LECTURE 1

INTRODUCTION & AXIAL LOAD CAPACITY

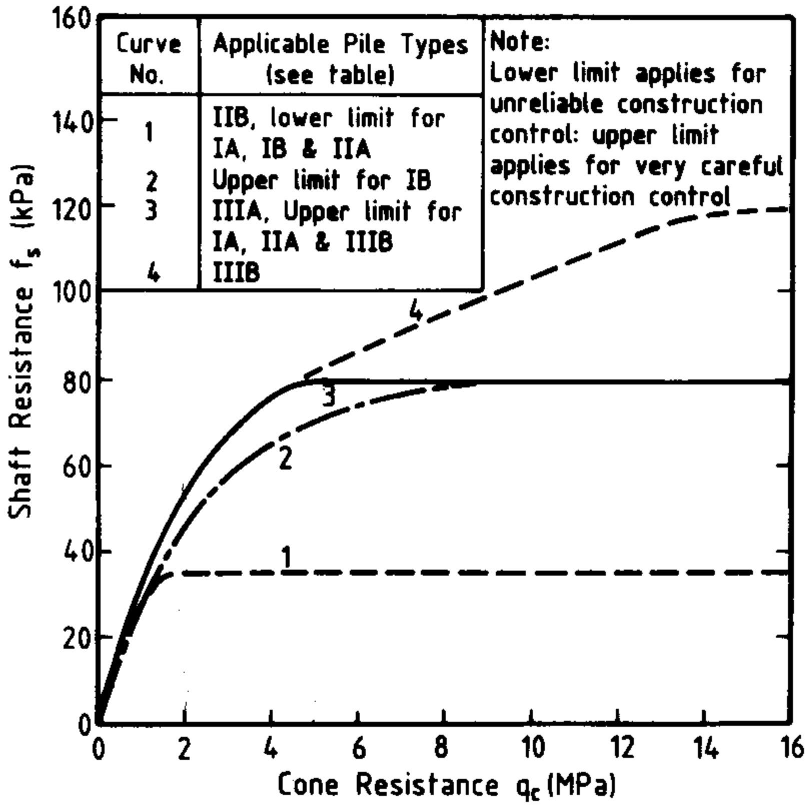


Fig. 25. Design values of shaft resistance for piles in clay (based on Bustamante & Gianeselli, 1982)

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

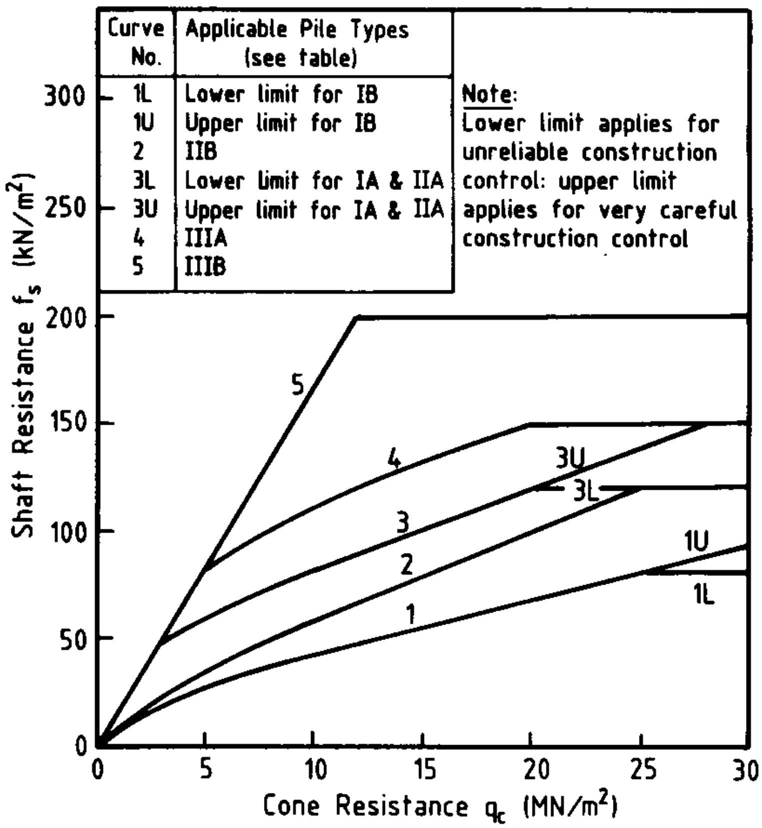
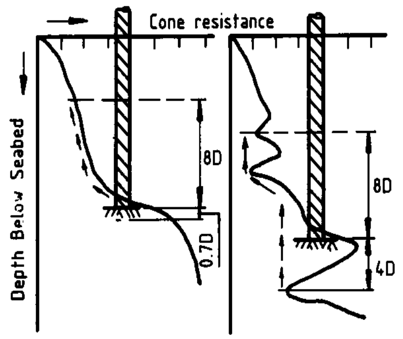


Fig. 26. Design values of shaft resistance for piles in sand (based on Bustamante & Gianeselli, 1982)

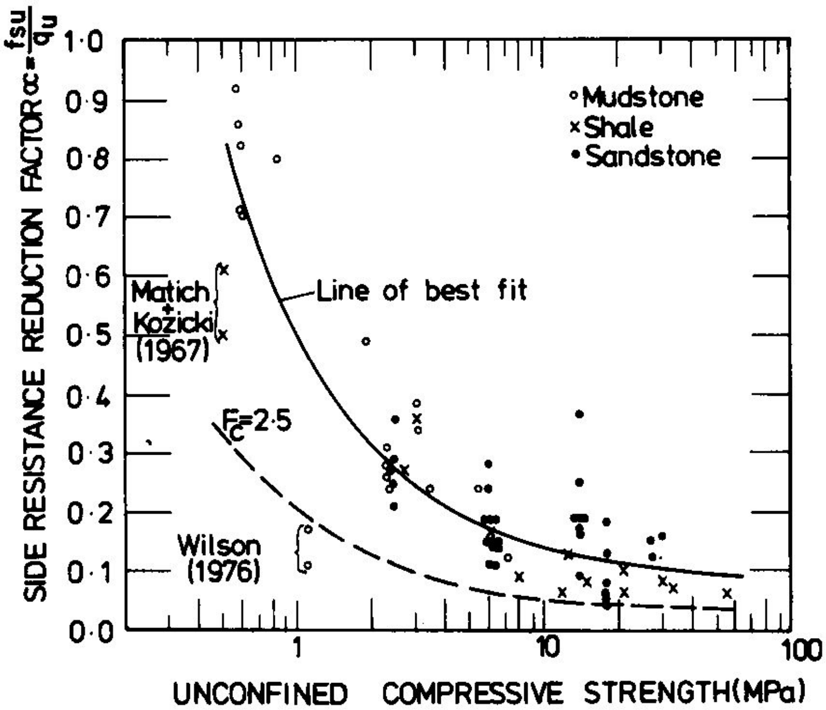


$$q_p = \frac{(A+B)/2+C}{2}$$

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).



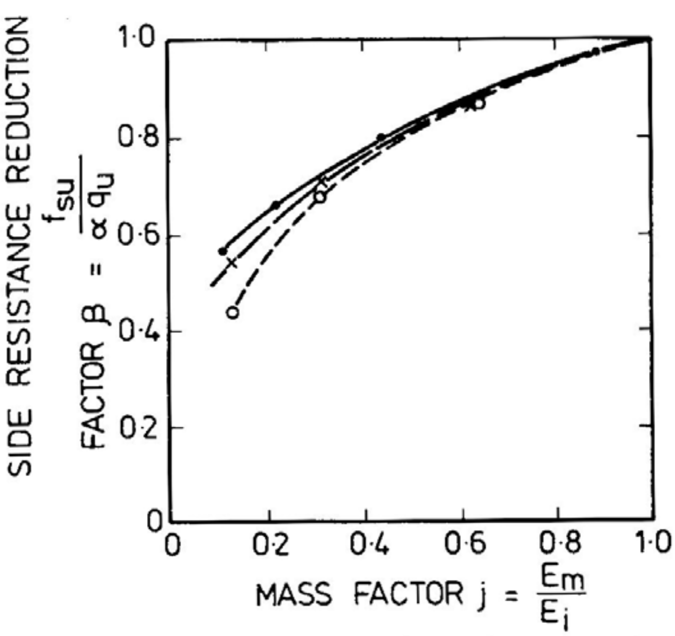


FIG. 7. Mass modulus factor β for Melbourne mudstone. NOTES: ●——● MW mudstone, normal roughness; ×——× HW mudstone roughened; ○- - - ○ HW mudstone, normal roughness.

LECTURE 2
SETTLEMENT OF SINGLE PILES AND PILE GROUPS

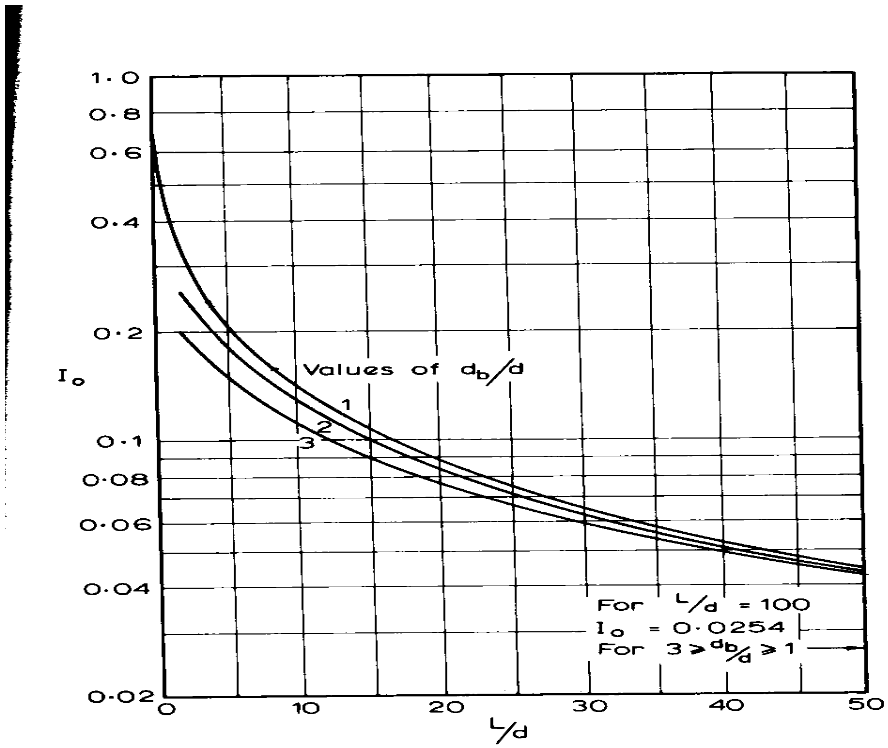


FIGURE 5.18 Settlement-influence factor, I_o.

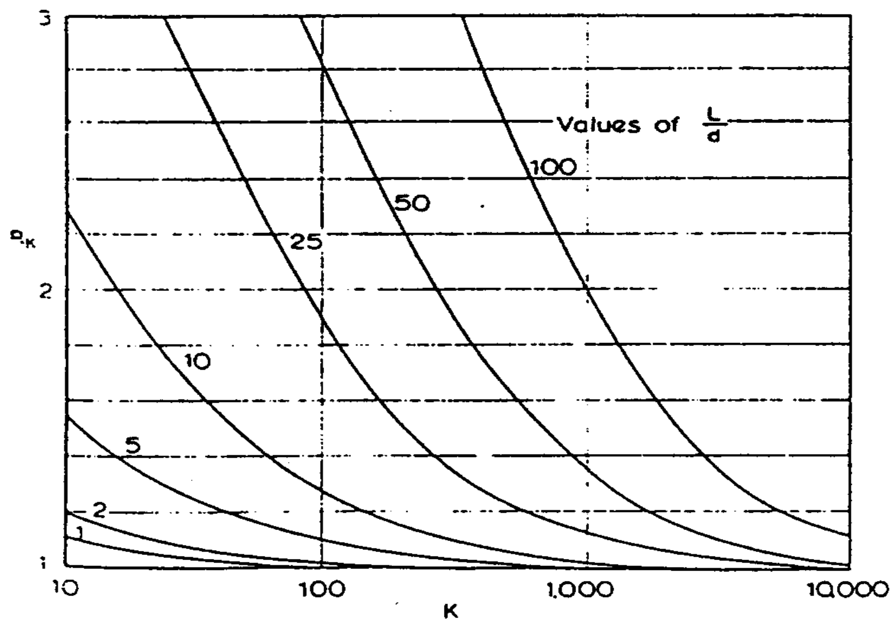


FIGURE 5.19 Compressibility correction factor for settlement, R_K.

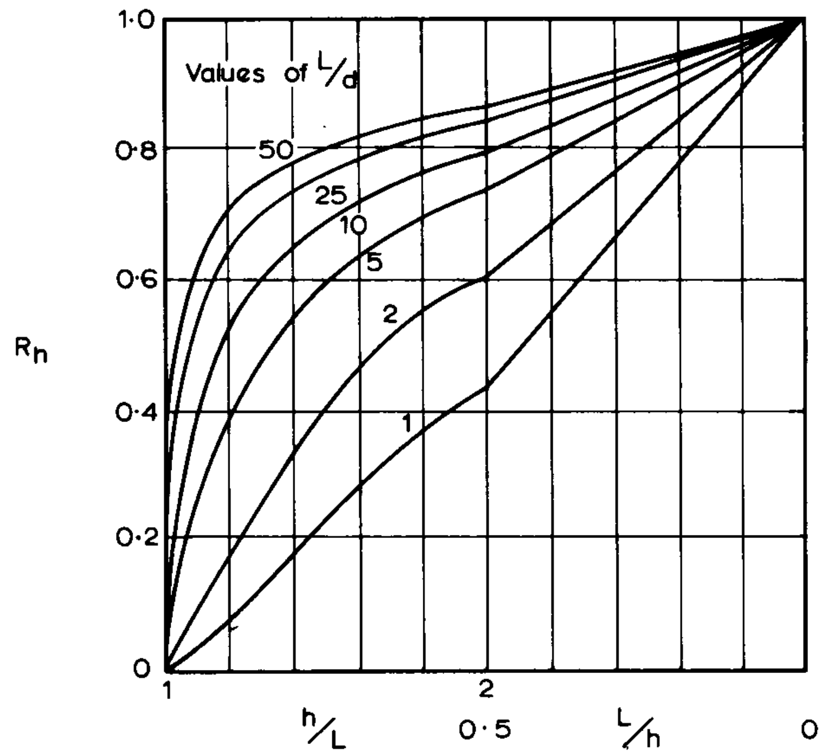
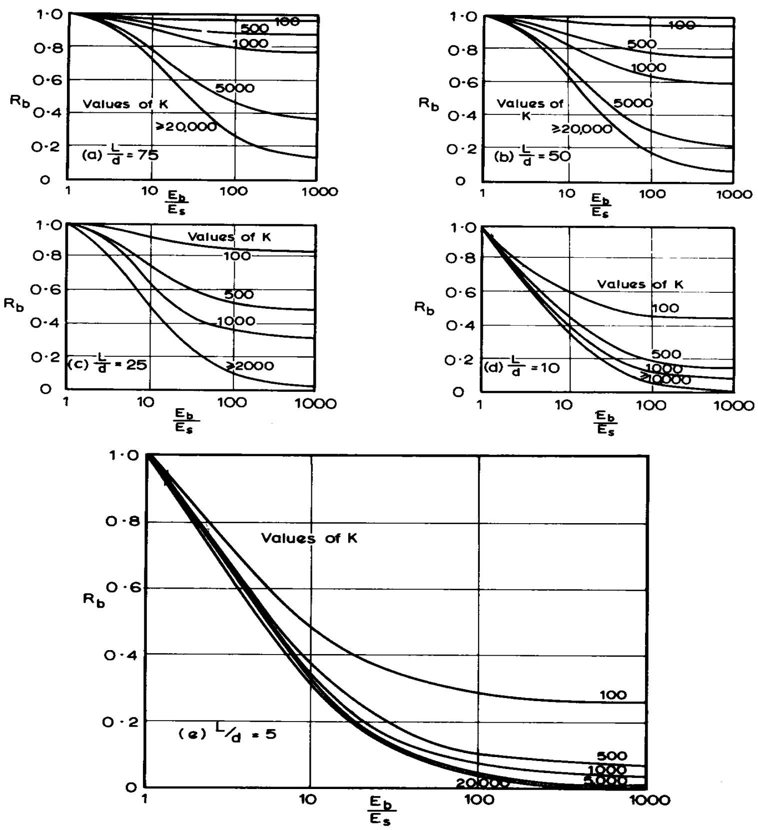


FIGURE 5.20 Depth correction factor for settlement, R_h.



