LECTURE 3

LATERAL LOADING OF PILES

OUTLINE

- Ultimate lateral capacity
- Lateral deflection
  - Methods of analysis
  - Solutions based on elastic continuum
  - Non-linear analyses
  - Assessment of parameters
- Interaction & group effects

ULTIMATE LATERAL CAPACITY OF PILES

SIGNIFICANCE

- Usually lateral deflections govern pile design for lateral loadings
- BUT, ultimate lateral resistance may be important for:
  - Short piers
  - Long slender piles
  - Non-linear analysis of deflections

FAILURE MODES & HEAD CONDITIONS

SHORT PILE MODE:

- Failure of the supporting soil

LONG PILE MODE:

- Structural failure or yielding of the pile itself.

Both modes need to be analyzed, and the more critical mode established.

HEAD CONDITIONS

1. Free or unrestrained head – no head restraint
2. Fixed head or restrained head – No rotation of head.

GENERAL PRINCIPLES OF ANALYSIS

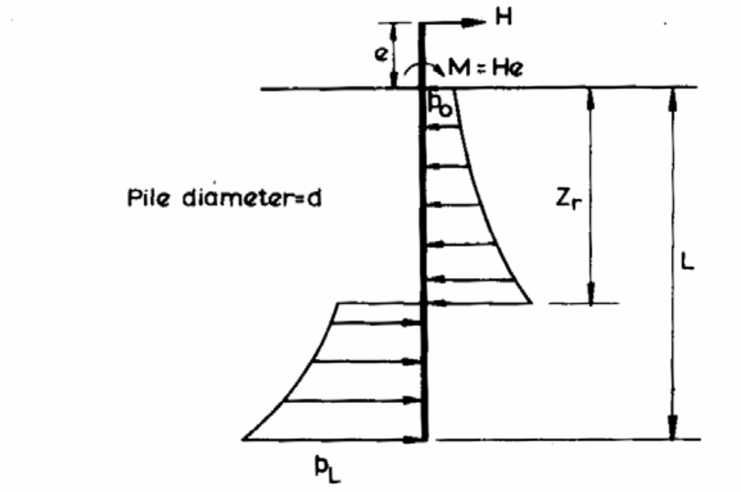
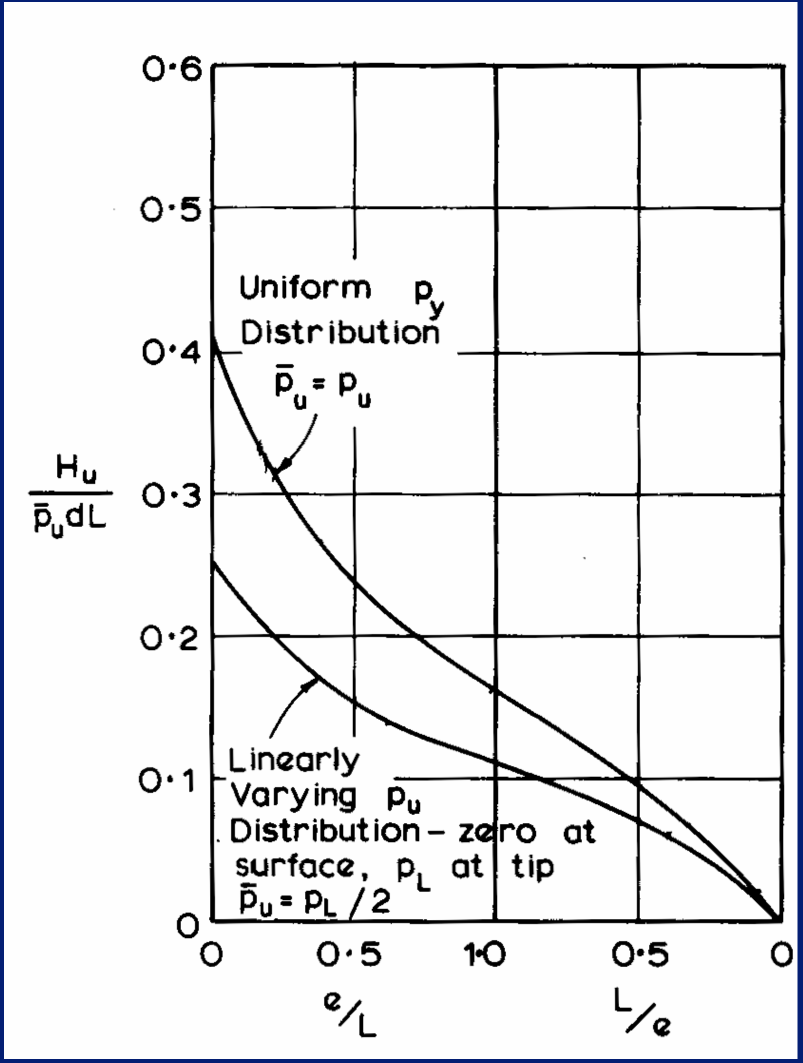


FIGURE 7.1 Unrestrained laterally-loaded pile.

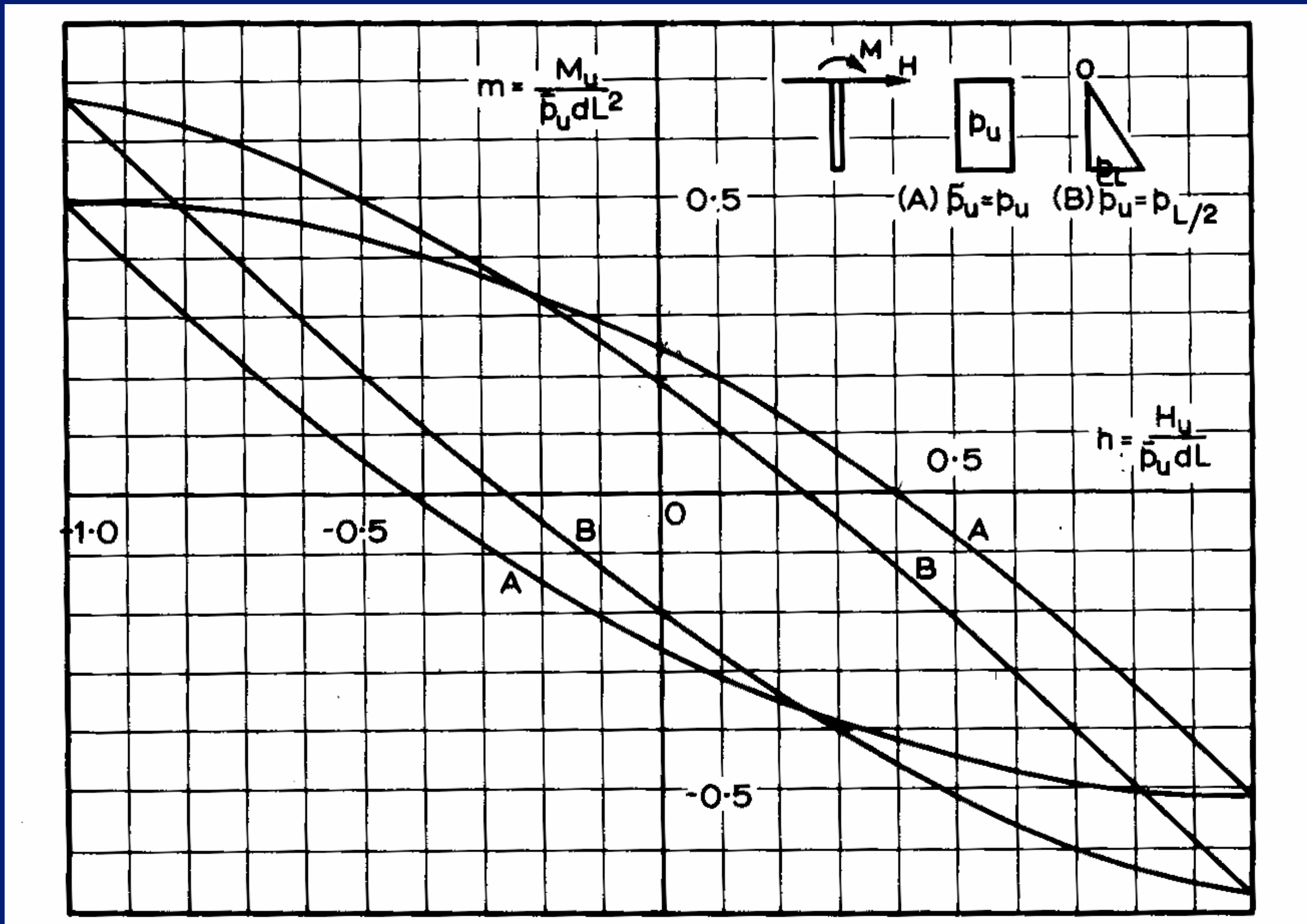
- Consider:
  - Horizontal load equilibrium
  - Moment equilibrium
- Need to specify:
  - Mode of failure
  - Distribution of ultimate lateral pile-soil pressure

SIMPLE CASES – FREE HEAD



- Constant p_y with depth (e.g. O/C clays)
- Linearly increasing p_y with depth (e.g. N/C clays, sands)

ALTERNATIVE PRESENTATION OF SIMPLE SOLUTIONS



H-M "yield" Surface

Rigid free-head pile

ULTIMATE LATERAL PILE-SOIL PRESSURE - CLAYS

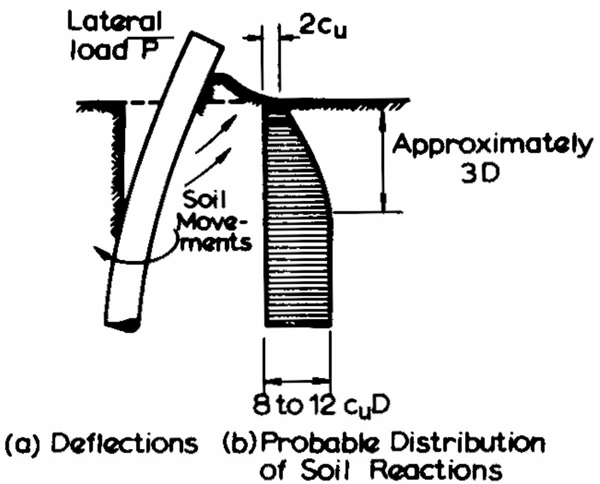


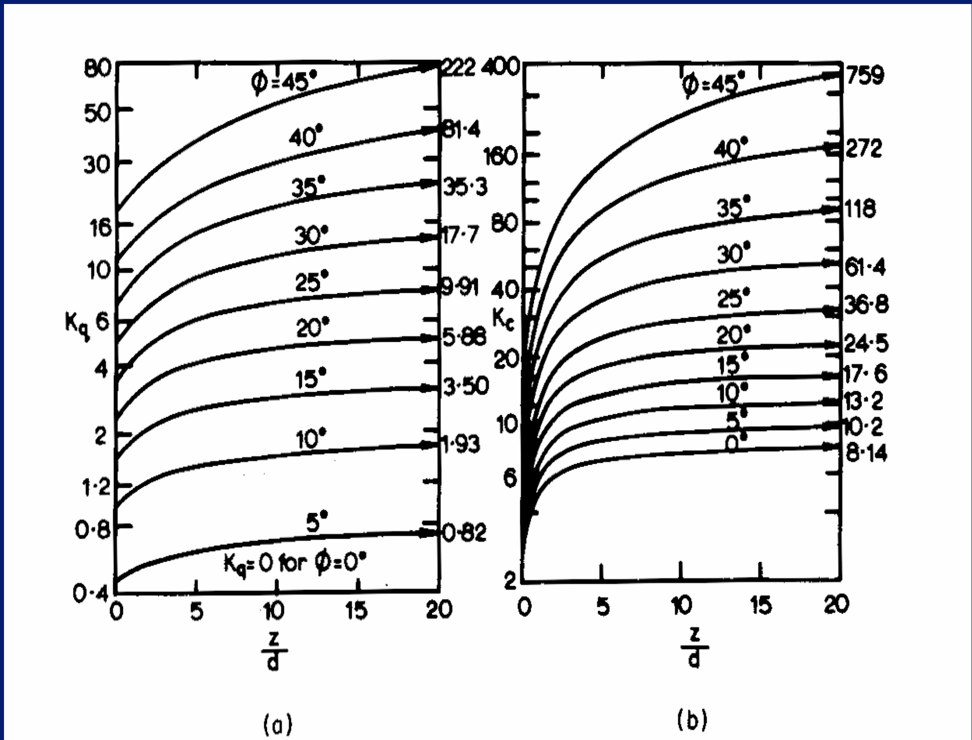
FIGURE 7.4 Distribution of lateral resistance.

Usually adopt

$$p_y = N_c . c$$

N_c varies from 2 at the surface to 8 – 12 at z/d greater than 3 (typically, 9 is used)

ULTIMATE LATERAL PILE-SOIL PRESSURE – SANDS & c-ϕ SOILS



Usually adopt

$p_y = F. p_p$

where F = 3 – 5

Alternatively, use theory of Brinch Hansen

$p_y = q.N_q + c. N_c$

BROMS' THEORY

- Probably the most widely-used method.
- Relies on statics.
- Considers short & long-pile failure modes.

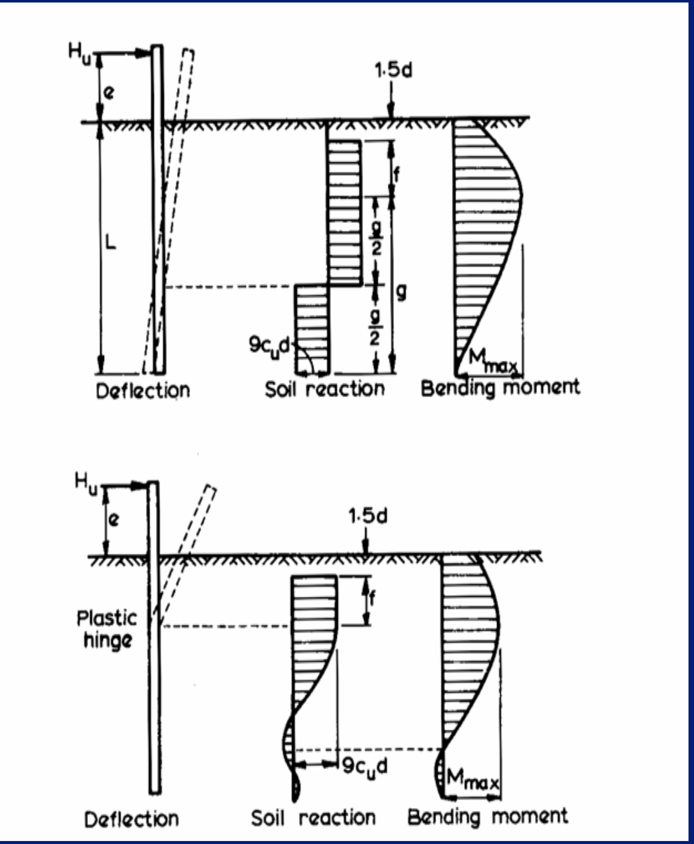
Analyses Available:

- **Cohesive soils**
  - Short & long pile failure – unrestrained head
  - Short & long-pile failure – restrained head
- **Cohesionless Soils**
  - Short & long pile failure – unrestrained head
  - Short & long-pile failure – restrained head

