PILES SUBJECTED TO DYNAMIC & EARTHQUAKE LOADINGS

OUTLINE

- Dynamic design criteria
- Response of simple dynamic systems
- Earlier experimental data
- Solutions for dynamic loading on single piles
- Pile groups
- Piles subjected to earthquakes
- Effects of liquefaction on pile response
- Axial effects from earthquake-induced settlements

MODES OF VIBRATION FOR A FOUNDATION

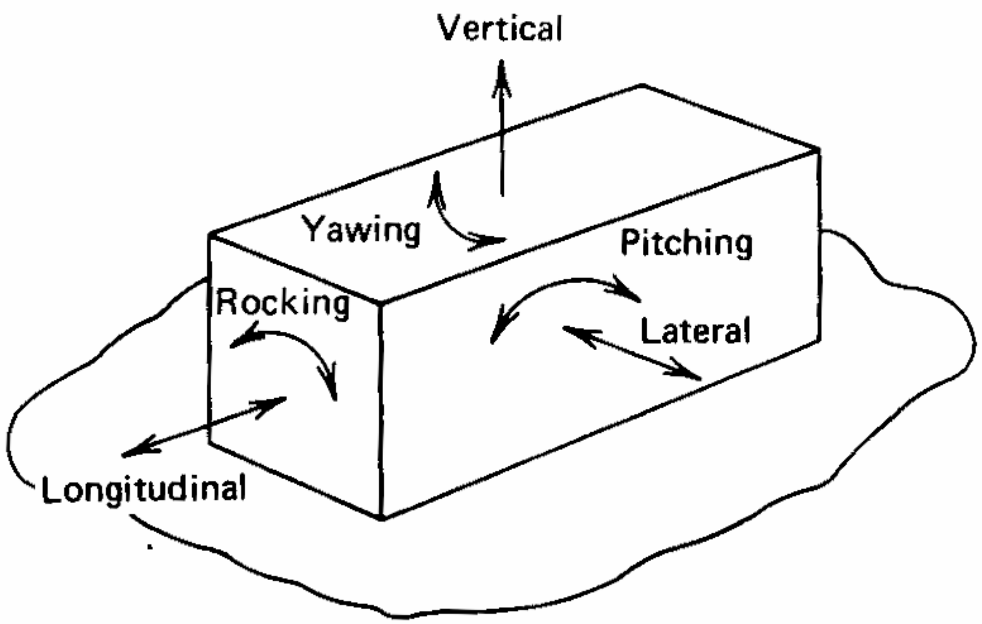
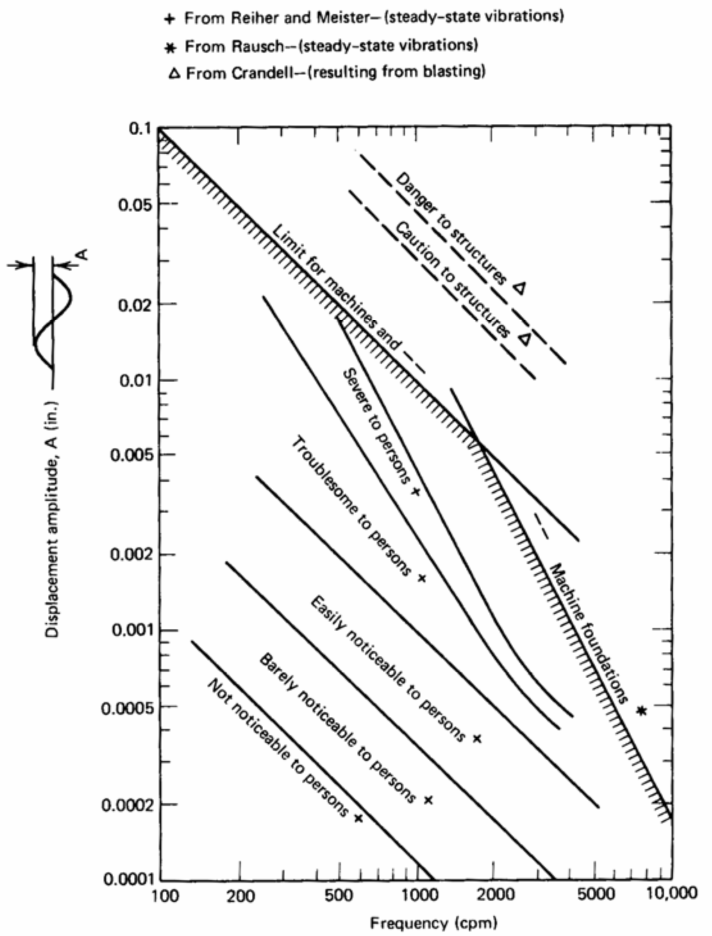


FIGURE 15.2 Six modes of vibration.

In general, there are 6 modes:
-3 translational,
-3 rotational.

ALLOWABLE VERTICAL VIBRATION AMPLITUDE vs FREQUENCY

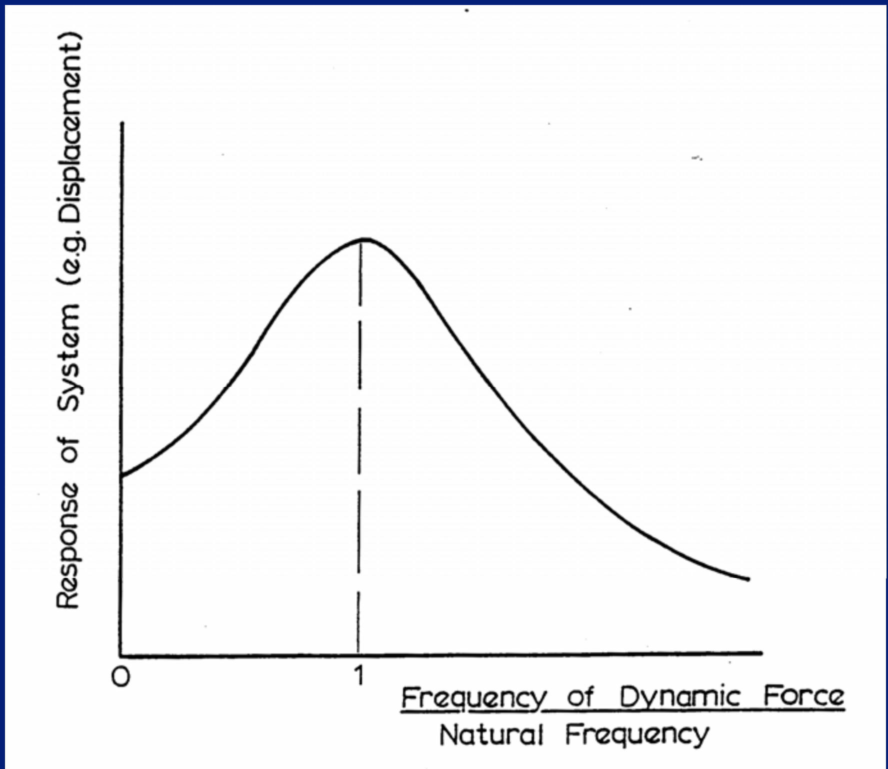


General limits of displacement amplitude for a particular frequency of vibration (from Richart, 1962).

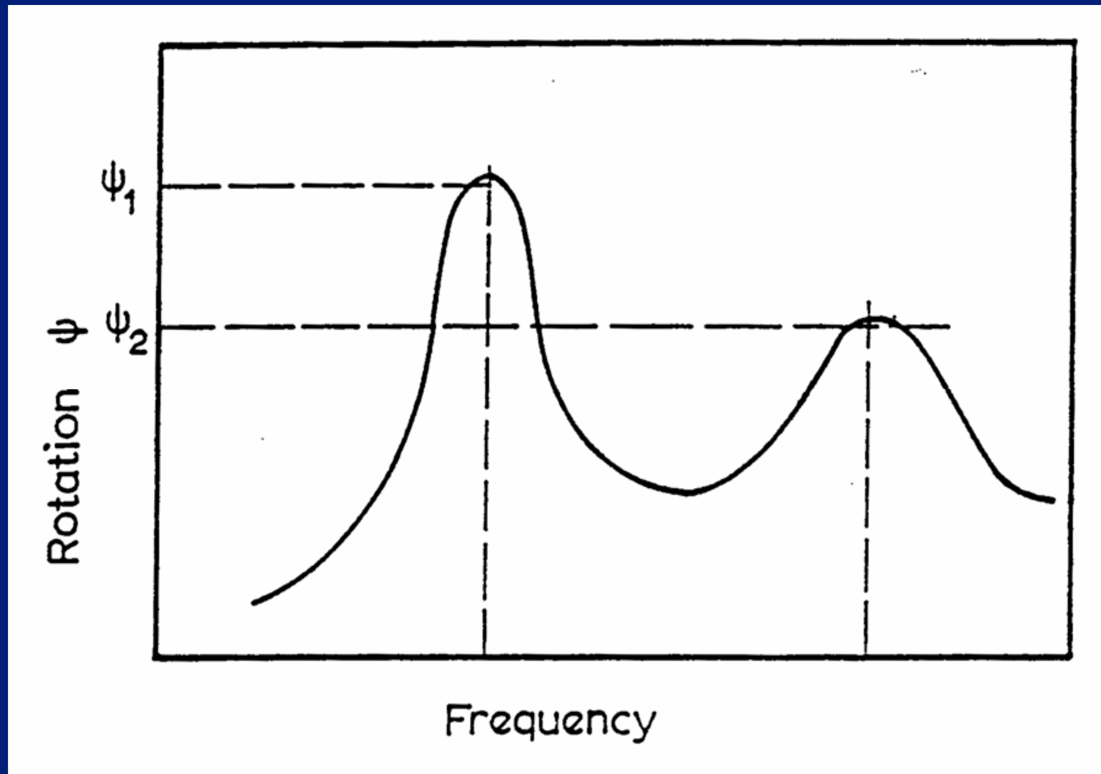
Note the **extremely small** allowable movement values, especially for high frequency loading

Can thus use elastic theory directly in this case

RESPONSE CURVES FOR DYNAMICALLY LOADED SYSTEMS

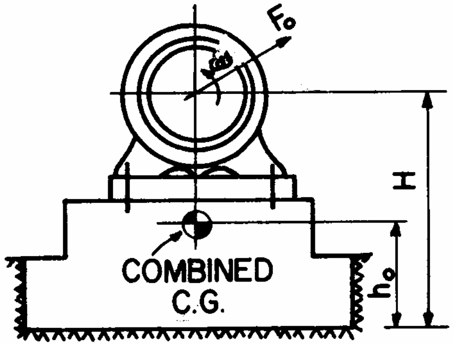


Single degree of freedom



Two degrees of freedom

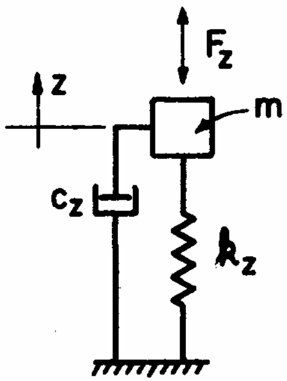
MODELLING OF A MACHINE FOUNDATION



Vibrating Machine Supported by Block-Type Foundation

A "lumped parameter" model is used. Characterized by:

- Mass
- Stiffness
- Damping



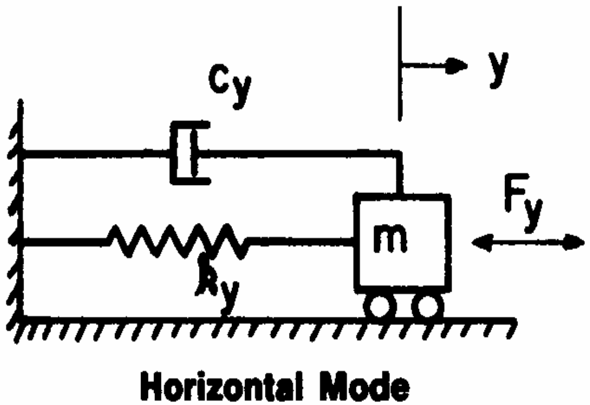
Vertical Mode

Vertical mode normally behaves independent of other modes. The mass (m) of machine and foundation is assumed to be concentrated on the vertical axis. Spring constant of soil (k_z), damping in soil (C_z), inertia of mass (m_z) and the forcing function (F_z) of the machine have their line of action coinciding with the vertical axis. Equation of motion:

$$m\ddot{z} + C_z\dot{z} + k_z z = F_z(t) \qquad (a)$$

Vertical Mode

MODELLING OF A MACHINE FOUNDATION

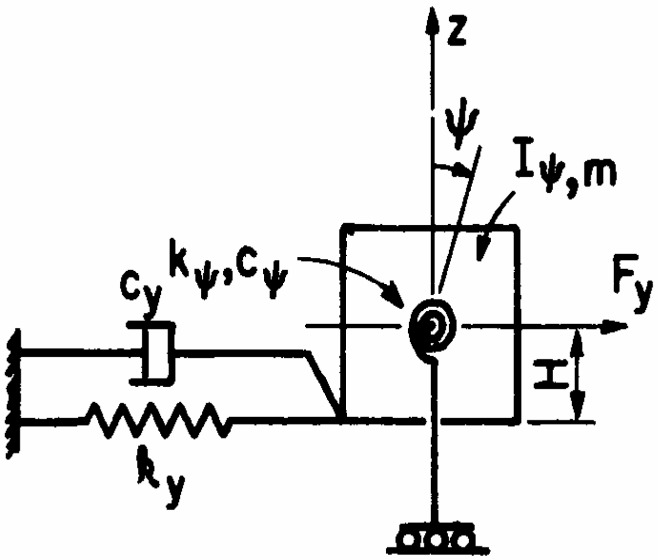


The representation for the horizontal mode involves an approximation. In this mode, contrary to the vertical mode, the masses do not lie on the same horizontal axis, nor the line of action of the forces coincides. Due to these reasons, this mode is normally coupled with the rocking mode. Equation of motion:

$$m\ddot{y} + C_y\dot{y} + k_y y = F_y(t) \tag{b}$$

Horizontal Mode

MODELLING OF A MACHINE FOUNDATION



This model is a better representation of the true dynamic behavior of the structure. However, the analytical solution is difficult to attempt due to coupling of horizontal and rocking motions. This coupling effect should be investigated for the case when the machine is located high above the founding level. Equations of motion:

$$m\ddot{y} + C_y\dot{y} + k_y(y - \psi h_0) - h_0 C_y \dot{\psi} = F_0 \cos \omega t \quad (c)$$

$$I_\psi \ddot{\psi} + (C_\psi + h_0^2 C_y)\dot{\psi} + (k_\psi + h_0^2 k_y)\psi - h_0 C_y \dot{y} - h_0 k_y y = F_y(t) H = T_\psi(t) = F_0 H \cos \omega t \quad (d)$$

Combined Horizontal & Rocking Mode

MODELLING OF A MACHINE FOUNDATION BY LUMPED PARAMETER SYSTEM

- **MASS & INERTIA** – are usually approximated by the mass & inertia of the machine + the foundation

- **STIFFNESS** – is dependent on the foundation & soil characteristics & frequency of vibration

- **DAMPING** – also dependent on foundation & soil characteristics & frequency of vibration

BASIC DYNAMICS EQUATIONS (1)

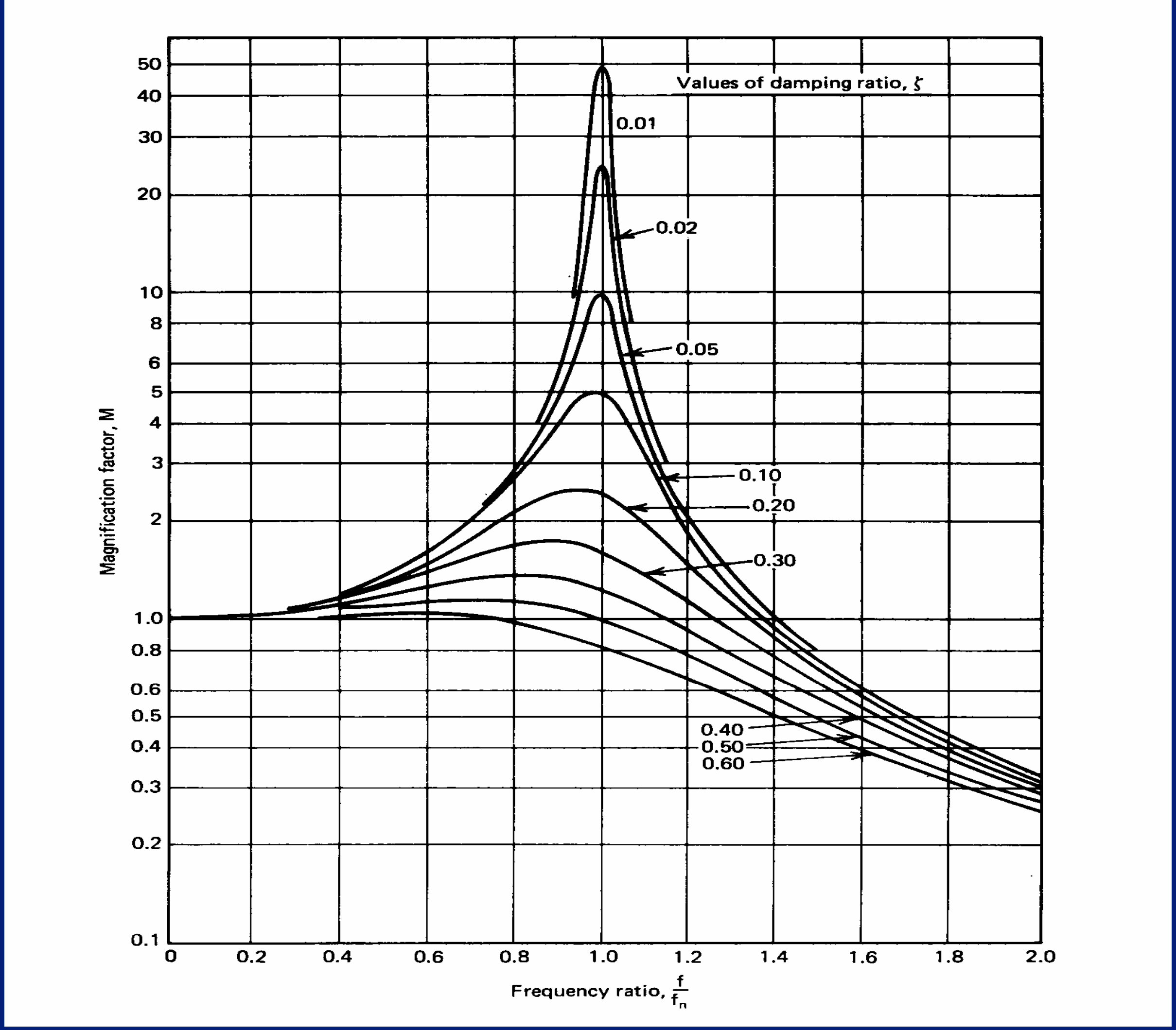
Summary of Relations for Single-Degree-of-Freedom Vibration
(z-coordinate chosen for illustration)

Critical Damping	c_c	=	$2\sqrt{km}$
Dampting Ratio	D	=	$\frac{c}{c_c}$
Undamped "Natural Frequency"	f_n	=	$\frac{1}{2\pi}\sqrt{\frac{k}{m}}$
Static Displacement	z_s	=	$\frac{Q_o}{k}$
Amplitude-Magnification Factor During Vibration	M	=	$\left[\left(1 - \frac{f^2}{f_n^2}\right)^2 + \left(2D\,\frac{f}{f_n}\right)^2\right]^{-½}$

BASIC DYNAMICS EQUATIONS (2)