GURE 5.32 Base modulus correction factor for settlement, R_b

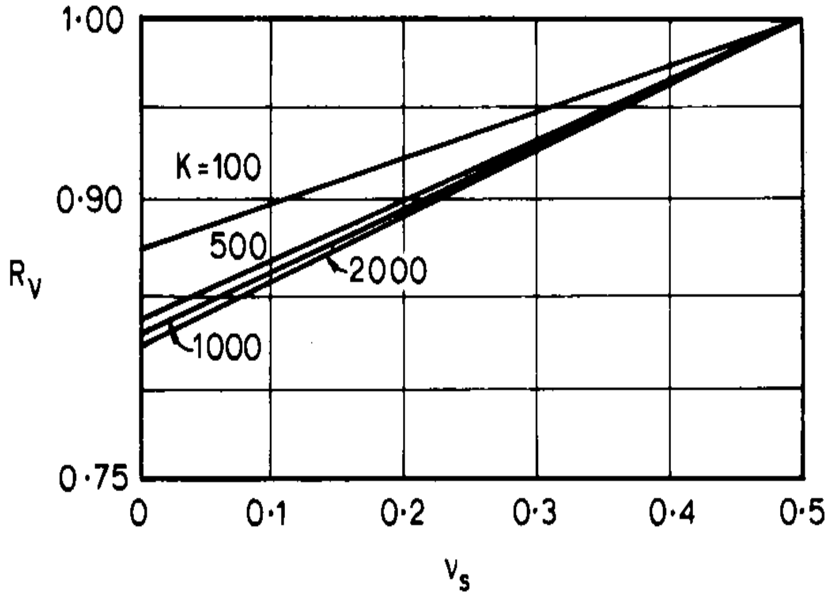
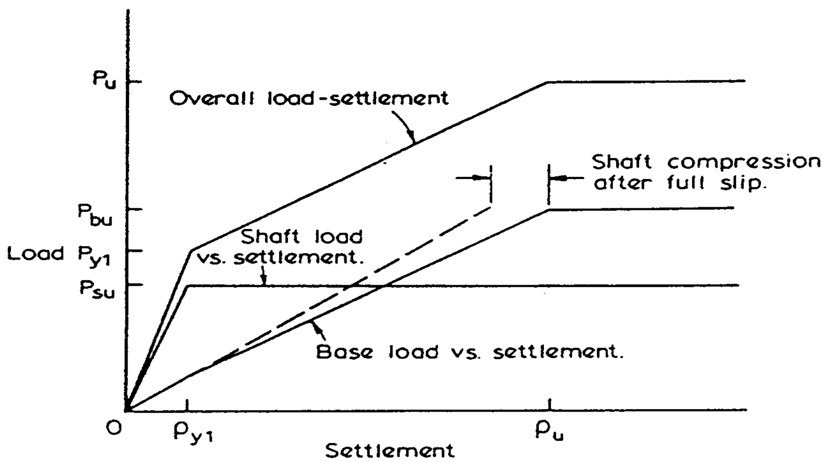


FIGURE 5.21 Poisson's ratio correction factor for settlement, R_ν.

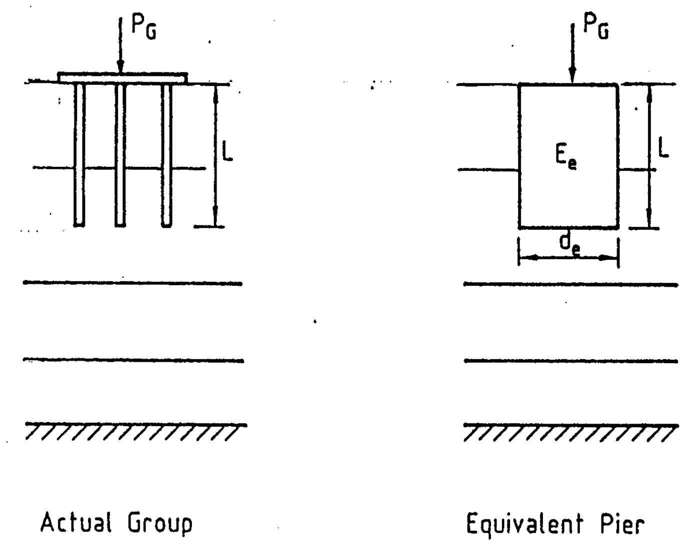
$$\rho_{y1} = \left(\frac{I}{E_s d}\right)\left(P_{y1}\right)$$

$$\rho_u = \left(\frac{I}{E_s d}\right)\left(\frac{P_{bu}}{\beta}\right) + \left[P_{bu} - \frac{P_{su}\beta}{(1-\beta)}\right]\frac{L}{A_p E_p}$$



Construction of load-settlement curve.

EQUIVALENT PIER APPROACH FOR PILE GROUPS



a) for predominantly friction piles:

$$d_e = 1.27\sqrt{A_G}$$

b) for predominantly end-bearing piles:

$$d_e = 1.13\sqrt{A_G}$$

where A_G = plan area of pile group.

The equivalent pier modulus, E_e, is approximated as:

$$E_e = E_p\frac{A_p}{A_G} + E_s\left(1-\frac{A_p}{A_G}\right)$$

where E_p = Young's modulus of piles
E_s = average Young's modulus of soil within the group
A_p = total cross-sectional area of the piles in the group.

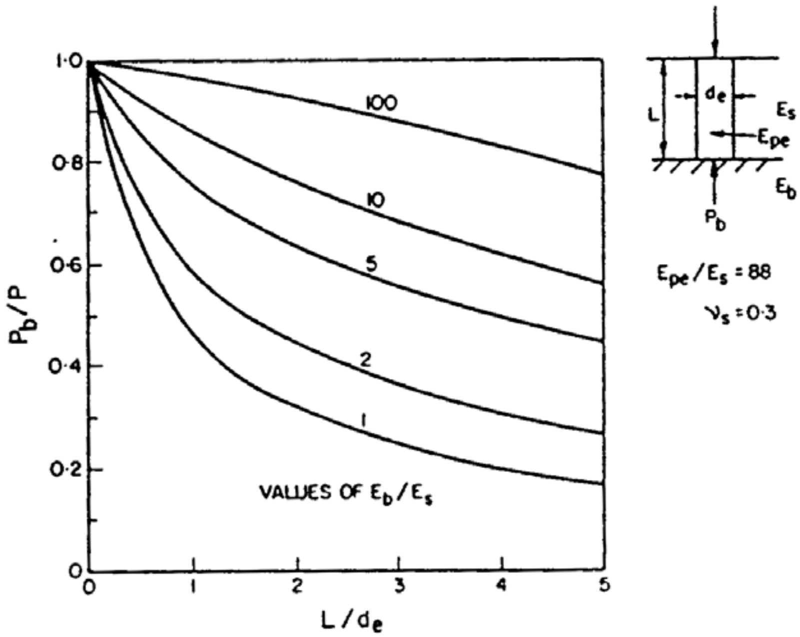


FIG. 5. Proportion of Base Load for Equivalent Pier

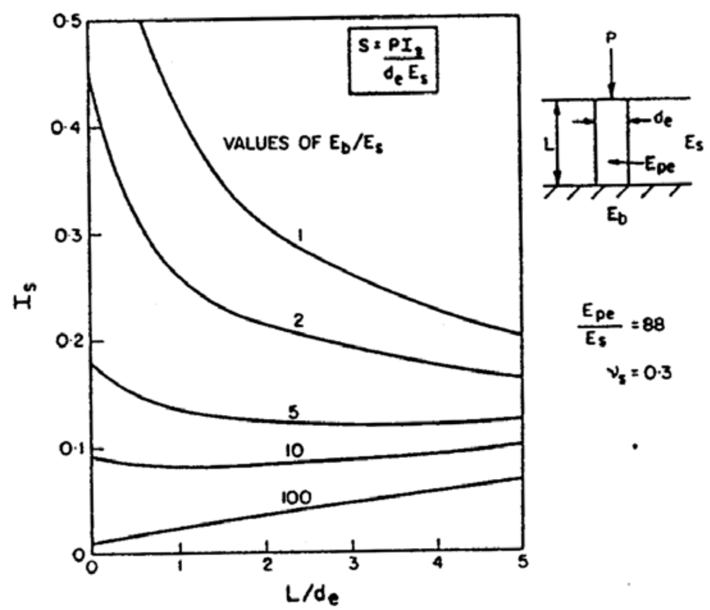
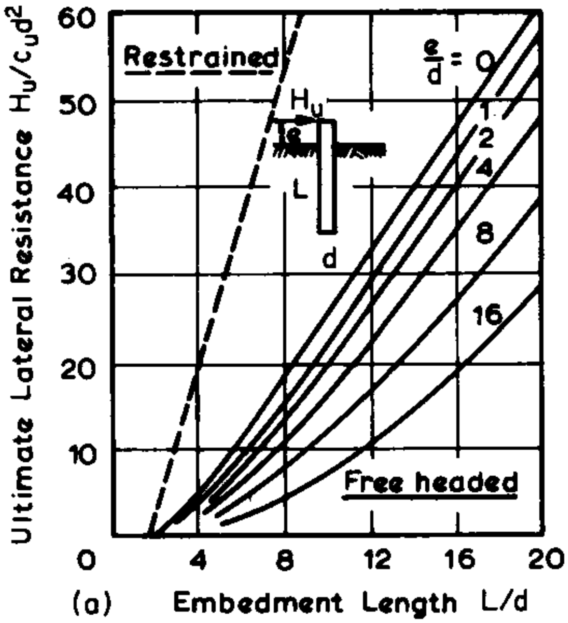
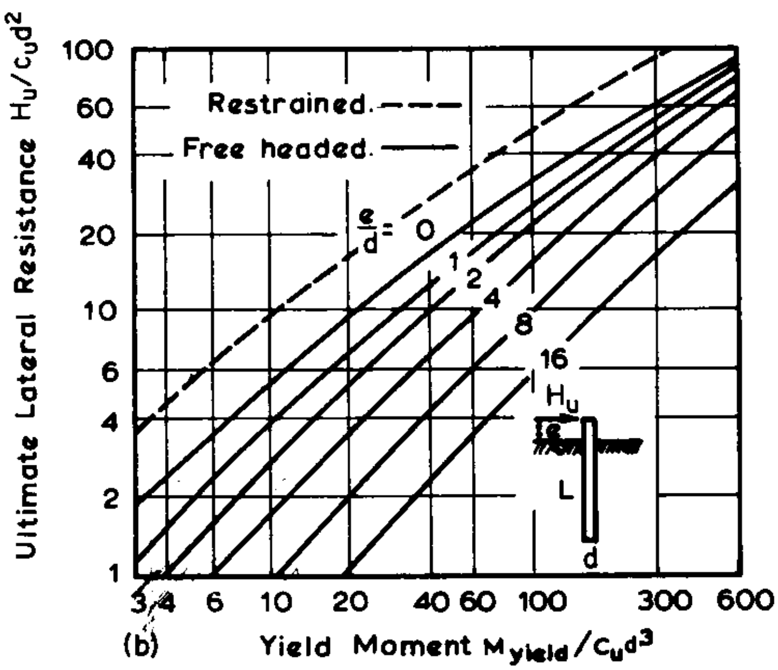


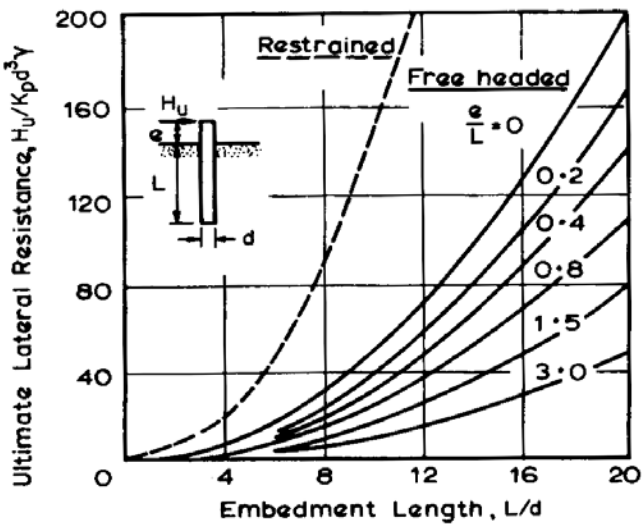
FIG. 4. Settlement of Equivalent Pier in Soil Layer

LECTURE 3
LATERALLY LOADED PILES

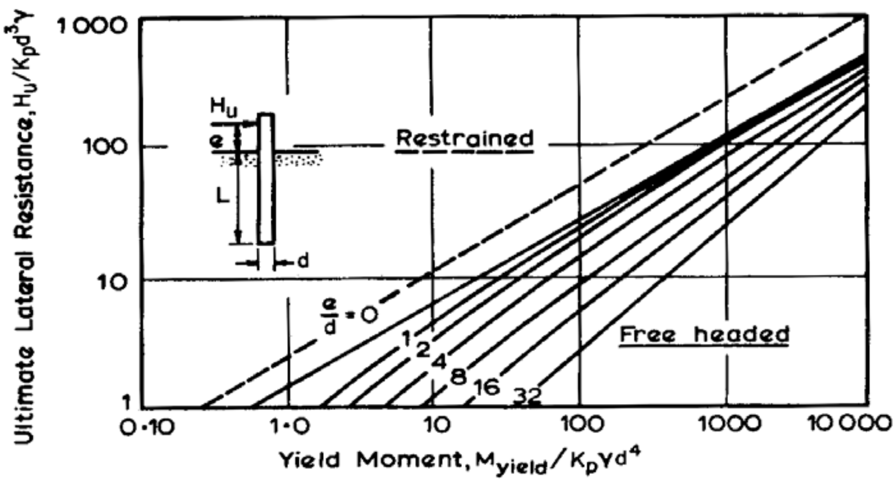




Piles in Clay



(a)



(b)