BROMS' THEORY FOR PILES IN CLAY

Short Pile

Long Pile



Failure modes for free-head pile

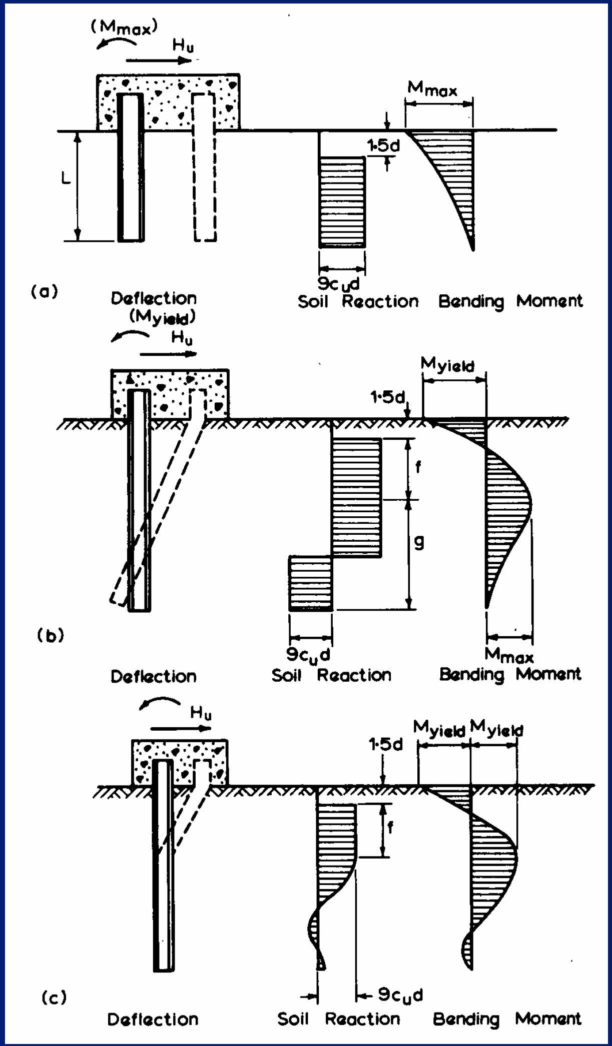
- Assume "dead" zone to 1.5d below surface
- Assume p_y = 9c below this depth
- Employ statics to obtain failure load for short-pile and long-pile cases

BROMS' THEORY FOR PILES IN CLAY

Short Pile

Intermediate Pile

Long Pile



Failure modes for fixed-head pile

BROMS' THEORY FOR PILES IN CLAY – ULTIMATE LATERAL LOAD

Need:

For soil -

c_u

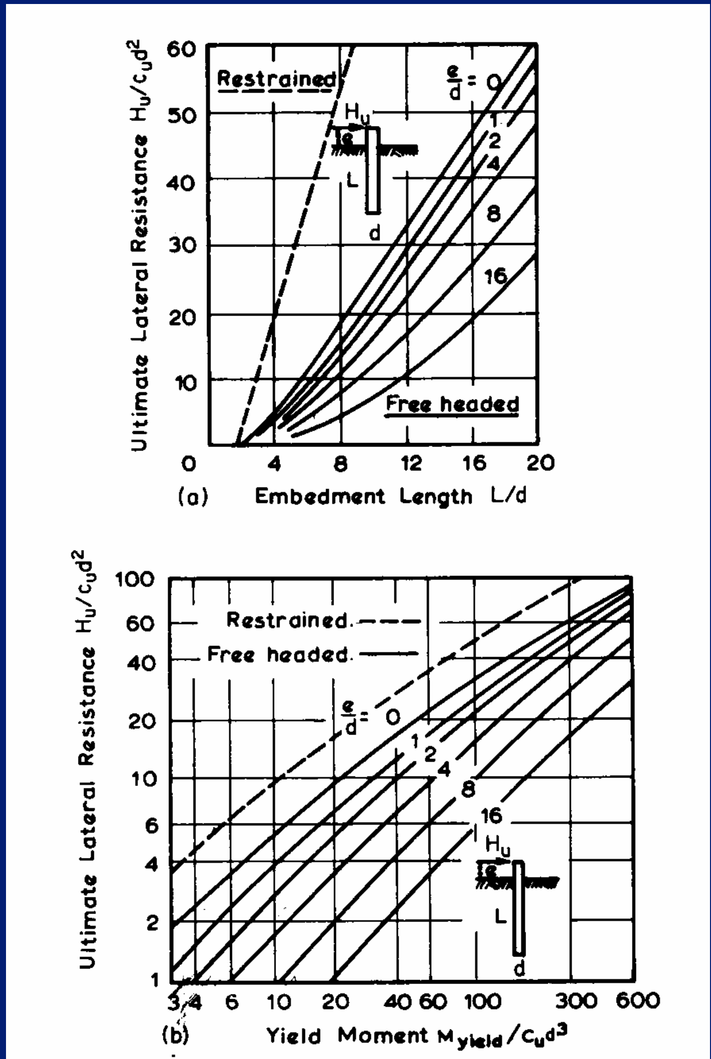
For pile -

M_{yield}

L

D

e



SHORT PILE FAILURE MODE

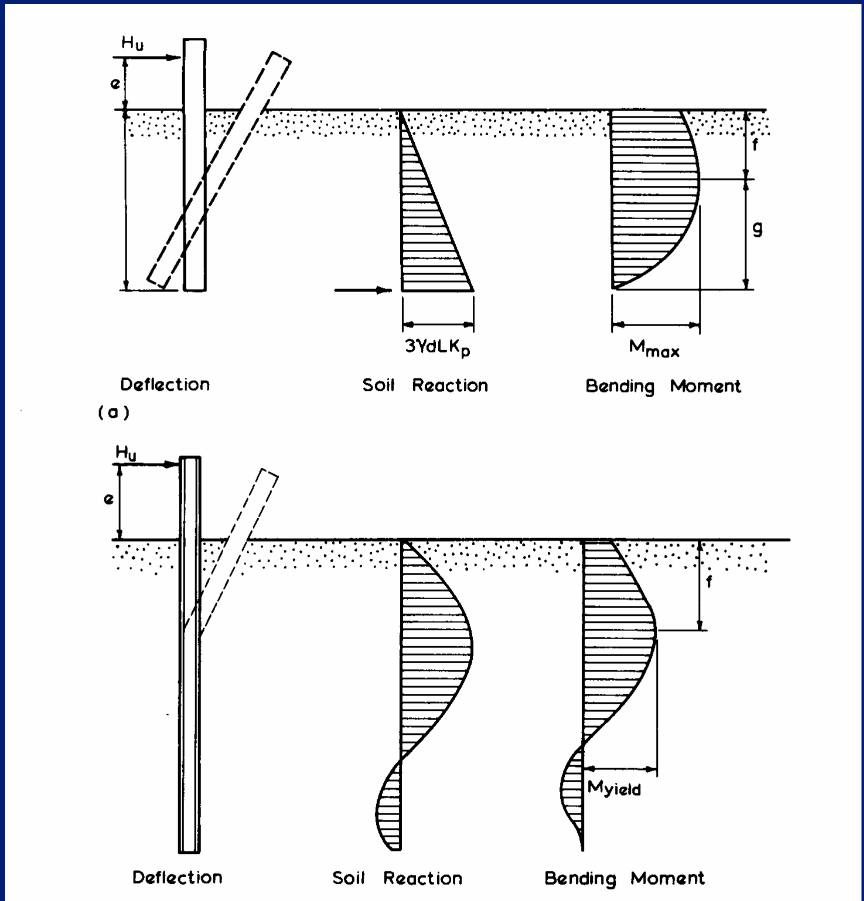
LONG-PILE FAILURE MODE

BROMS' THEORY FOR PILES IN COHESIONLESS SOILS

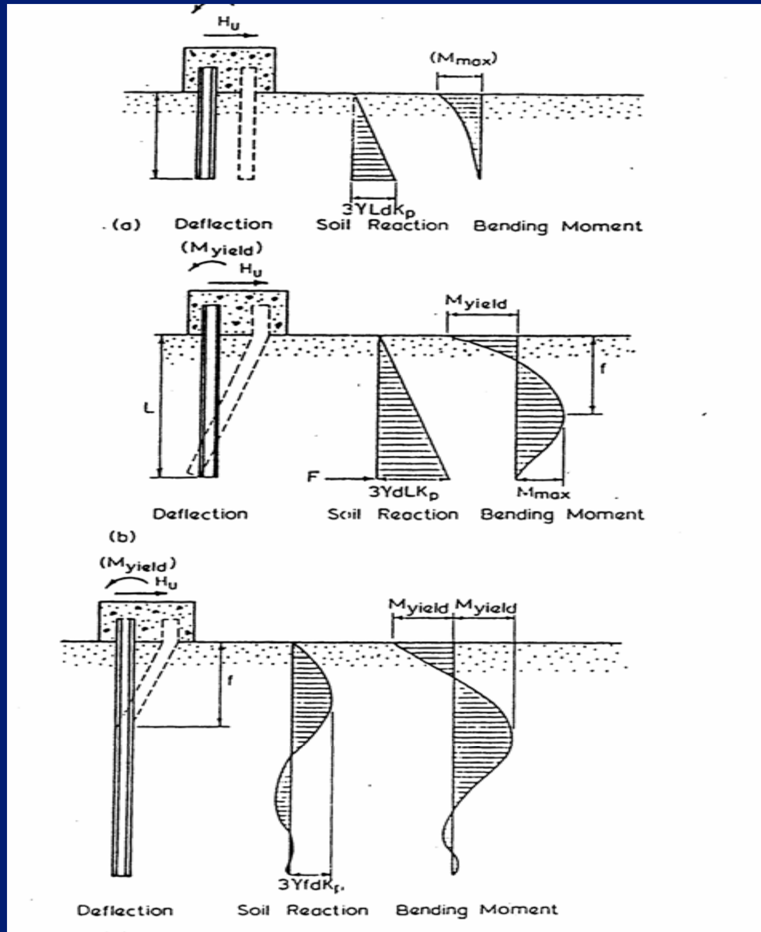
ASSUMPTIONS:

- Ignore active earth pressure behind wall
- Assume shape of pile section does not affect py
- Soil reaction below point of rotation is represented by a point load (to simplify algebra)
- $p_y = 3p_p$

BROMS' THEORY FOR PILES IN COHESIONLESS SOILS – FAILURE MODES



Free Head Pile



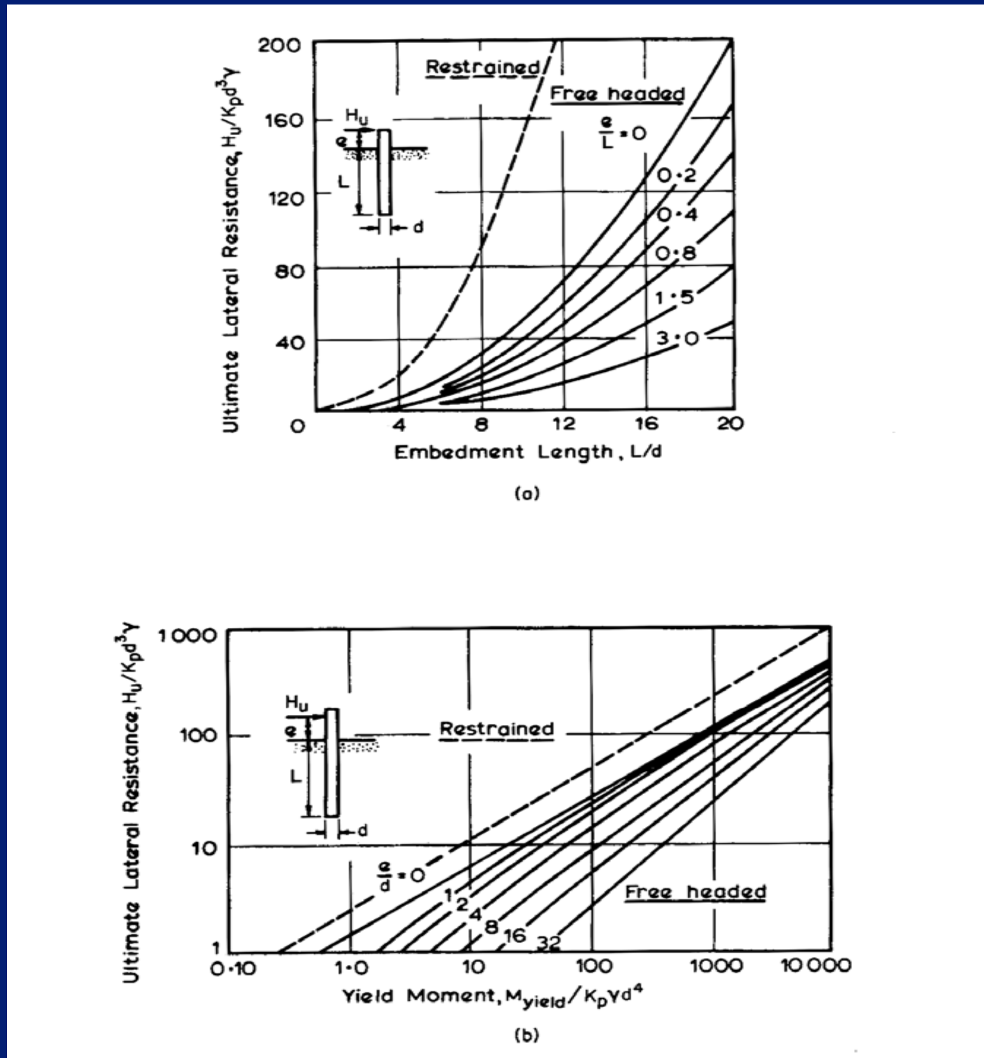
Fixed Head Pile

Coffey Geosciences

BROMS' THEORY FOR PILES IN COHESIONLESS SOILS – FAILURE LOAD

Short Pile Mode

Long Pile Mode



Need:

For soil -

K_p

γ

For pile -

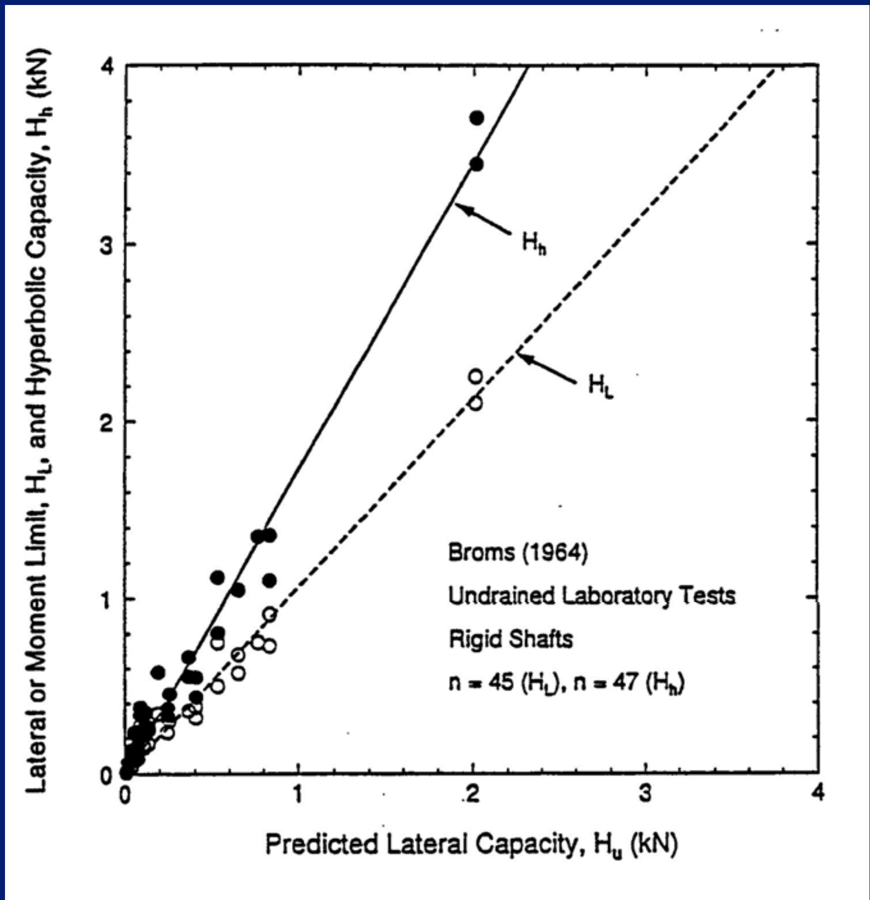
M_{yield}

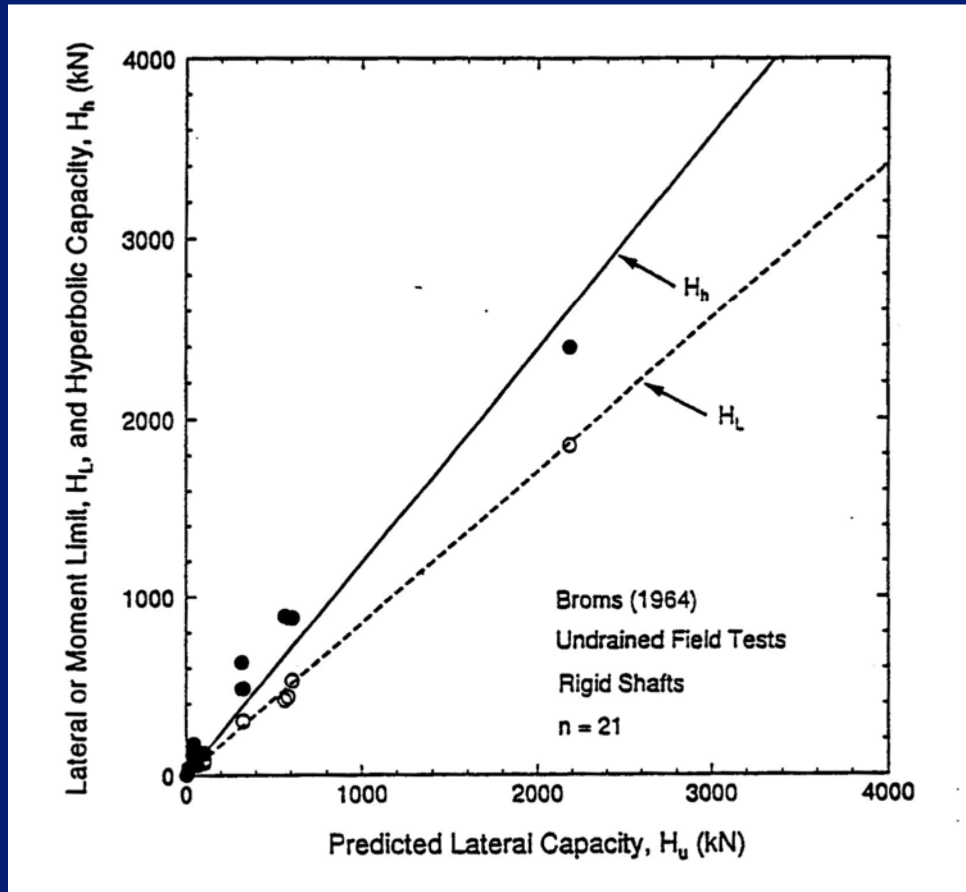
L

D

e

BROMS' THEORY – EVALUATION (Kulhawy & Chen, 1995)





BROMS' THEORY – EVALUATION (Kulhawy & Chen, 1995)

Data Type	*Calc./Msd Limit Load H_L*	*Calc. / Msd. Ult. Load H_h*
Undrained lab + field data (68 tests)	0.78	0.50
Drained lab + field data (65 tests)	1.38	0.89

H_L = load for initial failure or yield

H_h = hyperbolic extrapolation for ultimate load

In general, Broms' method appears to be moderately conservative

PILE IN LAYERED COHESIVE SOIL

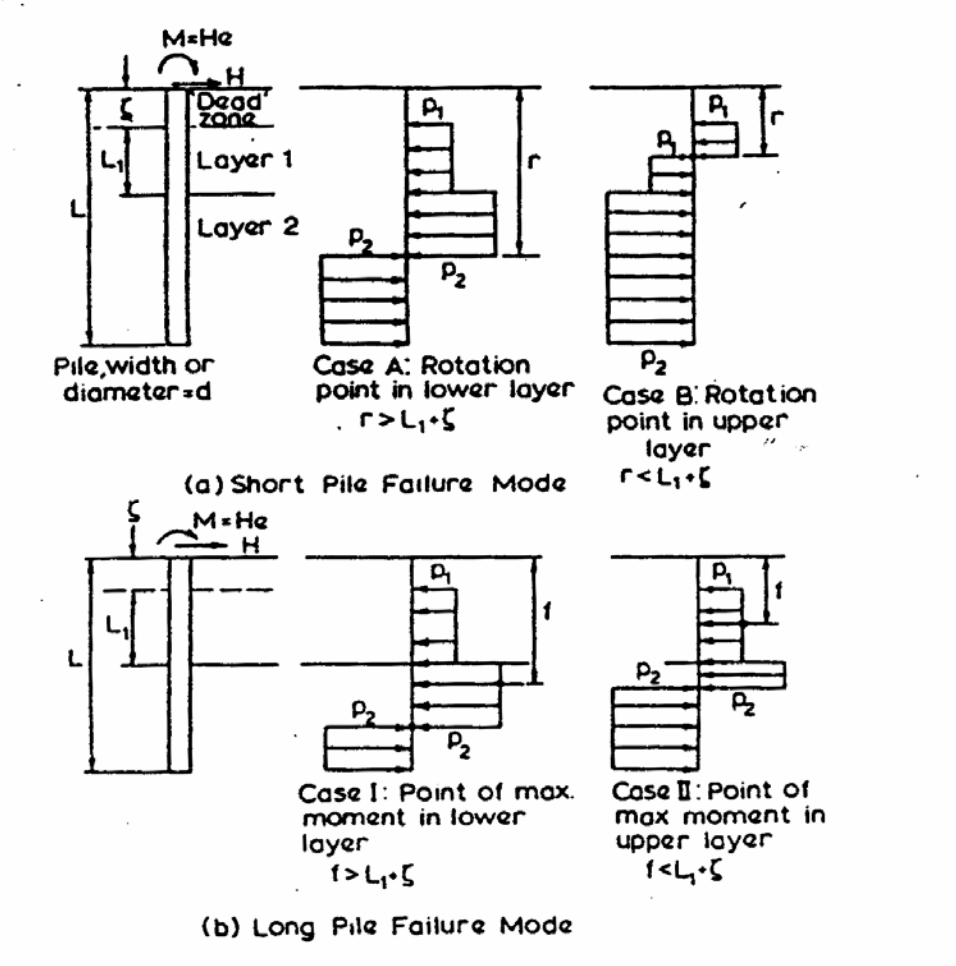


Fig. 1. Failure of Free Head Pile.

Again, consider short- and long-pile failure modes. Obtain failure load via statics.

PILE IN LAYERED COHESIVE SOIL

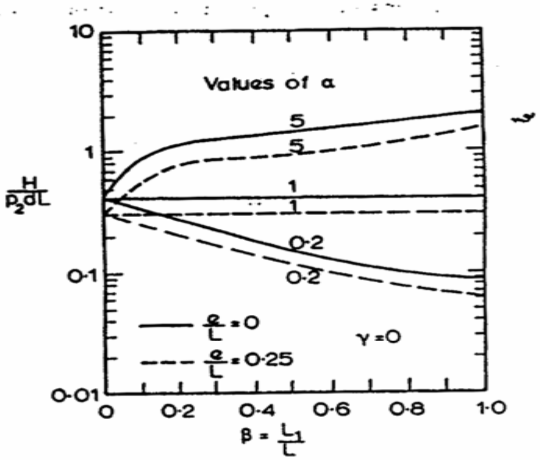


Fig. 5. Short-Pile Failure : Free-Head Pile.

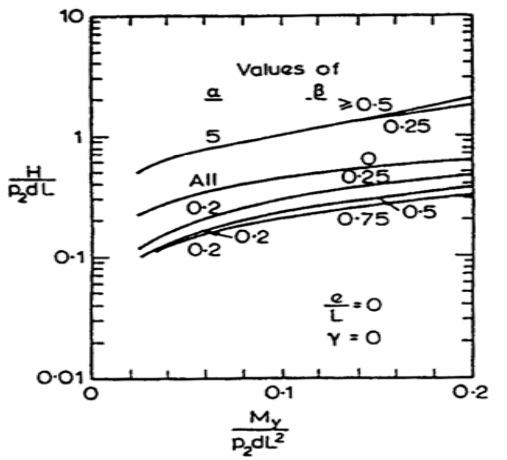


Fig. 6. Long-Pile Failure : Free-Head Pile.

Quadratic equation for ultimate lateral load capacity:

$$aH^{*2} + bH^{*} + C = 0$$

where $H^* = H / (p_2dL)$

$M^* = M_y / (p_2dL^2)$

$\alpha = p_1 / p_2$

$\beta = L_1 / L$

$\gamma = \zeta / L$

Coefficients a, b, c given by Poulos (1985)

PILE GROUPS

- Take group capacity as lesser of:
  - Sum of individual pile capacities;
  - Capacity of block containing piles + soil.
- Need to consider both short-pile and long-pile modes for single piles.
- Consider only short-pile mode for block.
- DO NOT USE Broms' solutions for blocks in clay! "Dead zone" of 1.5B is not realistic.
- Suggest use of solutions for uniform p_y, with a smaller "dead" zone (via an eccentricity of load), or solutions of Fleming et al (1992)..

LATERAL DEFLECTION OF PILES

METHODS OF ANALYSIS

1. SUBGRADE REACTION METHOD

Soil modelled as series of indepenedent springs (linear or nonlinear).

A point in the soil only deflects if it has stress acting on it. Therefore,this model does not consider stress transmission.

$$\rho(i, j) = 0 \qquad \text{for } i \neq j$$

$$\rho(i, j) = p(j) / k(j) \qquad \text{for } i = j$$

2. ELASTIC - BASED METHOD

Soil modelled as an elastic mass.
Allowance is made for soil yielding by specifying limiting values of pile - soil pressure at various points along the pile.

$$\rho(i, j) = p(j) . d . I(i, j) / Es(i)$$

where I (i, j) = displacement influence factor, evaluated from Mindlin's equations.

3. FINITE ELEMENT METHOD

Requires 3 - Dimensional analysis for proper nonlinear analysis.

METHODS OF ANALYSIS

ADVANTAGES AND DISADVANTAGES OF ALTERNATIVE METHODS OF ANALYSIS

METHOD	ADVANTAGES	DISADVANTAGES
Subgrade Reaction	Simpler analysis Any type of "p - y" pile - soil response can be analysed Considerable experience in use & evaluation of parameters	Soil "spring" stiffness is dimension - dependent Cannot consider group action
Elastic - based analysis	Can analyse interaction between piles and consider group behaviour can allow for effect of cut or slope near pile Can extend to batter piles Can use to analyse effect of external soil movements	Consideration of non-homogeneous soil profile is approximate Only limited experience in practical use Uncertainties remain in evaluation of relevant soil parameters

SUBGRADE REACTION THEORY

Linear Analysis

- Soil "spring" behaviour:
  - **$p = k.\rho$**
- **Usual cases:**
  - k = constant
  - $k = n_h.z / d$

Dimensionless Parameters:

1. k=constant

$\beta = (k_h.d / E_p I_p)^{1/4}$

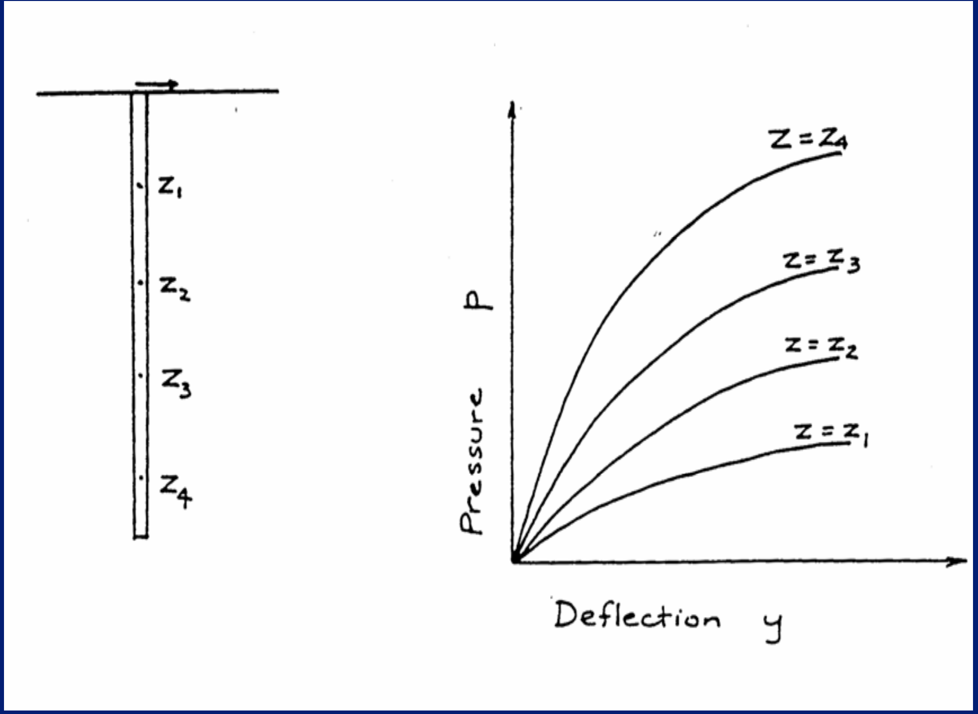
$Lcr = 2.5 / \beta$

2. Linearly Varying k

T = (EpIp / nh)1/5

Lcr = 4T

NONLINEAR “p-y” ANALYSES



- Widely used, both onshore and offshore
- Uses empirical relationships between pressure p and deflection r at a point on the pile
- These are used in a beam analysis to obtain load-deflection curves for the pile

TYPICAL “p-y” CURVES - CLAY

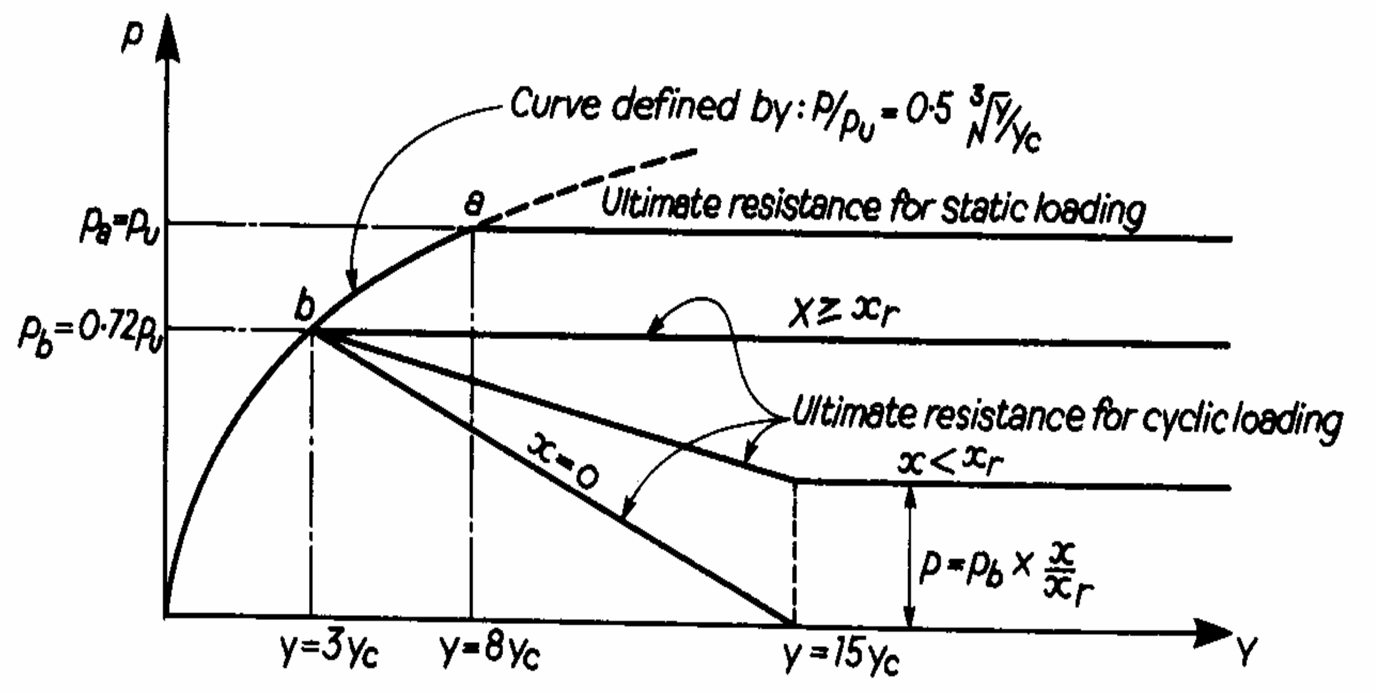


Fig. 6.42 Determining shape of p–y curve in soft to firm clay (after Matlock(6.16))