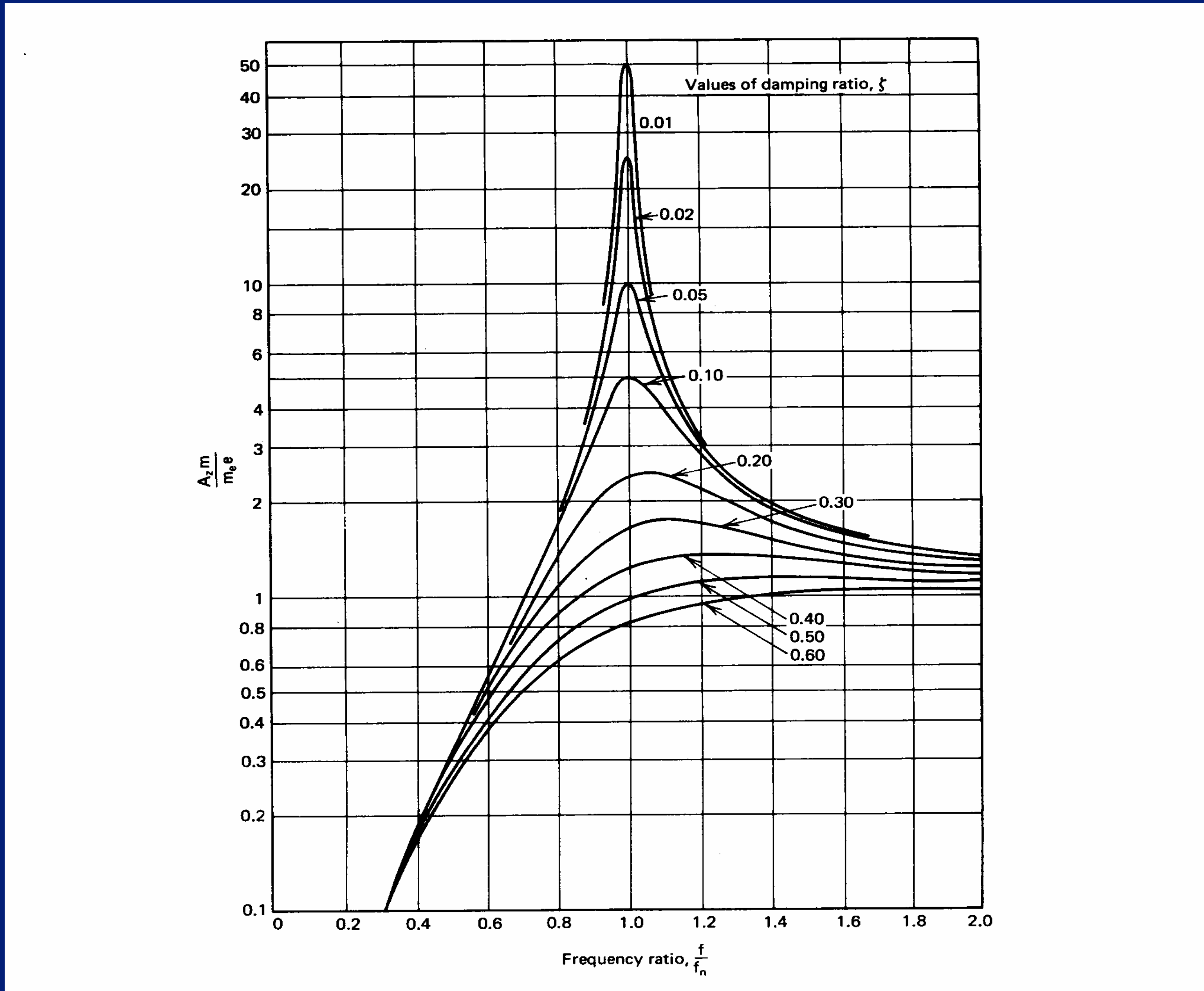
For Constant-Force Excitation (Q_o = constant)	For Rotating-Mass Excitation ($Q_o = m_e e\omega^2$)
Amplitude at Frequency f	
$A_z = \frac{Q_o}{k} M$	$A_z = \frac{m_e e}{m}\left(\frac{f}{f_n}\right)^2 M$
Maximum Amplitude of Vibration	
$A_{zm} = \frac{Q_o}{k} \frac{1}{2D\sqrt{1-D^2}}$	$A_{zm} = \frac{m_e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

DYNAMIC RESPONSE CURVES – Constant Force Excitation



Single Degree of Freedom System

DYNAMIC RESPONSE CURVES – Rotating Mass Excitation



Single Degree of Freedom System

MEASURED RESPONSE CURVES FOR H-PILE

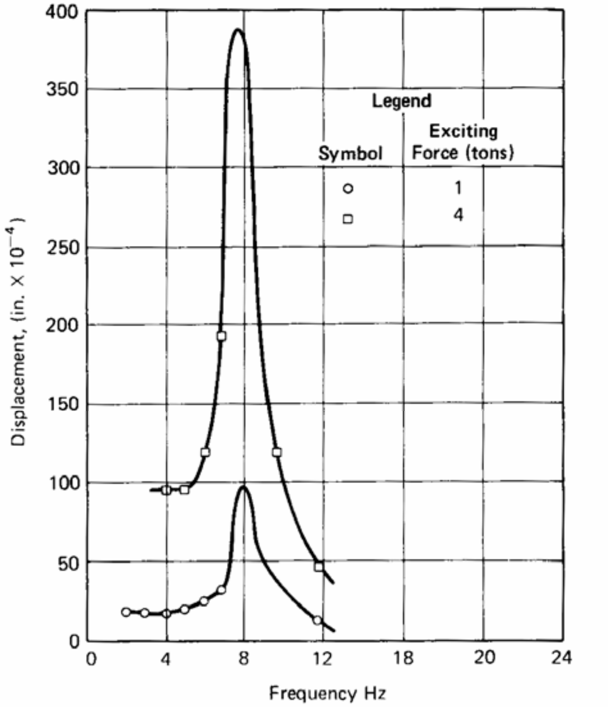


FIGURE 15.10 Results of constant-force test on uncased H-pile *D*-4 before excavation of soil under cap (Maxwell et al. 1969). (Reprinted by permission of the American Society for Testing and Materials, © 1969.)

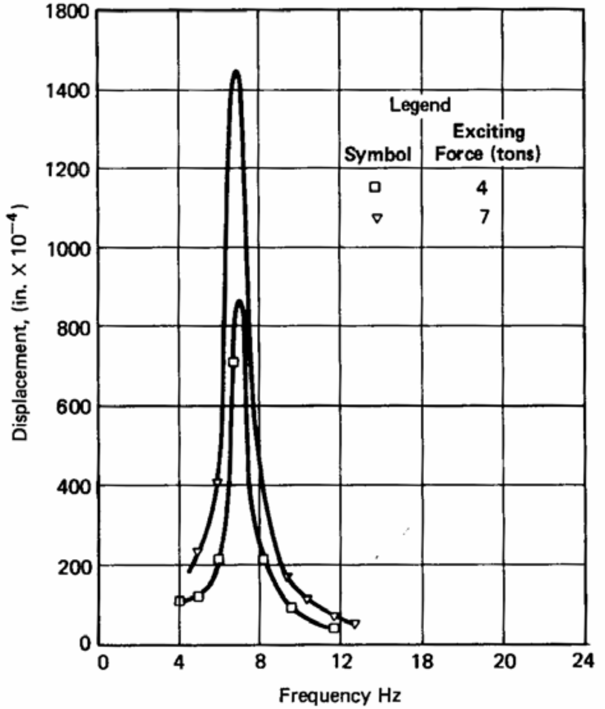


FIGURE 15.11 Results of constant-force test on uncased H-pile *D*-4 after excavation of soil under cap (Maxwell et al. 1969). (Reprinted by permission of the American Society for Testing and Materials, © 1969.)

Before excavation under cap

After excavation under cap

MEASURED RESPONSE CURVES FOR PIPE PILE

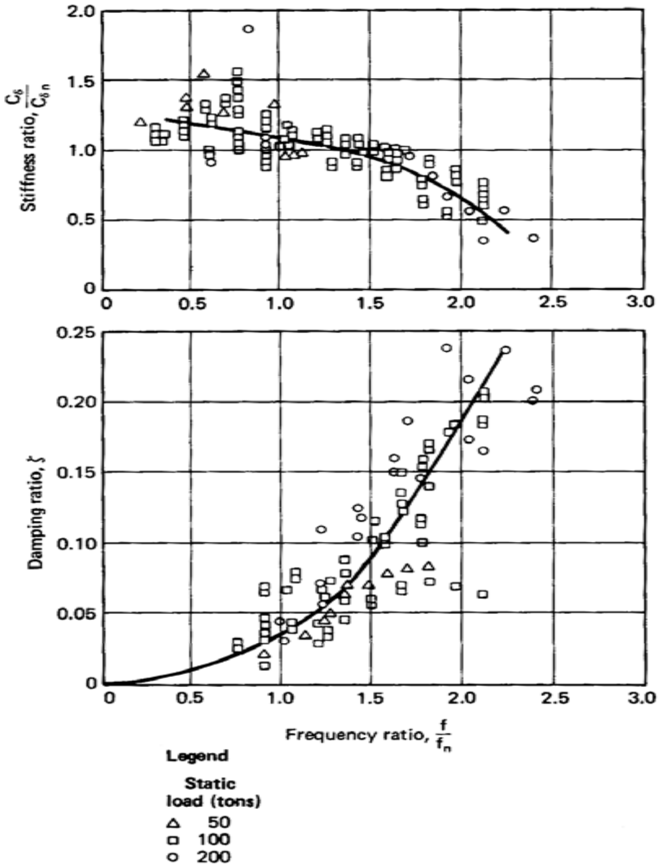


FIGURE 15.9 Stiffness and damping ratio vs. frequency ratio, pipe pile *D*-1 (Maxwell et al., 1969). (Reprinted by permission of the American Society for Testing and Materials, © 1969.)

- Note decrease in stiffness ratio and increase in damping ratio as frequency ratio increases

EFFECT OF FOOTING EMBEDMENT ON DYNAMIC SETTLEMENT AMPLITUDE

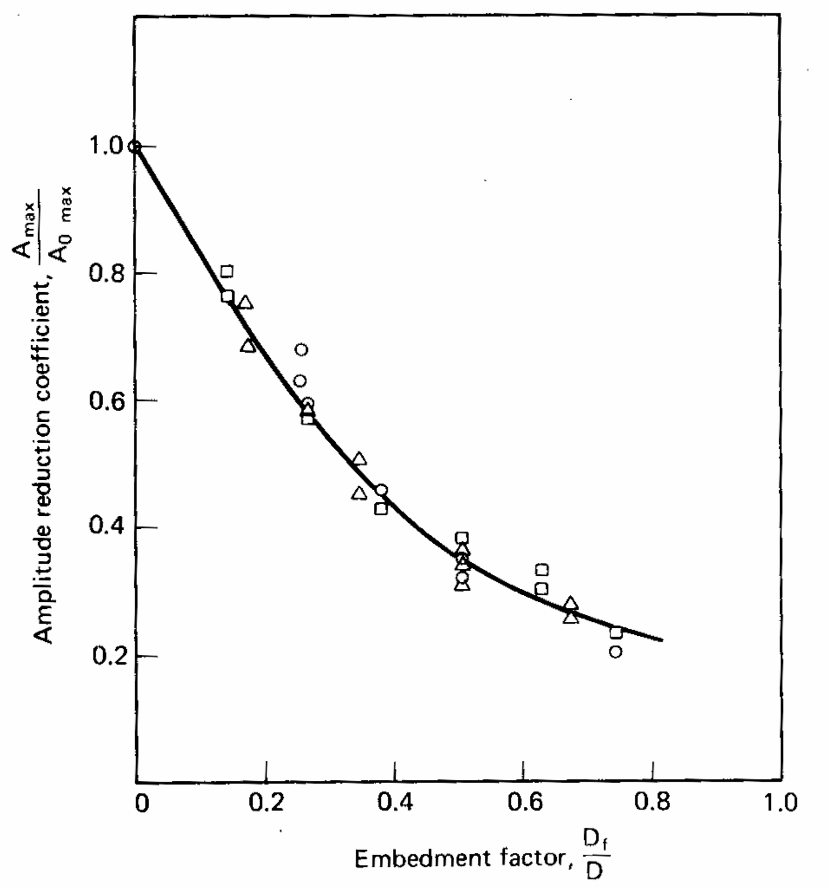


FIGURE 15.13 Amplitude reduction factor versus embedment factor for circular footings on sand (Chae, 1970).

Footing embedment *increases* stiffness and *decreases* movement amplitude

THEORETICAL SOLUTIONS FOR PILE STIFFNESS & DAMPING

FACTORS AFFECTING DYNAMIC RESPONSE

- Dimensionless frequency: $a_0 = \omega R/V_s$
- Relative stiffness of soil G / E_p or V_s / V_c

  (V_s = soil shear wave velocity, V_c = pile P-wave velocity)
- Slenderness ratio L/d or L/R
- Pile head fixity condition
- Pile tip condition
- Variation of soil stiffness with depth

NOVAK'S SOLUTIONS FOR A SINGLE PILE

Stiffness & damping coefficients can be expressed as follows:

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:

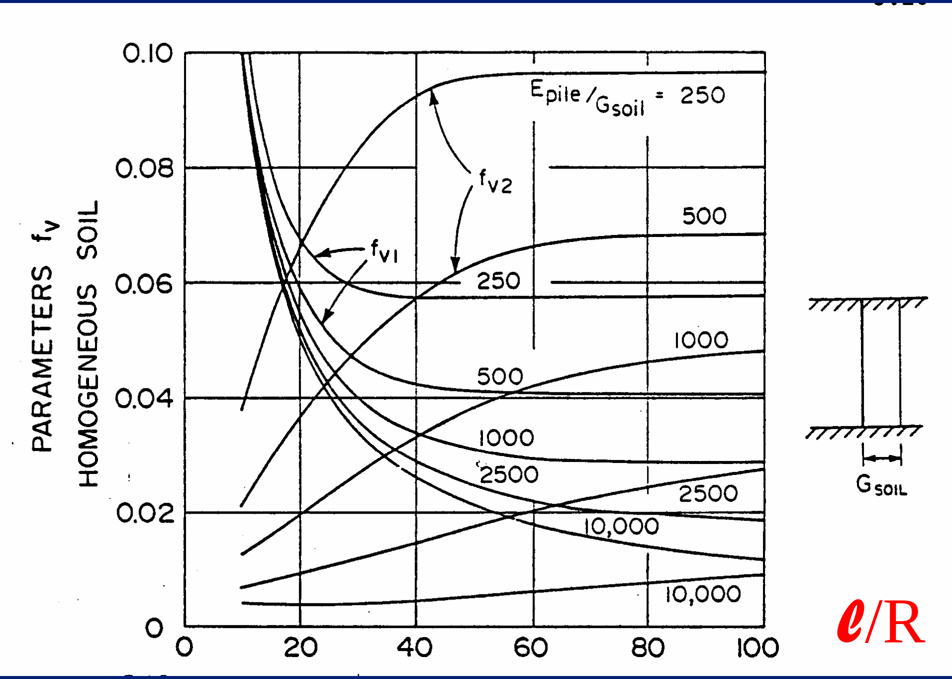
$$k_c = \frac{E_p I}{R^2} f_{c1} \quad , \quad c_c = \frac{E_p I}{R V_s} f_{c2}$$

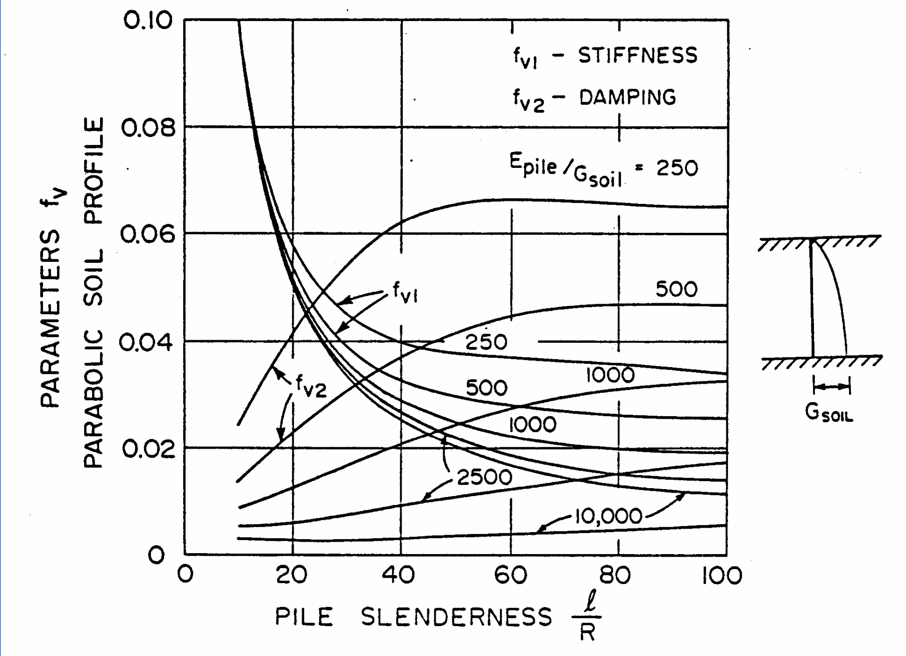
Torsion:

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

E_p=pile Young's modulus
A=pile cross-sectional area
R=pile radius
V_s=soil shear wave velocity
I=moment of inertia of pile section
G_pJ=torsional stiffness of pile section