Piles in Sand

LECTURE 5

PILES SUBJECTED TO GROUND MOVEMENTS

Vertical Ground Movements

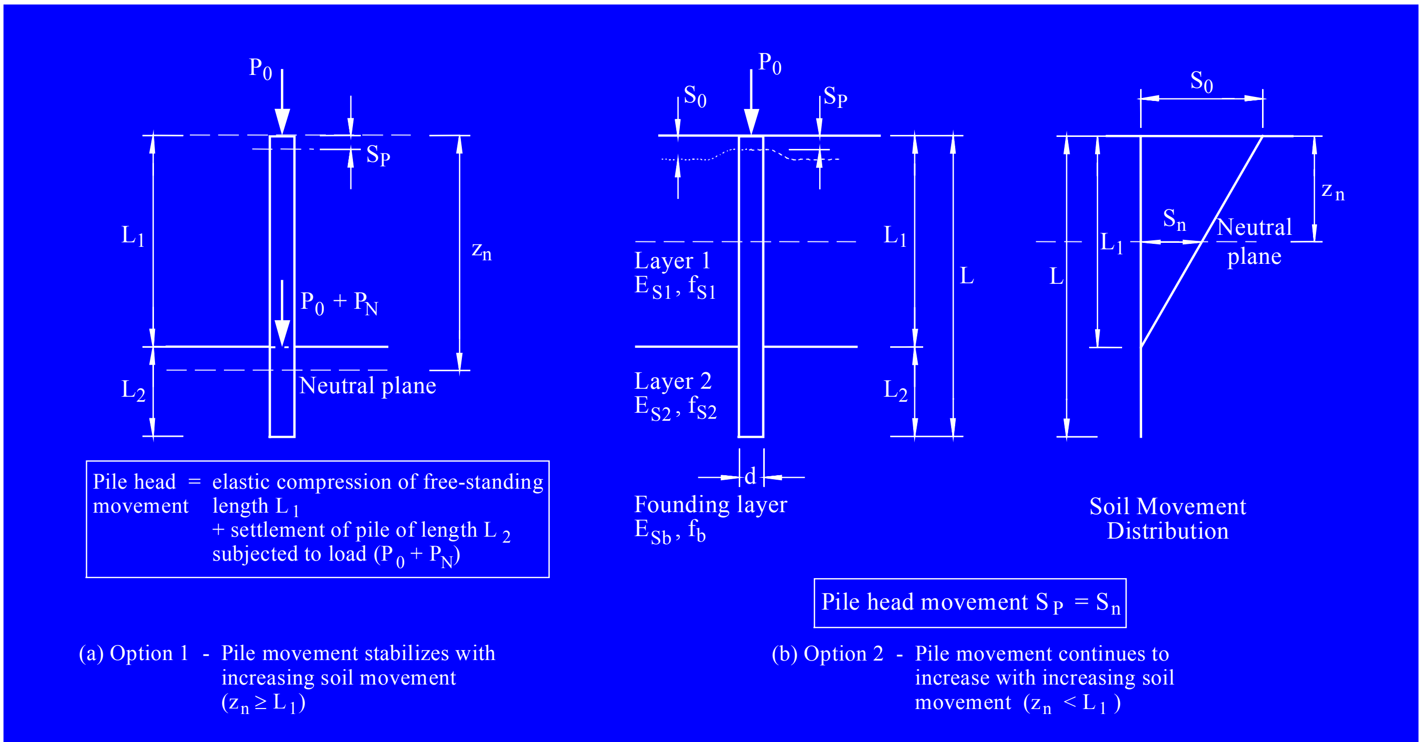


Fig. 2 Simplified approaches to estimating pile head movement

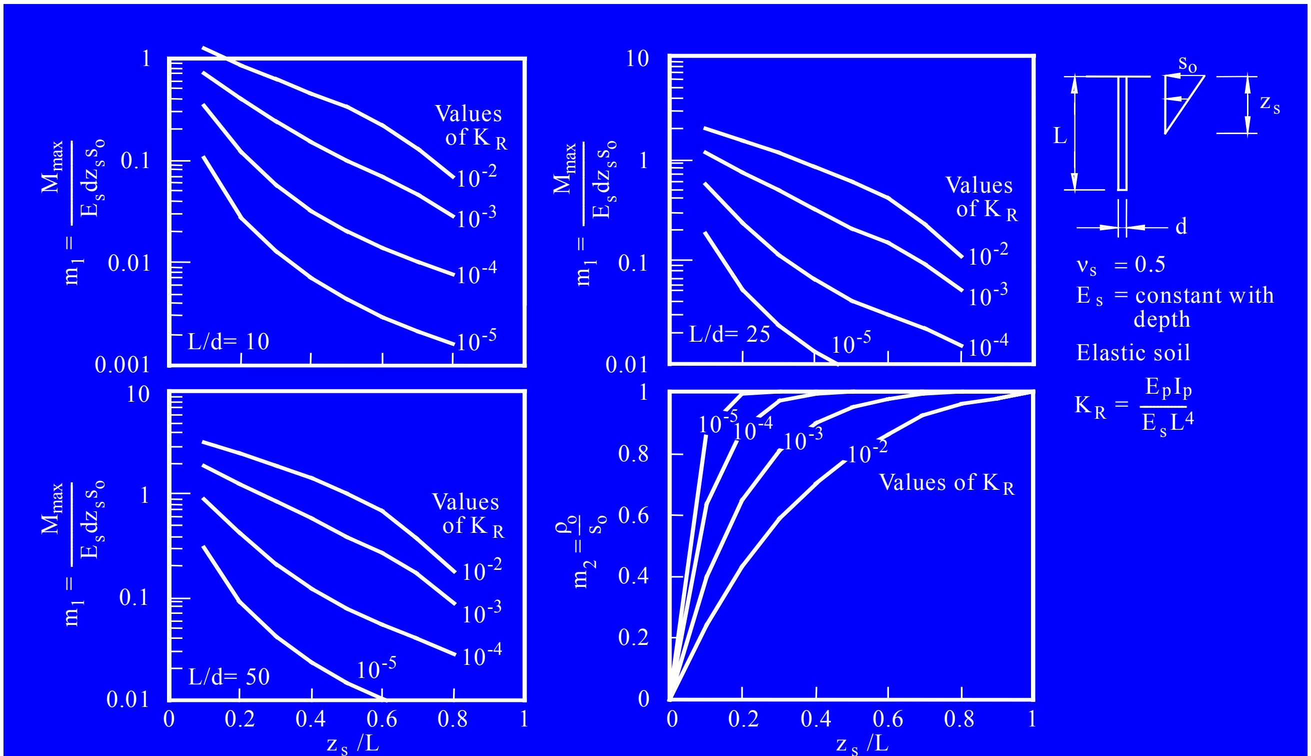


Fig. 5 Elastic solutions for unrestrained free-head pile in uniform soil (linear soil movement profile) (Chen and Poulos, 1997)

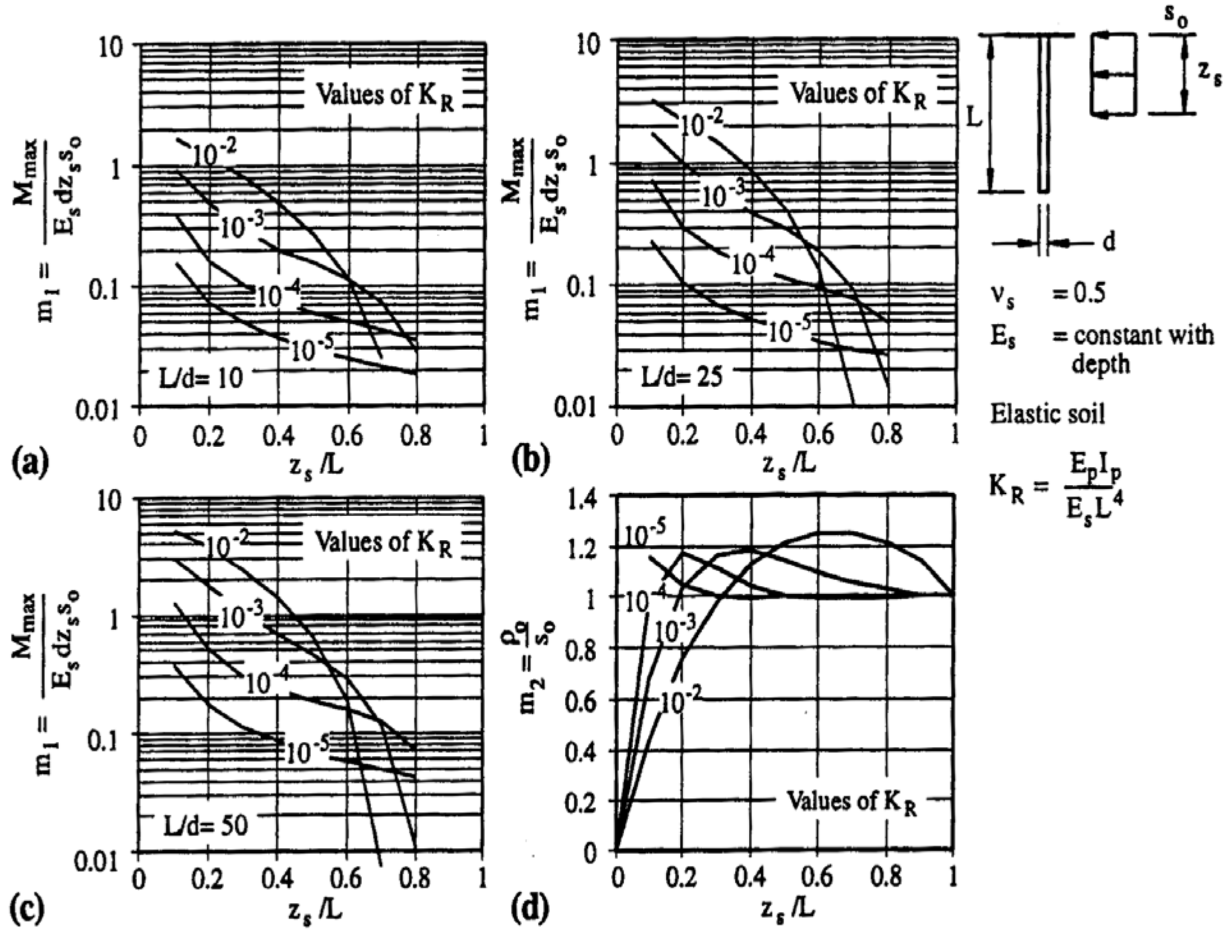


FIG. 4. Elastic Solutions for Unrestrained Free-Head Pile in Uniform Soil (Uniform Soil Movement Profile)

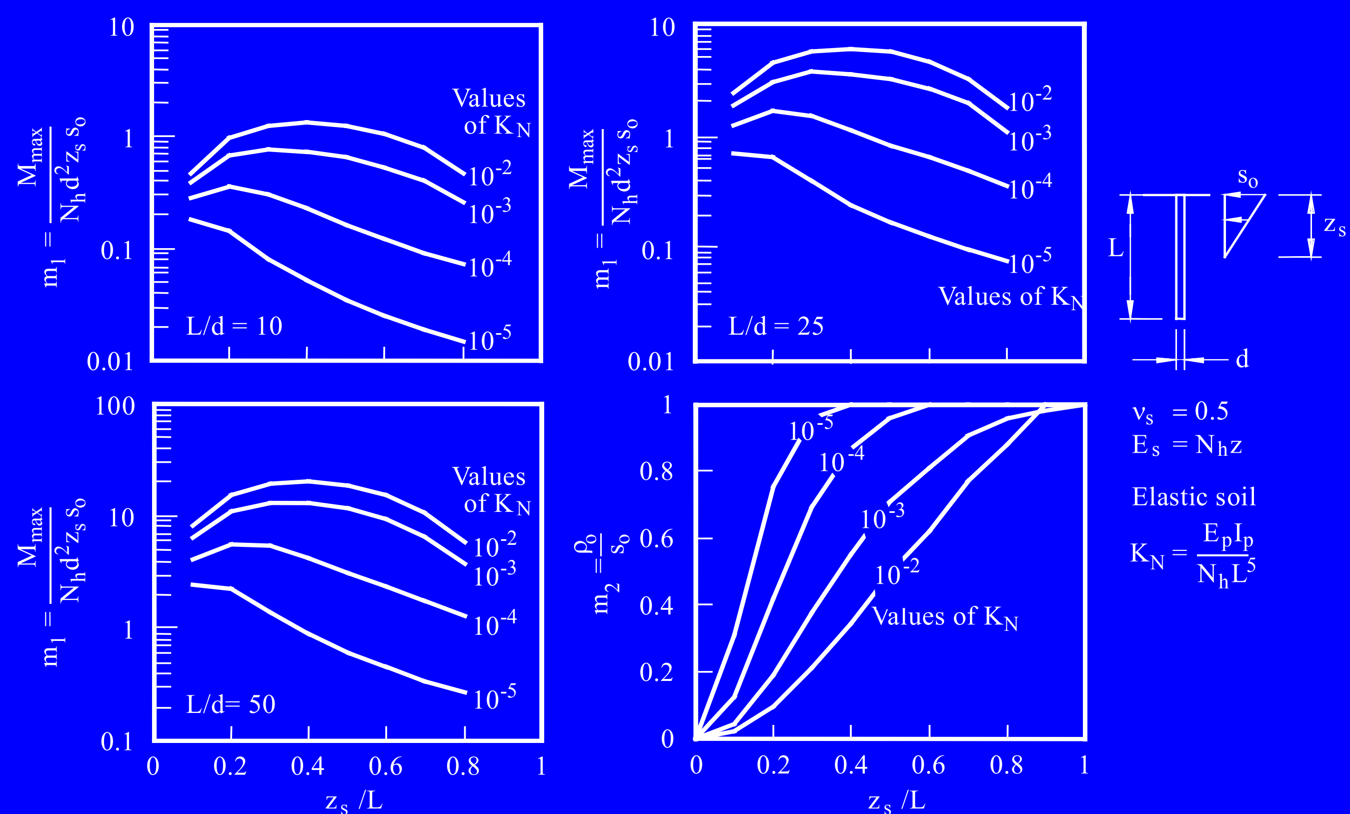


Fig. 6 Elastic solutions for unrestrained free-head pile in Gibson soil
(Linear soil movement profile) (Chen and Poulos, 1997)

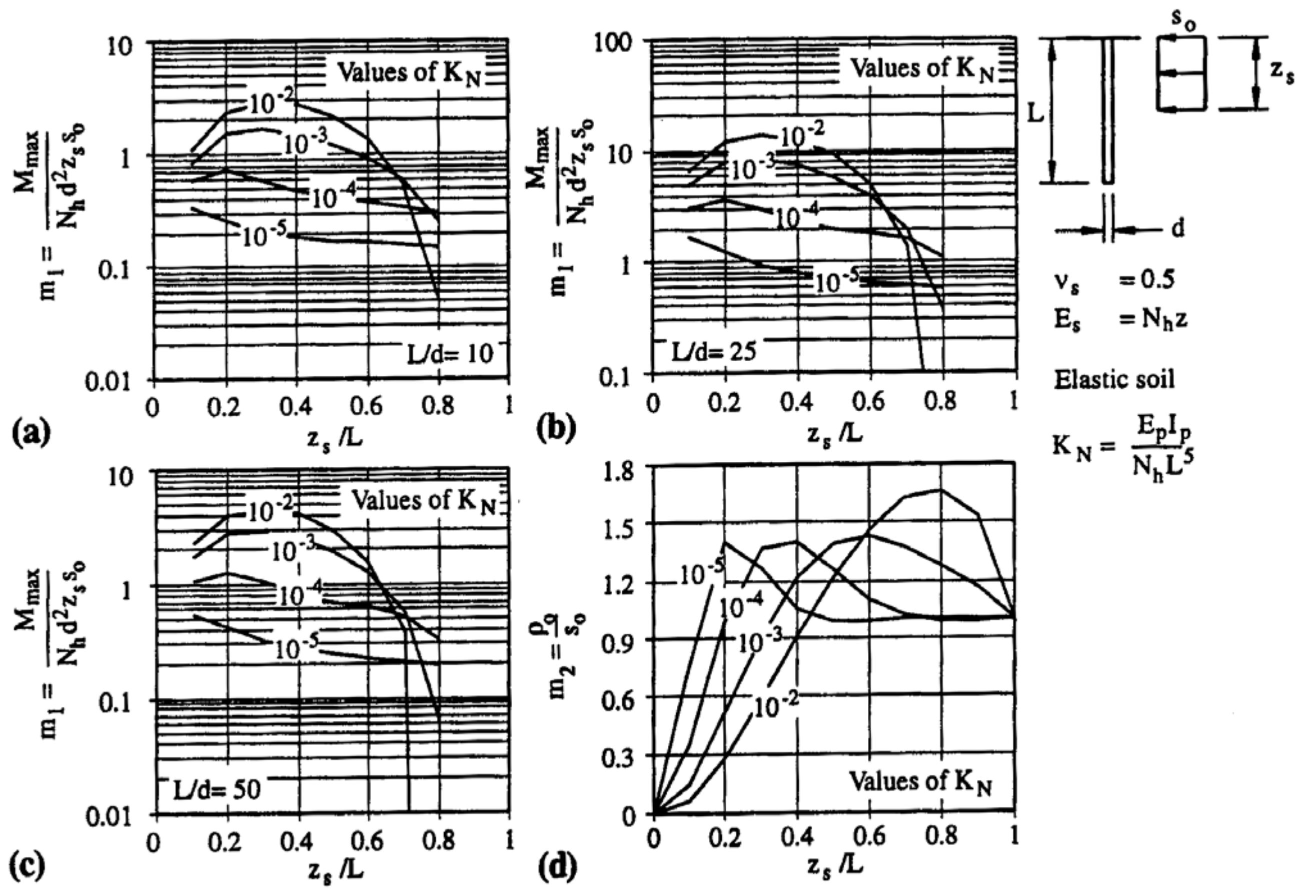


FIG. 5. Elastic Solutions for Unrestrained Free-Head Pile in Gibson Soil (Uniform Soil Movement Profile)