$x_r = 6d / (\gamma d/c_u + J)$

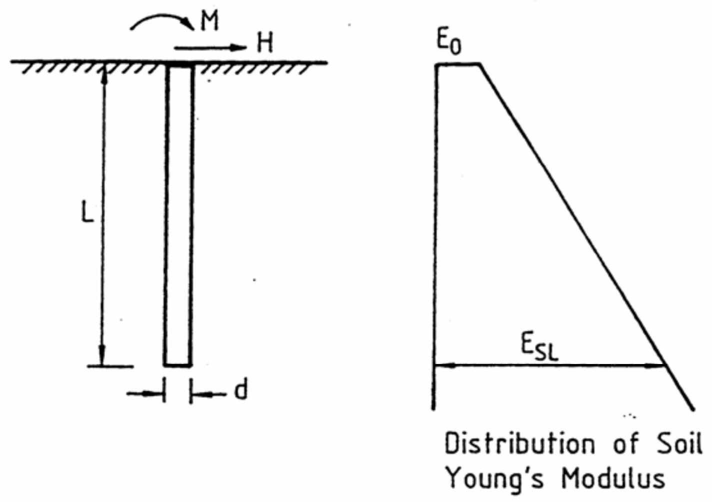
J=0.5 soft clay
=0.25 stiff clay

$p_u = N_c.c_u.d$

$y_c = 2.5\varepsilon_c.d$

ε_c=0.02 soft clay;
0.0005 for brittle & stiff clay

CONTINUUM SOIL MODEL



E_p = Pile modulus
I_p = Pile moment of inertia
E_s = Soil Young's modulus (uniform soil)
v_s = Soil Poisson's ratio
G = Soil shear modulus
k = Modulus of subgrade reaction
N_h = Rate of increase of Young's modulus with depth
n_h = Rate of increase of subgrade reaction modulus with depth

For linear theory:

- Components due to shear & moment can be superposed
- Critical length exists (as for subgrade reaction theory)
- Need accurate assessment of soil parameters near surface

SOLUTIONS FROM CONTINUUM SOIL MODEL

- Soil characterized by:
  - Young's modulus E_s
  - Ultimate lateral pressure p_y
- Cases:
  - Uniform E_s & p_y
  - Linearly varying E_s & p_y - ($Es = Nh.z$)
- Dimensionless Parameters:
  - Flexibility Factor
    - $K_R = E_p I_p / E_s L^4$
    - $K_N = E_p I_p / N_h L^4$
  - Critical Length
    - $L_c/L = 4.44.K_R^{1/4}$
    - $L_c/L = 3.30\ K_N^{1/5}$

SOLUTIONS FROM CONTINUUM SOIL MODEL

1. Constant Modulus With Depth

$$\rho = \frac{H}{E_s L} I_{\rho H} + \frac{M}{E_s L^2} I_{\rho M}$$

$$\theta = \frac{H}{E_s L^2} I_{\theta H} + \frac{M}{E_s L^3} I_{\theta M}$$

$$\rho_F = \frac{H}{E_s L} I_{\rho F} \quad \text{(Fixed-head)}$$

2. Linearly Increasing Modulus With Depth

$$\rho = \frac{H}{N_h L^2} I'_{\rho H} + \frac{M}{N_h L^3} I'_{\rho M}$$

$$\theta = \frac{H}{N_h L^3} I'_{\theta H} + \frac{M}{N_h L^4} I'_{\theta M}$$

$$\rho_F = \frac{H}{N_h L^2} I'_{\rho F} \quad \text{(Fixed-head)}$$

SOLUTIONS FROM CONTINUUM SOIL MODEL

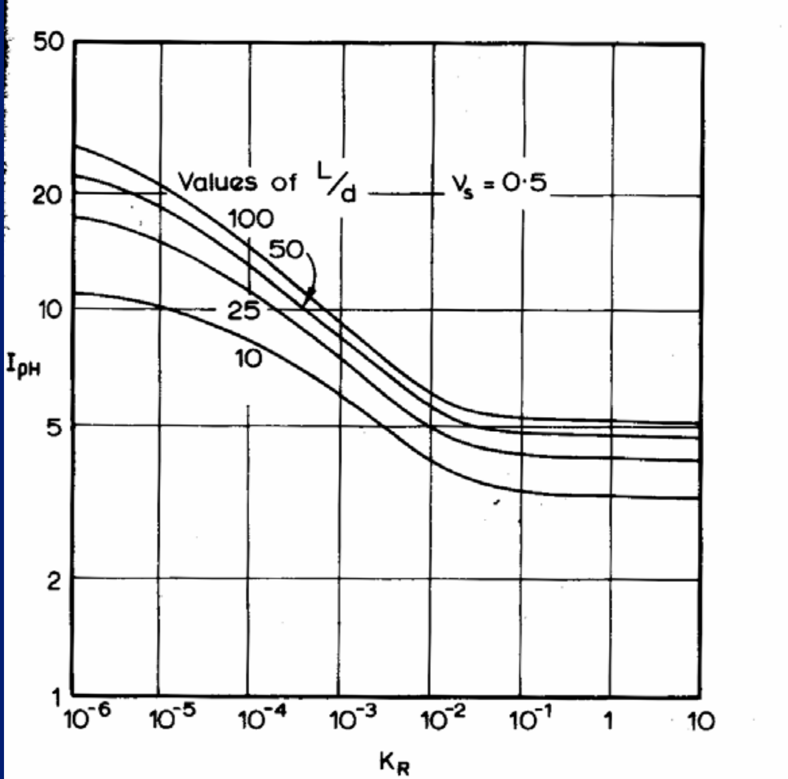


FIGURE 8.13 Values of $I_{\rho H}$–free-head floating pile, constant soil modulus.

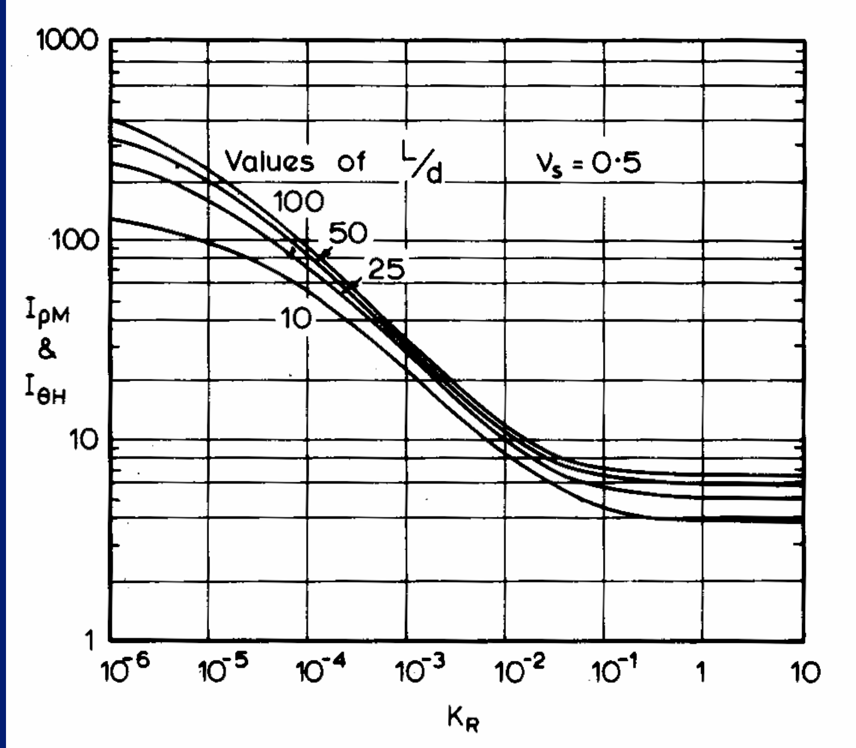


FIGURE 8.14 Values of $I_{\rho M}$ and $I_{\theta H}$–free-head floating pile, constant soil modulus.

SOLUTIONS FROM CONTINUUM SOIL MODEL

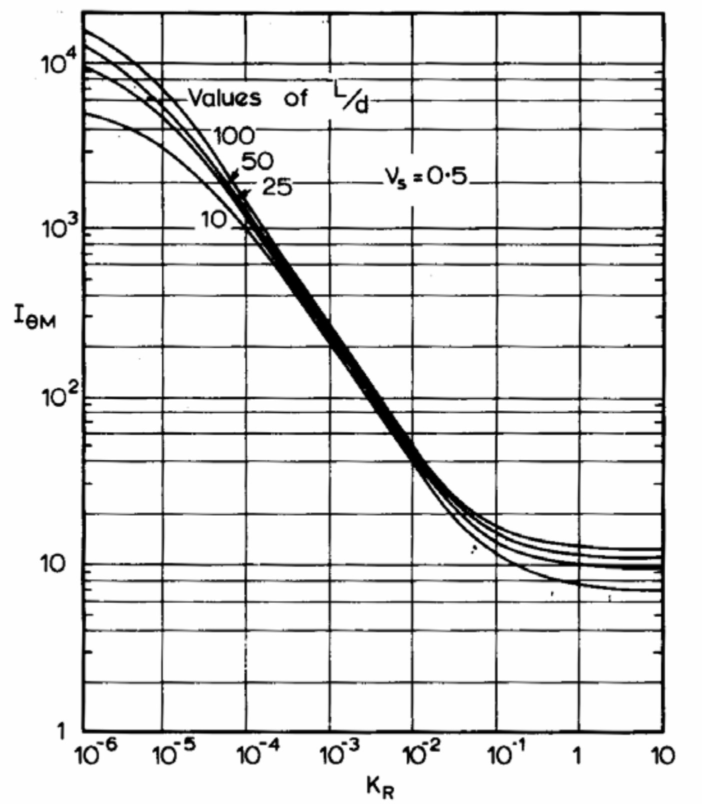
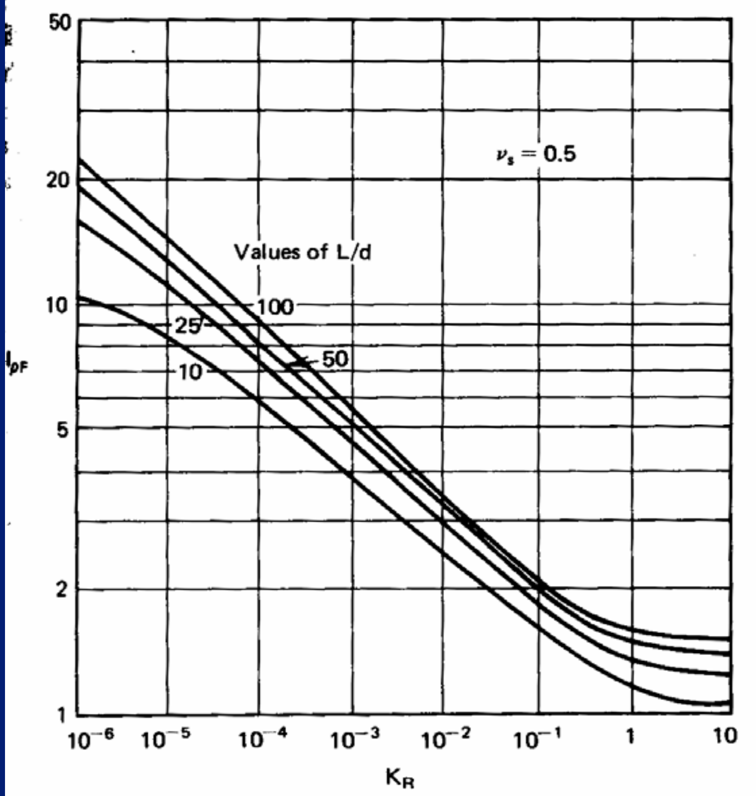


FIGURE 8.15 Values of $I_{\theta M}$–free-head floating pile, constant soil modulus.



IGURE 8.19 Influence factor $I_{\rho F}$–fixed-head floating pile, con-tant soil modulus.

SOLUTIONS FROM CONTINUUM SOIL MODEL (Poulos & Hull, 1989)

Table 3. Solutions For Lateral Pile Response (Linear Elastic Soil)

Pile Head Condition	Groundline Deflection u_e	Groundline Rotation θ_e	Pile Head Fixing Moment M_{Fe}
Free	$\frac{H}{E_e L_e} I_1 + \frac{M}{E_e L_e^2} I_2$	$\frac{H}{E_e L_e^2} I_2 + \frac{M}{E_e L_e^3} I_3$	-
Fixed	$\frac{H}{E_e L_e} I_4$	0	$- H L_e I_5$

- H = applied horizontal load at groundline
- M = applied moment at groundline
- E_e = soil Young's modulus at depth equal to effective length L_e of pile
- L_e = effective length of pile
  - = critical length L_c if $L > L_c$ (flexible pile)
  - = or actual length L if $L < L_c/3$ (rigid pile)
- I_1-I_5 = influence factors depending on L_e/d (where d = pile diameter or breadth) (see Table 4)

Table 4. Solutions For Pile Head Displacement and Rotation – Linear Elastic Response

		Uniform Soil		Gibson Soil	
Case	Factor	A	B	A	B
Flexible Piles	I_1	1.646	3.395	13.10	11.09
	I_2	5.520	9.082	34.63	18.03
$(L > L_c)$	I_3	64.98	37.95	156.1	37.14
	I_4	1.326	1.641	5.659	4.139
	I_5	0.09764	0.04245	0.2278	0.04402
Rigid Piles	I_1	0.976	2.196	3.181	9.701
	I_2	0.701	3.225	2.409	12.71
$(L < L_c/3)$	I_3	1.086	6.292	1.844	18.65
	I_4	0.539	0.545	0.773	1.081
	I_5	0.547	−0.0140	0.764	−0.347

For each factor, $I = A + B \log_{10}(L_e/d)$
Solutions are for soil Poisson's ratio $\nu_s = 0.5$.