STIFFNESS & DAMPING PARAMETERS –
END BEARING PILES

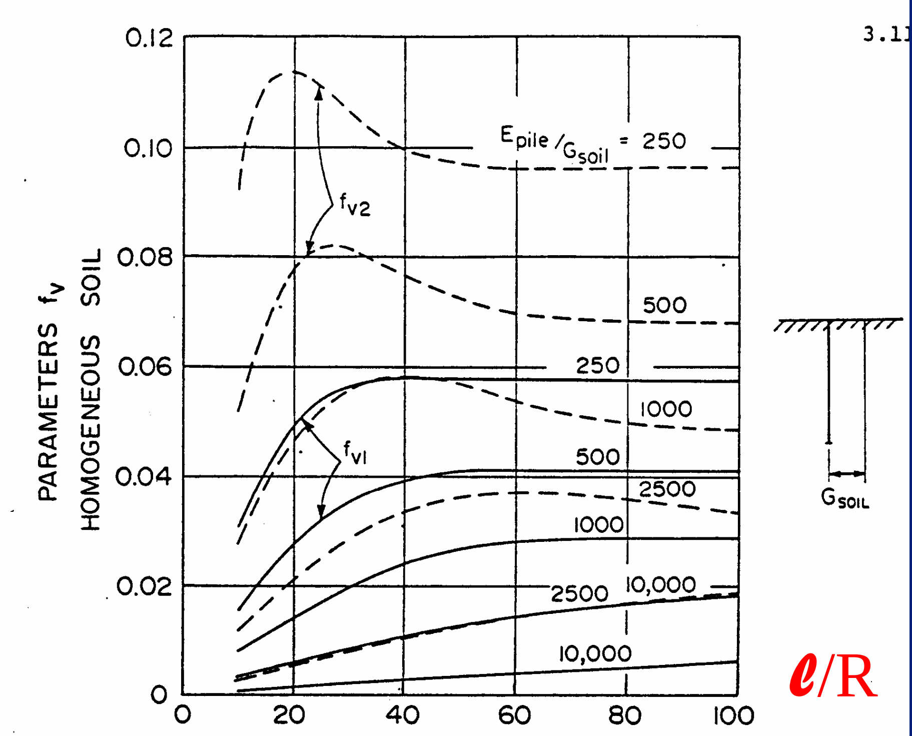




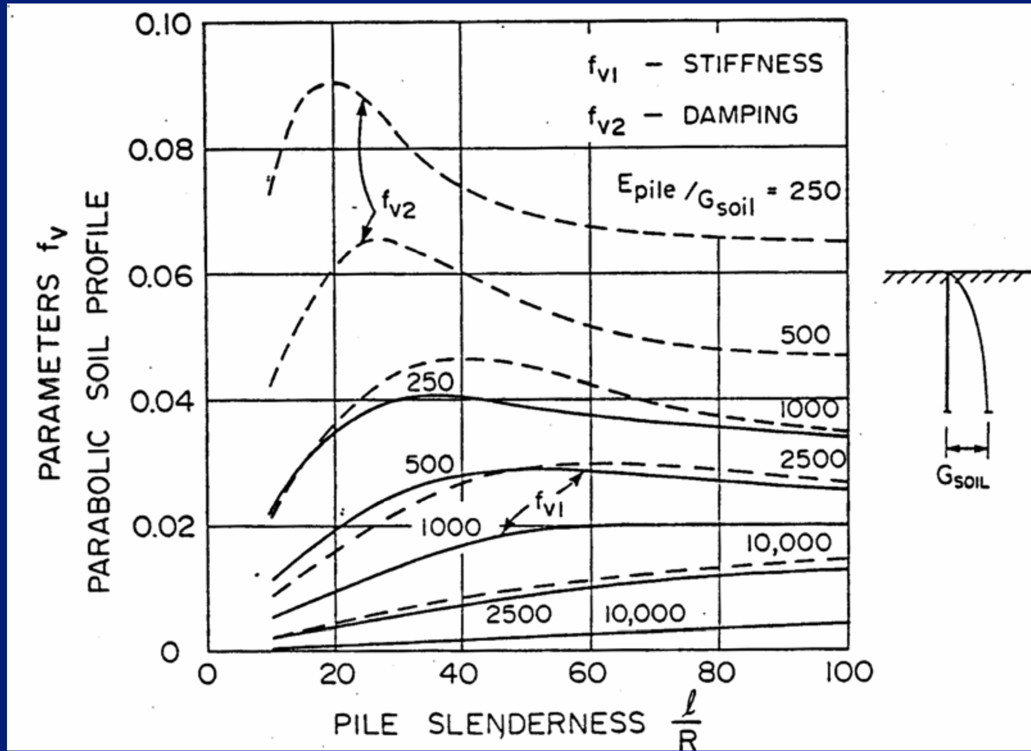
Uniform soil – Constant Shear Modulus

Soil with Parabolically Increasing Shear Modulus

STIFFNESS & DAMPING PARAMETERS –
FLOATING PILES



Uniform soil – Constant Shear Modulus



Soil with Parabolically Increasing Shear Modulus

COMPARISON BETWEEN MEASURED & CALCULATED HORIZONTAL RESPONSE

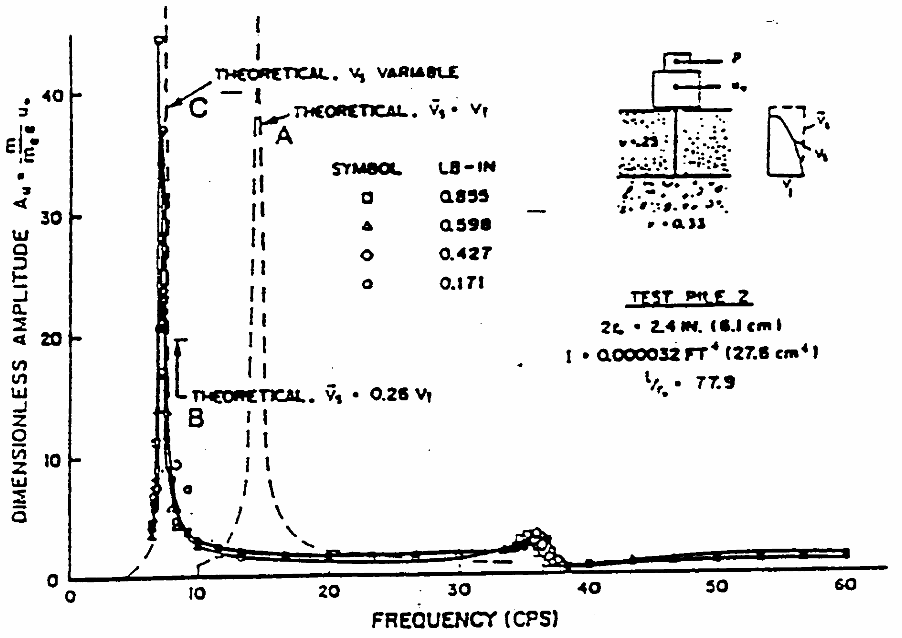


Figure 5 Comparison of experimental horizontal response of steel test pile with theoretical predictions (Novak and Sheta, 1982)

- Note the importance of a realistic distribution of soil shear modulus with depth
- Near-surface distribution is critical to lateral response (as for static case)

PILE GROUPS – INTERACTION EFFECTS

Three approaches have been taken:

1. Ignore interaction
2. Use static interaction factors
3. Use dynamic interaction factors

Use of approaches 1 & 2 can be misleading!

Static interaction may be adequate IF:

- Frequencies of interest are low, especially if they are lower than the natural frequency of the soil deposit

CHARACTERISTICS OF DYNAMIC INTERACTION

- Interaction factors display an oscillatory nature – they depend greatly on both frequency and spacing
- The presence of a "weakened" zone around the pile can be important; it reduces the interaction "peaks" and appears to give more realistic results.
- Solutions for dynamic interaction factors (DIF) are available.
- $$\text{DIF} = \frac{\text{dynamic displacement of pile 2}}{\text{static displacement of pile 1}}$$

SOME SOLUTIONS FOR DYNAMIC INTERACTION FACTORS

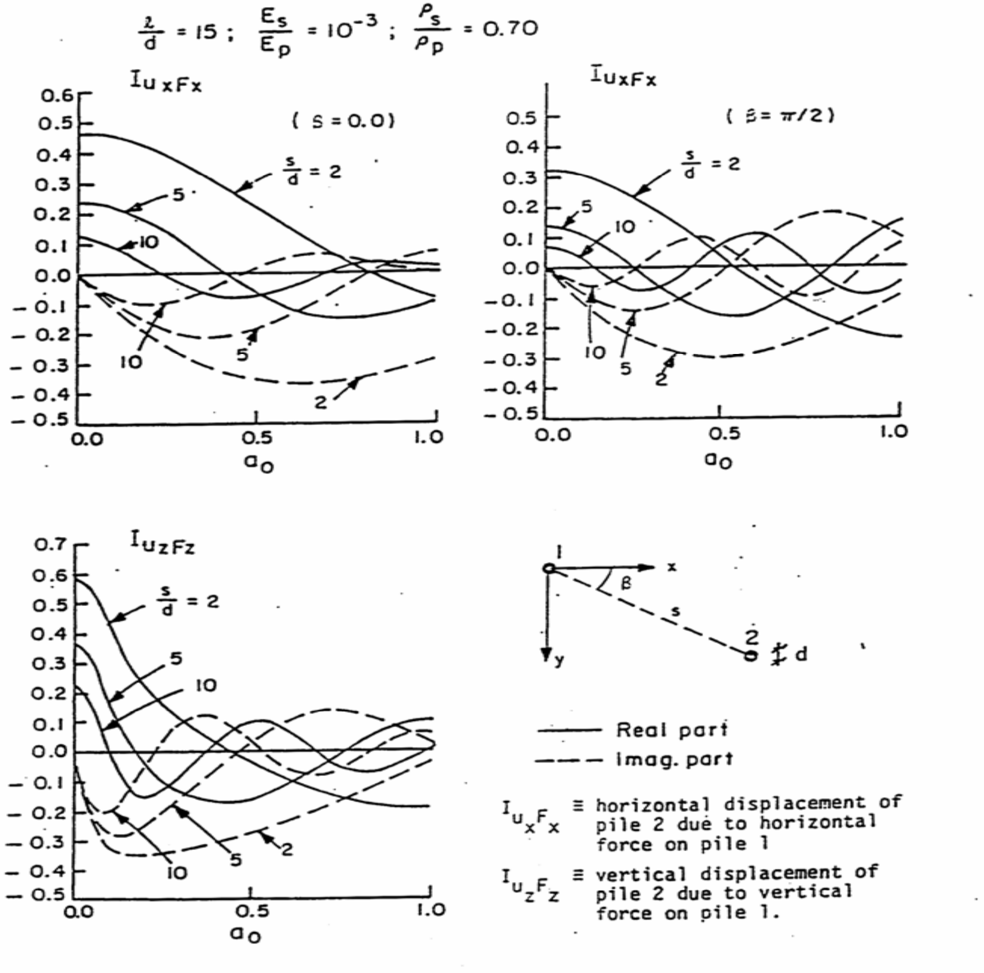


Fig. 3.16 Interaction Curves for Horizontal and Vertical Displacement of Pile 2 Due to Horizontal and Vertical Force on Pile 1 (Kaynia and Kausel, 1982)

Note:

•DIF can be positive or negative

•Depends on s/d and a_0

•Damping factors generally negative (i.e. interaction reduces damping)

2-PILE GROUP EFFICIENCY FACTORS

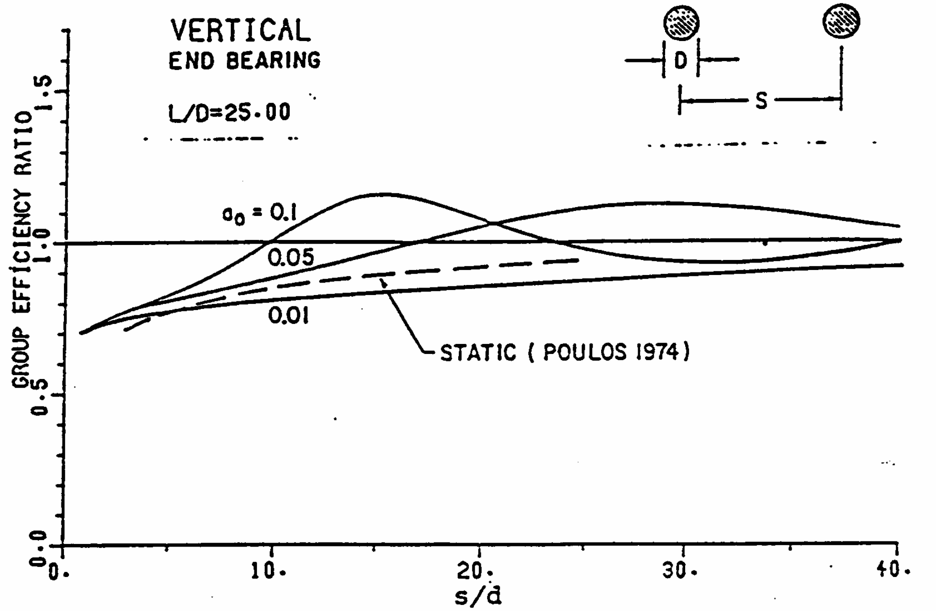


Fig. 3.10 Group efficiency of vertical stiffness of two end bearing piles for varying pile separation (s/d) and different frequencies (a_o) (Sheta and Novak, 1982)

Note:

- Group efficiency depends on s/d and frequency (a_0)
- Use of static interaction factors gives good approximation for low a_0
- Not so good for higher frequencies

2-PILE GROUP EFFICIENCY FACTORS – EFFECT OF WEAKENED ZONE

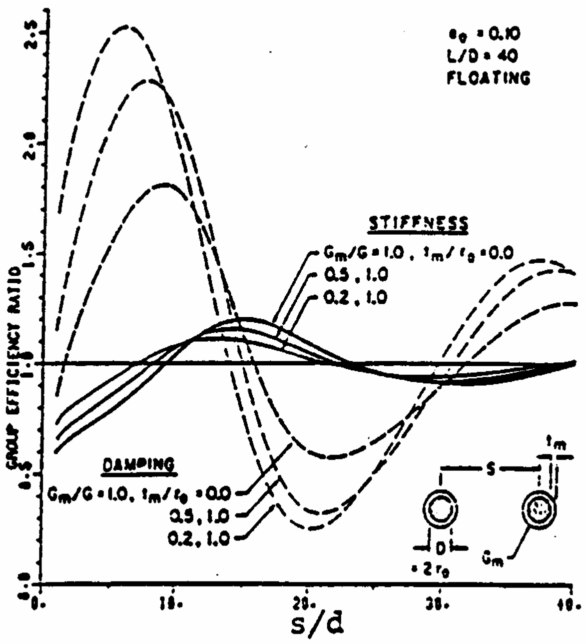


Fig. 3.11 Group efficiency of vertical stiffness and damping of two floating piles for different separations and weakened zones around piles (r_o=R)

Note:

Weakened zone around each pile tends to reduce interaction

THEORY vs EXPERIMENT FOR 2x2 GROUP

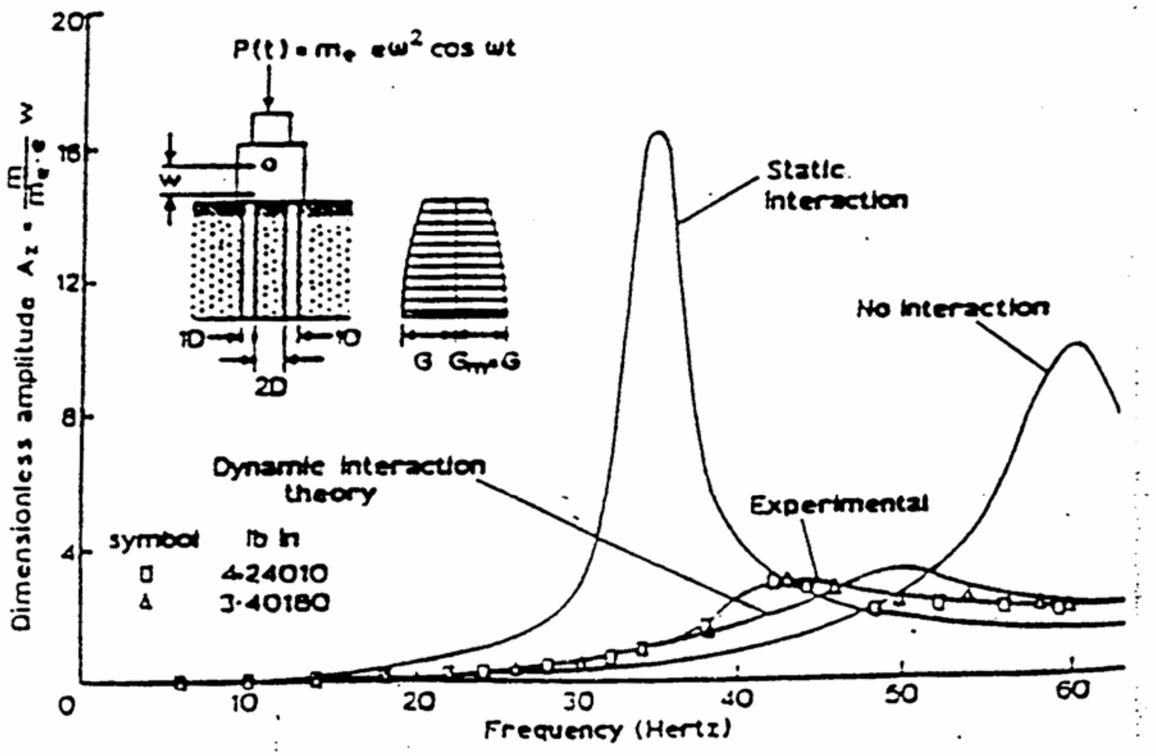


Figure 25 Vertical response of 2x2 group of closely spaced piles: theory vs. experiment (Sheta & Novak, 1982; L = 3.4 m, d = 60.3 mm)

Note:

Poor results for theory using both no interaction & static interaction

GAZETAS' APPROXIMATE SOLUTIONS

Gazetas (1991) has developed simplified closed-form expressions for dynamic stiffness & damping coefficients for piles, and for dynamic interaction factors. These are summarized tabular form, for three distributions of soil stiffness with depth.

Note that:

1. **the "impedance" (ratio of dynamic force to displacement) is a complex, frequency-dependent quantity, and is given by:**

   $$K(\omega) = \bar{K}(\omega) + i\,\omega\,C(\omega)$$

   where $\bar{K}(\omega)$ **= dynamic stiffness (real component)**

   $C(\omega)$ **= dashpot coefficient**

   ω **= circular frequency =** $2\pi f$**, with** f **= frequency in Hz**

2. **In the tables, the dashpot coefficient includes hysteretic damping**
3. **The soil modulus to be employed is a small-strain value.**

GAZETAS' APPROXIMATE SOLUTIONS

Definition of Parameters:

L = pile length

d = pile diameter

E_p = pile modulus

E_s = soil modulus

$\tilde{E}_s$ = rate factor for increase of Youngs modulus with depth

V_s = shear wave velocity in soil

V_{la} = Lysmer analog wave velocity

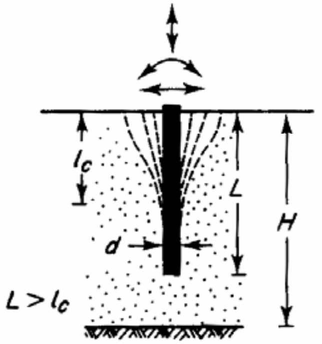
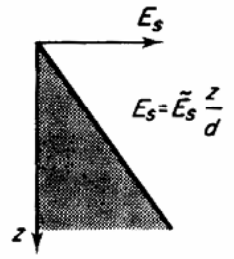
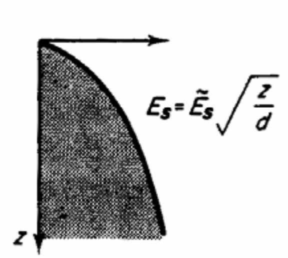
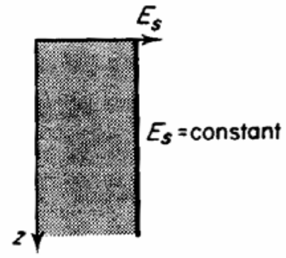
$= 3.4 * V_s / \pi(1-\nu)$

S = centre-to-centre spacing between two piles

β = soil hysteretic damping (typically between 2 & 6% for soils)

The subscript L indicates the value at the level of the pile tip.

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	$K_z \simeq 1.8E_{sL}d\left(\frac{L}{d}\right)^{0.55}\left(\frac{E_p}{E_{sL}}\right)^{-(L/d)(E_p/E_{sL})}$ $E_{sL} = \bar{E}_s \cdot (L/d)$	$K_z \simeq 1.9E_{sL}d\left(\frac{L}{d}\right)^{0.6}\left(\frac{E_p}{E_{sL}}\right)^{-(L/d)(E_p/E_{sL})}$ $E_{sL} = \bar{E}_s \cdot \sqrt{(L/d)}$	$K_z \simeq 1.9E_s d\left(\frac{L}{d}\right)^{2/3}\left(\frac{E_p}{E_s}\right)^{-(L/d)(E_p/E_s)}$
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, $f_r \approx f_c \approx \bar{V}_{La}/4H$ and $k_z(f_r) \approx 0.8$ for material soil damping $\beta = 0.05$. $\bar{V}_{La}$ is the average V_{La} over the whole stratum depth.		
Axial radiation dashpot coefficient	$C_z = \frac{2}{3}a_0^{-1/3}\rho V_{sL}\pi d L r_d$ for $f > 1.5f_r$ 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 $f_r < f < 1.5f_r$	$C_z \approx \frac{3}{4}a_0^{-1/4}\rho V_{sL}\pi d L r_d$ for $f > 1.5f_r$ 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 $f_r < f < 1.5f_r$	$C_z \approx a_0^{-1/5}\rho V_s \pi d L r_d$ for $f > 1.5f_r$ 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 $f_r < f < 1.5f_r$
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	$\alpha_{HH}(90°) \approx (3/4)\alpha_z$ $\alpha_{HH}(0°) \approx 0.5\left(\frac{S}{d}\right)^{-1/2} \cdot e^{-\beta\omega S/V_{La}} \cdot e^{-i\omega S/V_{La}}$ $\alpha_{HH}(\theta°) \approx \alpha_{HH}(0°)\cos^2\theta + \alpha_{HH}(90°)\sin^2\theta$
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.