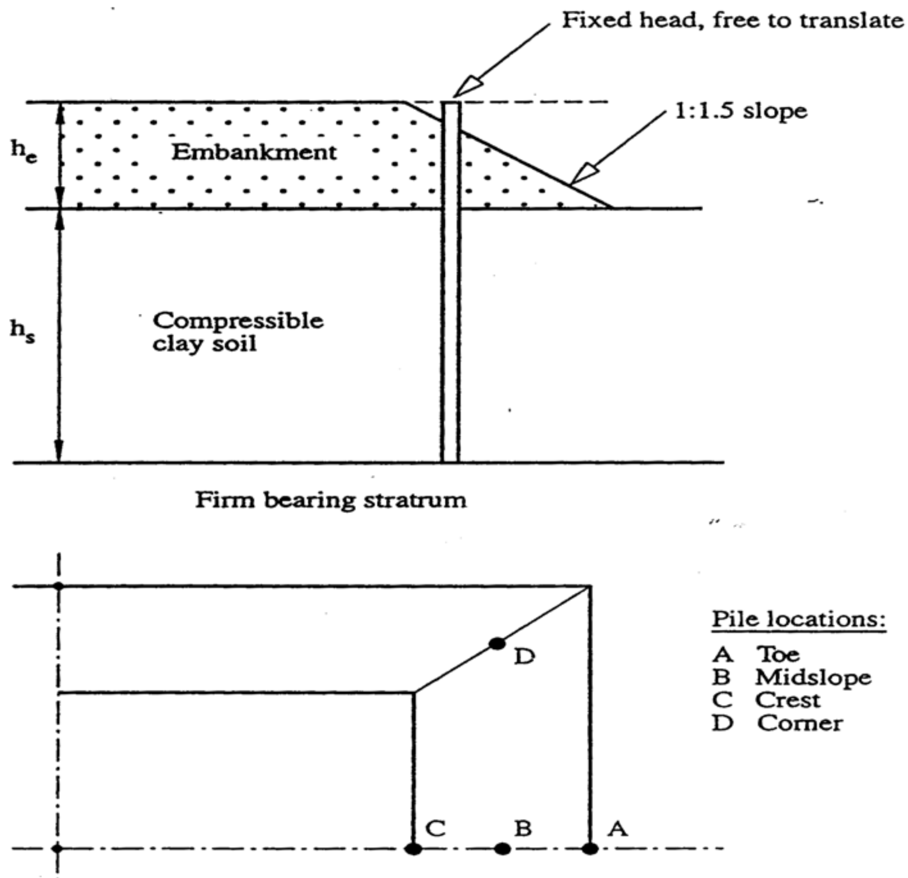


FIG.9 PROBLEM OF PILE WITHIN OR NEAR EMBANKMENT

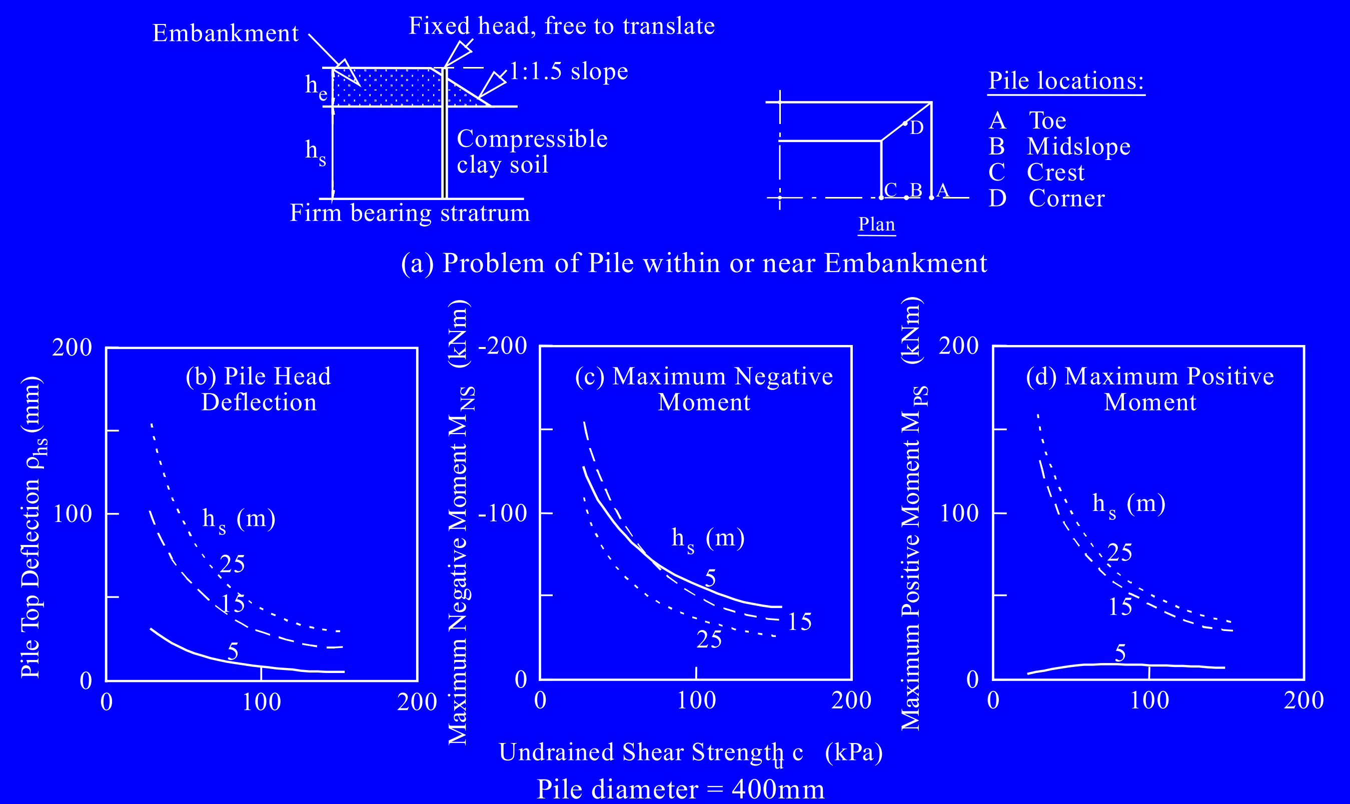


Fig. 9 Design charts for pile at crest of embankment - Embankment height = 8m

LECTURE 6

PILES NEAR TUNNELS AND EXCAVATIONS

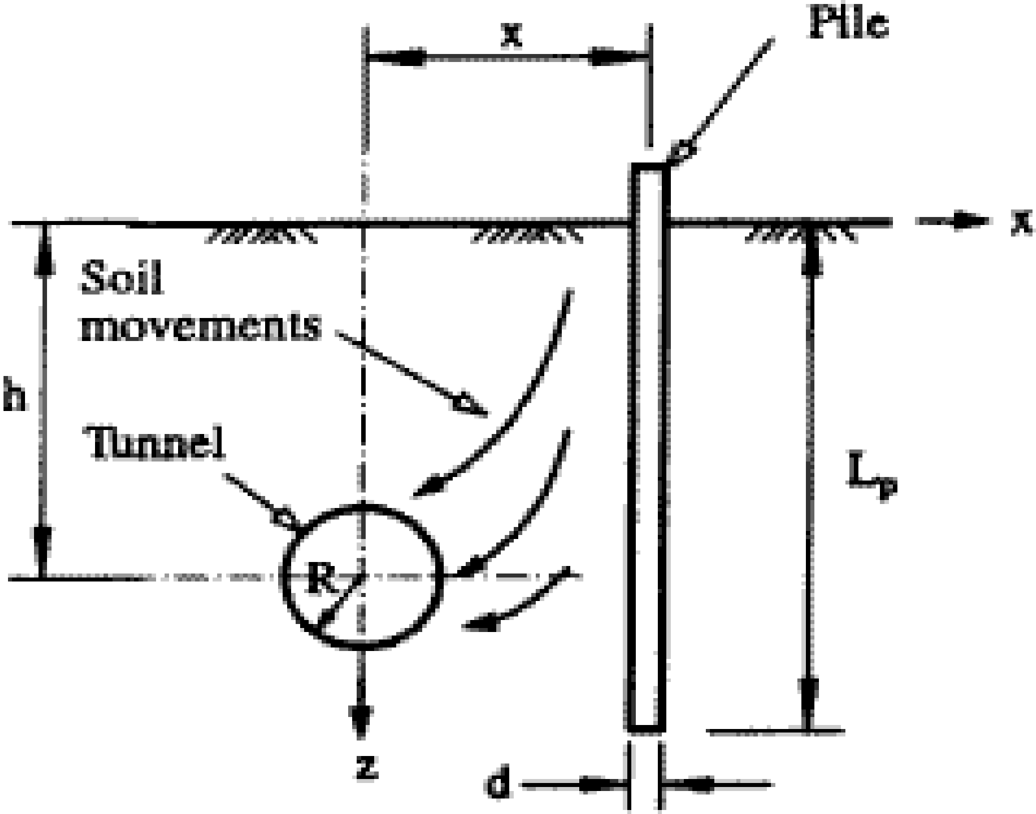


FIG. 1. Pile Adjacent to Tunneling—Basic Problem Analyzed

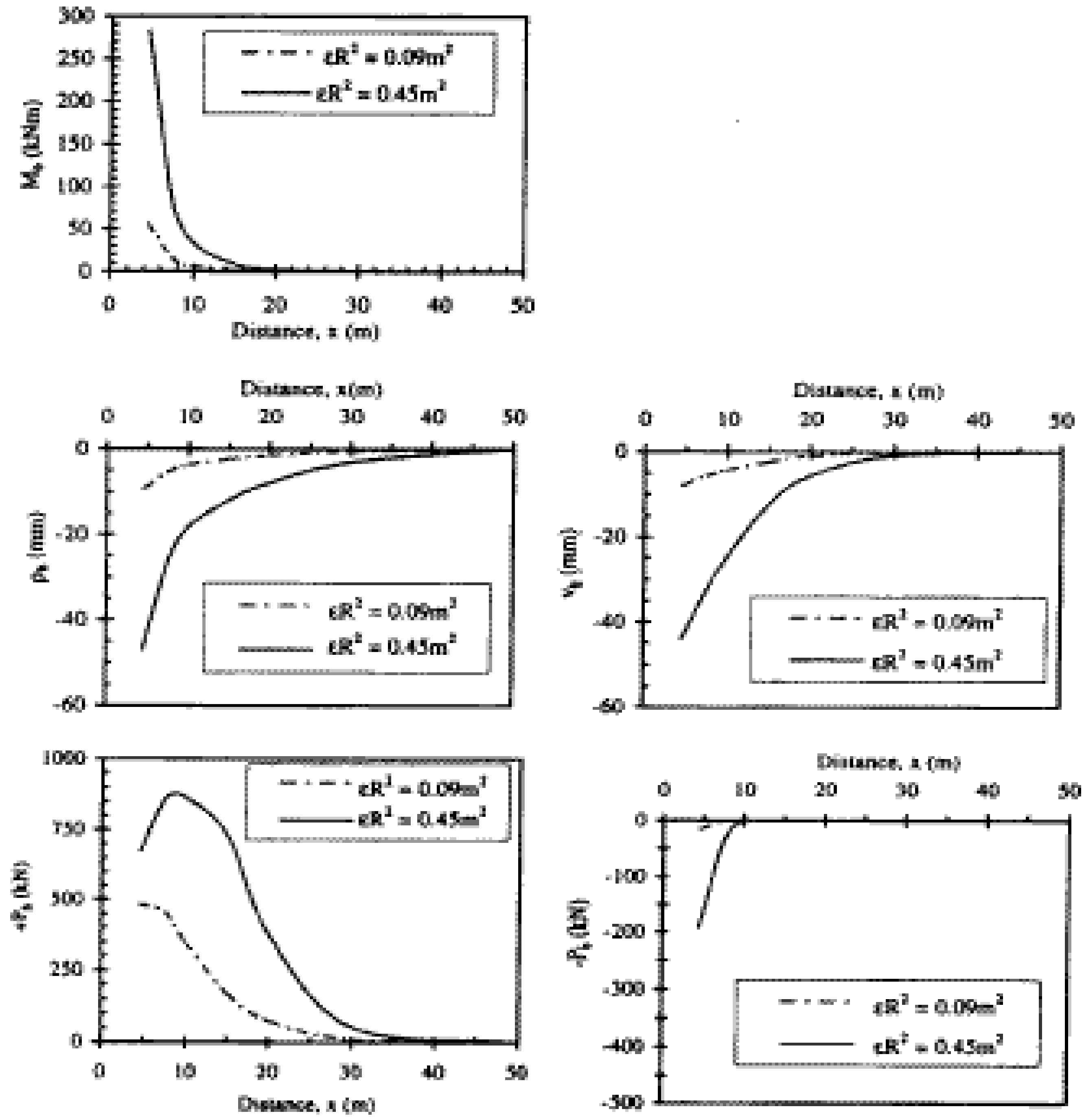


FIG. 4. Maximum Pile Responses versus Distance *x* for Long Pile Case