MOMENTS INDUCED IN PILE

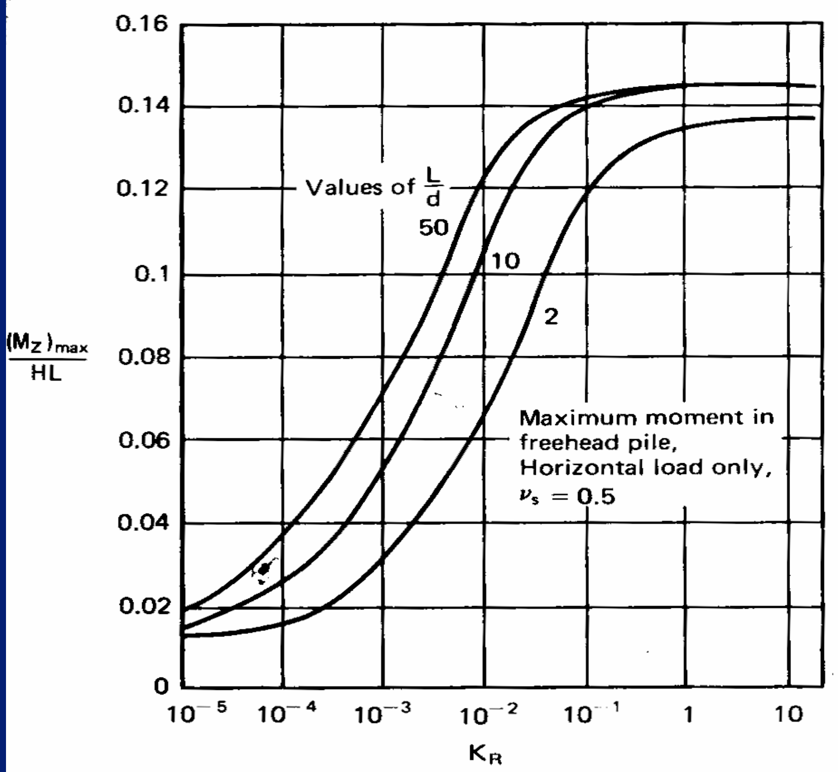


FIGURE 8.27 Maximum moment in free-head pile.

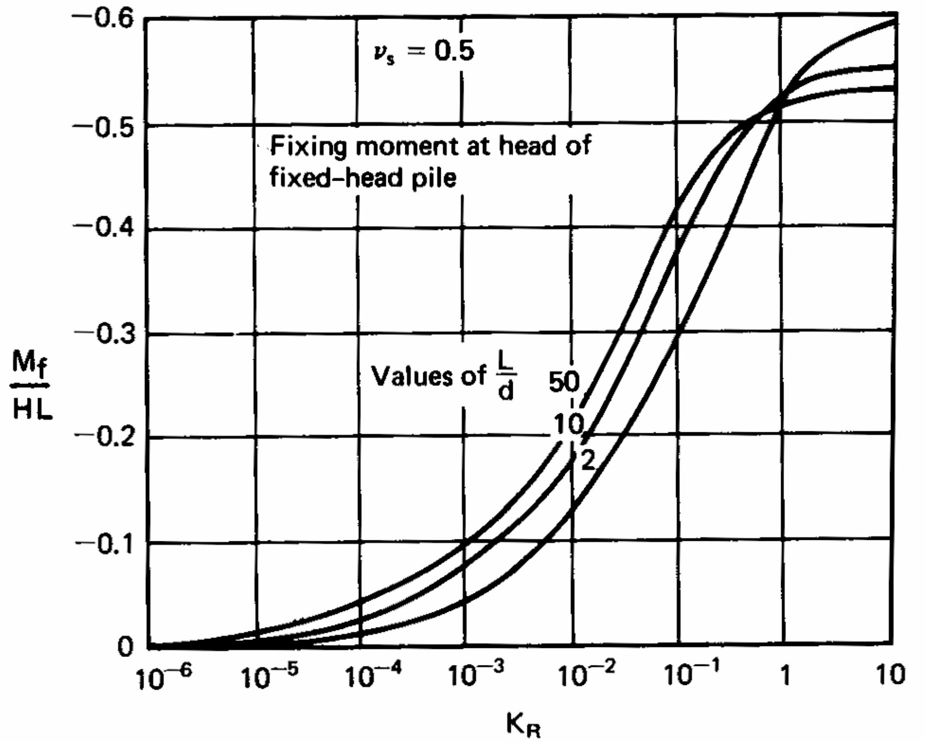


FIGURE 8.28 Fixing moment at head of fixed-head pile.

COMPARISONS BETWEEN THEORIES (Linear)

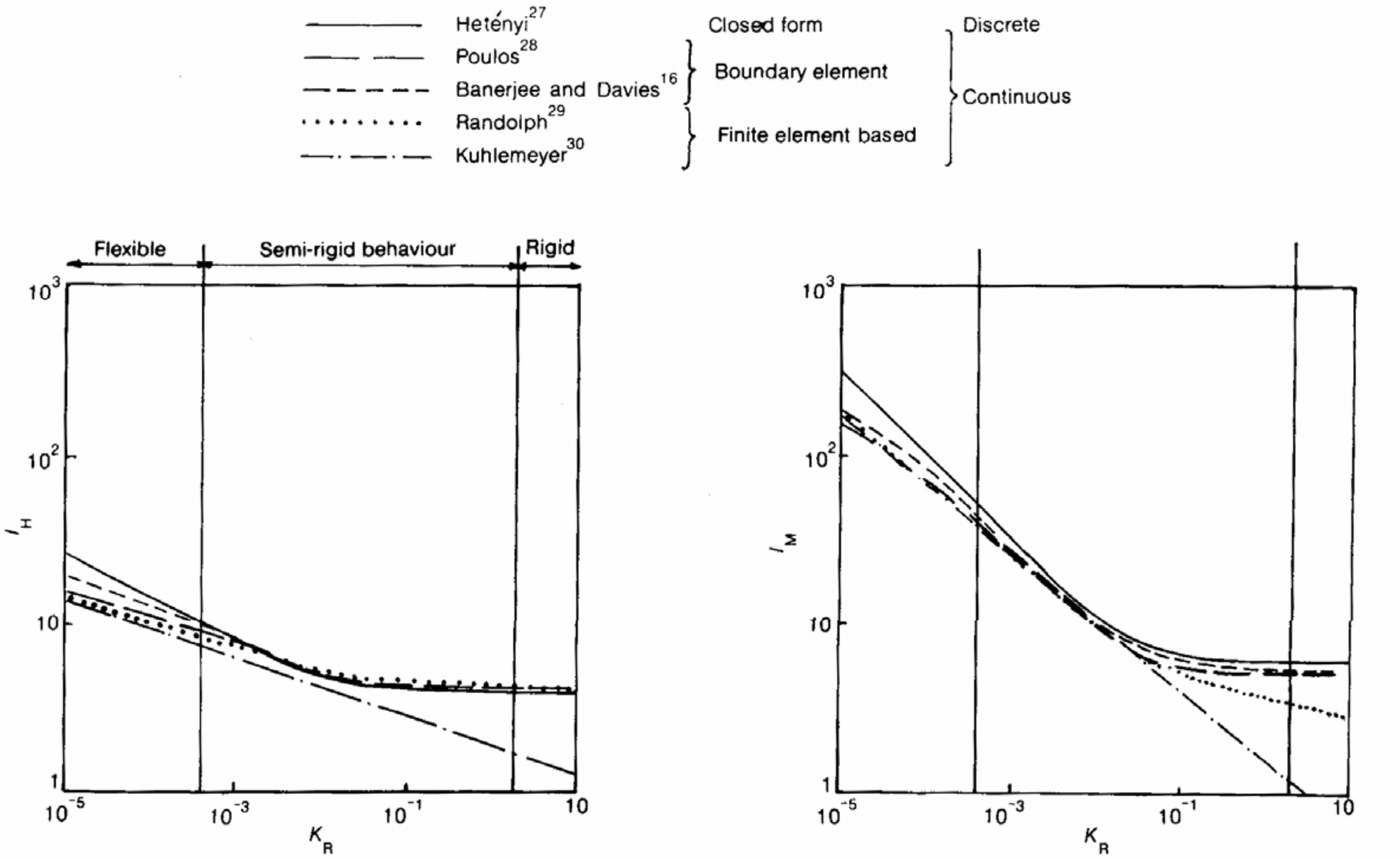


Fig. 17. Constant modulus: influence coefficients for horizontal load, I_H

Fig. 18. Constant modulus: influence coefficients for moment, I_M

COMPARISONS BETWEEN THEORIES (Non-Linear)

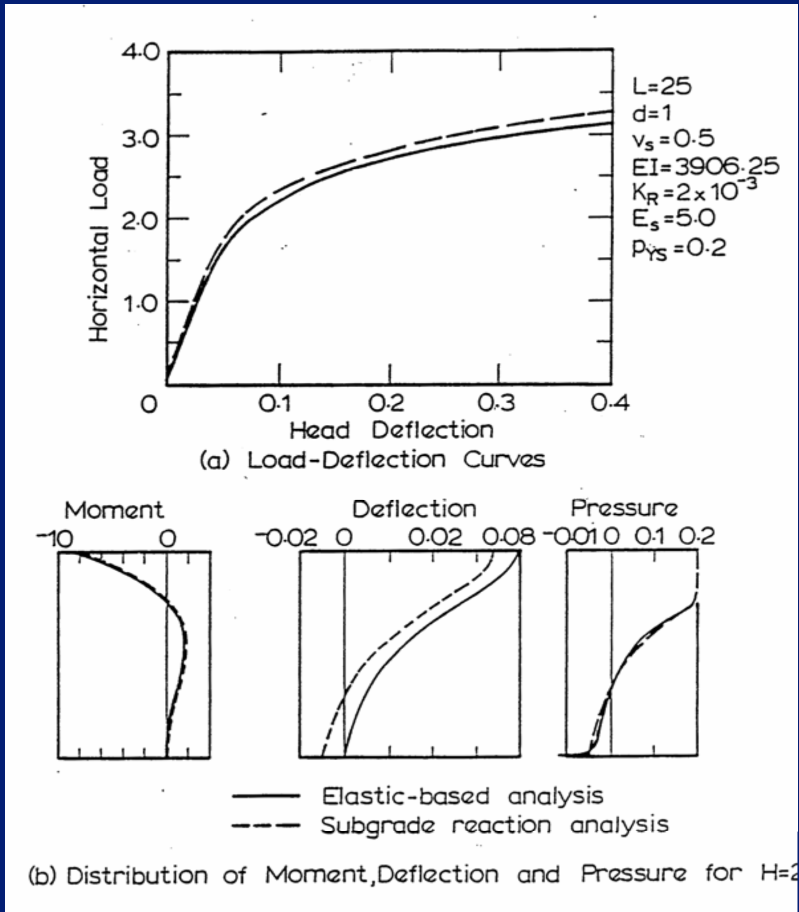


FIG.5.7 INFLUENCE OF ANALYSIS ON STATIC RESPONSE OF FIXED-HEAD PILE

- Subgrade reaction & elastic continuum analyses are quite similar
- Elastic solution shows general additional movement due to pile-soil interaction through the soil
- Bending moments are very close, as are pressures

MODIFICATIONS FOR NON-LINEAR SOIL-PILE BEHAVIOUR

Can allow for non-linear lateral response by modifying elastic solutions via yield factors obtained from non-linear analysis.

Lateral Displacement:

$$\rho = \rho_e / F_u$$

Fixing Moment at Pile Head:

$$M_f = M_{fe} / F_m$$

ρ_e, M_{fe} are calculated from elastic theory.

The yield correction factors F_u, F_m depend on:

- **Load level**
- **Relative flexibility of pile.**

Non-linearity can be significant for:

- **Piles in stiff clay**
- **Relatively flexible piles.**

Solutions available for:

- **Uniform E_s & p_y (except near surface) - relevant to stiff clay**
- **Linearly varying E_s & p_y ("Gibson soil") - relevant to soft clay**

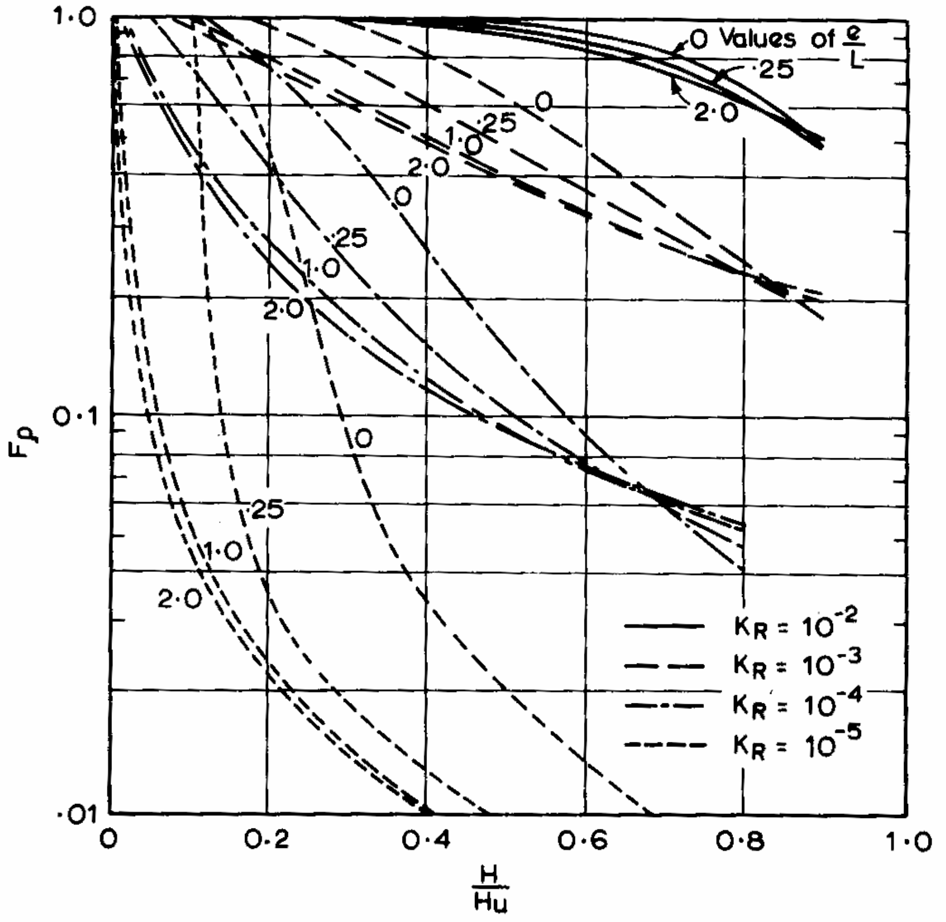


FIGURE 8.16 Yield-displacement factor F_ρ—free-head floating pile, uniform E_s and p_y.