SEISMIC RESPONSE OF PILES

- Effects of earthquakes
- Method of analysis of lateral response
- Comparisons with existing solutions
- Important factors affecting pile response
- Liquefaction assessment & effects
- Practical design methods
- Aspects of vertical response

EFFECTS OF EARTHQUAKES

- Ground movements generated by bedrock accelerations
- Lateral movements are usually dominant
- Vertical movements also occur
- Movement profiles with depth are time-dependent
- Existing piles will be forced to interact with the soil
- Additional forces and moments will be developed in the piles

EFFECTS OF EARTHQUAKES
Pile Foundations

EFFECTS OF EARTHQUAKES

Building Settlements

EFFECTS OF EARTHQUAKES
Lateral Spreading

EVEN AUSTRALIA IS NOT IMMUNE

Scenes from Newcastle, December 1989

THE FOUNDATION DESIGNER'S CHALLENGE

- Estimate the maximum movements of the piles

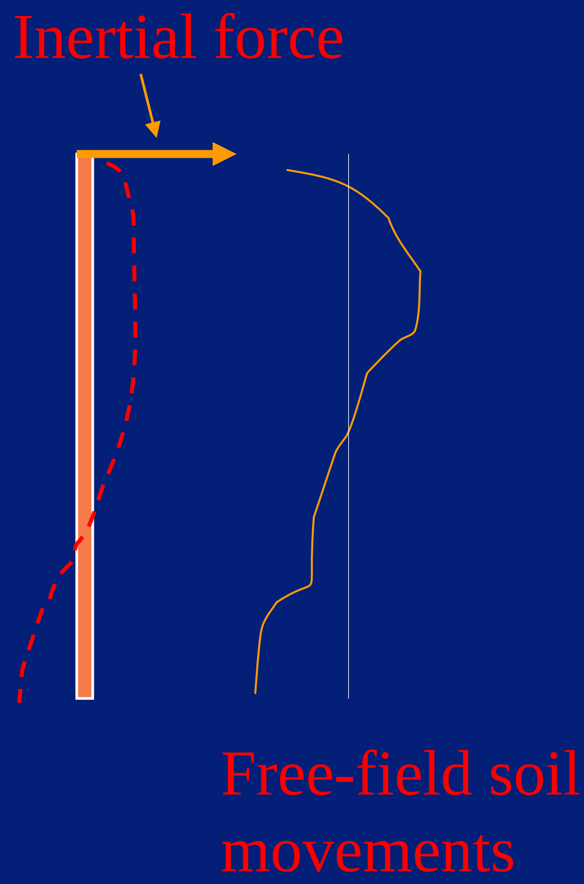
- Allow for the possibility of liquefaction

- Estimate the additional forces and moments which will be generated in a pile by the design earthquake

- Design the piles to be able to withstand these forces & moments

TWO MAIN ASPECTS OF PILE RESPONSE

- **"Inertial" effects**
  - Forces induced at pile head by lateral excitation of structure
  - Generally incorporated in design as an additional lateral load.
- **"Kinematic" effects**
  - Lateral ground movements generated by earthquake interact with the piles
  - Generally ignored or overlooked in design.



ANALYSIS METHODOLOGY- NO LIQUEFACTION

- Use ground response analysis to compute ground lateral movements vs depth vs time.
  Use 1-D response analysis for simplicity (ERLS)

- Use pile-soil interaction analysis to compute distributions of lateral movement, rotation, moment , shear and pressure along the pile at various times.
  Use boundary element analysis (ERCAP)

- Implemented via a combined program **SEPAP**

GROUND RESPONSE ANALYSIS

Allow for :

- Layered soil profile
- Strain dependent modulus and damping
- Input time-acceleration history of earthquake motion at bedrock level
- Damping of bedrock

GROUND RESPONSE ANALYSIS

Basic equation of motion:

$$[M]\{d^2x/dt^2\} + [C]\{dx/dt\} + [K]\{x\} = -[M][I]\{\ddot{u}\}$$

[M] = mass matrix, [C] = damping matrix,
[K] = stiffness matrix, [I] = unit matirix
x = lateral movement rel. to base
{ü} = ground movement vector at time t

Solve numerically by time - marching process

PILE RESPONSE ANALYSIS

Allow for :

- Layered soil profile
- Strain dependent modulus
- Limiting pile-soil pressures along pile shaft
- Input time-soil movement history along pile shaft (kinematic effects) from ground response analysis
- Applied head force versus time (inertial effects) from mass x pile head acceleration

PILE RESPONSE ANALYSIS

Basic equation:

$$[D + I^{-1}/K_R n^4]\ \{\Delta\rho\} = [I^{-1}]\ \{\Delta\rho_e(t)\} / K_R n^4$$

where D = pile action matrix

I^{-1} = inverted soil influence matrix

$\Delta\rho$ = incremental pile displacements

$\Delta\rho_e(t)$ = incremental ground displacements

K_R = pile stiffness factor

n = number of pile elements

PSEUDOSTATIC DESIGN APPROACH

1. Model superstructure as SDOF system with same natural frequency.
2. Perform free-field site response analysis to obtain time history of surface motion and maximum displacement of soil along pile length location.
3. Treat MAXIMUM values of soil displacements at each depth are treated as static soil movements.
4. Compute spectral acceleration, a_s, of structure, and compute inertial force as (cap mass* a_s).
5. Analyze pile subjected to combined inertial force & soil displacements, to obtain maximum shear and moment induced in pile.

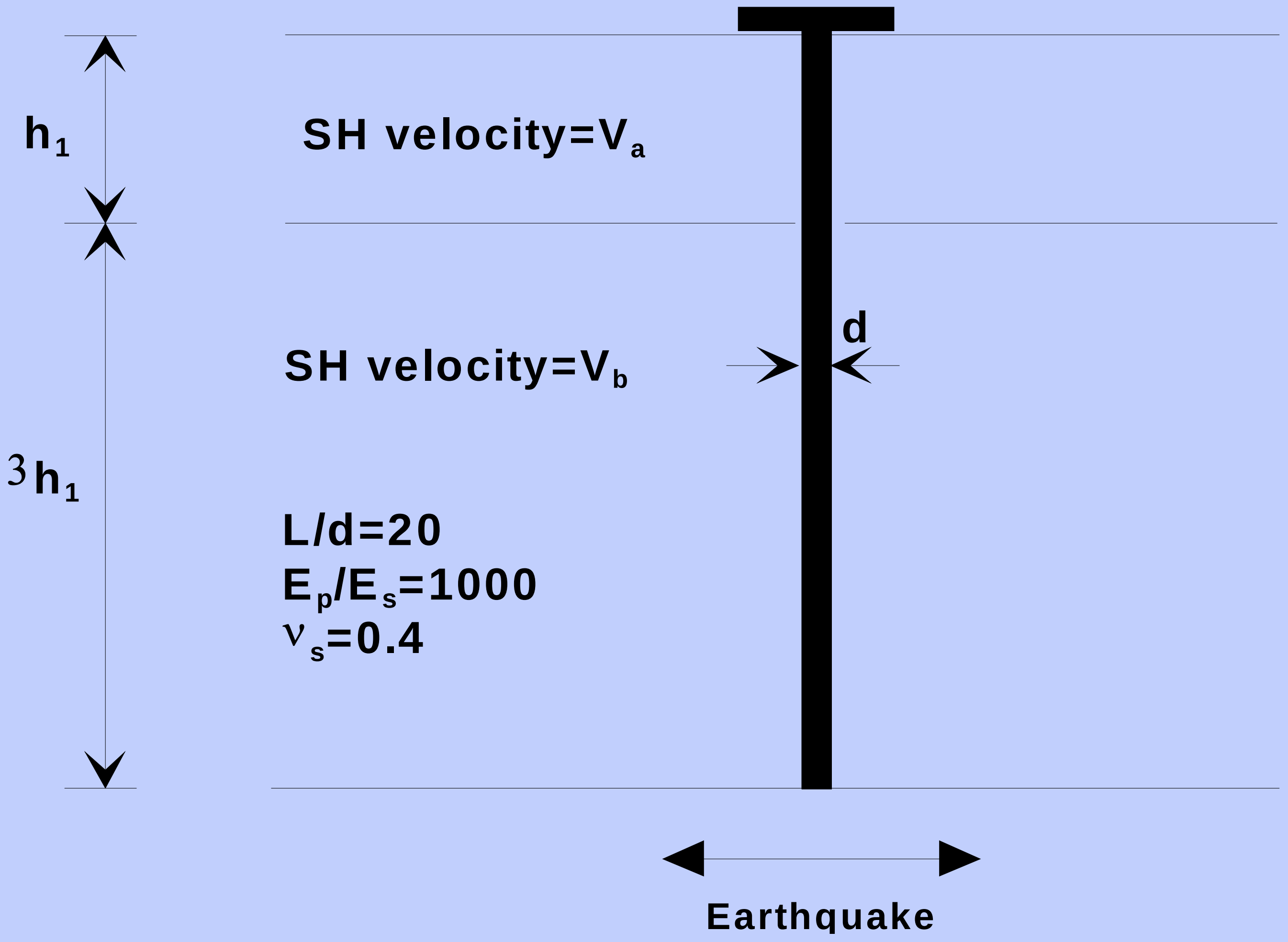
ANALYSIS VERIFICATION

Alternative solutions available:

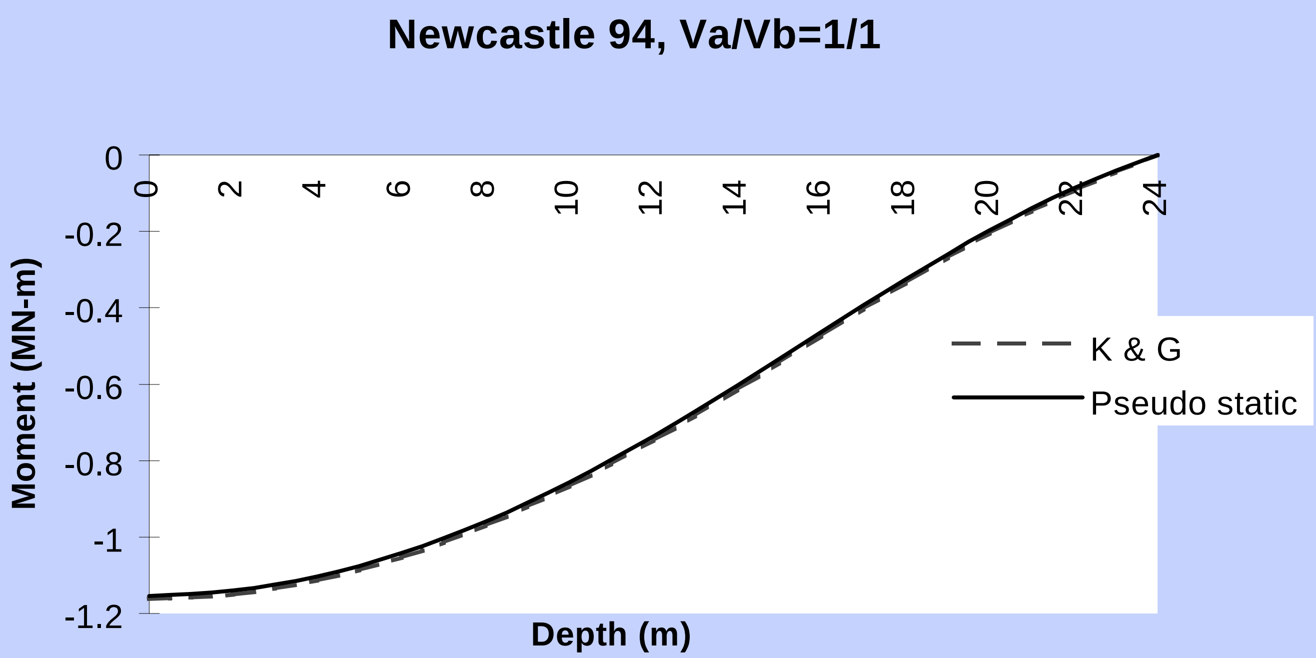
- Kaynia (1982) – relatively rigorous method
- Kavvadas & Gazetas (1993) – use of beam on elastic foundation
- Ke Fan (1992) – used Kaynia’s approach.

Above methods assume linear soil behaviour but are useful for comparison purposes.