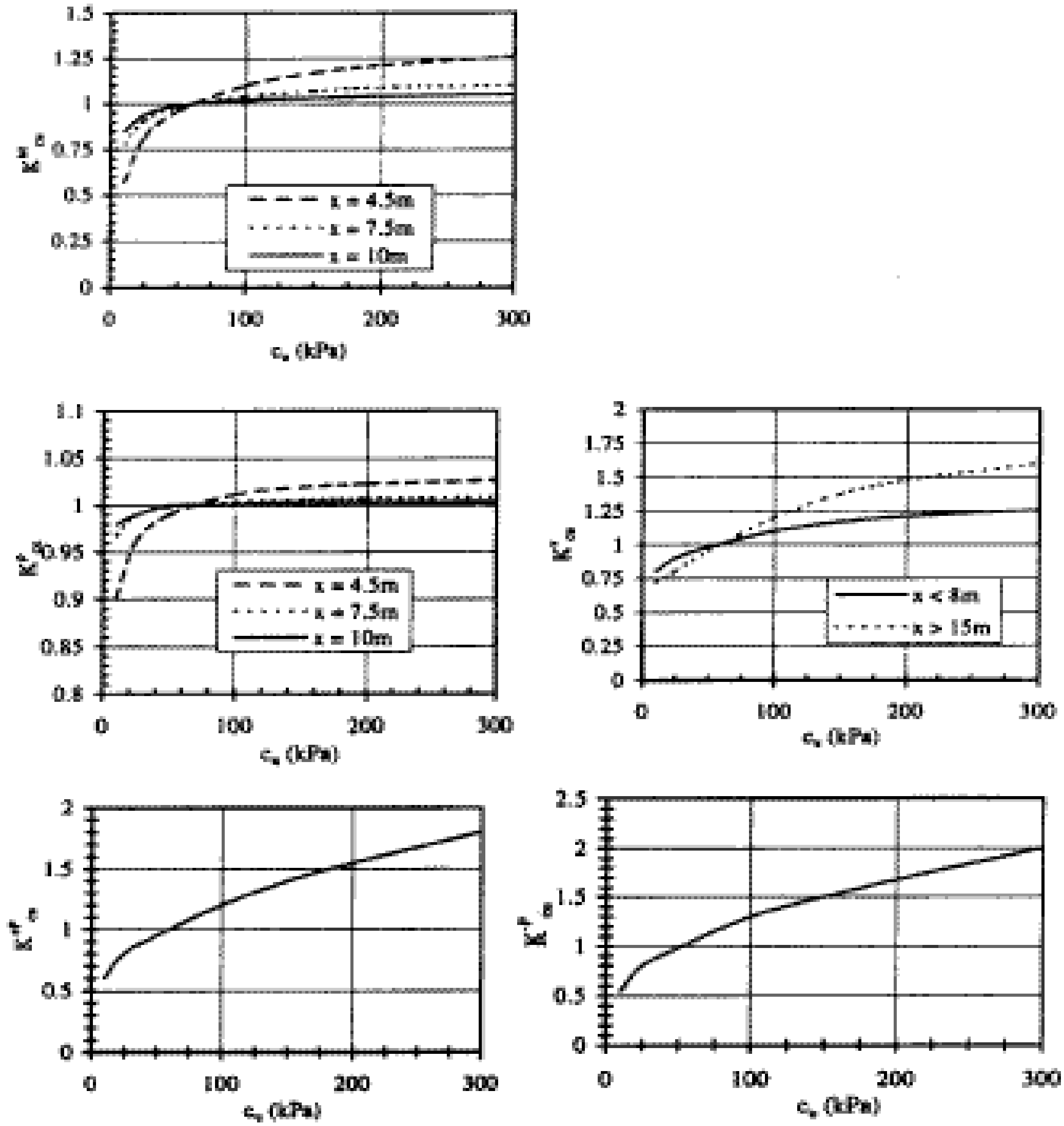


FIG. 6. Correction Factors for c_u

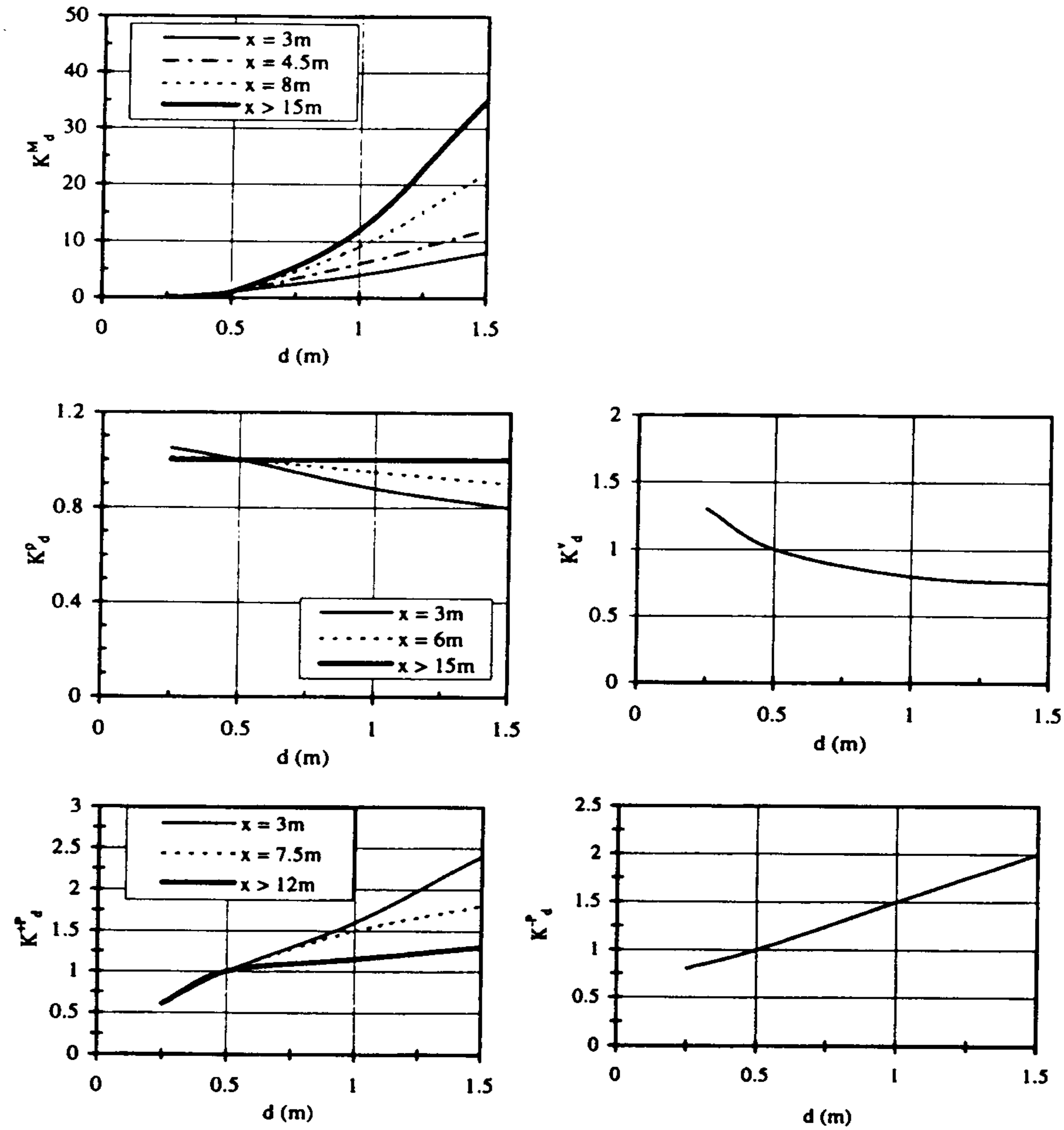


FIG. 7. Correction Factors for *d*

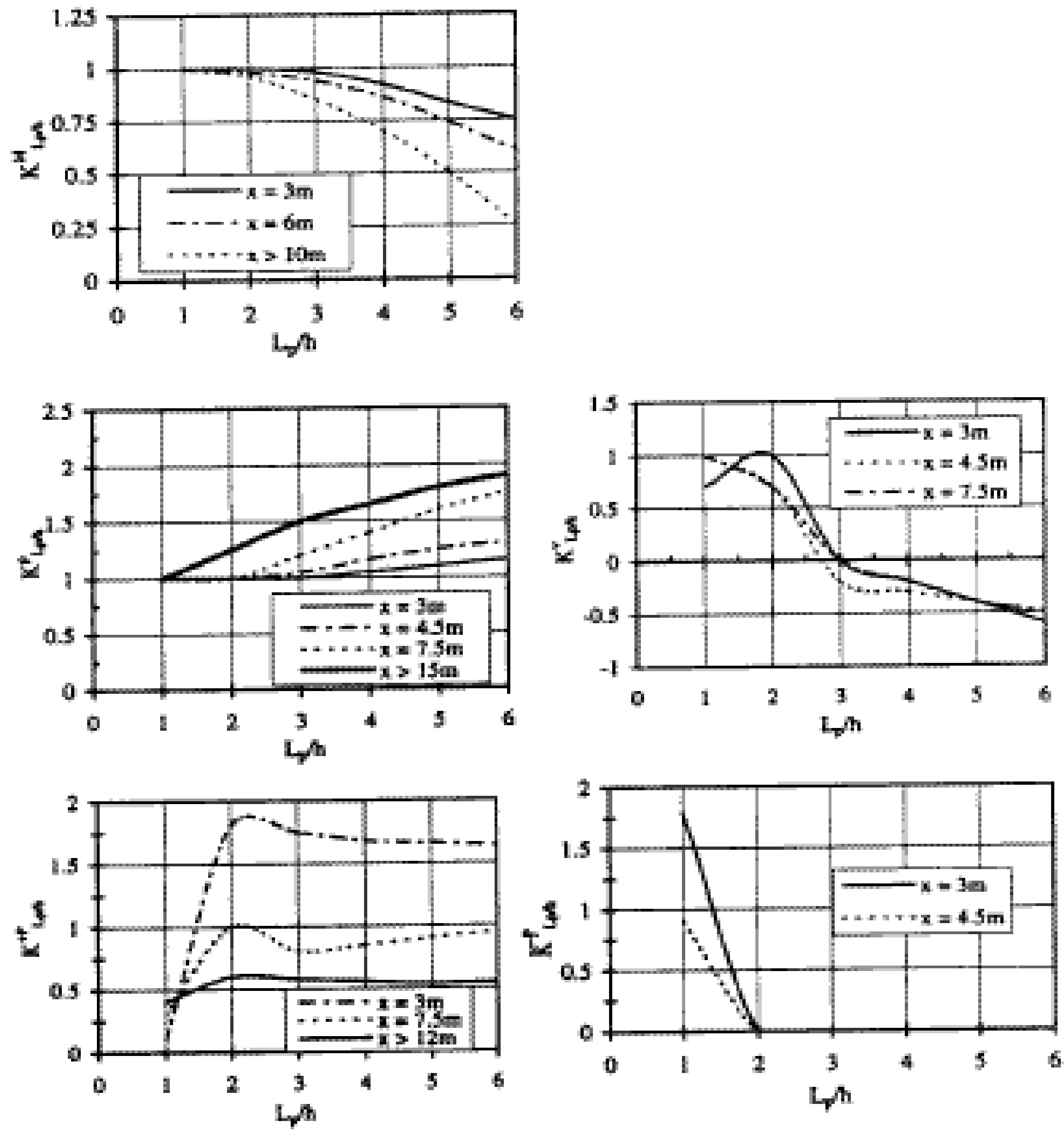


FIG. 8. Correction Factors for L_y/h

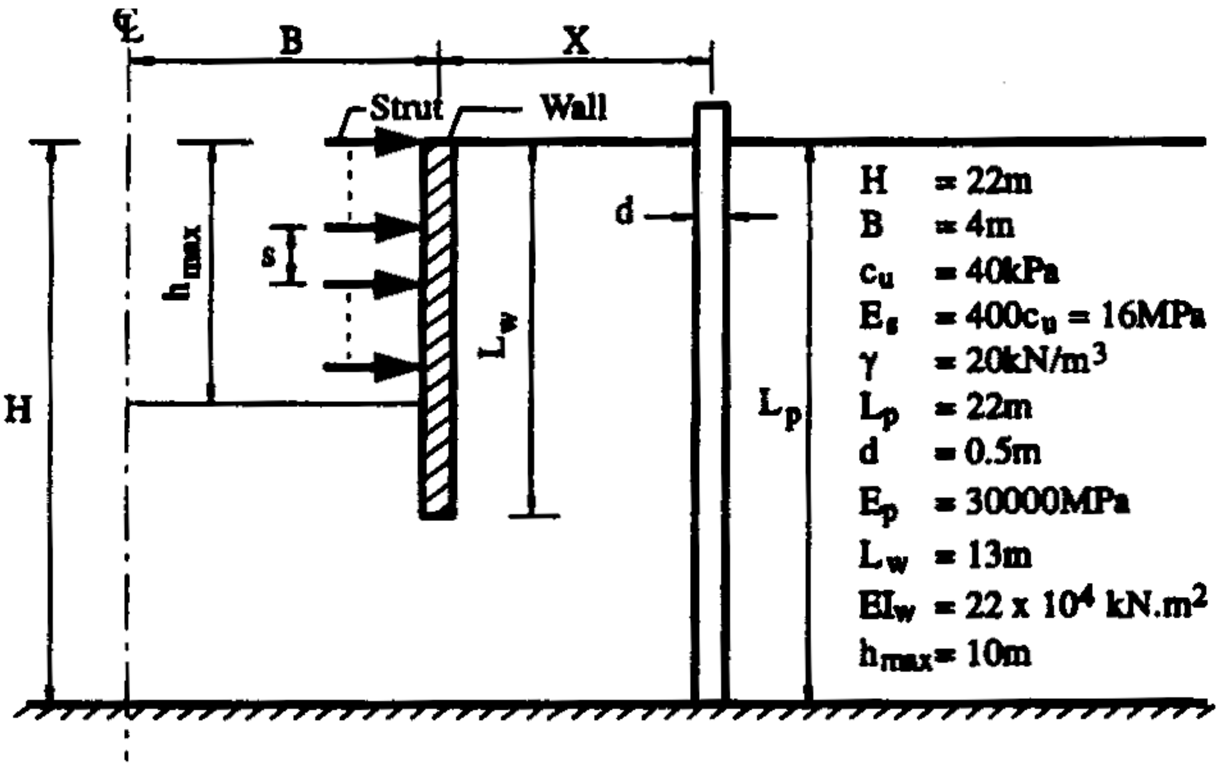


FIG. 1. Basic Problem Analyzed

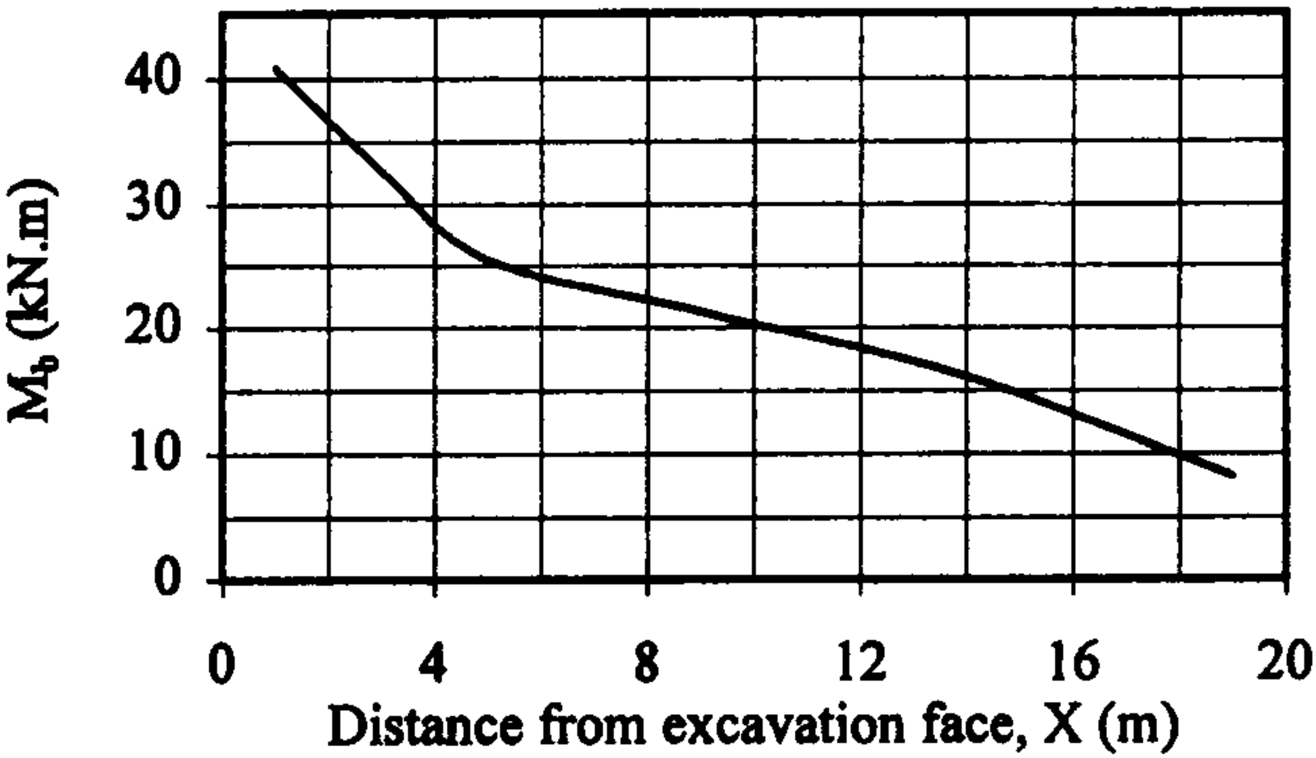


FIG. 7. Basic Bending Moment versus Distance from Excavation Face

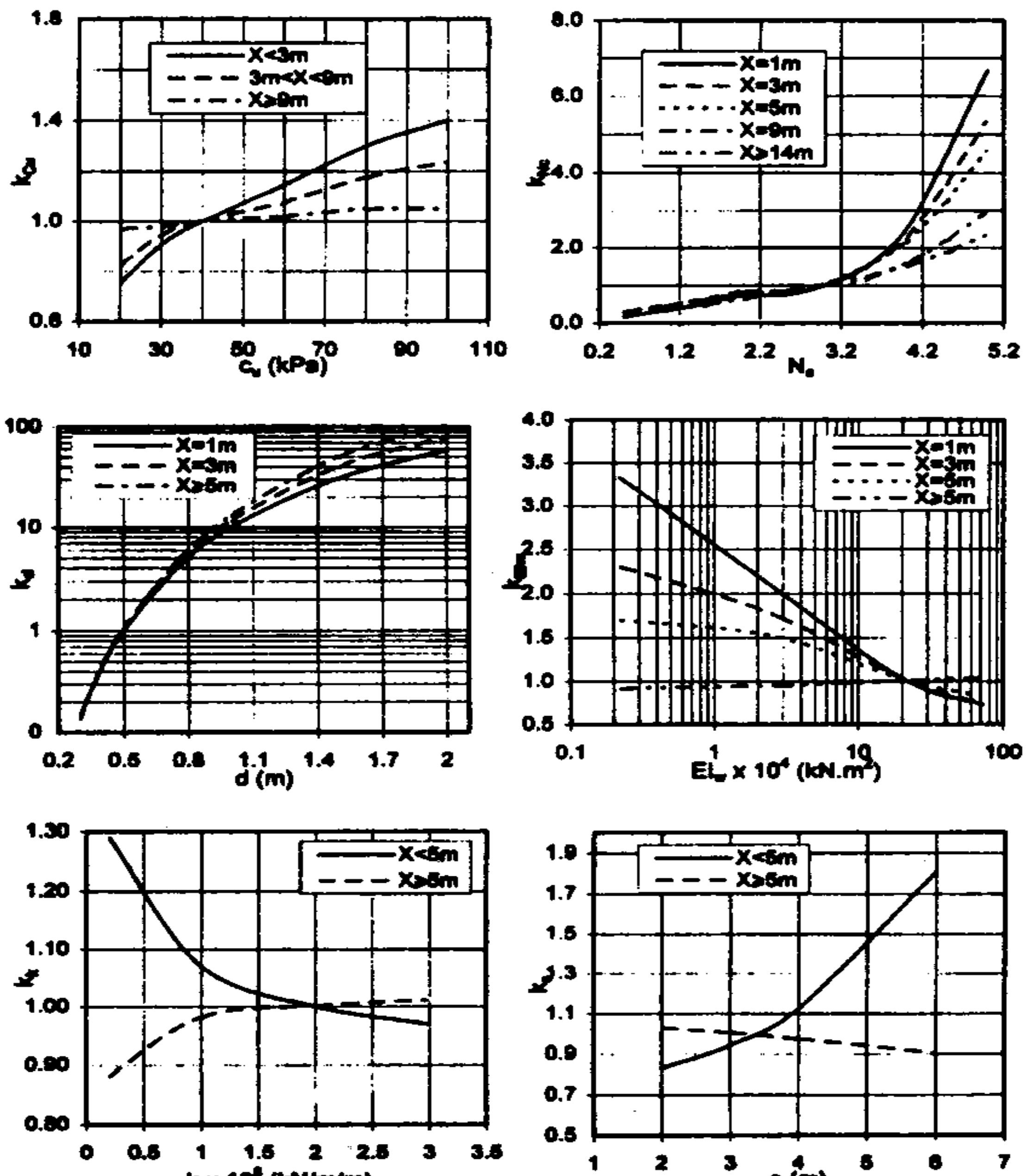