MODIFICATIONS FOR NON-LINEAR SOIL-PILE BEHAVIOUR

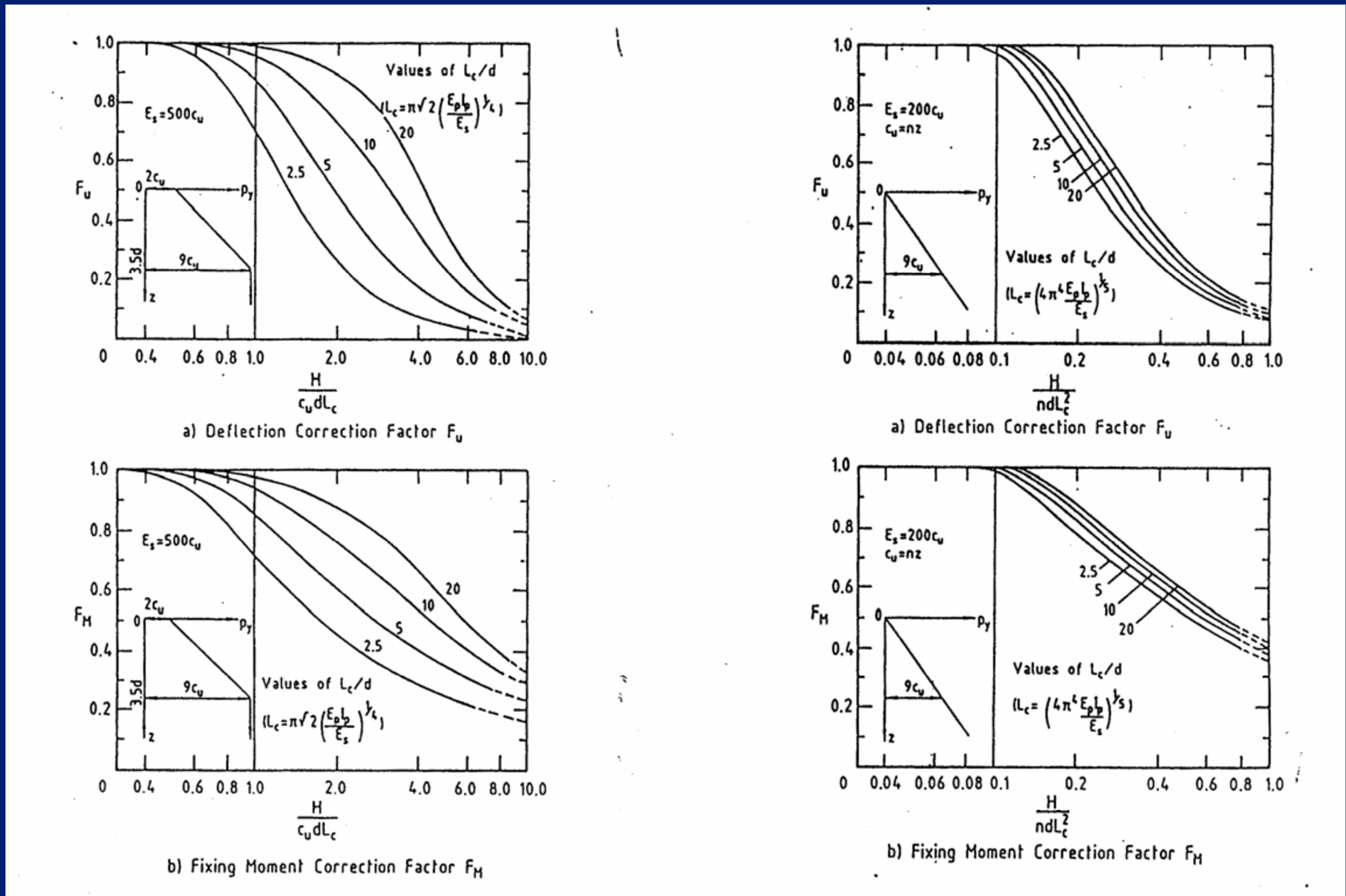


Figure 2. Nonlinear Correction Factors for a Flexible Fixed-Head Pile in Stiff Clay

Figure 3. Nonlinear Correction Factors for a Flexible Fixed-Head Pile in Soft Clay

Fixed-head Piles (Poulos & Hull, 1989)

COMPARISONS BETWEEN MEASURED AND CALCULATED BEHAVIOUR

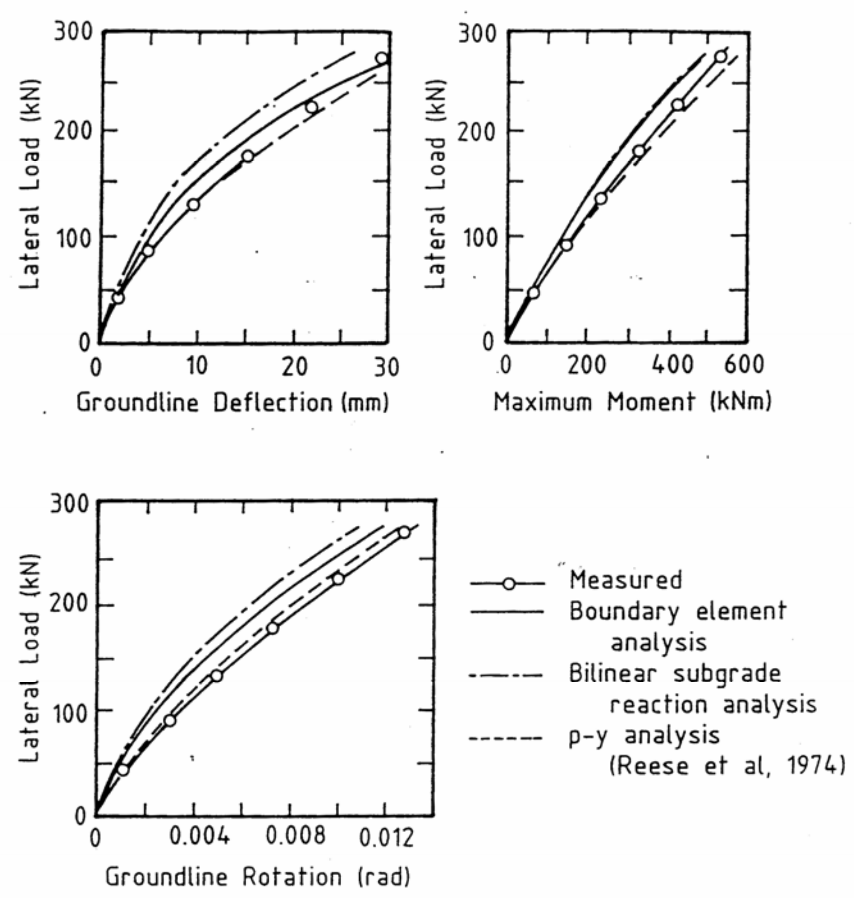


FIG.5 MUSTANG ISLAND TESTS. COMPARISONS BETWEEN MEASURED AND CALCULATED BEHAVIOUR

- All methods show acceptable agreement
- p-y analysis agrees closely because it was used in calibration of method
- Fair results with elastic –plastic subgrade reaction model

ESTIMATION OF SOIL PARAMETERS (MODULUS)

1) INTERPRETATION OF PILE LOAD TESTS

Fit observed deflections to theory

2) INSITU TESTS

eg pressuremeter
plate load tests (lateral)

3) EMPIRICAL CORRELATIONS

a) Clays

$E_s / c_u = 300 \pm 100$

$p_y = 9\ c_u$

b) Sands

$E_s = 1.6\ N$ MPa (Kishida & Nakai)

$N_h = 0.19\ D_r^{1.16}$ MPa / m

(tangent value , Reese et al, 1974)

CORRELATIONS FOR MODULUS IN CLAYS

Table 7.12 Empirical correlations for Young's modulus in clays, for laterally loaded piles.

Relationship	Theory	Reference	Remarks
$E_{si}/c_u = 300–600$	non-linear subgrade reaction	Jamiolkowski & Garassino (1977)	initial tangent modulus for driven piles in soft clays
$E_{si}/c_u = 180–450$	non-linear boundary element	Poulos (1973)	tangent modulus from model tests on jacked piles
$E_{si}/c_u = 280–400$	non-linear subgrade reaction	Kishida & Nakai (1977)	tangent modulus
$E_s/c_u = 100–180$	linear boundary element	Banerjee (1978)	secant value
c_u (kPa) / N_{hi} (MPa m_{-1}) 12–25 / 8 25–50 / 27 50–100 / 80 100–200 / 270 200–400 / 800	non-linear subgrade reaction	Sullivan *et al.* (1979)	tangent values of rate of modulus increase: $E_{si} = N_{hi}\, z$

CORRELATIONS FOR MODULUS IN SANDS

COFFEY

Table 7.13 Empirical correlations for Young's modulus in sands, for laterally loaded piles.

Relationship	Theory	Reference	Remarks
$N_{hi} = 0.19 D_R^{1.16}$ MPa m^{-1} where D_R = relative density (%)	non-linear subgrade reaction	Jamiolkowski & Garassino (1977)	tangent value for driven piles in saturated sands
Condition / N_{hi} MP a/m: Loose 5.4; Medium 16.3; Dense 34.0	non-linear subgrade reaction	Reese *et al.* (1974)	tangent value for driven piles in submerged sands
N_h = 8–19 MPa m^{-1} (av. 10.9)	linear boundary element	Banerjee (1978)	secant value
$E_{si} = 1.6N$ MPa where N = SPT value	non-linear subgrade reaction	Kishida & Nakai (1977)	tangent value

CYCLIC LOADING

MAIN EFFECTS

- Cyclic deflection increases
- Sustainable cyclic load decreases
- Degradation effects more severe for stiffer soils
- Failure can be quite abrupt for small increases in cyclic load level

ANALYTICAL PROCEDURES

1. p-y analyses with modified curves
2. Matlock et al degradation model
   - Degradation occurs when have reversal of plastic strain
3. Modified boundary element analysis: soil modulus & p_y degrade, depending on cyclic deflection & no. of cycles

CYCLIC LOADING APPROACHES

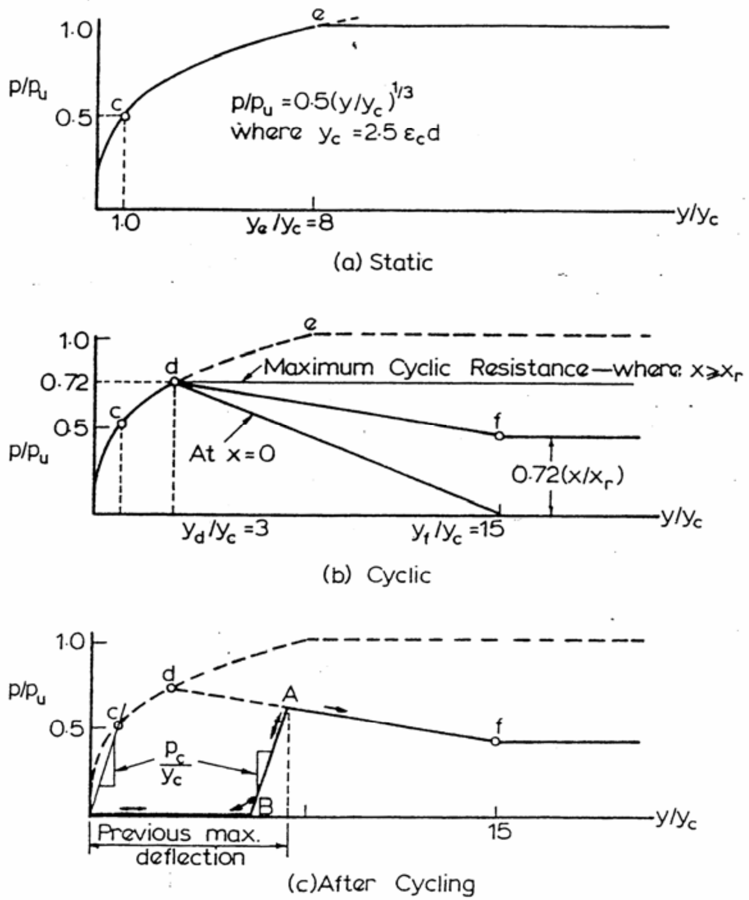


FIG.12 CRITERIA FOR PREDICTING p-y CURVES FOR (a) SHORT-TIME STATIC LOADING,(b) EQUILIBRIUM UNDER INITIAL CYCLIC LOADING AND (c) RELOADING AFTER CYCLING
(Matlock,1970)

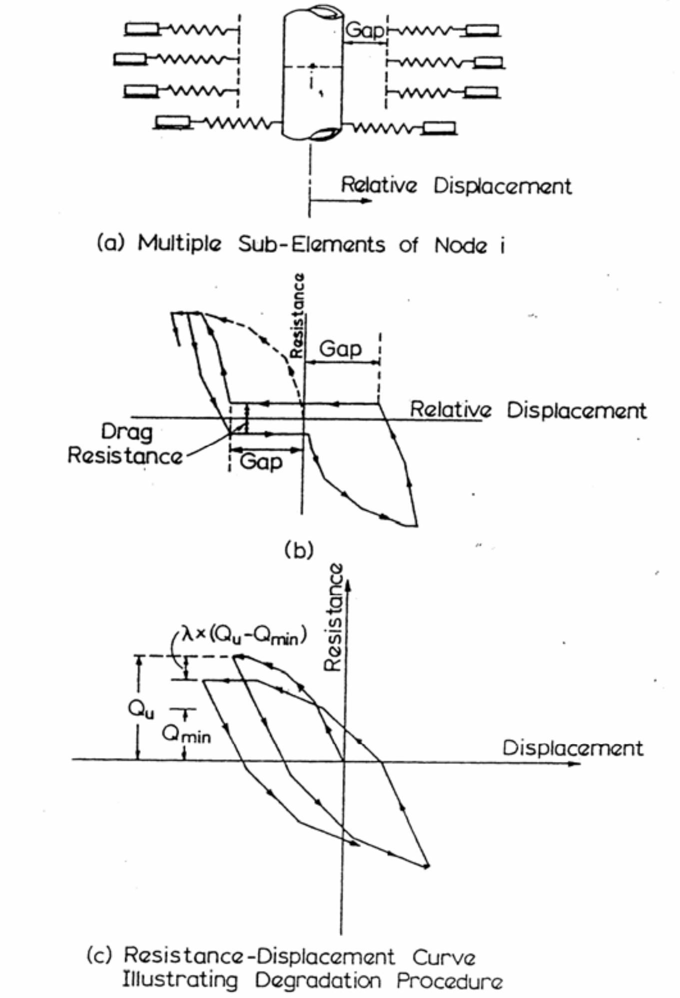


FIG.13 MODEL DEVELOPED BY MATLOCK ET AL (1978)

CYCLIC LOADING APPROACH VIA CONTINUUM ANALYSIS

1. **Compute static lateral deflection for maximum lateral loading, ρ_{c1}**

2. **Estimate:**
   - **Critical strain ε_{cr}**
   - **Ultimate lateral load capacity under static loading (H_{uR})**
   - **Number of cycles N**

3. **Look up curves to obtain ratio of ρ_c / ρ_{c1}, where ρ_c = maximum deflection under cyclic loading, and ρ_{c1} = static lateral deflection**

4. **Compute $\rho_c = \rho_{c1} \cdot \rho_c / \rho_{c1}$**

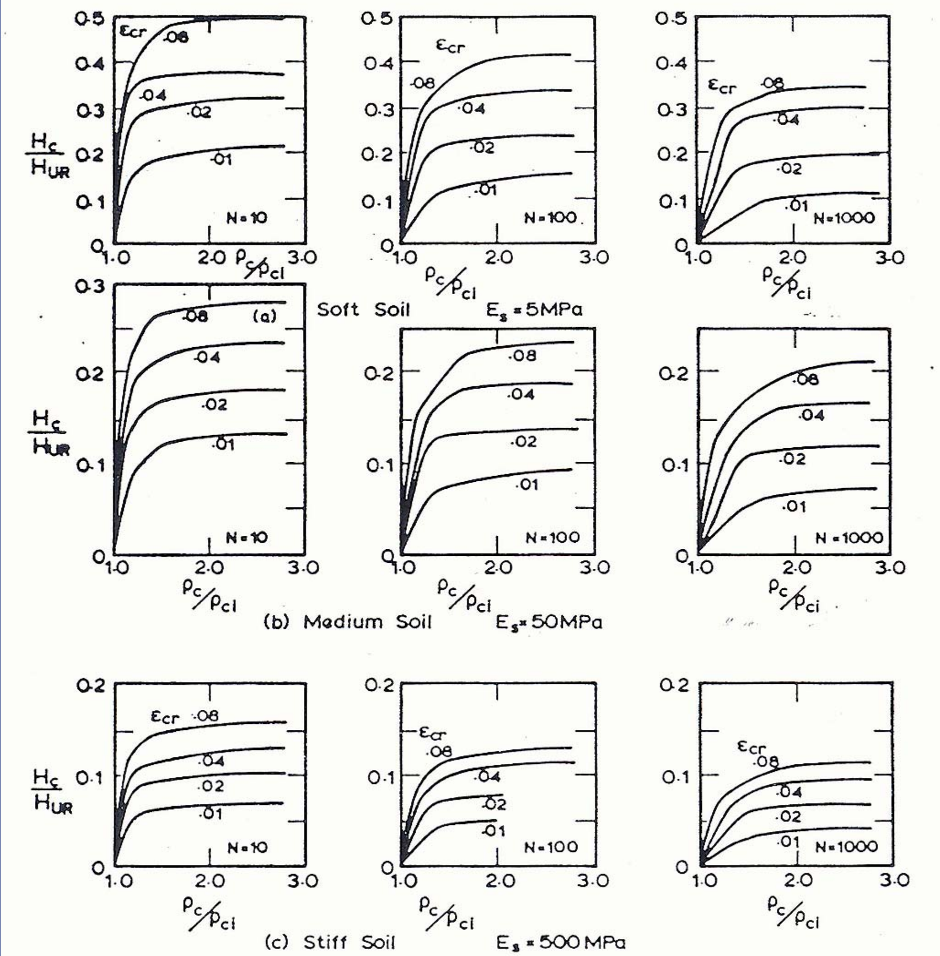


FIG.9 NORMALISED CYCLIC LOAD-DEFLECTION CURVES. HOMOGENEOUS SOIL.