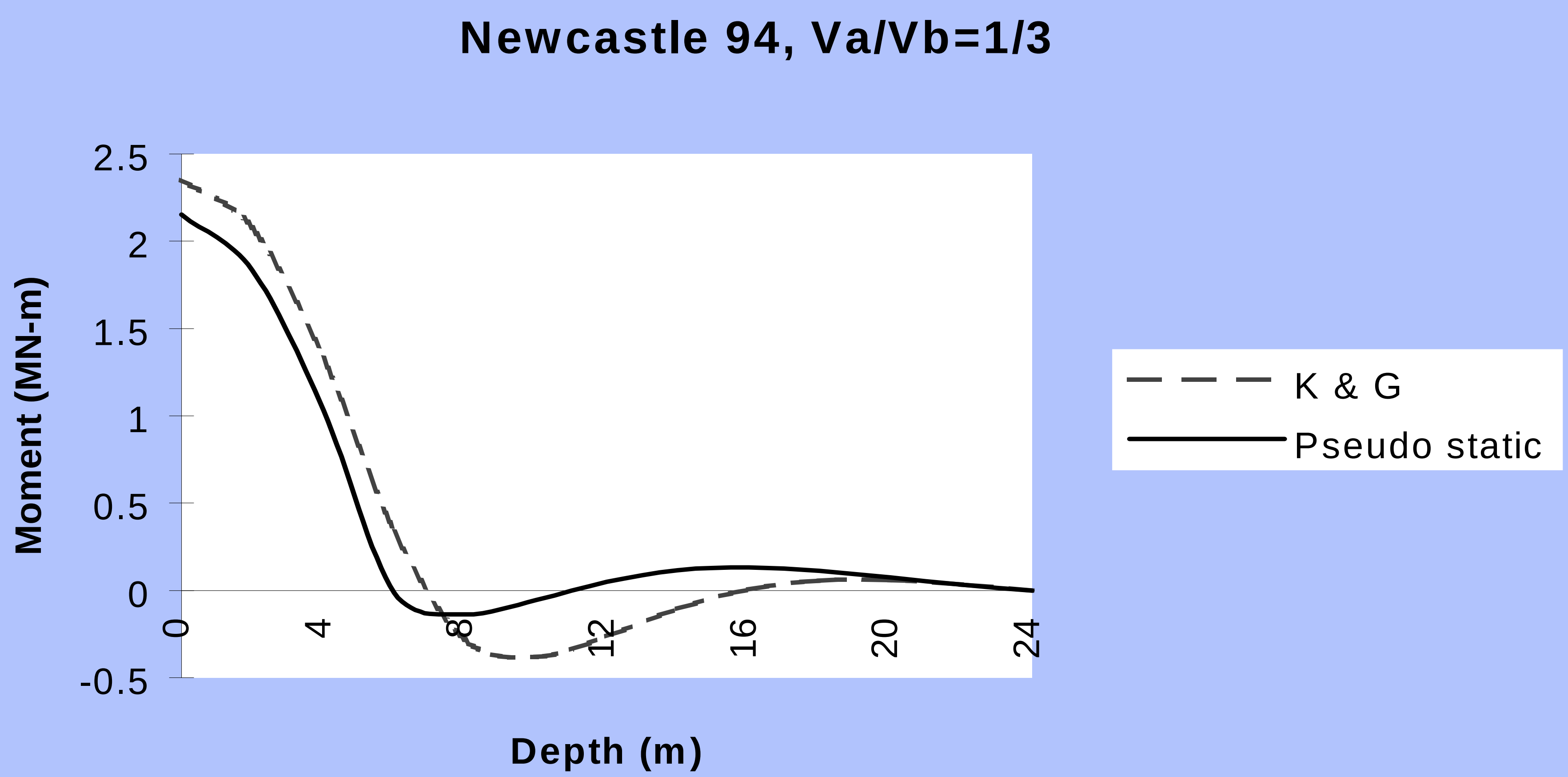
EXAMPLE FOR COMPARISON OF METHODS



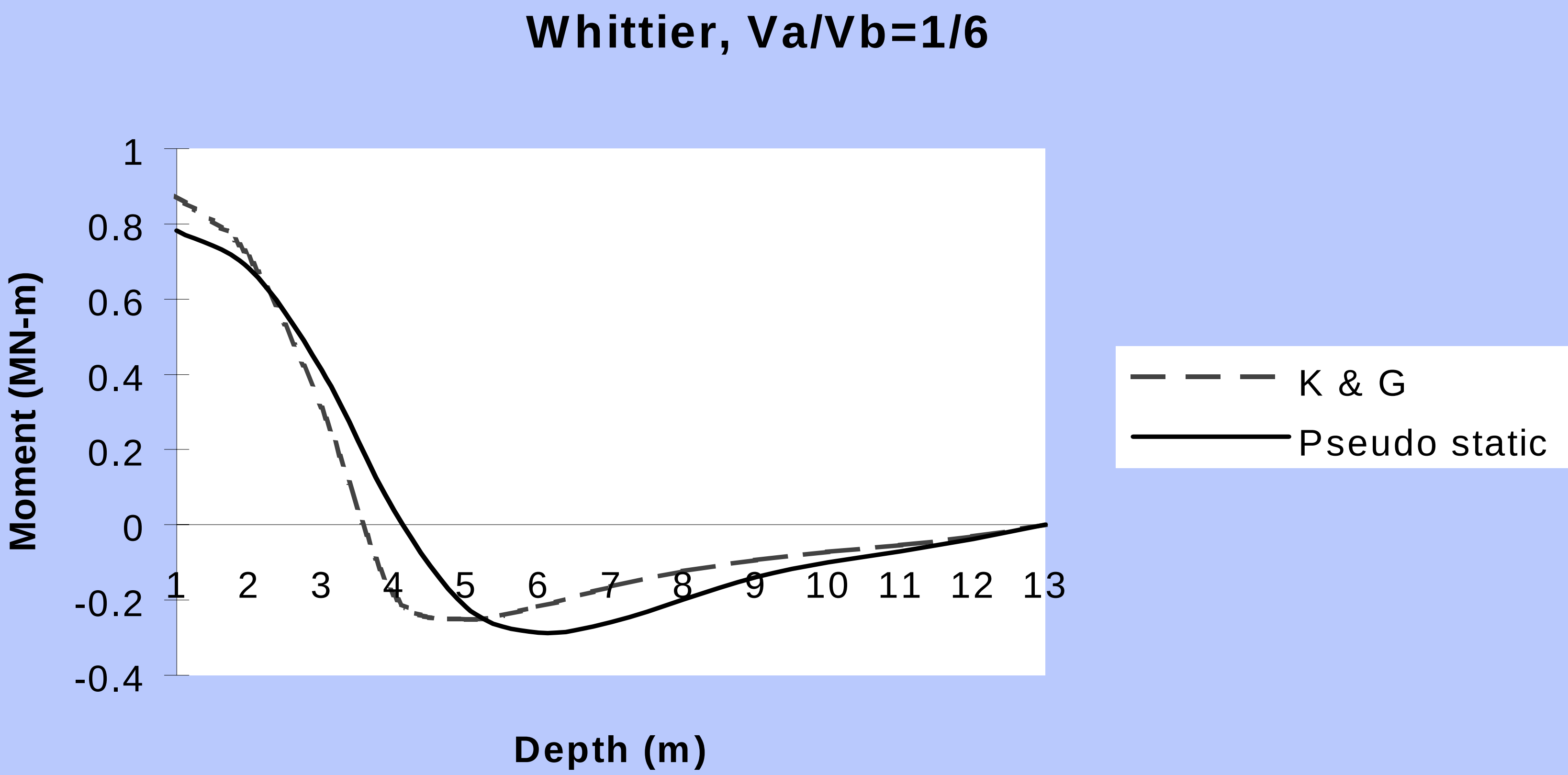
COMPARISON FOR MAXIMUM MOMENTS FOR UNIFORM LAYER



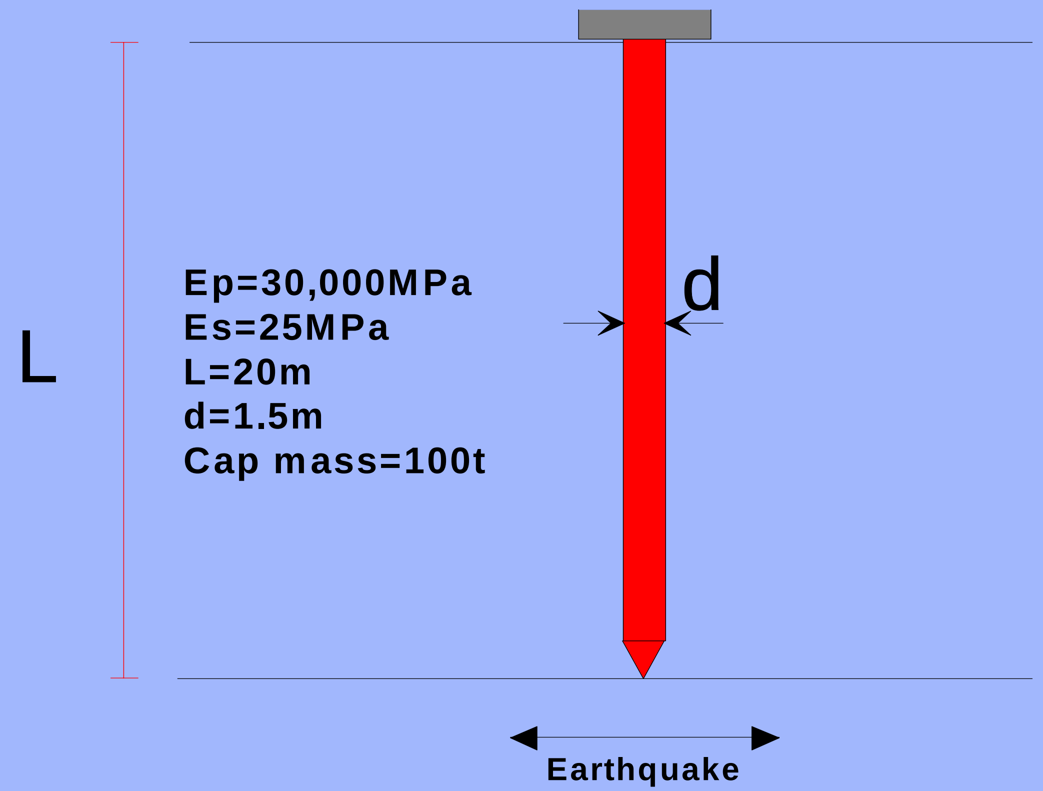
COMPARISON OF SOLUTIONS FOR LAYERED SOIL PROFILE



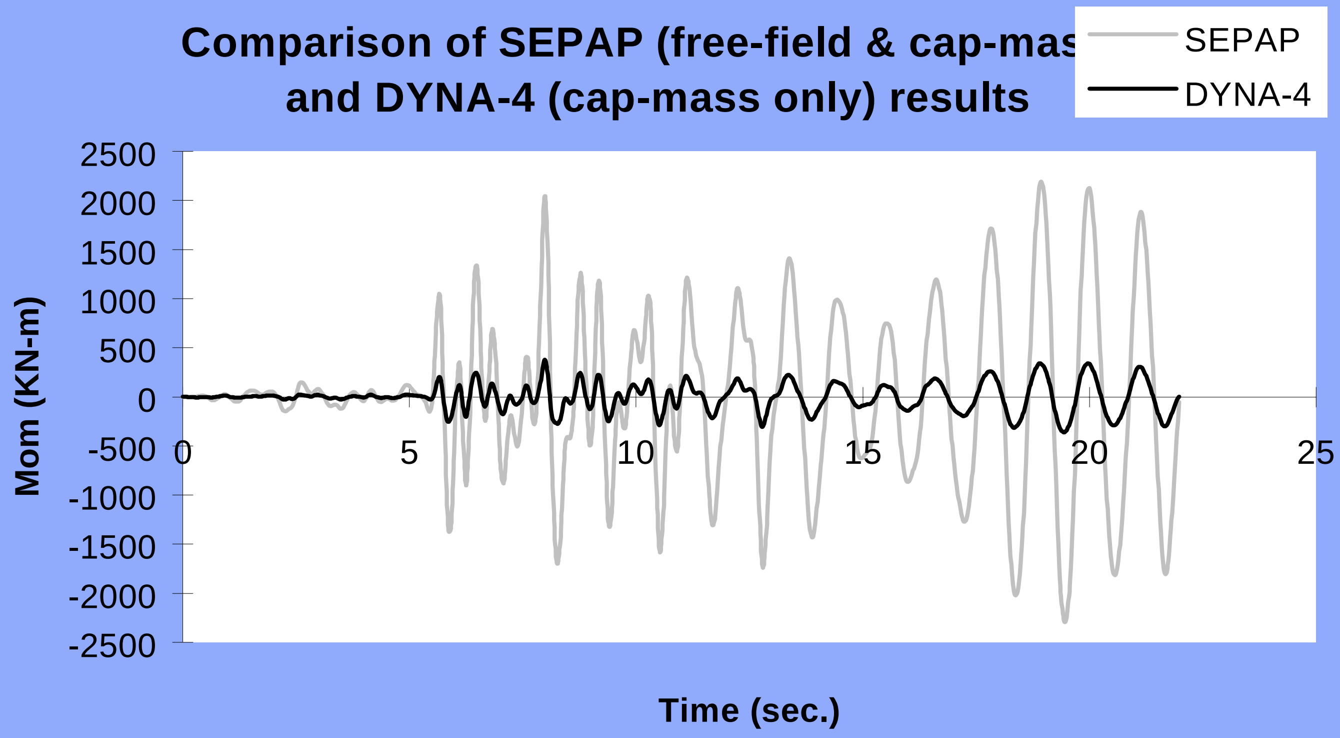
COMPARISON OF PSEUDO-STATIC & DYNAMIC SOLUTIONS FOR HIGHLY NON-HOMOGENEOUS PROFILE



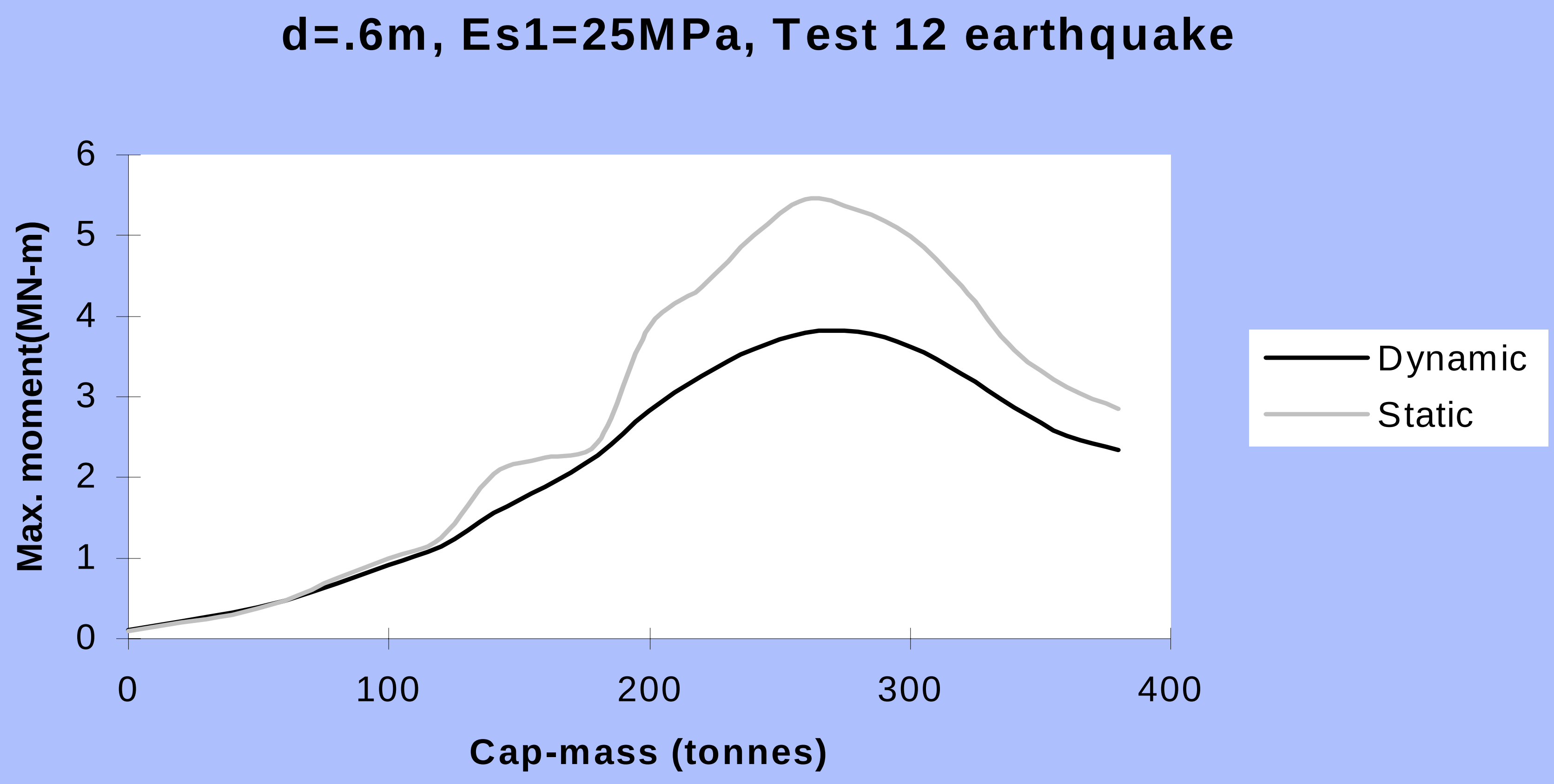
THE IMPORTANCE OF COMBINED INERTIAL & KINEMATIC EFFECTS



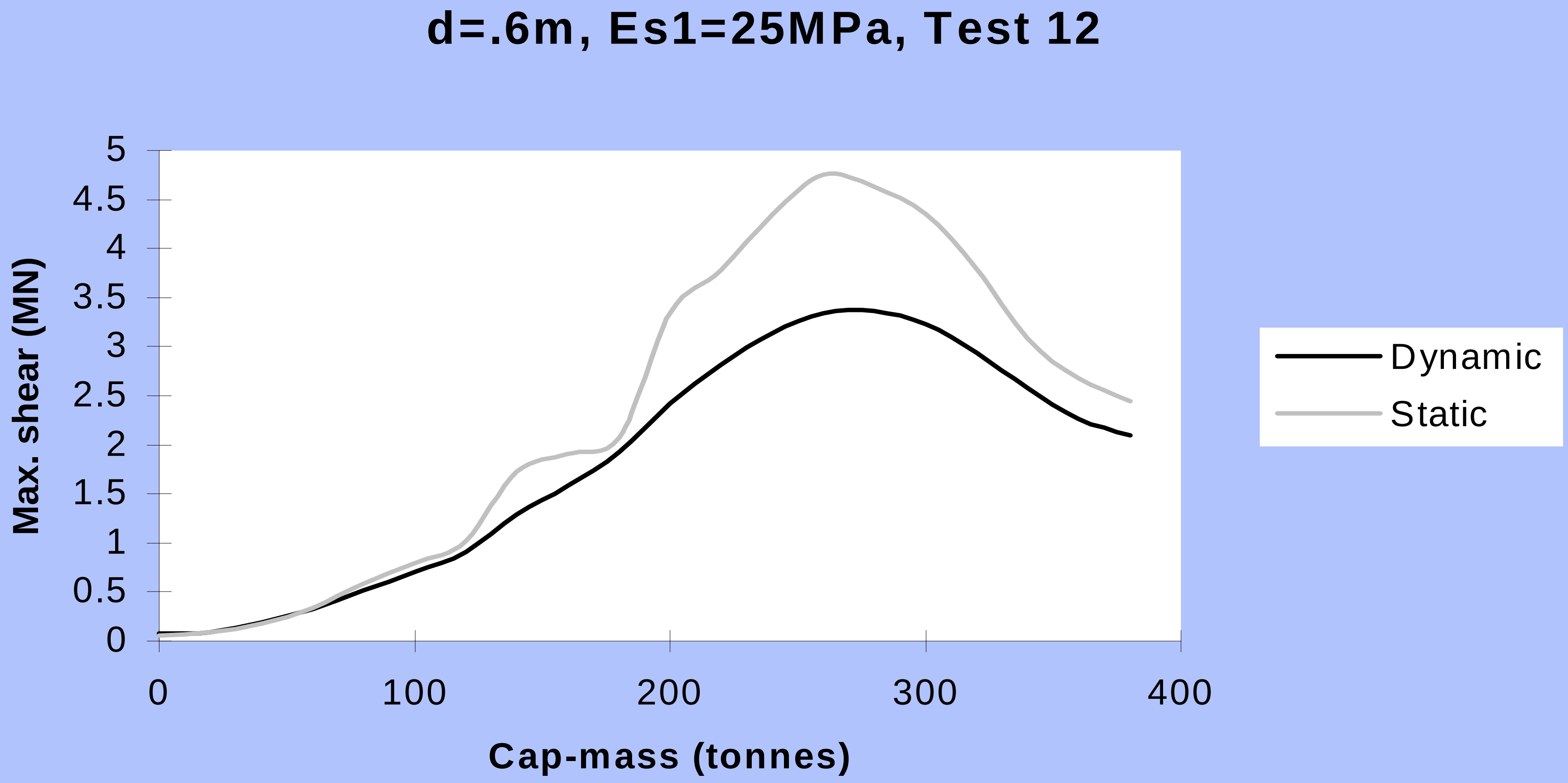
TIME-MOMENT RESPONSES FOR INERTIAL AND INERTIAL + KINEMATIC EFFECTS



DYNAMIC vs PSEUDO-STATIC SOLUTIONS FOR MAXIMUM MOMENT



DYNAMIC vs PSEUDO-STATIC SOULTIONS FOR MAXIMUM PILE SHEAR



FINDINGS

- Cap mass has a very important effect
- There is a cap mass that gives the maximum pile response
- The pseudo-static approach tends to over-estimate moment & shear compared to dynamic analysis
- However, is adequate for most practical purposes

FACTORS AFFECTING SEISMIC PILE RESPONSE

Pile length:

- Can have a critical length where induced moments are largest.

- Depends on frequency content of earthquake and depth of soil.

FACTORS AFFECTING SEISMIC PILE RESPONSE

Pile diameter :

- Larger diameter piles suffer greater induced moments.

FACTORS AFFECTING SEISMIC PILE RESPONSE

Soil modulus :

- Can be important via effects on natural frequency of soil profile.
- Also influences magnitude of ground motions.

FACTORS AFFECTING SEISMIC PILE RESPONSE

Cap Mass :

- Most important – greater mass gives greater inertia force, which induces greater moments in pile.

FACTORS AFFECTING SEISMIC PILE RESPONSE

Nature of Earthquake :

- Important, as site response depends on frequency content and acceleration – time history, as well as maximum acceleration.

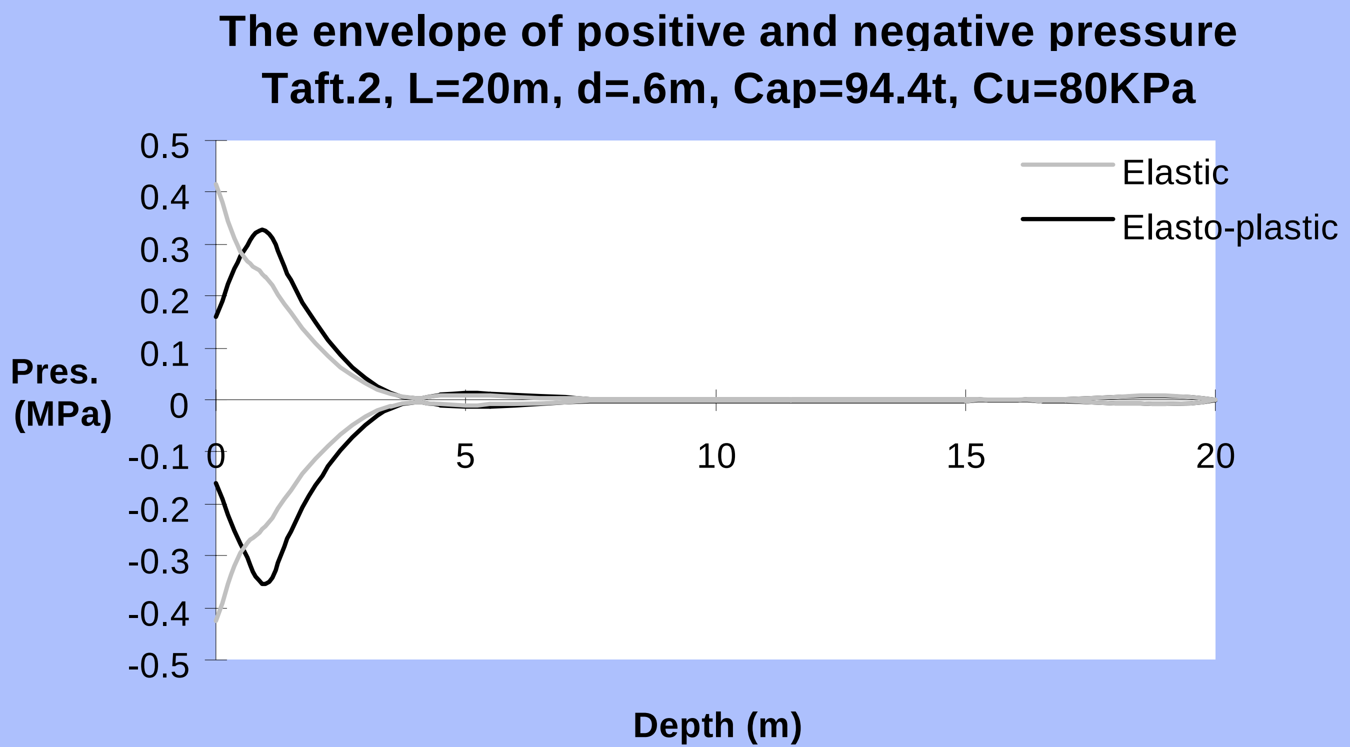
- Effect depends on natural frequency of soil layer, compared to predominant frequency of earthquake.