FIG. 8. Correction Factors for Bending Moment

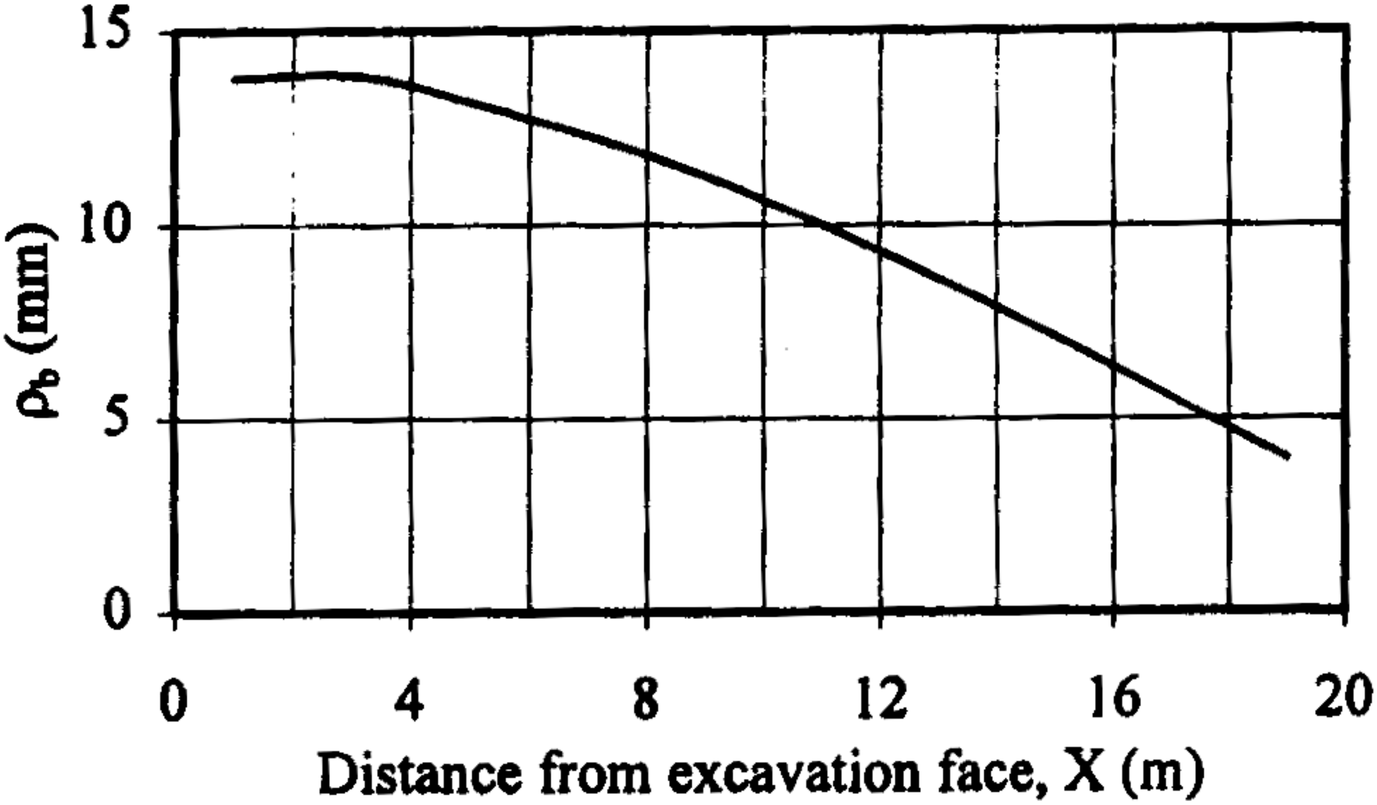


FIG. 9. Basic Deflection versus Distance from Excavation Face

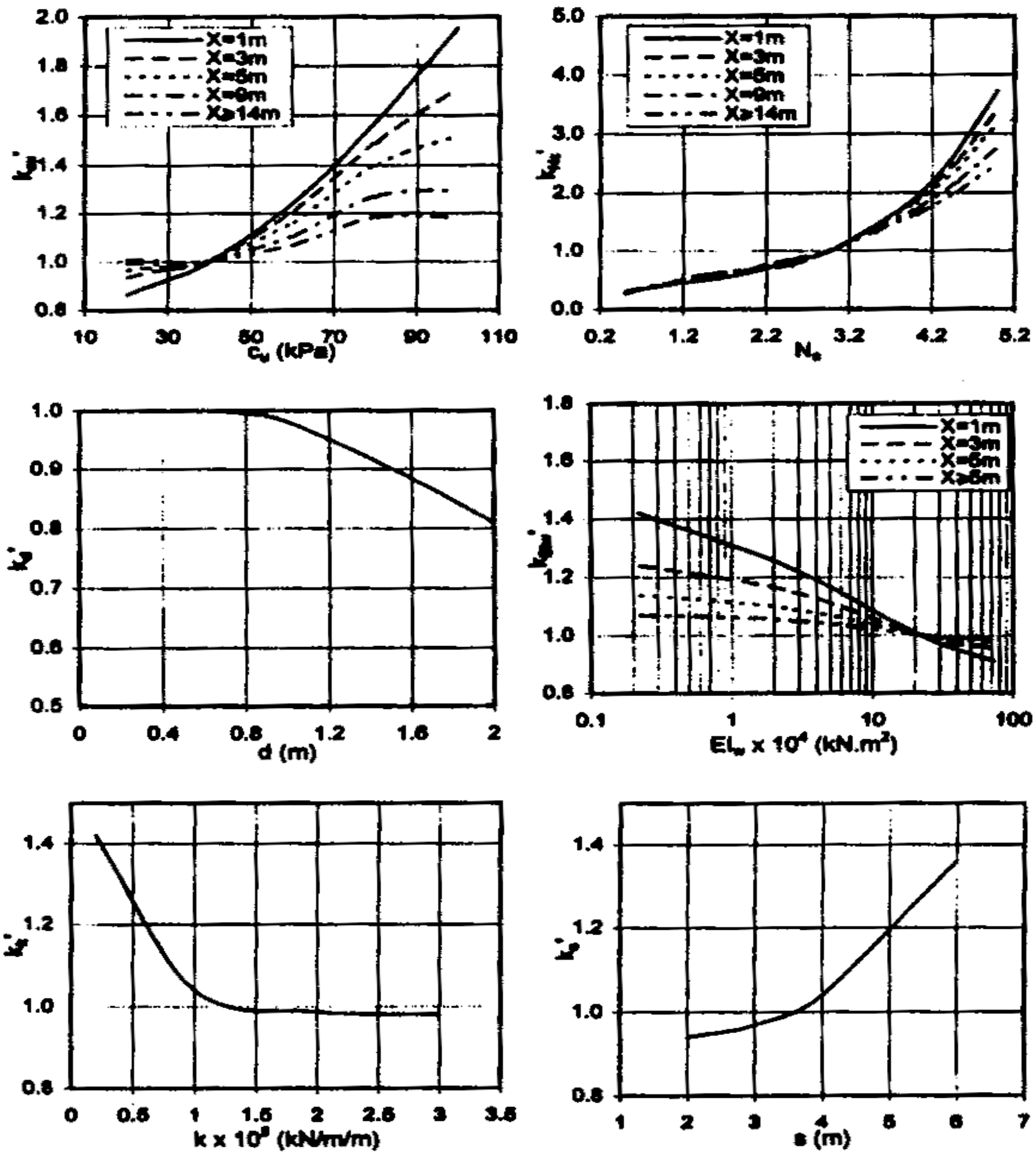
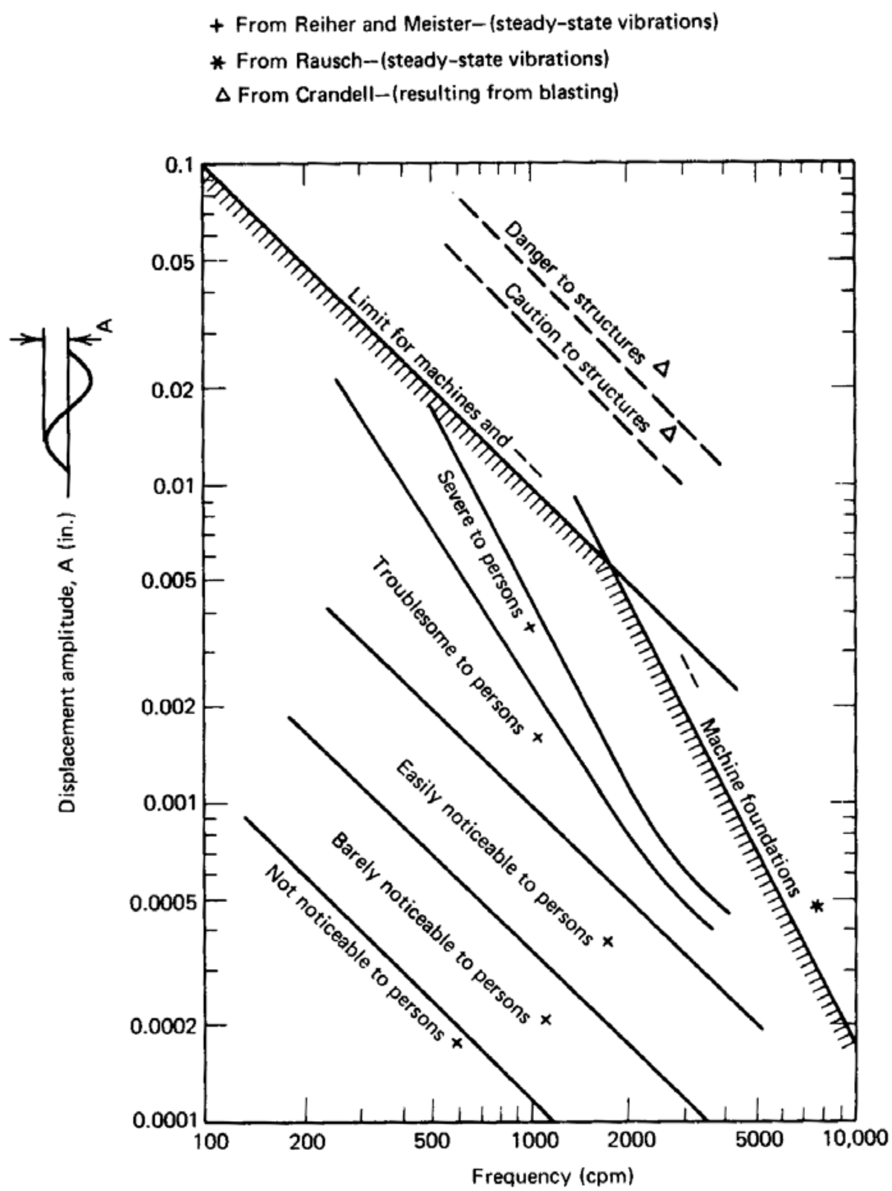


FIG. 10. Correction Factors for Deflection

LECTURE 7

PILES SUBJECTED TO DYNAMIC & EARTHQUAKE LOADING

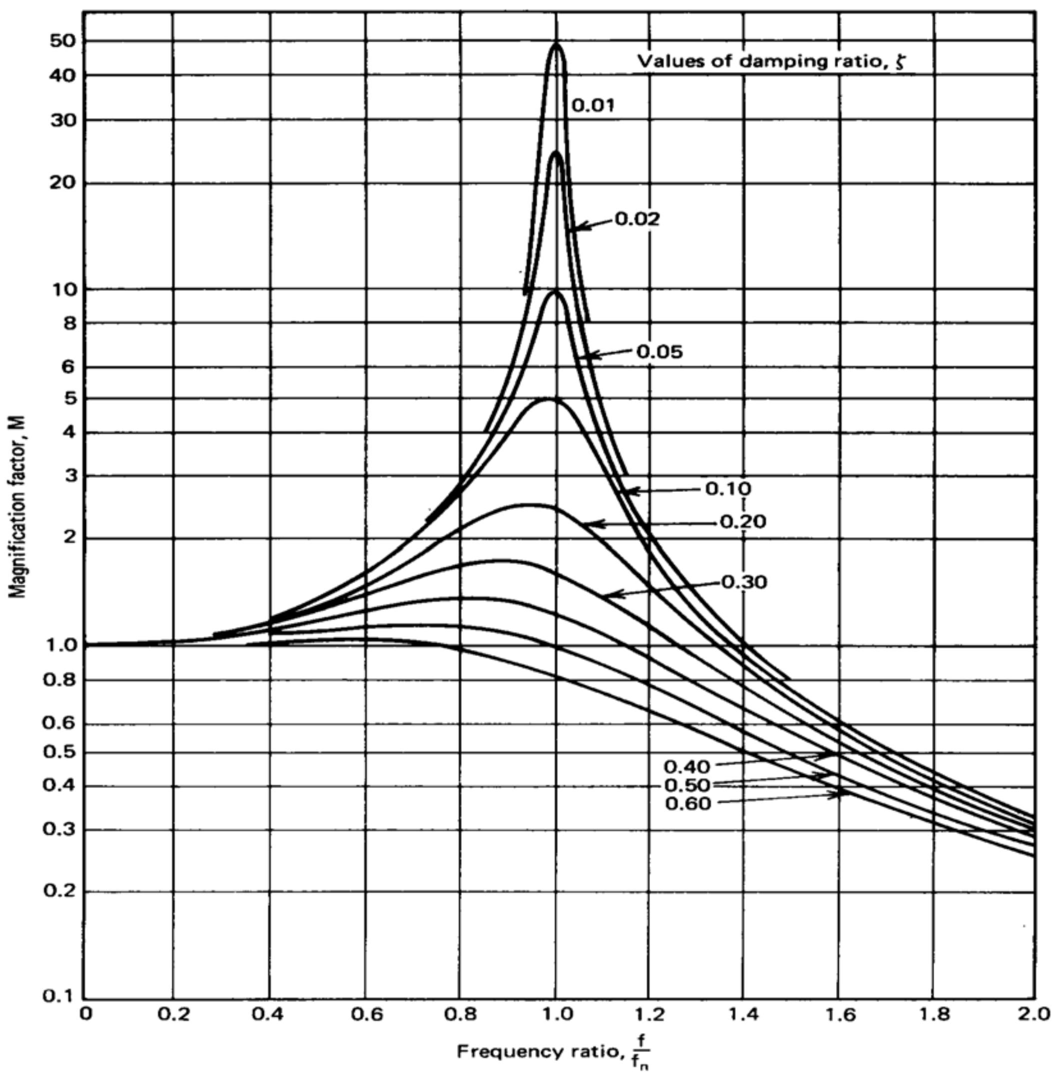


General limits of displacement amplitude for a particular frequency of vibration (from Richart, 1962).