MEASURED vs CALCULATED BEHAVIOUR

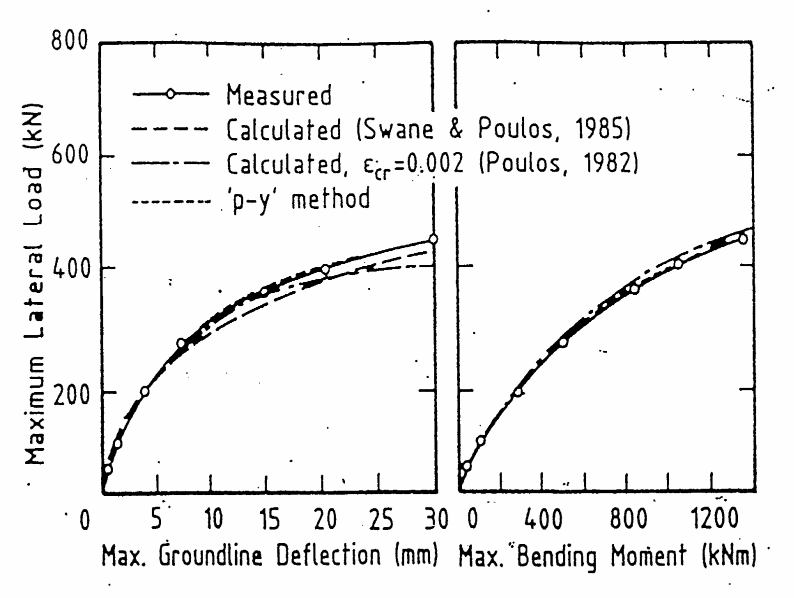
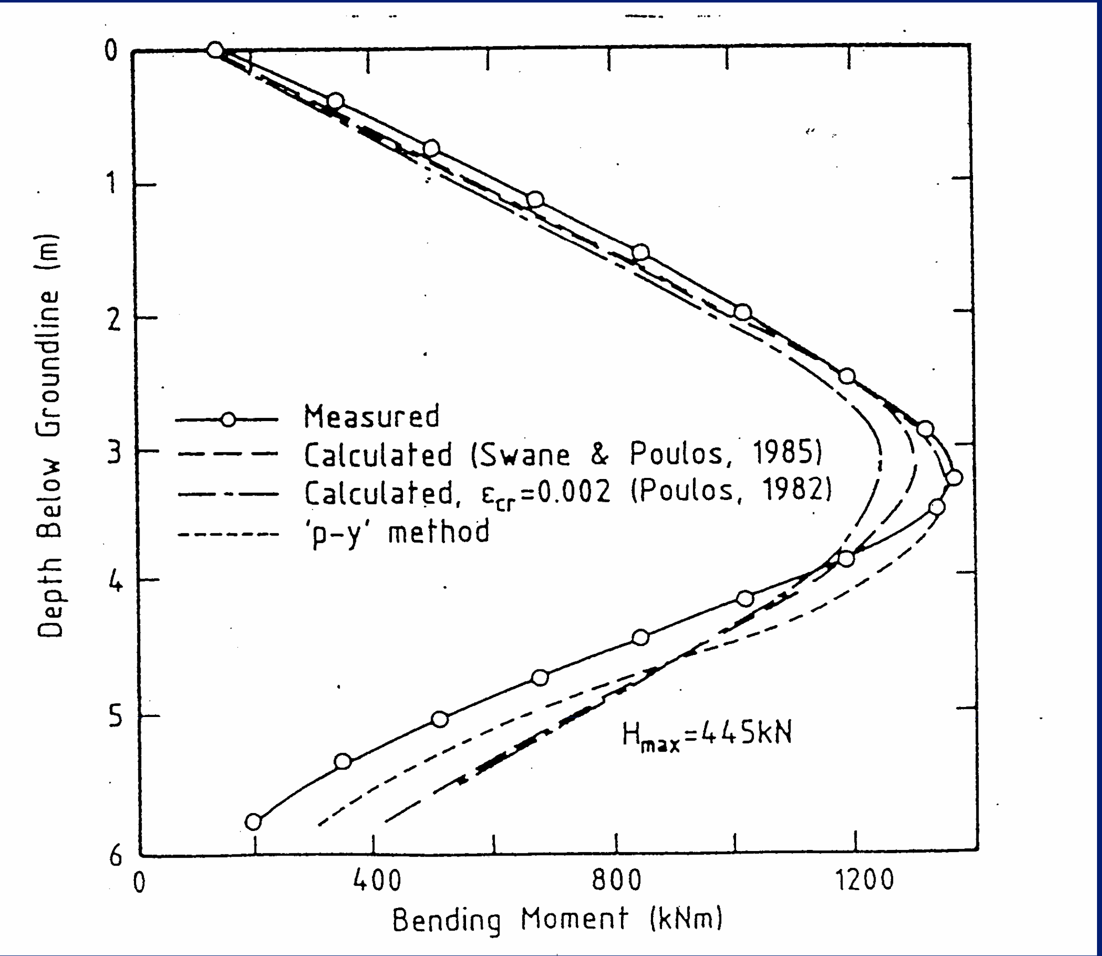


FIG.22 MEASURED AND CALCULATED MAXIMUM PILE DEFLECTION AND BENDING MOMENT FOR CYCLIC LOADING (Tests by Reese et al, 1975)

Load-deflection &
Load-maximum moment



Bending moment distribution along pile

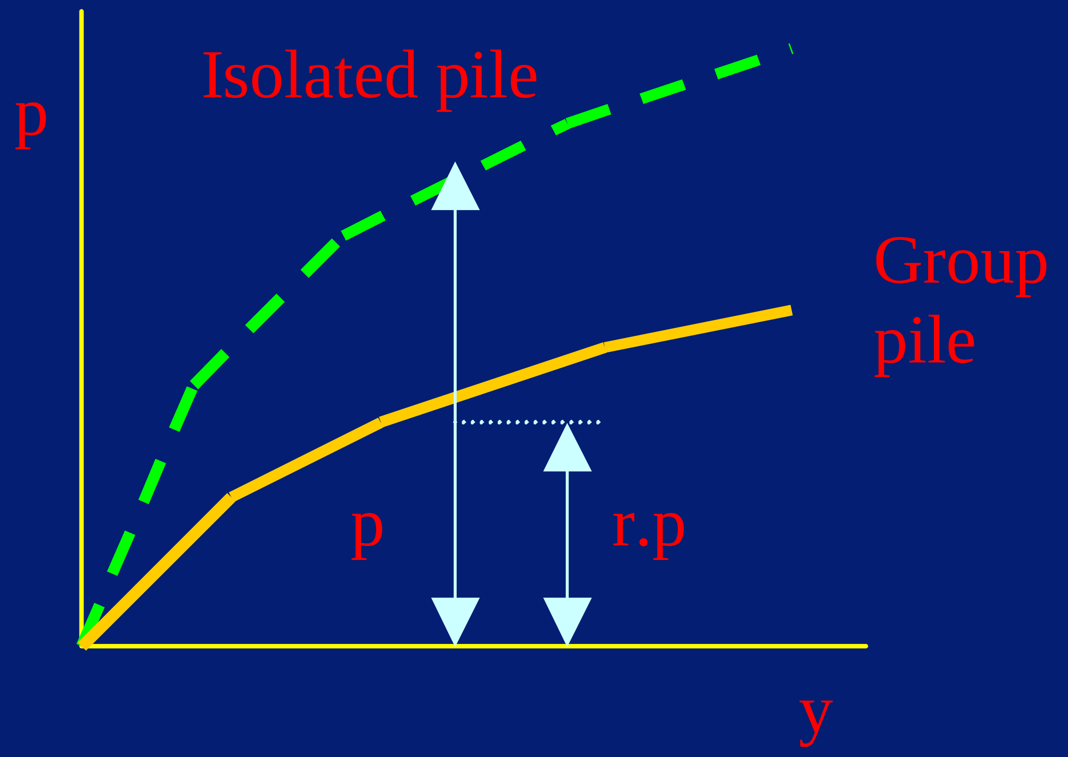
Coffey Geosciences

ALLOWANCE FOR GROUP EFFECTS

- US practice employs the concept of “p-multipliers”.
- These scale down the p-y curves for a single pile to allow for pile-soil-pile interaction in piles within a group.
- Typical example shown in table, from program FLPIER.

Row	*p-multiplier*
Lead	0.8
1st trail	0.4
2nd trail	0.2
3rd trail	0.3

ALLOWANCE FOR GROUP EFFECTS – "p-Multiplier"



Apply multiplier (≤1) to p-y curve for each pile in group. Multiplier derived empirically in most cases.

LATERAL INTERACTION BETWEEN PILES

- Can adopt the interaction factor approach, as for settlement of groups.
- BUT, have some additional problems:
  - Have 5 interaction factors:
    - $\alpha_{\rho H}$, $\alpha_{\rho M}$, $\alpha_{\theta H}$, $\alpha_{\theta M}$,$\alpha_{\rho F}$ (but $\alpha_{\rho M} = \alpha_{\theta H}$ from reciprocal theorem)
  - Interaction factors depend both on spacing (s/d) and "departure angle" between direction of loading & piles
- Values plotted in Poulos & Davis (1980.
- Useful approximations by Randolph (1981).

TYPICAL LATERAL INTERACTION FACTOR DIAGRAM

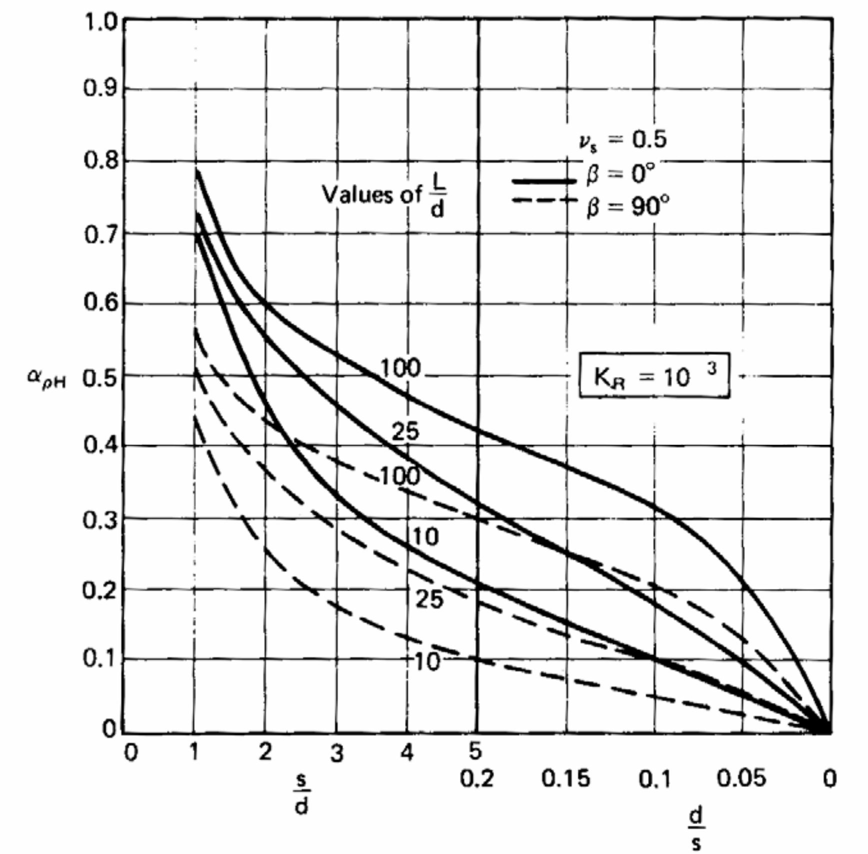
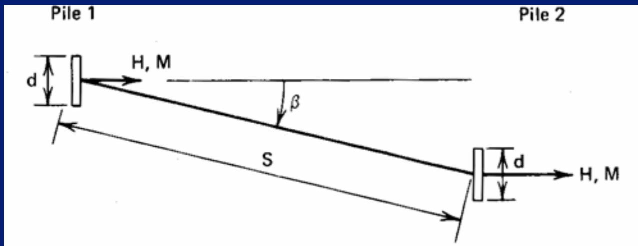
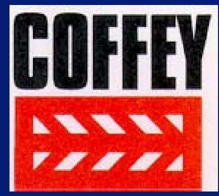


FIGURE 8.63 $\alpha_{\rho H}$ for $K_R = 10^{-3}$.



(a) Forces and pressures acting on piles

APPROXIMATIONS FOR LATERAL INTERACTION FACTORS

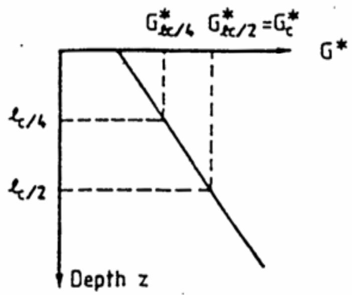


APPROXIMATIONS FOR LATERAL INTERACTION FACTORS

(Randolph, Geotechnique, June 1981)

1 Non-Homogeneity Factor

$$\rho_c = \frac{G^*_{lc/4}}{G^*_{lc/2}}$$



$$G^* = G(1 + \frac{3\nu_s}{4})$$

$$G_c = G^*_{lc/2}$$

2 Critical length

$$l_c = 2r_o \left(\frac{E_p}{m^* r_o}\right)^{2/9}$$

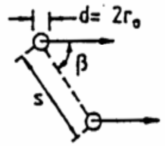
where $m^* = m\,(1 + \frac{3\nu_s}{4})$

$m = dG/dz$

= rate of increase of shear modulus with depth

r_o = pile radius

E_p = pile modulus



3 Interaction Factors

i)

$$\alpha_{\rho F} = 0.6\rho_c \left(\frac{Ep}{Gc}\right)^{1/7} \frac{r_o}{s} (1 + \cos^2\beta)$$

If $\alpha_{\rho F} > 0.5$ from this expression

use $\alpha_{\rho F} = 1 - (4\alpha_{\rho F})^{-1}$

ii)

$$\alpha_{\rho H} = 0.5\,\rho_c \left(\frac{Ep}{Gc}\right)^{1/7} \frac{r_o}{s} (1 + \cos^2\beta)$$

If $\alpha_{\rho H} > 0.5$ from this expression

use $\alpha_{\rho H} = 1 - (4\,\alpha_{\rho H})^{-1}$

iii) $\alpha_{\rho M} = \alpha_{\theta H} = \alpha^2_{\rho H}$

iv) $\alpha_{\theta m} = \alpha^3_{\rho H}$

EQUATIONS FOR LATERALLY LOADED GROUP

1. **Lateral Deflections**

$$\{ \rho \} = [A_{\rho H}] \{H\} + [A_{\rho M}] \{M\}$$

2. **Rotations**

$$\{ \theta \} = [A_{\theta H}] \{H\} + [A_{\theta M}] \{M\}$$

3. **Equilibrium**

$$H_G = \Sigma H_i$$
$$M_G = \Sigma M_i$$

4. **Boundary Conditions**

For rigid cap:

$$\{ \rho \} = \rho_G \{ 1 \}$$

$$\{ \theta \} = \theta_G \{ 1 \}$$

Note: These equations hold only for pinned - head piles or fixed - head piles, where lateral response can be de-coupled from the axial response.