EFFECTS OF SOIL YIELDING

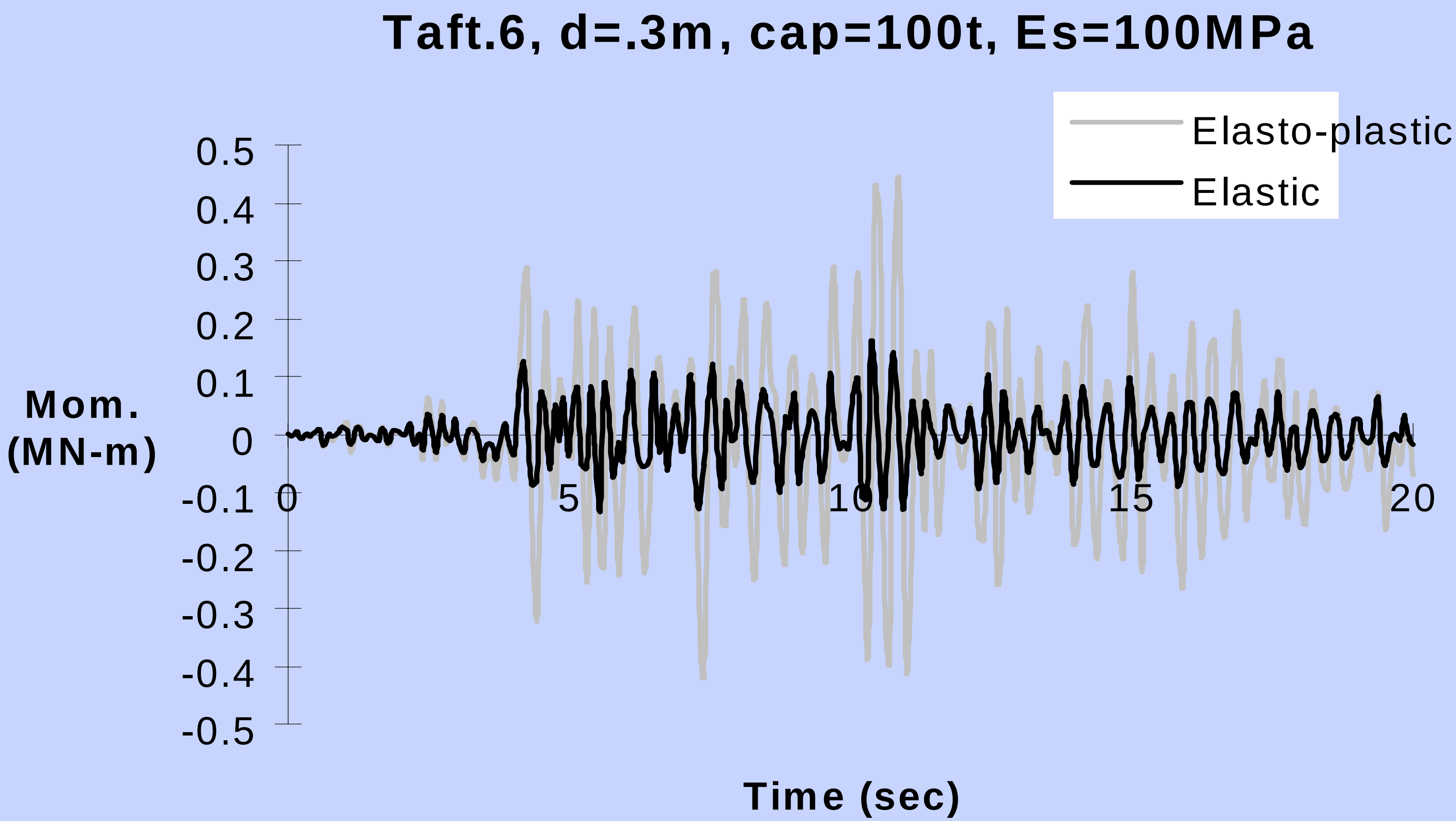
- Soil yielding generally leads to:
  - an increase in maximum relative pile displacement;
  - An increase in maximum pile moment;
  - A reduction in the maximum shear.

- Soil yielding is more likely to occur:
  - When a cap mass is present;
  - When the pile is relatively slender and/or flexible;
  - When the soil modulus is constant with depth, rather than increasing.

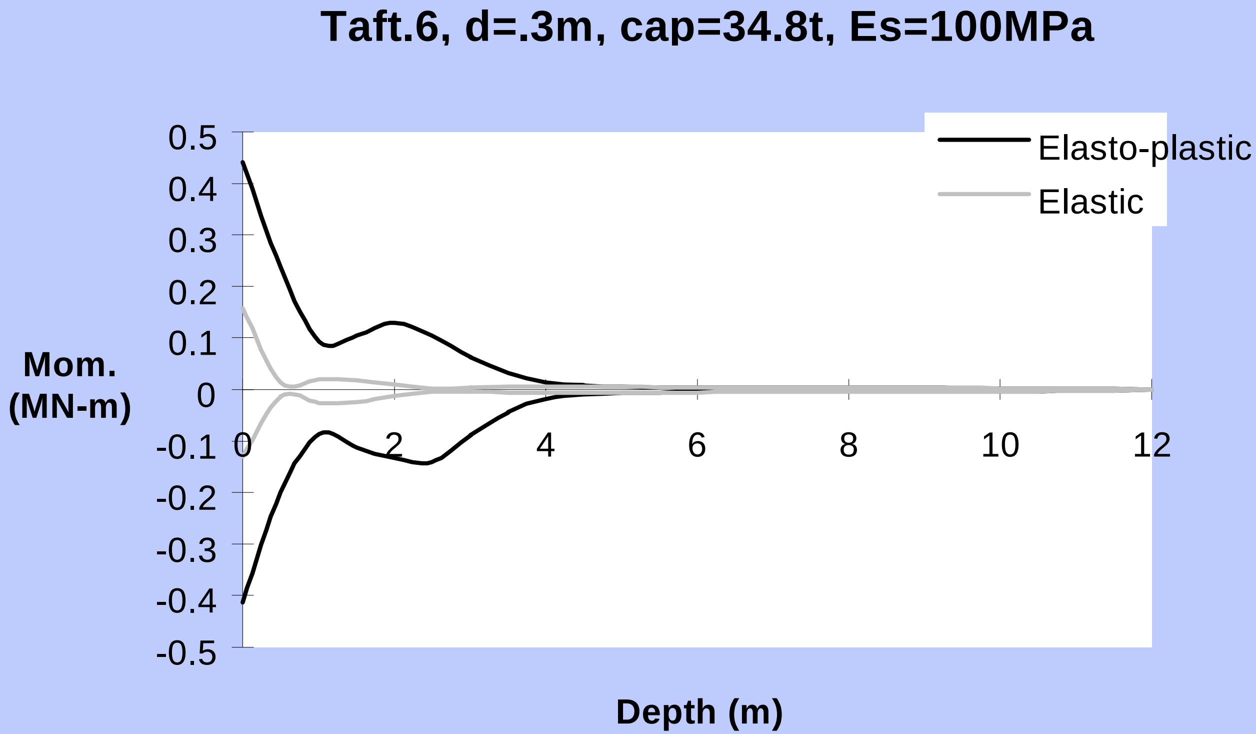
EFFECT OF SOIL NON-LINEARITY ON MAXIMUM MOMENT ENVELOPES



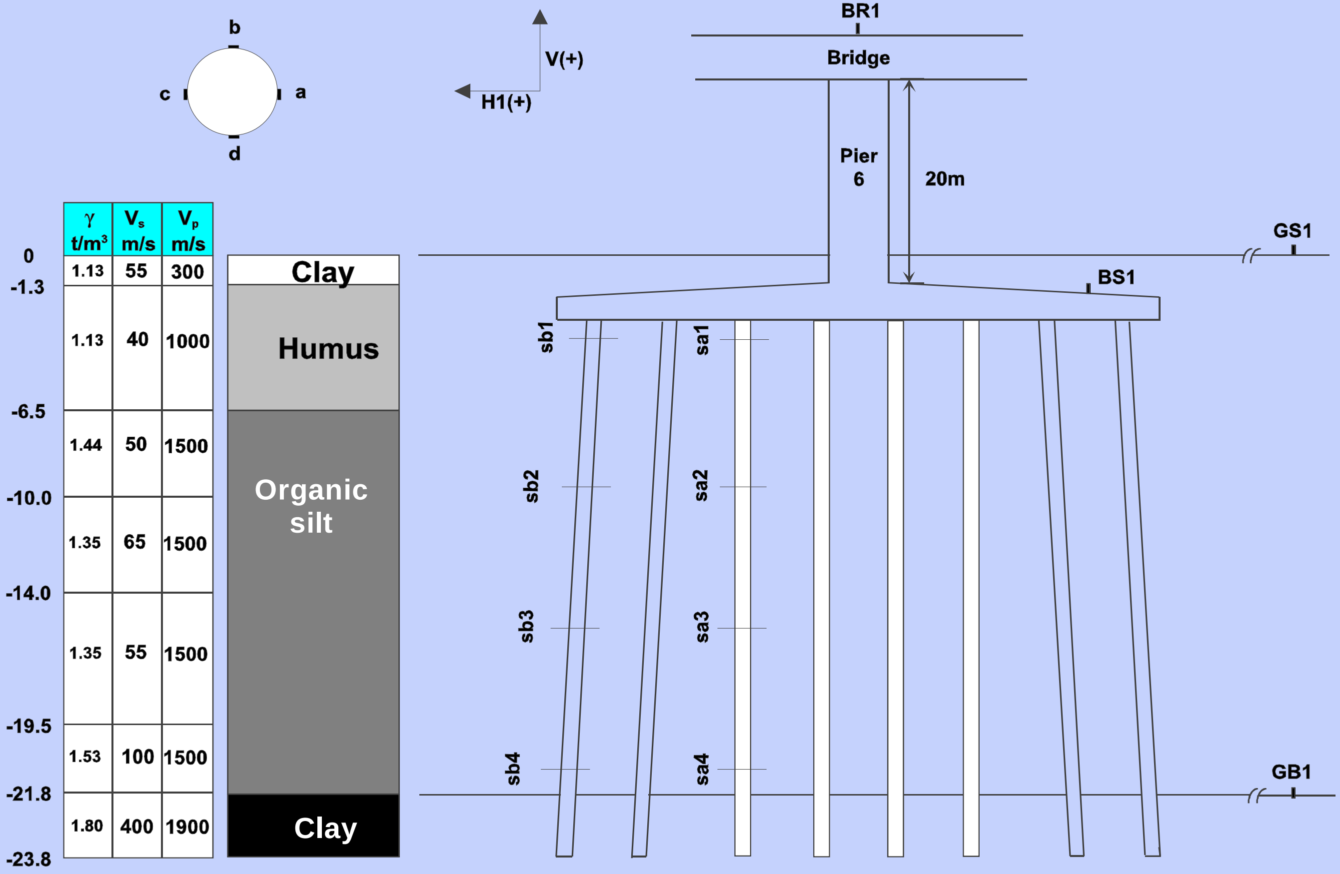
EFFECT OF SOIL NON-LINEARITY ON MOMENT-TIME RESPONSE



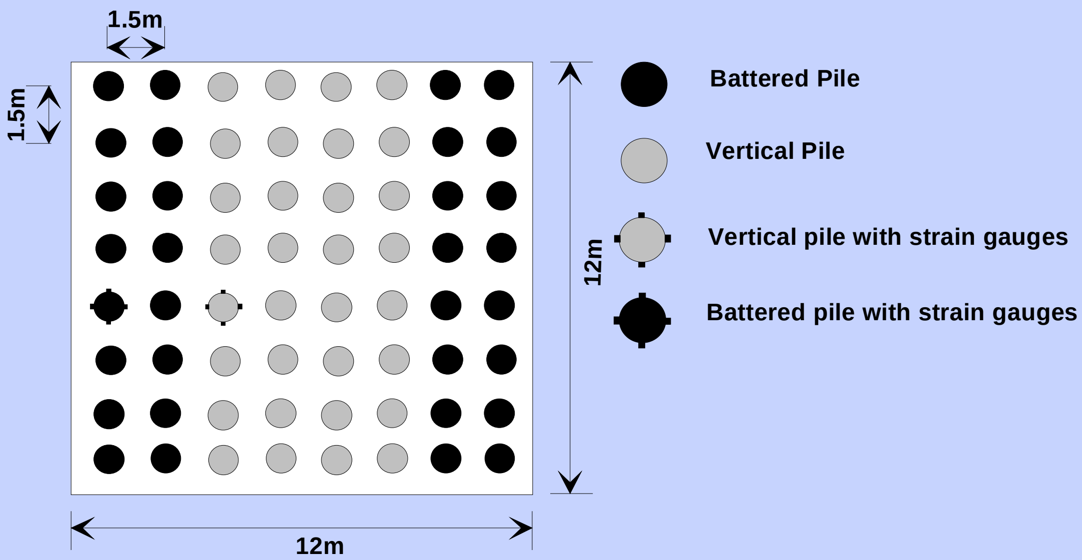
EFFECT OF SOIL NON-LINEARITY ON MAXIMUM MOMENT ENVELOPE ALONG PILE



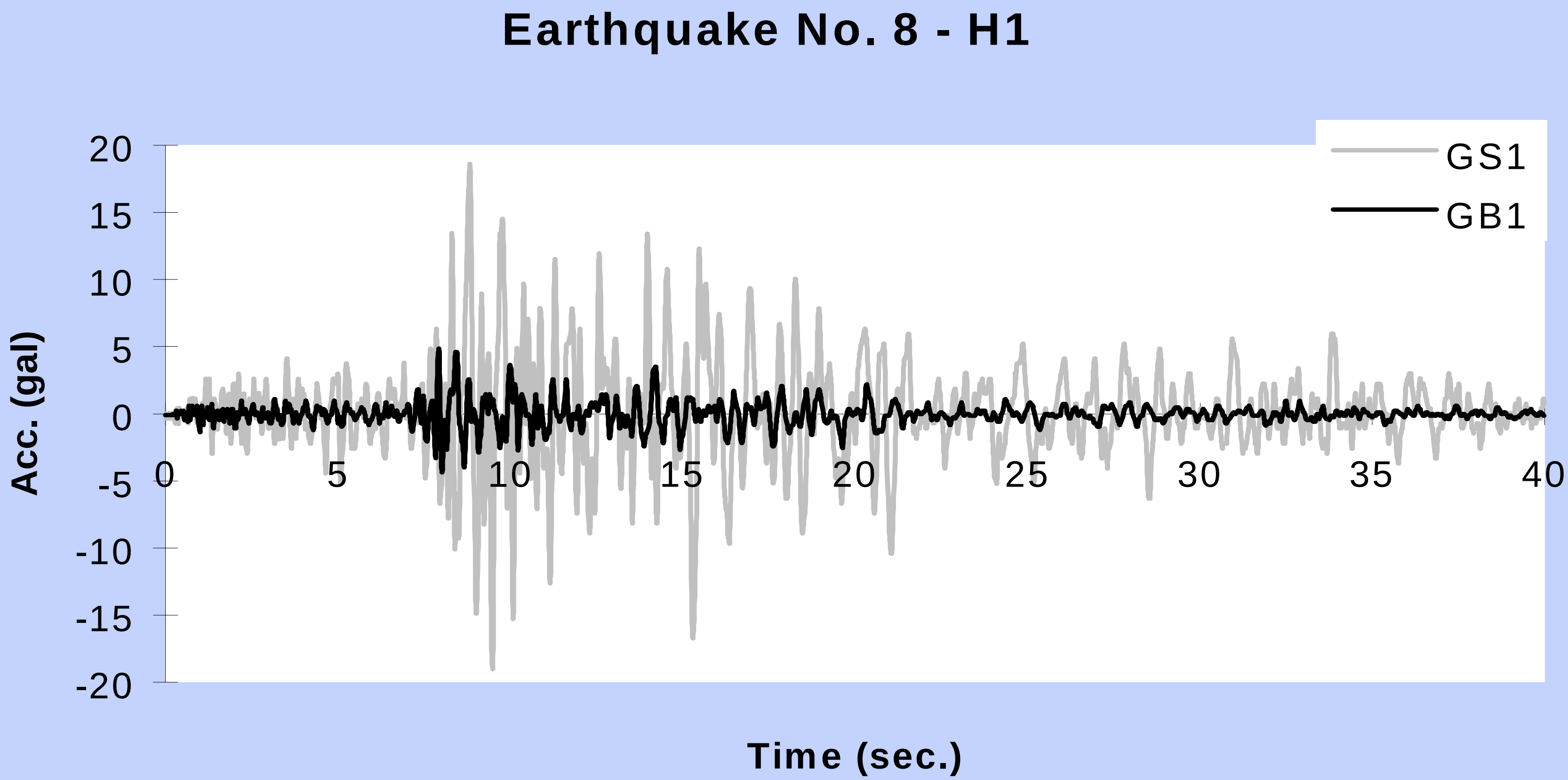
DETAILS OF OHBA-OHASHI BRIDGE



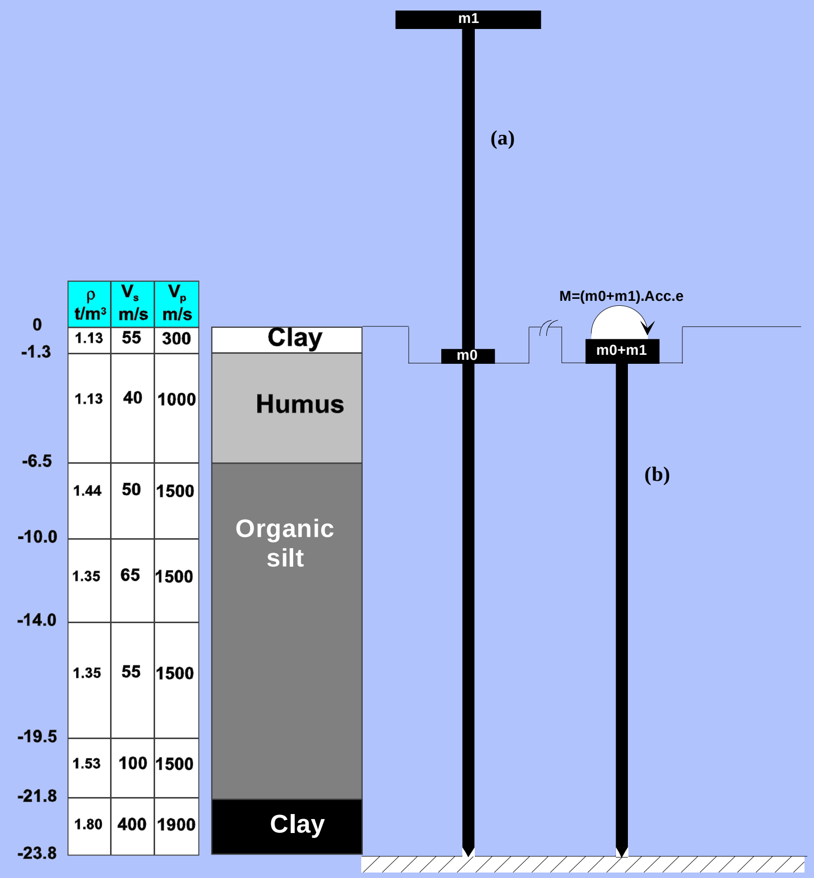
LAYOUT OF PILES



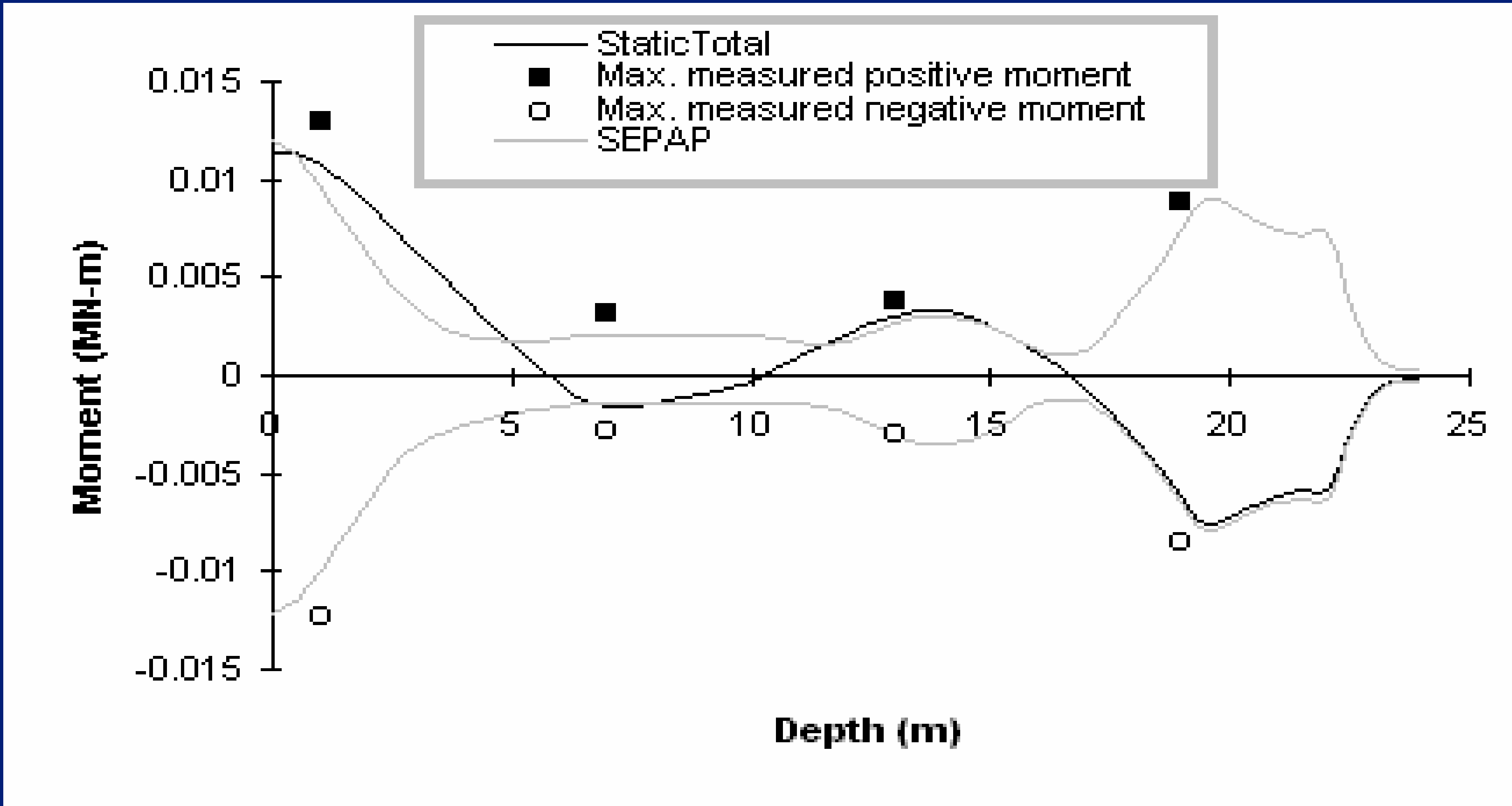
MEASURED BASE & SURFACE TIME-ACCELERATION HISTORIES



SIMPLIFIED COMPUTATIONAL MODEL



MEASURED & COMPUTED MAXIMUM MOMENTS



EFFECTS OF SOIL LIQUEFACTION

Increase in pore pressure degrades soil stiffness and strength → Reduces ability of pile foundations to support structures

- For safe and economic design of pile foundations, numerical simulation of the pile behaviour in liquefying soil is important.