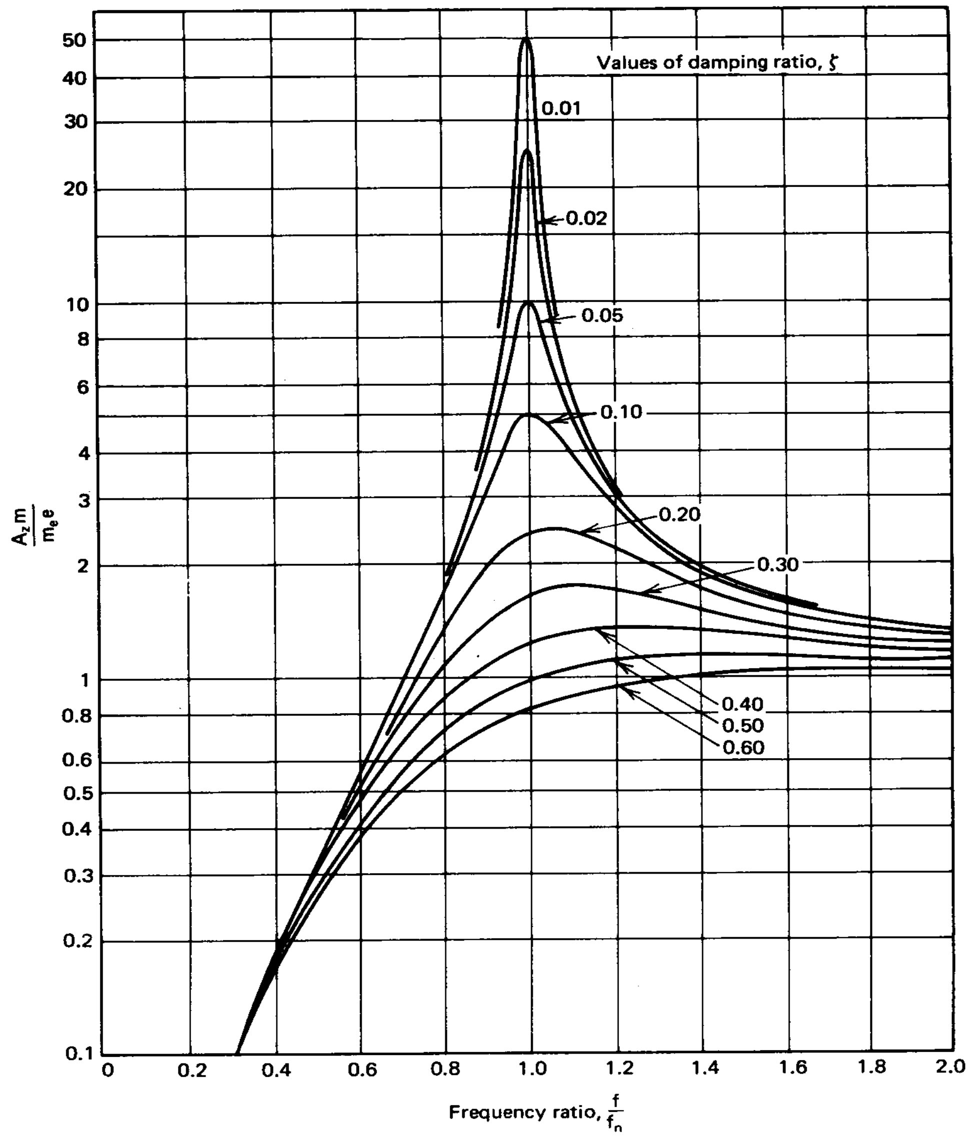
Summary of Relations for Single-Degree-of-Freedom Vibration
(z-coordinate chosen for illustration)

Critical Damping	c_c	=	$2\sqrt{km}$
Dampting Ratio	D	=	$\dfrac{c}{c_c}$
Undamped "Natural Frequency"	f_n	=	$\dfrac{1}{2\pi}\sqrt{\dfrac{k}{m}}$
Static Displacement	z_s	=	$\dfrac{Q_o}{k}$
Amplitude-Magnification Factor During Vibration	M	=	$\left[\left(1 - \dfrac{f^2}{f_n^2}\right)^2 + \left(2D\,\dfrac{f}{f_n}\right)^2\right]^{-1/2}$

For Constant-Force Excitation (Q_o = constant)	For Rotating-Mass Excitation ($Q_o = m_e e \omega^2$)
Amplitude at Frequency f	
$$A_z = \frac{Q_o}{k} M$$	$$A_z = \frac{m_e e}{m}\left(\frac{f}{f_n}\right)^2 M$$
Maximum Amplitude of Vibration	
$$A_{zm} = \frac{Q_o}{k}\frac{1}{2D\sqrt{1-D^2}}$$	$$A_{zm} = \frac{m_e e}{m}\frac{1}{2D\sqrt{1-D^2}}$$
Frequency for Maximum Amplitude	
$$f_m = f_n\sqrt{1-2D^2}$$	$$f_m = f_n\frac{1}{\sqrt{1-2D^2}}$$



Constant Force Excitation - Response Curves



Rotating Mass Excitation - Response Curves

PILE RESPONSE TO DYNAMIC LOADING - SOLUTIONS OF NOVAK (1974)

Vertical translation:

$$k_v = \frac{E_p A}{R} f_{v1} \quad , \quad c_v = \frac{E_p A}{V_s} f_{v2}$$

Horizontal translation:

$$k_u = \frac{E_p I}{R^3} f_{u1} \quad , \quad c_u = \frac{E_p I}{R^2 V_s} f_{u2}$$

Rotation of the pile head in the vertical plane:

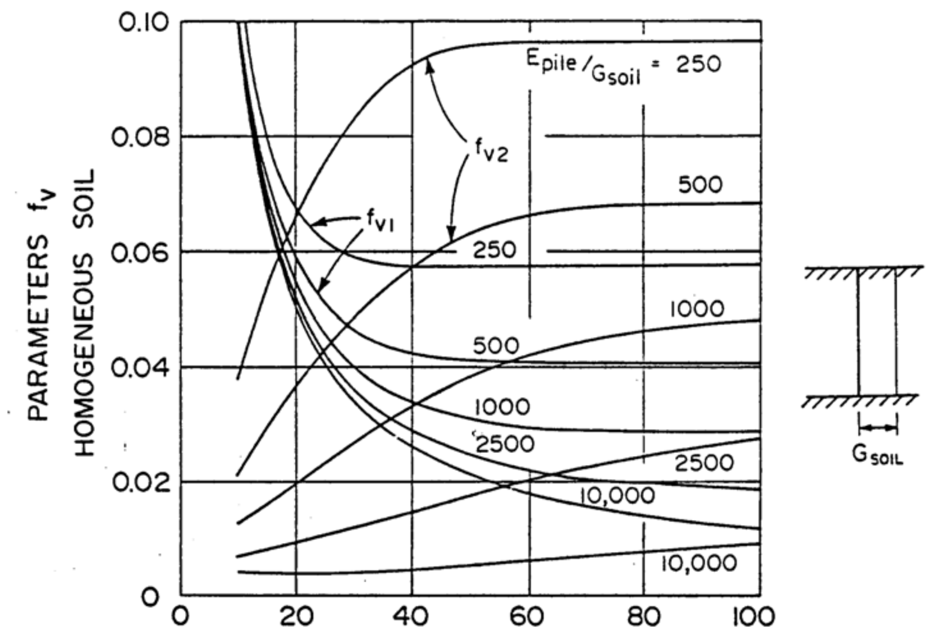
$$k_\psi = \frac{E_p I}{R} f_{\psi 1} \quad , \quad c_\psi = \frac{E_p I}{V_s} f_{\psi 2}$$

Coupling between horizontal translation and rotation:

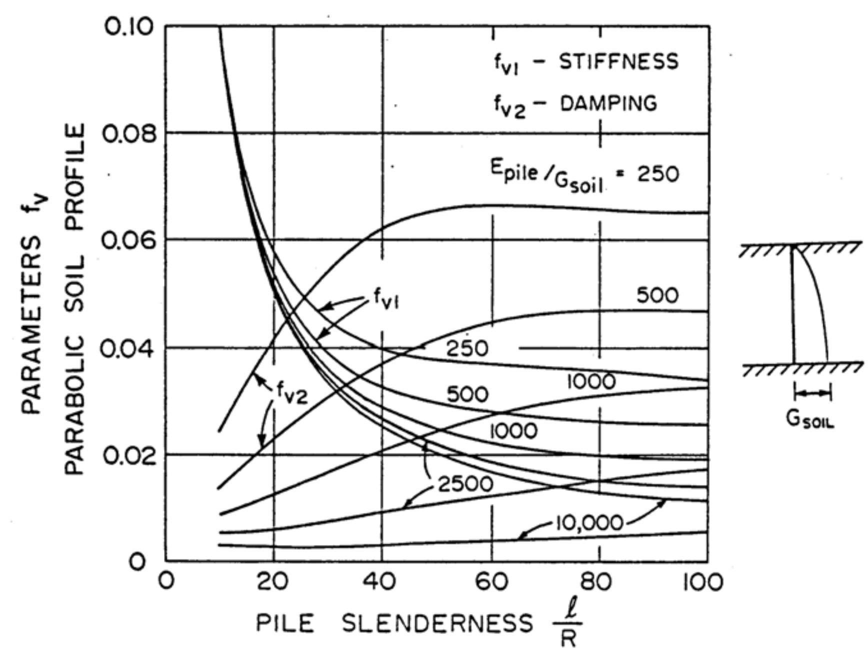
$$k_c = \frac{E_p I}{R^2} f_{c1} \quad , \quad c_c = \frac{E_p I}{R V_s} f_{c2}$$

Torsion:

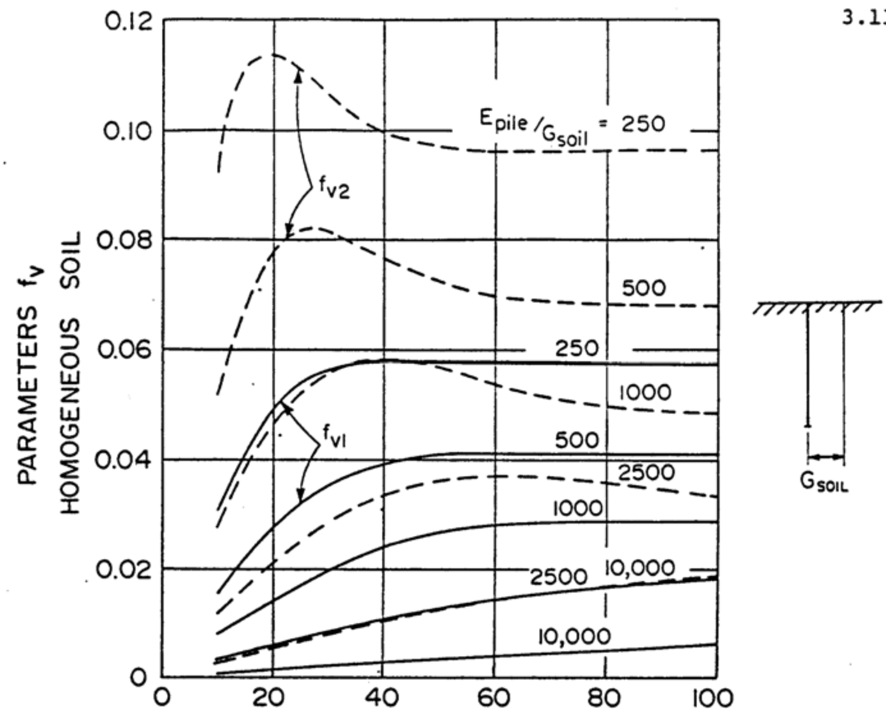
$$k_\eta = \frac{G_p J}{R} f_{\eta 1} \quad , \quad c_\eta = \frac{G_p J}{V_s} f_{\eta 2}$$



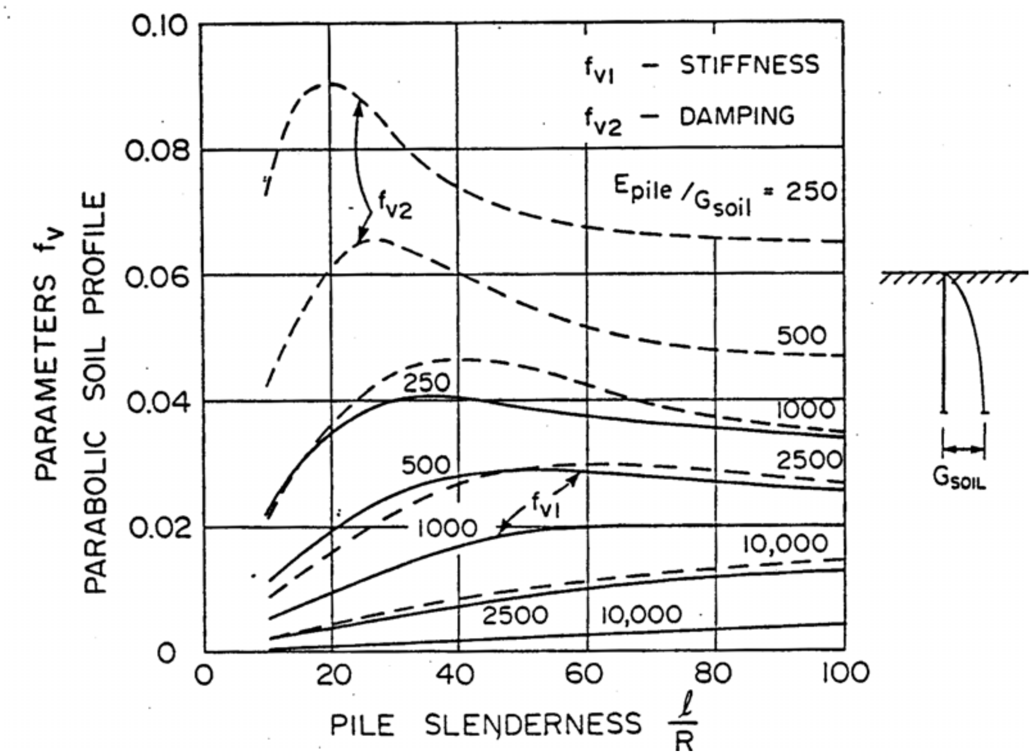
Uniform Soil Modulus



Parabolically Increasing Soil Modulus



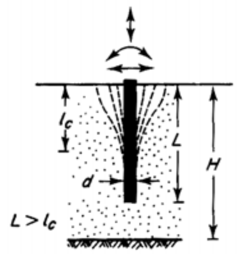
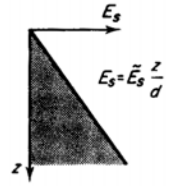
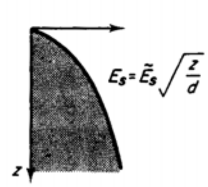
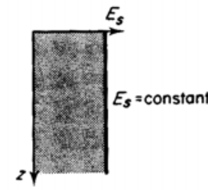
Uniform Soil Modulus



Parabolically Increasing Soil Modulus

SIMPLIFIED SOLUTIONS FOR PILE STIFFNESS, DAMPING AND INTERACTION (GAZETAS, 1991)

TABLE 15.6 DYNAMIC STIFFNESSES AND DAMPING COEFFICIENTS FOR FLEXIBLE PILES ($L > l_c$).