SIMPLIFIED APPROACH FOR ESTIMATING LATERAL GROUP DEFLECTION

FIXED HEAD GROUP

$$\rho_{hF} = \rho_{F1} \cdot H_{av} \cdot R_{\rho F}$$

where

ρ_{hF} = single pile deflection / unit load

H_{av} = average lateral load

$R_{\rho F}$ = group deflection factor $\approx n^{\omega L}$

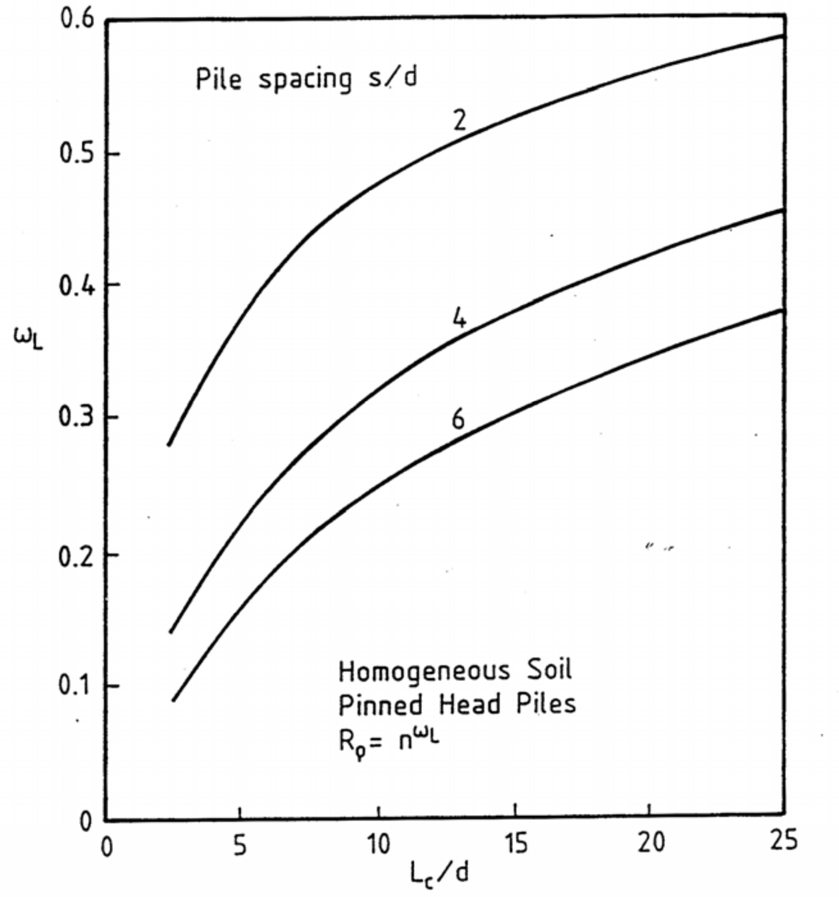


FIG.4.5 GROUP FACTOR EXPONENT ω_L FOR LATERALLY LOADED GROUPS. HOMOGENEOUS SOIL

SOME PRACTICAL DIFFICULTIES WITH THEORETICAL LATERAL INTERACTION FACTORS

- Elastic theory suggests that interaction between 2 piles is the same, for the same spacing & orientation
- Experiments indicate this is NOT SO.
- Interaction is smaller for ζ = 0º than for ζ=180º.

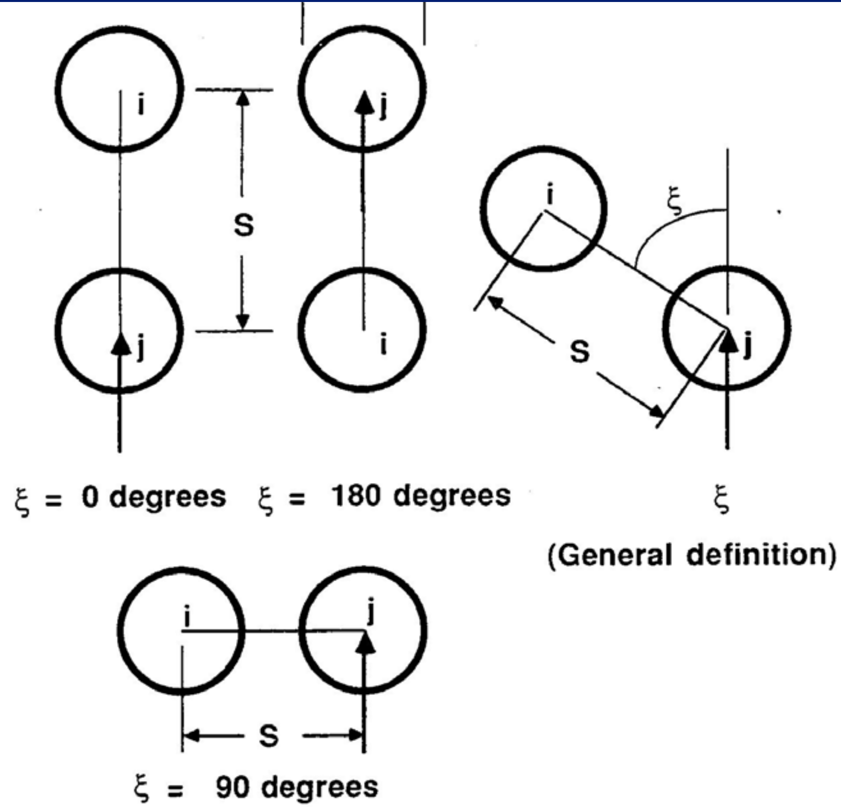


FIG. 7. Definitions of ξ, S, and D

SOME PRACTICAL DIFFICULTIES WITH THEORETICAL LATERAL INTERACTION FACTORS

- Also, field tests indicate that interaction factors increase with increasing load level, but decrease with increasing number of cycles

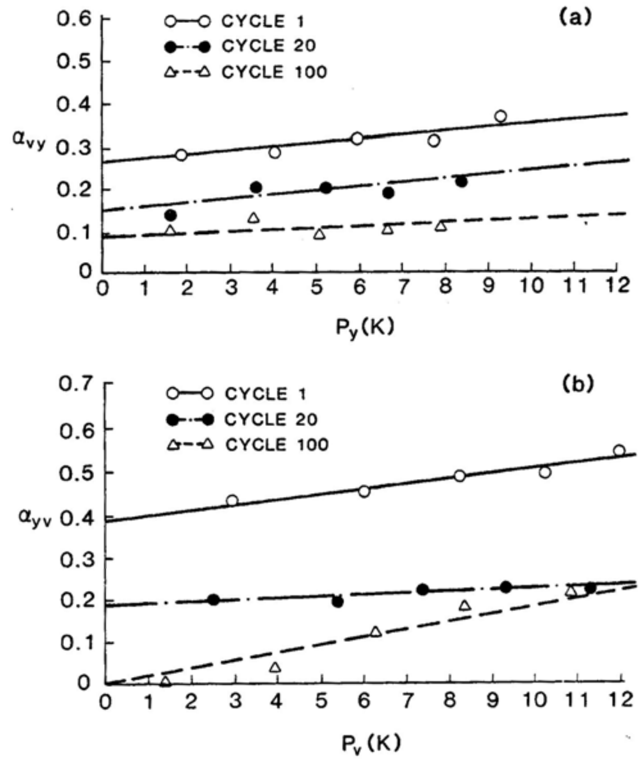


FIG. 8. Interaction Factors for Two-Pile Test on Piles V and Y: (*a*) $\xi = 0°$, $S/D = 3$, Loading North; (*b*) $\xi = 0°$, $S/D = 3$, Loading South (1 K = 4.45 kN)

SOME PRACTICAL DIFFICULTIES WITH THEORETICAL LATERAL INTERACTION FACTORS

- The conventional interaction factors can be adjusted to produce more realistic group behaviour.
- Scaling process described by Ochoa and O'Neill (1989).
- Factor α_{ij} for $\zeta = 180^o$ should only be applied when pile j is in leading row of group.
- Otherwise, use factor for $\zeta = 0^o$ when $\zeta = 180^o$.

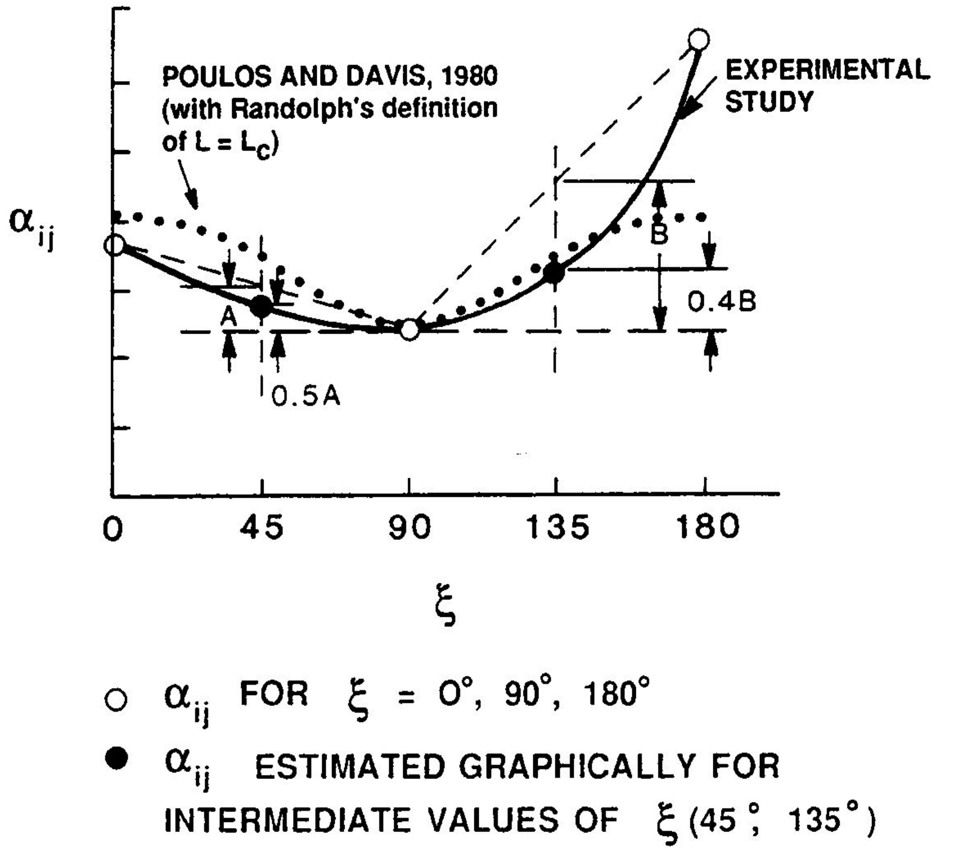
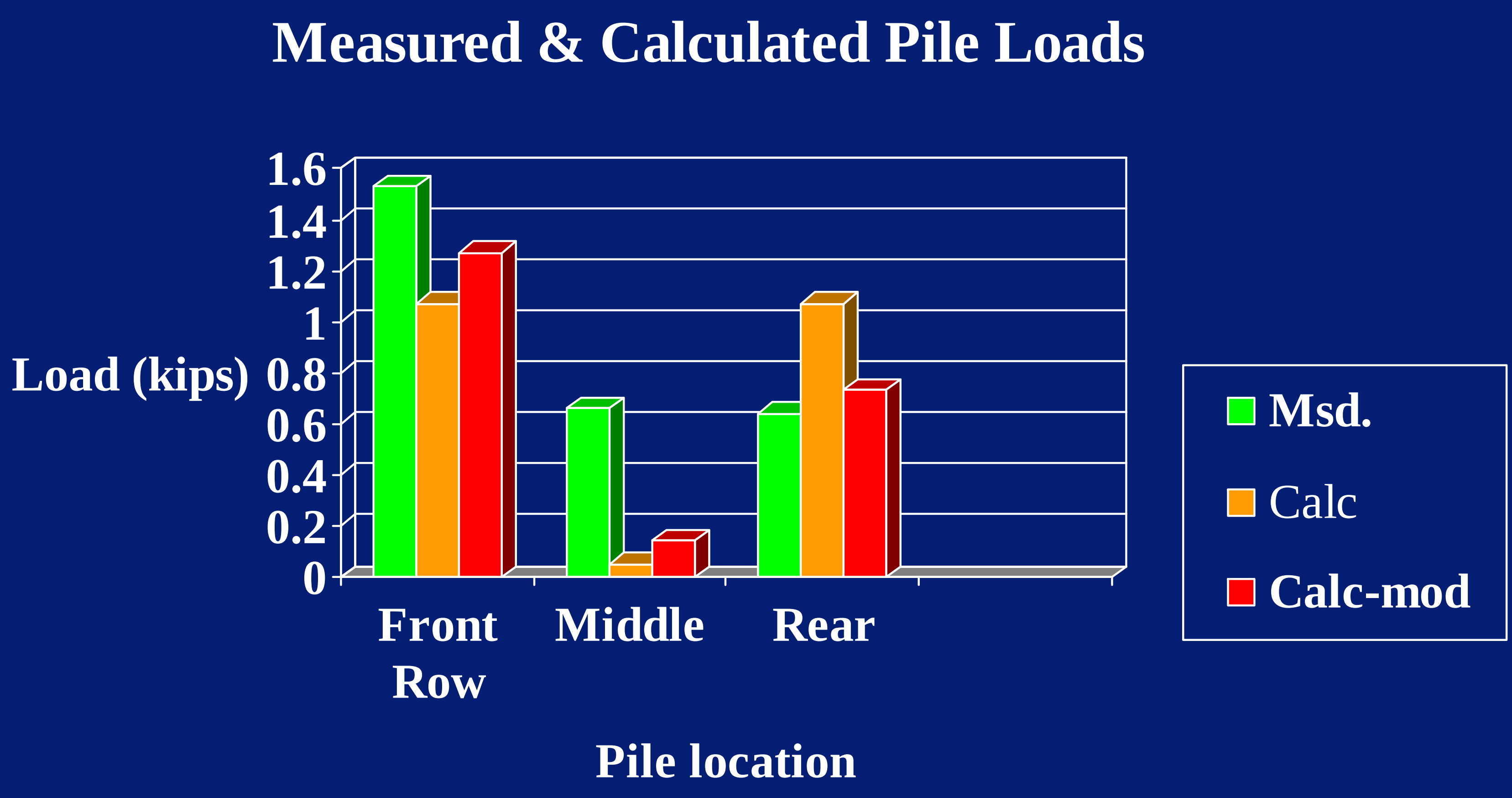


FIG. 14. Interpolation Relation

MEASURED & COMPUTED PILE LOADS IN 9-PILE GROUP (Ochoa & O'Neill, 1989)



SUMMARY

- In reality, the leading piles of a laterally loaded group tend to be stiffer and take a larger proportion of the lateral load than the inner or rear piles.
- This effect can be reproduced by the theoretical analysis if the adjustments are made to the interaction factors, as proposed by Ochoa & O'Neill (1989).