- Verified numerical procedures are required to predict

  ground → displacements and excess pore pressures

  Pile → bending moments and displacements.

Numerical Model

(1) Ground response analysis

(2) Pile analysis

Free field displacements, degraded soil stiffness and effective vertical stress of the soil due to pore pressure generation obtained from an effective stress based ground response analysis are used for the seismic analysis of the pile.

SUMMARY OF NUMERICAL PROCEDURE

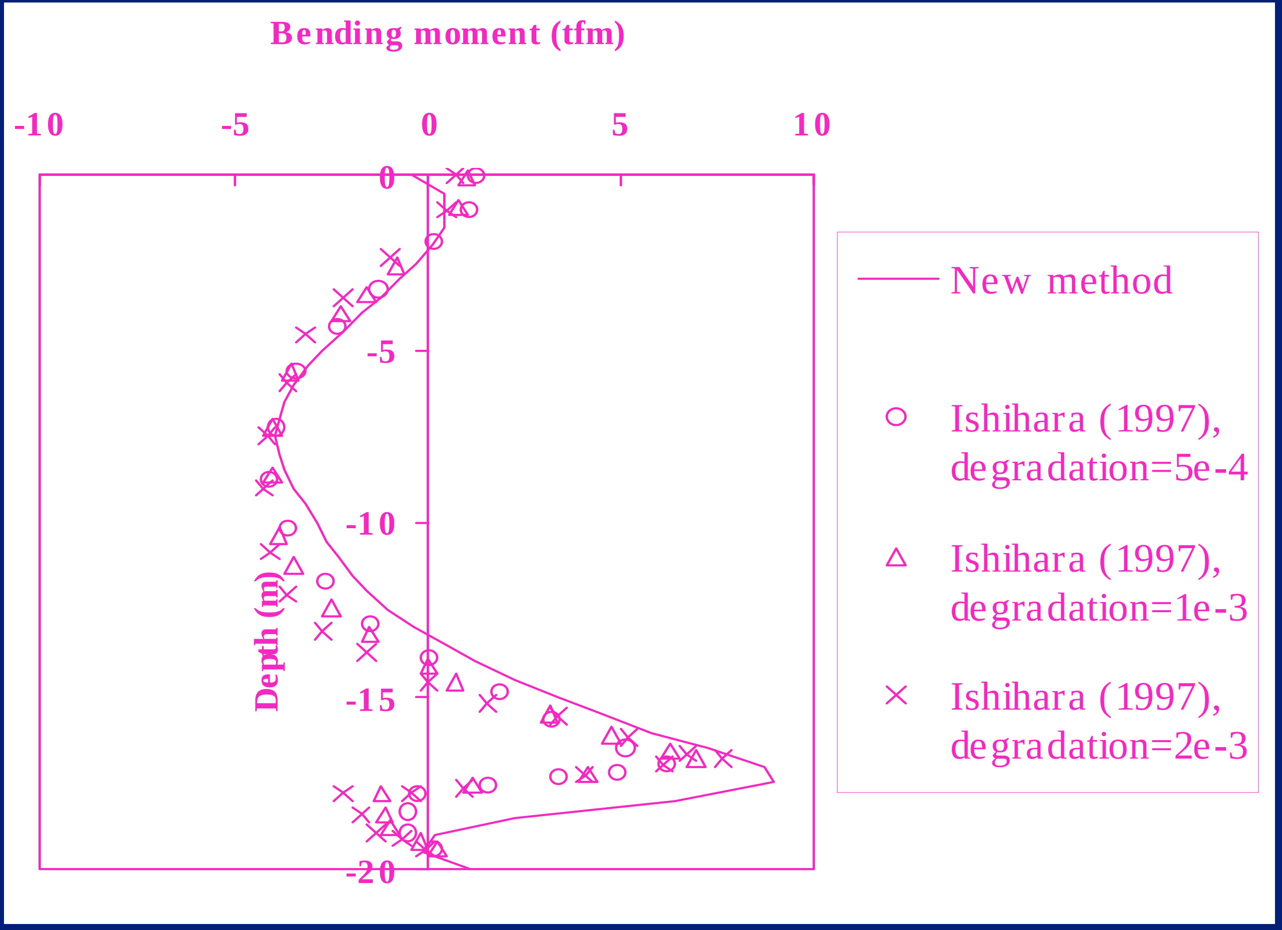
- **Free field ground displacements and degraded soil stiffness obtained from an effective stress based ground response analysis are used to obtain pile performance.**

- **Spring coefficients for the Winkler model used for the pile-soil interface are derived from Mindlin's solution.**

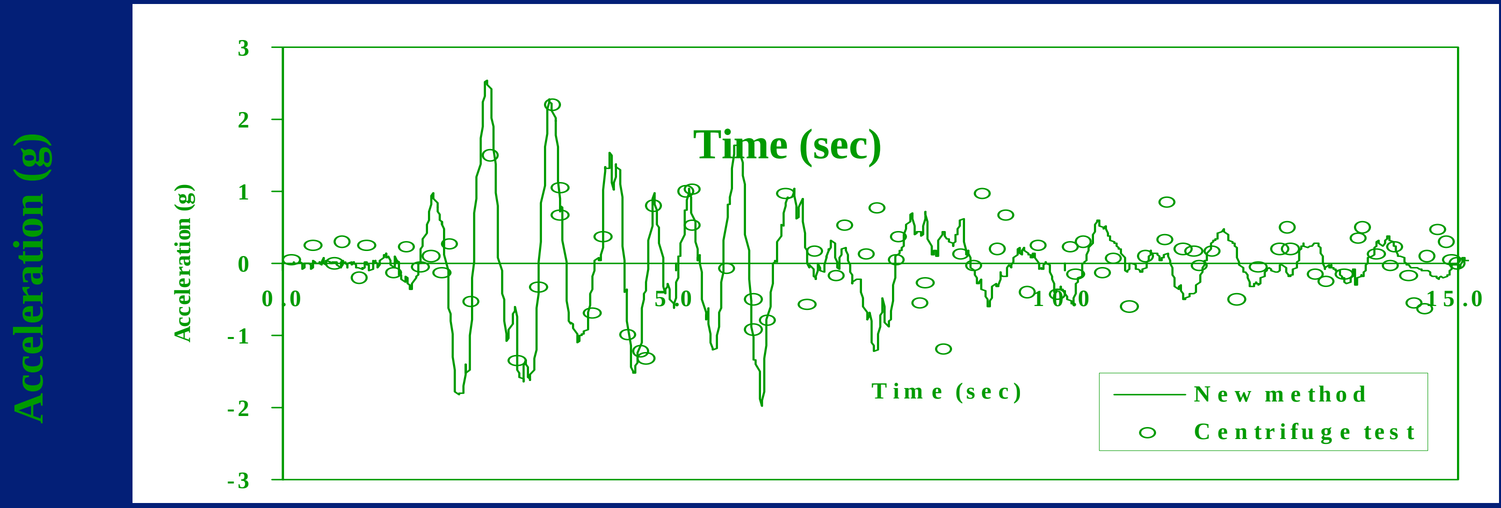
- **Centrifuge tests simulated from the new method shows excellent agreement with the observed behaviour.**

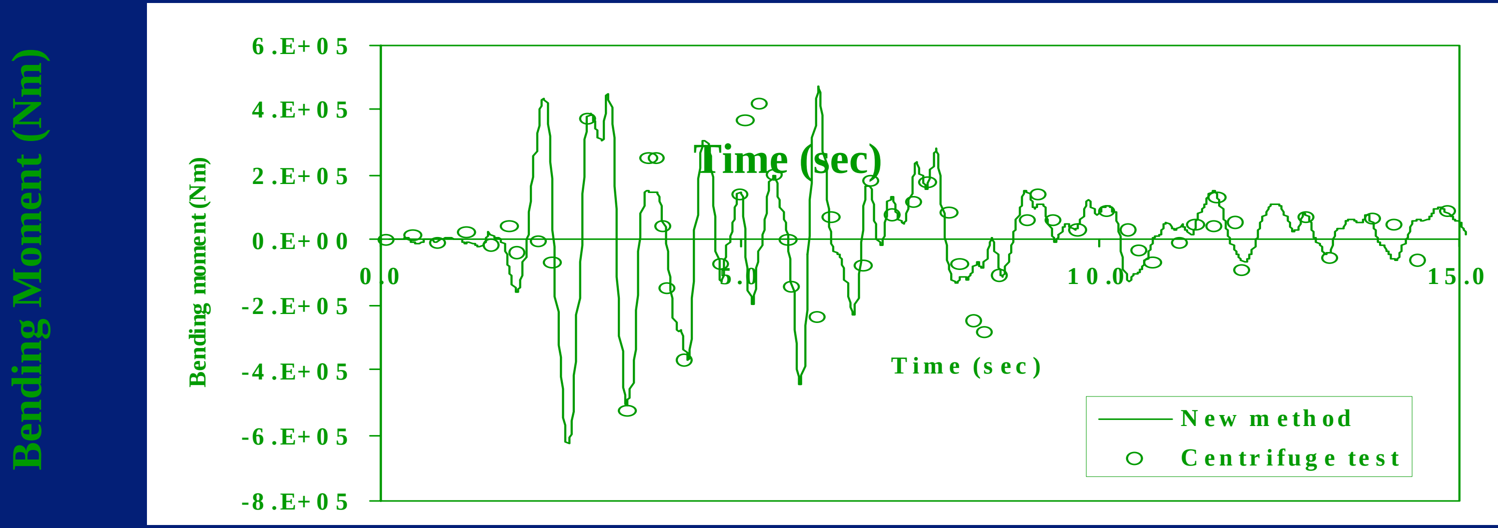
- **Good agreement also with the method developed by Ishihara in Japan.**

Comparison of pile bending moments with results given by Ishihara (1997)

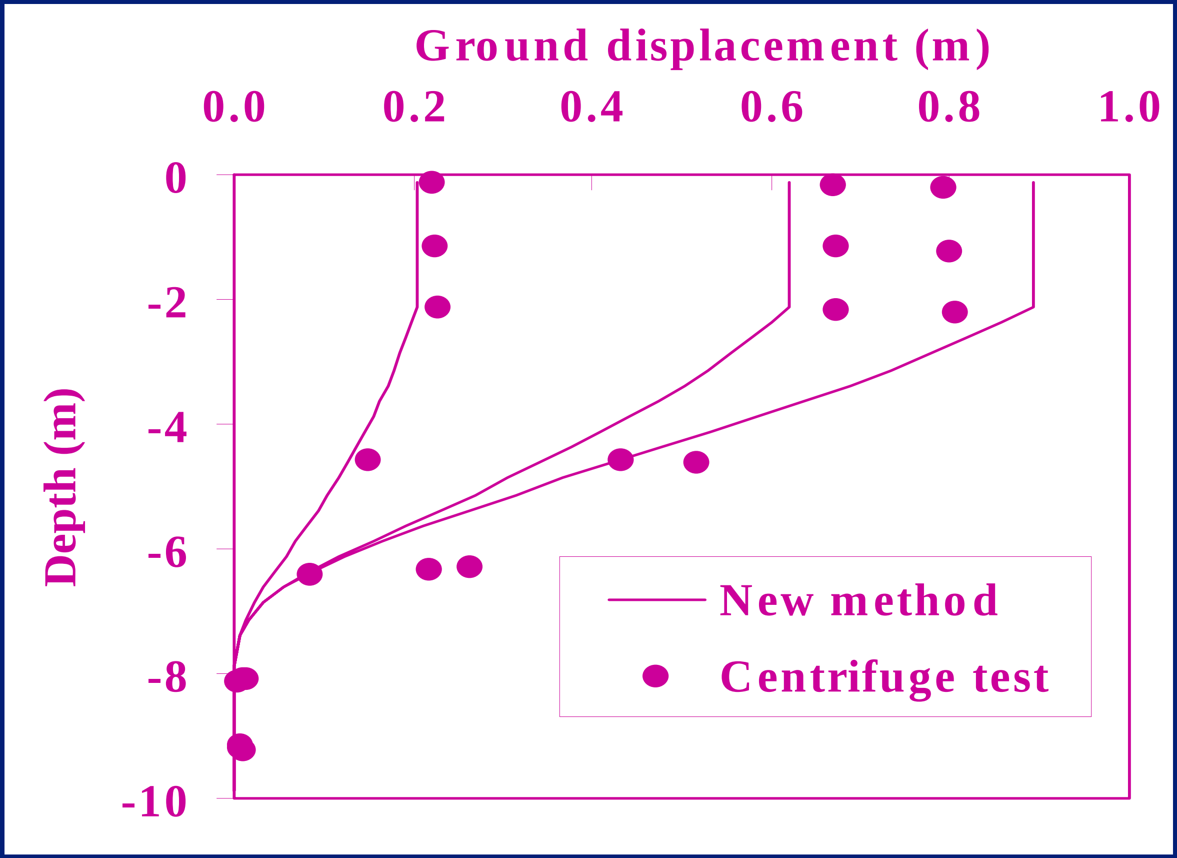


Comparison of superstructure acceleration and pile bending moment at 2.3 m depth with centrifuge test by Wilson *et al*., 1999.

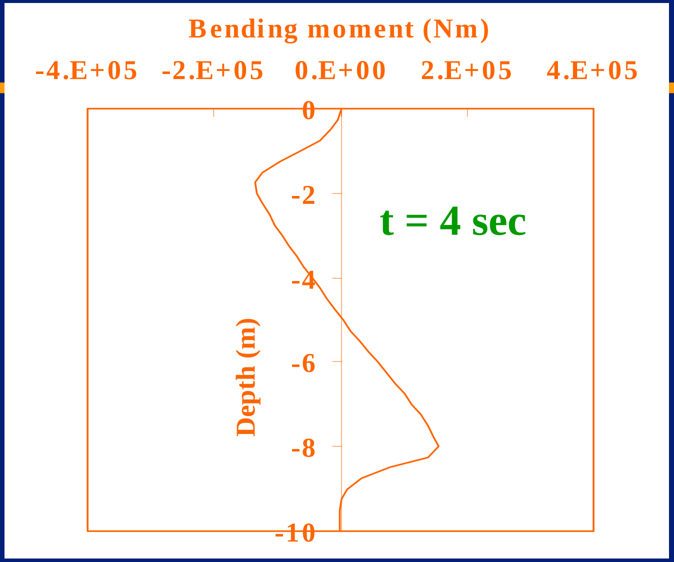


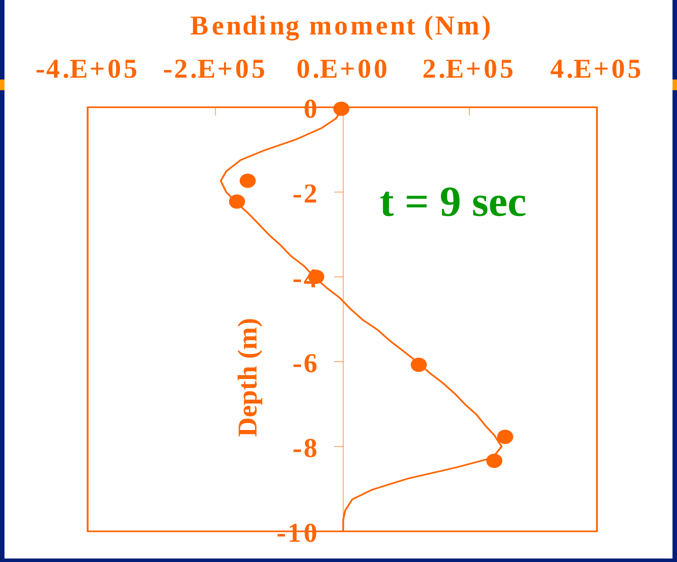


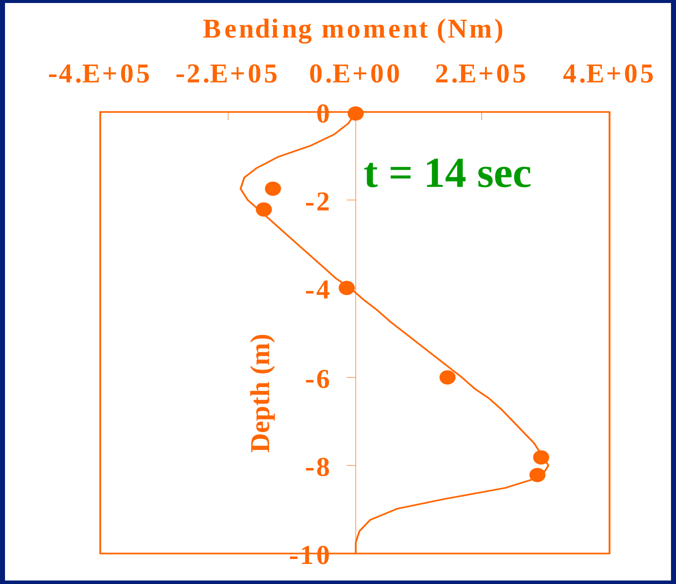
Comparison of ground displacement profiles with centrifuge test by Abdoun *et al*., 1997.