	*Linear Increase of Soil Modulus with Depth**  $\tilde{V}_s = [\tilde{E}_s / 2\rho(1+\nu)]^{\frac{1}{2}}$	*Parabolic Increase of Soil Modulus with Depth**  $\tilde{V}_s = [\tilde{E}_s / 2\rho(1+\nu)]^{\frac{1}{2}}$	*Constant Soil Modulus at All Depths* 
"Active length"	$l_c \approx 2d(E_p/\tilde{E}_s)^{0.20}$	$l_c \approx 2d(E_p/\tilde{E}_s)^{0.22}$	$l_c \approx 2d(E_p/E_s)^{0.25}$
Natural shear frequency of deposit	$f_s = 0.19V_{sH}/H$ where V_{sH} = the S-wave velocity at depth $z = H$ (bottom of stratum)	$f_s = 0.223V_{sH}/H$ where V_{sH} = the S-wave velocity at depth $z = H$ (bottom of stratum)	$f_s = 0.25V_s/H$
Static lateral (swaying) stiffness	$K_{HH} = 0.6d\tilde{E}_s(E_p/\tilde{E}_s)^{0.35}$	$K_{HH} = 0.8d\tilde{E}_s(E_p/\tilde{E}_s)^{0.28}$	$K_{HH} = dE_s(E_p/E_s)^{0.21}$
Lateral (swaying) stiffness coefficient	$k_{HH} \approx 1$	$k_{HH} \approx 1$	$k_{HH} \approx 1$
Lateral (swaying) coefficient: $C_{HH} = 2K_{HH}D_{HH}/\omega$	$D_{HH} \approx 0.60\beta + 1.80fd\tilde{V}_s^{-1}$, for $f > f_s$ $D_{HH} \approx 0.60\beta$, for $f \leq f_s$	$D_{HH} \approx 0.70\beta + 1.20fd(E_p/\tilde{E}_s)^{0.08}\tilde{V}_s^{-1}$, for $f > f_s$ $D_{HH} \approx 0.70\beta$, for $f \leq f_s$	$D_{HH} \approx 0.80\beta + 1.10fd(E_p/E_s)^{0.17}V_s^{-1}$, for $f > f_s$ $D_{HH} \approx 0.80\beta$, for $f \leq f_s$
Static rocking stiffness	$K_{MM} = 0.15d^3\tilde{E}_s(E_p/\tilde{E}_s)^{0.80}$	$K_{MM} = 0.15d^3\tilde{E}_s(E_p/\tilde{E}_s)^{0.77}$	$K_{MM} = 0.15d^3E_s(E_p/E_s)^{0.75}$
Rocking stiffness coefficient	$k_{MM} \approx 1$	$k_{MM} \approx 1$	$k_{MM} \approx 1$
Rocking dashpot coefficient: $C_{MM} = 2K_{MM}D_{MM}/\omega$	$D_{MM} \approx 0.20\beta + 0.40fd\tilde{V}_s^{-1}$, for $f > f_s$ $D_{MM} \approx 0.20\beta$, for $f \leq f_s$	$D_{MM} \approx 0.22\beta + 0.35fd(E_p/\tilde{E}_s)^{0.10}\tilde{V}_s^{-1}$, for $f > f_s$ $D_{MM} \approx 0.22\beta$, for $f \leq f_s$	$D_{MM} \approx 0.35\beta + 0.35fd(E_p/E_s)^{0.20}V_s^{-1}$, for $f > f_s$ $D_{MM} \approx 0.25\beta$, for $f \leq f_s$
Static swaying–rocking cross-stiffness	$K_{HM} = K_{MH} = -0.17d^2\tilde{E}_s(E_p/\tilde{E}_s)^{0.60}$	$K_{HM} = K_{MH} = -0.24d^2\tilde{E}_s(E_p/\tilde{E}_s)^{0.53}$	$K_{HM} = K_{MH} = -0.22d^2E_s(E_p/E_s)^{0.50}$
Swaying–rocking cross-stiffness coefficient	$k_{HM} = k_{MH} \approx 1$	$k_{HM} = k_{MH} \approx 1$	$k_{HM} = k_{MH} \approx 1$
Swaying–rocking dashpot coefficient: $C_{HM} = 2K_{HM}D_{HM}/\omega$	$D_{HM} \approx 0.30\beta + fd\tilde{V}_s^{-1}$, for $f > f_s$ $D_{HM} \approx 0.30\beta$, for $f \leq f_s$	$D_{HM} \approx 0.60\beta + 0.70fd(E_p/\tilde{E}_s)^{0.05}\tilde{V}_s^{-1}$, for $f > f_s$ $D_{HM} \approx 0.35\beta$, for $f \leq f_s$	$D_{HM} \approx 0.80\beta + 0.85fd(E_p/E_s)^{0.18}V_s^{-1}$, for $f > f_s$ $D_{HM} \approx 0.50\beta$, for $f \leq f_s$

	The axial stiffness of a pile depends not only on its relative compressibility (E_p/E_s) but also on the slenderness ratio L/d and the tip support conditions (end-bearing versus floating). See the pertinent geotechnical literature for a proper estimation of the static stiffness. The expressions given herein are *only* for estimates of the axial stiffness of floating piles in a homogeneous stratum of total thickness $H \approx 2L$.		
Static axial stiffness	$K_z \simeq 1.8 E_{sL} d \left(\frac{L}{d}\right)^{0.55} \left(\frac{E_p}{E_{sL}}\right)^{-(L/d)(E_p/E_{sL})}$ $E_{sL} = \bar{E}_s \cdot (L/d)$	$K_z \simeq 1.9 E_{sL} d \left(\frac{L}{d}\right)^{0.6} \left(\frac{E_p}{E_{sL}}\right)^{-(L/d)(E_p/E_{sL})}$ $E_{sL} = \bar{E}_s \cdot \sqrt{(L/d)}$	$K_z \simeq 1.9 E_s d \left(\frac{L}{d}\right)^{2/3} \left(\frac{E_p}{E_s}\right)^{-(L/d)(E_p/E_s)}$
Axial dynamic stiffness coefficient	$k_z \approx 1$ (for $a_0 = \omega d/V_{sL} < 0.5$, where V_{sL} is the S-wave velocity at depth L)	• $L/d < 20$: $k_z \approx 1$ • $L/d \geqslant 50$: $k_z \approx 1 + \frac{1}{3}\sqrt{a_0}$ interpolate in between (for $a_0 = \omega d/V_{sL} < 0.5$)	• $L/d < 15$: $k_z \approx 1$ • $L/d \geqslant 50$: $k_z \approx 1 + \sqrt{a_0}$ interpolate in between (for $a_0 = \omega d/V_s < 1$)
	In all cases, k_z shows a narrow valley at the resonant frequency f_r of the soil stratum; as a first approximation, $f_r \approx f_c \approx \bar{V}_{Ls}/4H$ and $k_z(f_r) \approx 0.8$ for material soil damping $\beta = 0.05$. $\bar{V}_{Ls}$ is the average V_{Ls} over the whole stratum depth.		
Axial radiation dashpot coefficient	$C_z = \frac{4}{3} a_0^{-1/3} \rho V_{sL} \pi d L r_d$ for $f > 1.5 f_r$ where: $r_d \approx 1 - e^{-2(E_p/E_{sL})(L/d)^{-2}}$ $C_z \approx 0$ for $f \leqslant f_r$ linearly interpolate for $f_r < f < 1.5 f_r$	$C_z \approx \frac{3}{2} a_0^{-1/4} \rho V_{sL} \pi d L r_d$ for $f > 1.5 f_r$ where: $r_d \approx 1 - e^{-1.5(E_p/E_{sL})(L/d)^{-2}}$ $C_z \approx 0$ for $f \leqslant f_r$ linearly interpolate for $f_r < f < 1.5 f_r$	$C_z \approx a_0^{-1/5} \rho V_s \pi d L r_d$ for $f > 1.5 f_r$ where: $r_d \approx 1 - e^{-(E_p/E_s)(L/d)^{-2}}$ $C_z \approx 0$ for $f \leqslant f_r$ linearly interpolate for $f_r < f < 1.5 f_r$
Pile-to-Pile Interaction Factors for Assessing the Response of Floating Pile Groups			
Interaction factor α_z for axial in-phase oscillations of the two piles	$\alpha_z \approx \sqrt{2}\left(\frac{S}{d}\right)^{-3/4} \cdot e^{-0.5\beta\omega S/V_{sL}} \cdot e^{-i\omega\sqrt{2}S/V_{sL}}$	$\alpha_z \approx \sqrt{2}\left(\frac{S}{d}\right)^{-2/3} \cdot e^{-(2/3)\beta\omega S/V_{sL}} \cdot e^{-i\omega\sqrt{2}S/V_{sL}}$	$\alpha_z \approx \sqrt{2}\left(\frac{S}{d}\right)^{-1/2} \cdot e^{-\beta\omega S/V_s} \cdot e^{-i\omega S/V_s}$
	V_{sL} = the S-wave velocity at depth $z = L$; $\bar{V}_s = V_s$ at pile mid-length; S = axis-to-axis pile separation; β = soil hysteretic damping. Note: although α_z are complex numbers their use is identical to the familiar use of static interaction factors introduced by Poulos.		
Interaction factor α_{HH} for lateral in-phase oscillation	Very little information presently available	Very little information presently available	$\alpha_{HH}(90°) \approx (3/4)\alpha_z$ $\alpha_{HH}(0°) \approx 0.5\left(\frac{S}{d}\right)^{-1/2} \cdot e^{-\beta\omega S/V_{Ls}} \cdot e^{-i\omega S/V_{Ls}}$ $\alpha_{HH}(\theta°) \approx \alpha_{HH}(0°)\cos^2\theta + \alpha_{HH}(90°)\sin^2\theta$
Interaction factors: α_{MM} for in-phase rocking, and α_{HM} for swaying–rocking	$\alpha_{MM} \approx \alpha_{MH} \approx 0$	$\alpha_{MM} \approx \alpha_{MH} \approx 0$	$\alpha_{MM} \approx \alpha_{MH} \approx 0$

* $\bar{E}_s$ and $\bar{V}_s$ (for the two inhomogeneous deposits) denote Young's modulus and S-wave velocity, respectively, at depth.

ANALYSIS OF PILE DRIVING

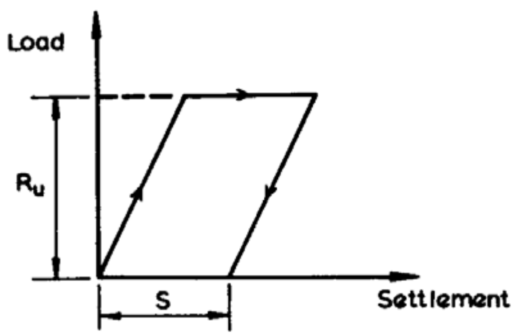


FIGURE 4.1 Assumed load-settlement curve for pile.

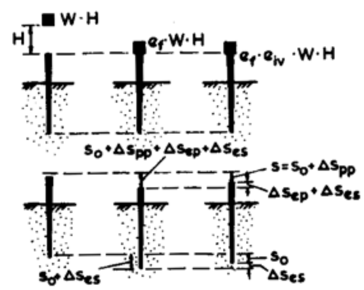


FIGURE 4.2 Transfer of energy and penetration of pile during one blow of the pile-driving hammer.

S = pile penetration for last blow, or "set"
ΔS_{pp} = plastic deformation of pile
ΔS_{ep} = elastic deformation of pile
ΔS_{es} = elastic deformation of soil
S_0 = $S - \Delta S_{pp}$
W = Weight of hammer
H = drop of hammer
e_f = efficiency factor for hammer
e_{iv} = efficiency factor for impact
W_p = weight of pile
A = cross-section of pile
L = pile length
E_p = modulus of elasticity of pile
v = hammer velocity before impact
u = hammer velocity after impact
v_p = pile velocity before impact
u_p = pile velocity after impact
g = gravitational acceleration
R_u = load capacity of pile (just after driving)
E_1 = energy reaching pile
E_2 = energy left after impact.

TABLE 4.1 SUMMARY OF PILE-DRIVING FORMULAS

Formula	Equation for R_u	Remarks
Sanders	$\frac{WH}{S}$	
Engineering News	$\frac{WH}{S+C}$	C = 1.0 in. for drop hammer 0.1 in. for steam hammer 0.1 W_p/W in. for steam hammer on very heavy piles
Eytelwein (Dutch)	$\frac{WH}{S} \cdot \frac{W}{W+W_p}$	
Weisbach	$-\frac{SAE_p}{L} + \sqrt{\left(\frac{2WHAE_p}{L}\right) + \left(\frac{SAE_p}{L}\right)^2}$	
Hiley	$\frac{e_f WH}{S+\frac{1}{2}(C_1+C_2+C_3)} \cdot \frac{W+n^2 W_p}{W+W_p}$	See Tables 4.2, 4.3 and 4.4 for values of e_f, C_1, C_2, C_3, and n.
Janbu	$\left(\frac{1}{k_u}\right)\left(\frac{WH}{S}\right)$	$k_u = C_d(1+\sqrt{1+\gamma e/C_d})$ $C_d = 0.75+0.15\,W_p/W$ $\lambda e = WHL/AES^2$
Danish	$\frac{e_f WH}{S+(2e_f WHL/AE_p)^{1/2}}$	See Table 4.2 for e_f values.
Gates	$5.6\sqrt{e_f WH}\log_{10}(10/S)$	Units are inches and tons (short).
	$4.0\sqrt{e_f WH}\log_{10}(25/S)$	Units are metric tons (1000 kg) and centimeters.

$$D(m, t) = D(m, t-1) + \Delta t \cdot V(m, t-1) \quad (4.16)$$

$$C(m, t) = D(m, t) - D(m+1, t) \quad (4.17)$$

$$F(m, t) = C(m, t) \cdot K(m) \quad (4.18)$$

$$R(m, t) = [D(m, t) - D'(m, t)] \cdot K'(m) \cdot [1 + J(m) \cdot V(m, t-1)] \quad (4.19)$$

$$V(m, t) = V(m, t-1) + [F(m-1, t) + W(m) - F(m, t) - R(m, t)] \cdot \frac{g\Delta t}{W(m)} \quad (4.20)$$

where

$C(m, t)$ = compression of internal spring m at time t
$D(m, t)$ = displacement of element m at time t
$D'(m, t)$ = plastic displacement of external spring m at time t
$F(m, t)$ = force in internal spring m at time t
g = acceleration caused by gravity
$J(m)$ = soil-damping constant at element m
$K(m)$ = spring constant for internal spring m
$K'(m)$ = spring constant for external spring m
$R(m, t)$ = force exterted by external spring m on element m at time t
$V(m, t)$ = velocity of element m at time t
$W(m)$ = weight of element m

In performing the computations, the following sequence of operations is carried out:

1. The initial velocity, v_r, is calculated from Eq. (4.23). Other time-dependent quantities are initialized at zero or to produce equilibrium of forces under gravity.
2. The displacements $D(m,t)$ are calculated from Eq. (4.16) where for the first time-step, $V(1,0)$ is the initial velocity of the ram.
3. The total plastic deformation of the soil, $D'(\dot{m}, t)$, remains constant [starting at $D'(\dot{m}, t) = 0$] unless it is changed by the following condition (see Fig. 4.7*a*):

$$D'(\dot{m}, t) \nless D(m, t) - Q(m) \tag{4.27a}$$
$$D'(\dot{m}, t) \ngtr D(m, t) + Q(m) \tag{4.27b}$$

These comparisons are made in each time-interval, Δt, and $D'(\dot{m}, t)$ is adjusted accordingly.

4. The plastic deformation of the pile tip, $D'(\dot{p}, t)$, remains constant, starting at zero, unless changed by the condition (see Fig. 4.7*b*)

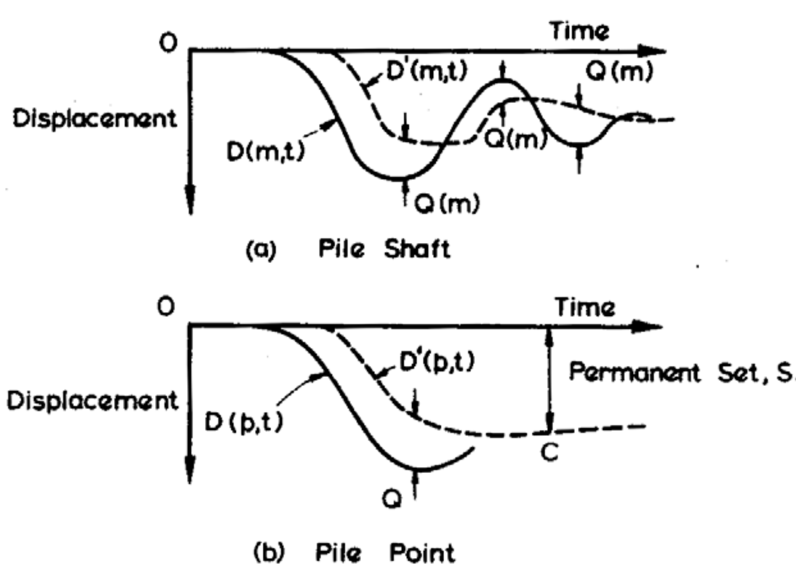


FIGURE 4.7 Displacements of pile vs. time.