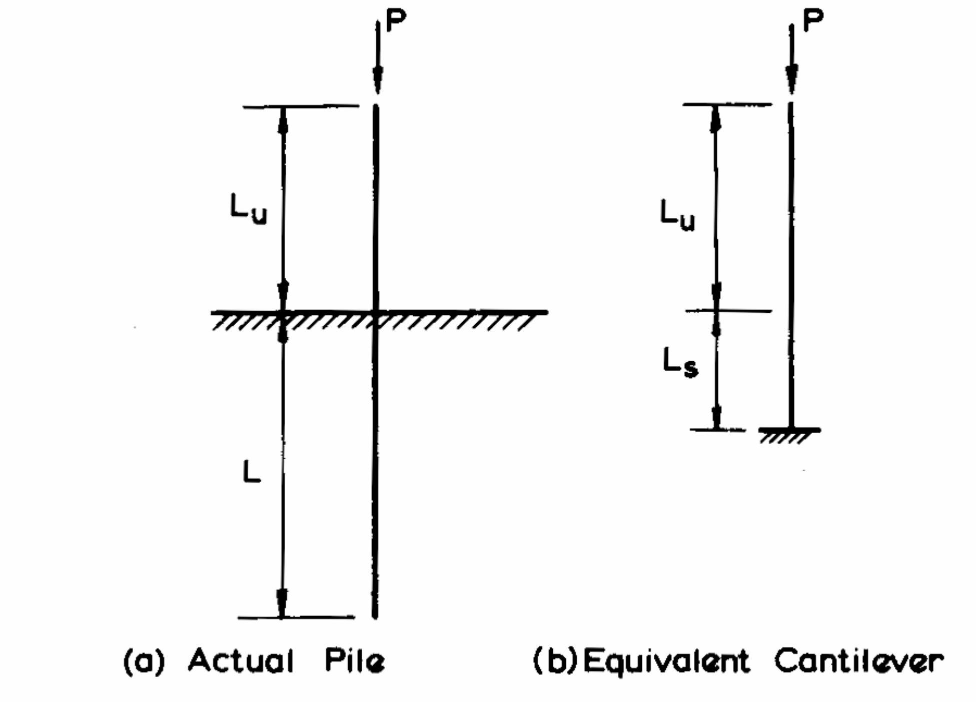


FIGURE 14.5 Partially embedded piles.

PILE BUCKLING

Constant k_h

$k_h = n_h z/d$

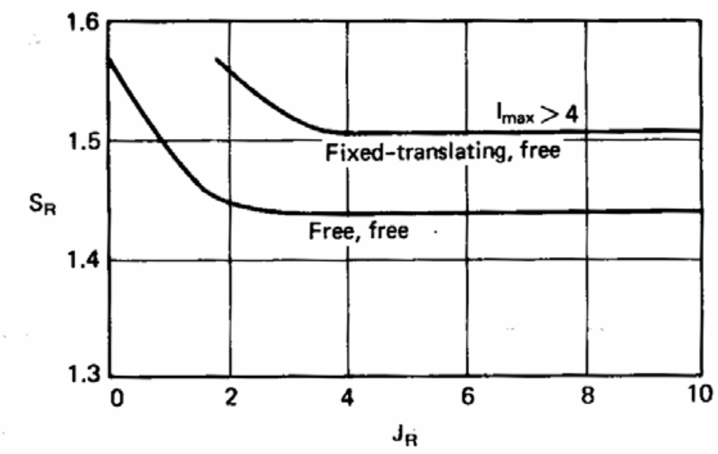


FIGURE 14.6 Dimensionless depth of fixity for buckling. Constant k_h (after Davisson and Robinson, 1965). (© Canada, 1965, by University of Toronto Press.)

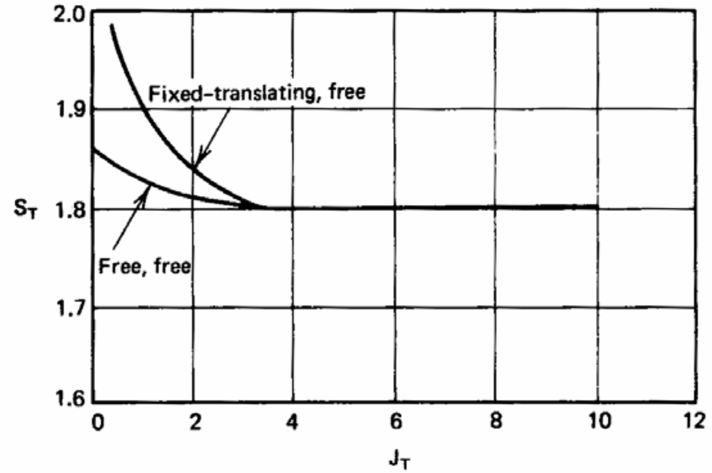


FIGURE 14.7 Dimensionless depth of fixity for buckling. Linearly varying k_h (after Davisson and Robinson, 1965). (© Canada, 1965, by University of Toronto Press.)

$S_R = L_s/R$

$J_R = L_u/R$

$R = (E_p I_p / k_h d)^{0.25}$

$S_T = L_s/T$

$J_T = L_u/T$

$T = (E_p I_p / n_h)^{0.20}$

DURABILITY ISSUES
TYPICAL CORROSION RATES FOR STEEL PILES

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

DURABILITY ISSUES
COUNTER MEASURES FOR STEEL PILES

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