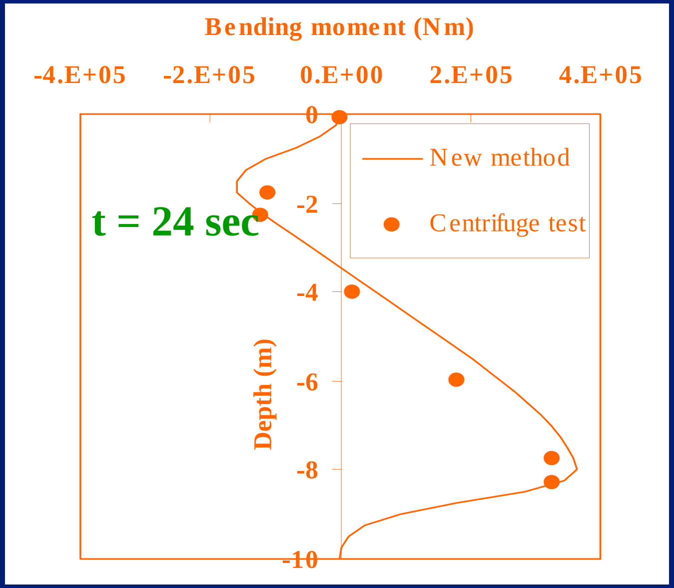


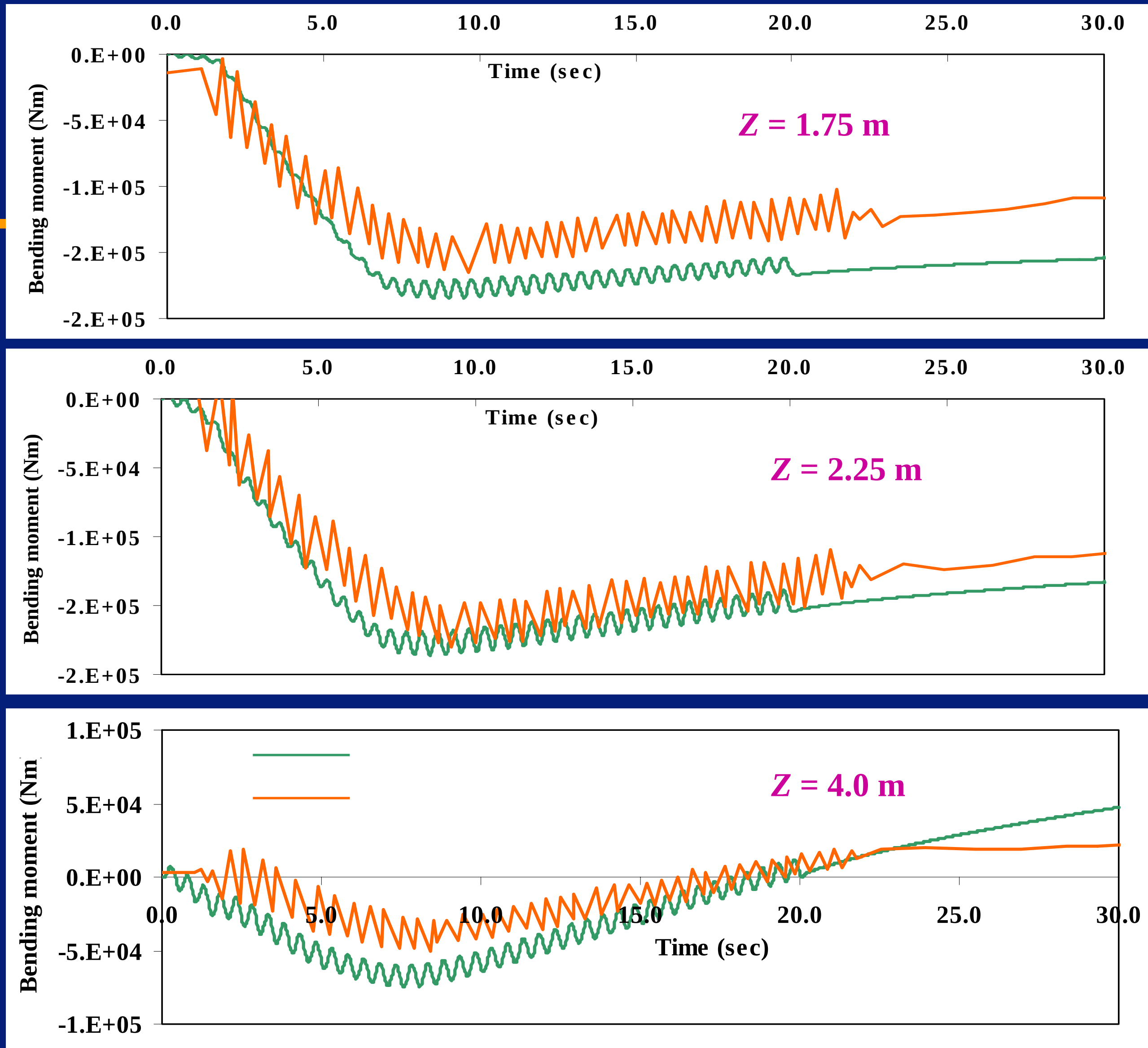
Comparison of bending moment profiles with centrifuge test by Abdoun et al., 1997.



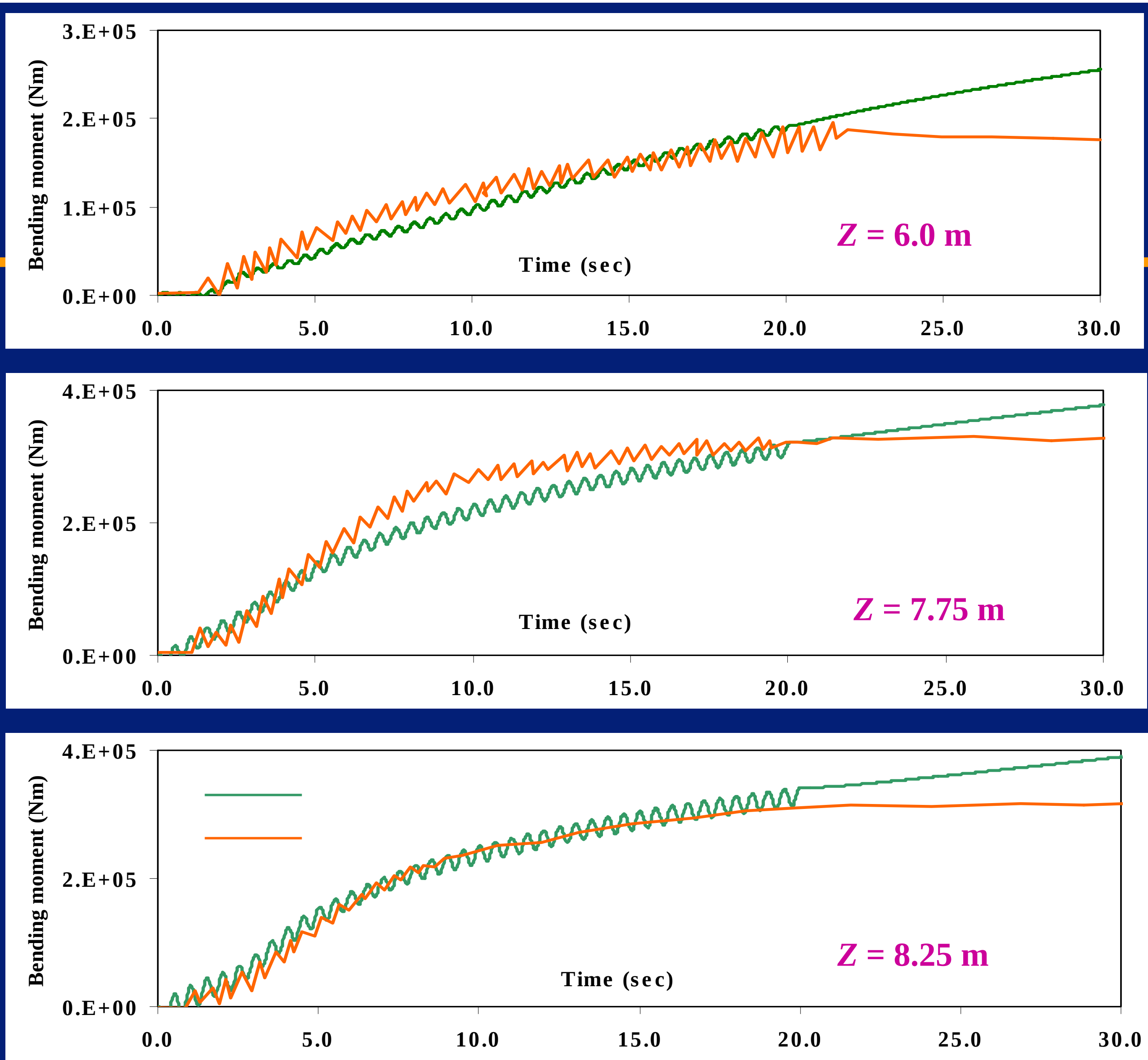








Comparison of time histories of bending moments with centrifuge data by Abdoun et al., 1997.



Comparison of time histories of bending moments with centrifuge data by Abdoun et al., 1997.

CONVENIENT DESIGN METHODS; JAPAN

- **Design Standards:**

  Subgrade reaction modulus reduced in liquefied zone, depending on:
  - Safety factor against liquefaction
  - Depth below ground surface
  - SPT value, adjusted for overburden pressure & fines content
  - Correction factors between 0 and 1.0.

CONVENIENT DESIGN METHODS; JAPAN

Table 3.11. Reduction factor of coefficient of subgrade reaction in horizontal direction (recommendations for design of building foundations).

Range of safety factor against liquefaction F_L	depth from ground surface z (m)	reduction factor r_k by which coefficient of subgrade reaction in horizontal direction is multiplied			
		$N_a \leq 8$	$8 < N_a \leq 14$	$14 < N_a \leq 20$	$20 < N_a$
$F_L \leq 0.5$	$0 \leq z \leq 10$	0	0	0.05	0.1
	$10 < z \leq 20$	0	0.05	0.1	0.2
$0.5 < F_L \leq 0.75$	$0 \leq z \leq 10$	0	0.05	0.1	0.2
	$10 < z \leq 20$	0.05	0.1	0.2	0.5
$0.75 < F_L \leq 1.0$	$0 \leq z \leq 10$	0.05	0.1	0.2	0.5
	$10 < z \leq 20$	0.1	0.2	0.5	1.0

CONVENIENT DESIGN METHODS; JAPAN

- **Ishihara & Cubrinovski (1998):**
  - Backfigured data from Kobe (1995) event.
  - Reduction factor in liquefied zone typically between $2*10^{-4}$ and $2*10^{-2}$.

CONVENIENT DESIGN METHODS; NIKOLAOU et al, 2001

- For simple first estimates, the procedures developed by Nikolaou et al can be used.
- Critical locations for induced moment are at layer interfaces.
- Procedure developed for kinematic bending of pile.

SIMPLIFIED DESIGN METHOD (1)

1. Compute characteristic shear stress τ_c:
   $\tau_c = a_s . \rho_1 . h_1$
2. Estimate relative layer stiffness G_2/G_1, pile-soil stiffness ratio E_p/E_1, and pile length-to- ratio L/d.
3. Compute shear wave velocities V_1, V_2, from G_1, G_2

SIMPLIFIED DESIGN METHOD (2)

4. Maximum Moment is approximated as:

$$0.042.\tau_c.d^3. (L/d)^{0.30} (E_p/E_1)^{0.65} (V_2/V_1)^{0.50}$$

Modified approach developed by Mylonakis & Nikolaou (2002). Gives similar results.

SIMPLIFIED DESIGN METHOD (3)

The simplified approach can also be used for liquefied deposits.

The challenge is to estimate the shear modulus ratio at the interface between the liquefied and non-liquefied deposits.

SIMPLIFIED DESIGN METHOD - MODIFIED PSEUDOSTATIC APPROACH

The simplified pseudostatic approach developed for non-liquefying soil profiles can also be used for liquefied deposits.

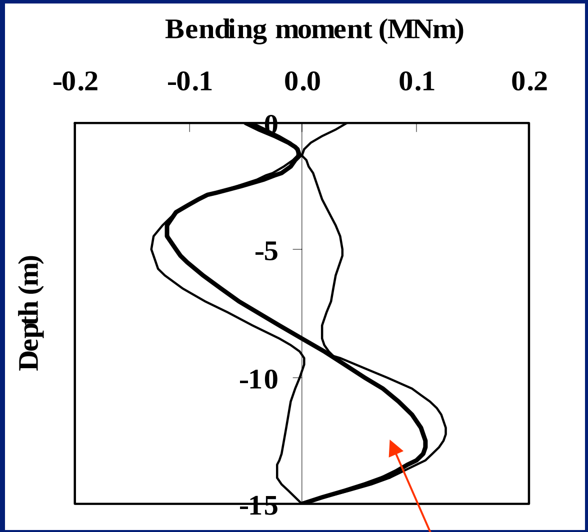
1. Carry out free-field site response analysis taking account of pore pressure generation and dissipation.
2. Model superstructure as concentrated mass at pile head.
3. Apply static lateral force to pile head

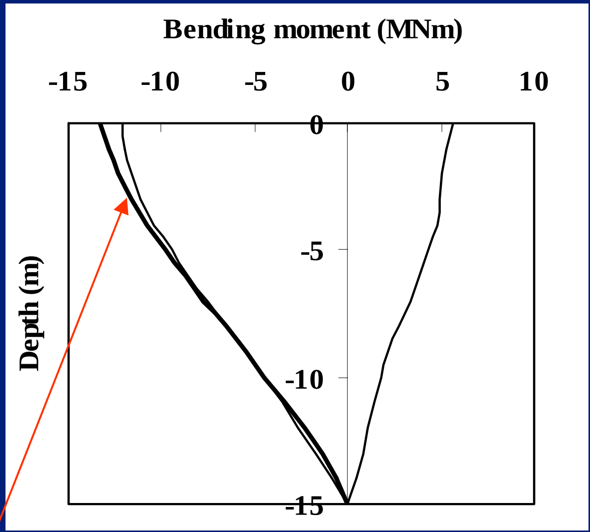
   = cap mass* maximum ground acceleration

SIMPLIFIED DESIGN METHOD - MODIFIED PSEUDOSTATIC APPROACH (2)

4. Apply static lateral ground deflection at each point along pile, equal to maximum computed deflection from ground response analysis.
5. Reduce soil modulus and ultimate lateral pile-soil pressure according to computed maximum pore pressures from ground response analysis.
6. Carry out non-linear static analysis of pile, to obtain profiles of maximum pile deflection, moment and shear.

CALCULATED MOMENT PROFILES FROM DYNAMIC & PSEUDO-STATIC ANALYSES





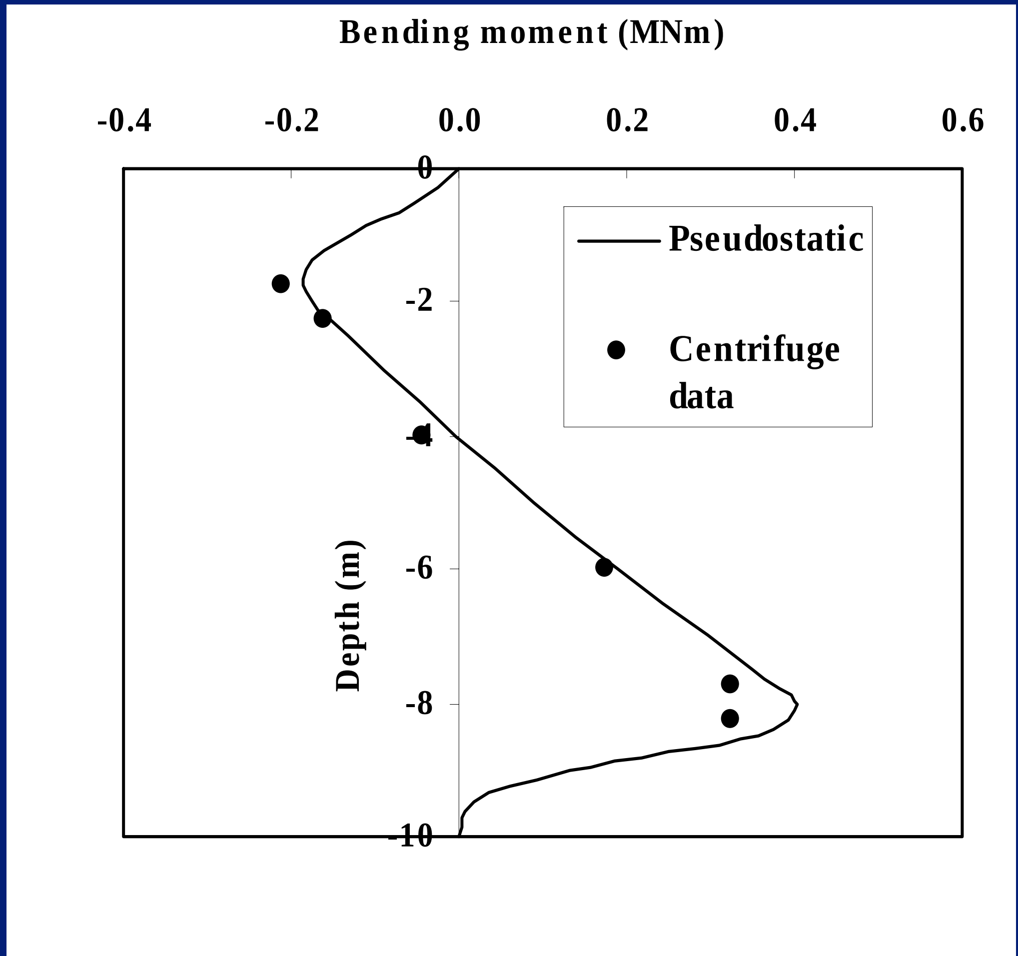
d=0.3 m

d=1.2 m

Pseudo-static

Coffey Geosciences

MEASURED vs CALCULATED MAXIMUM BENDING MOMENT FOR CENTRIFUGE TEST OF ABDOUN ET AL (1997)



OBSERVED CRACKING IN PILE 211

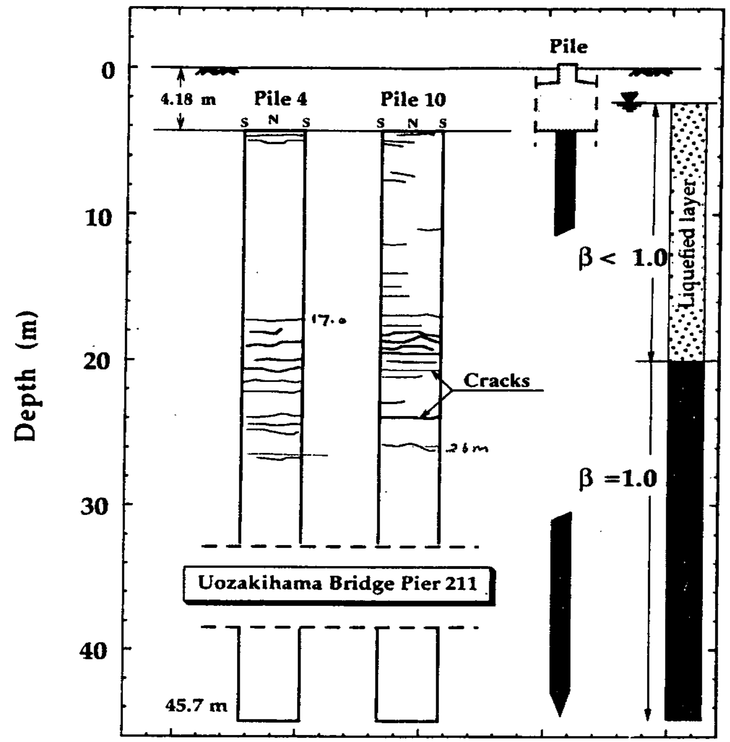
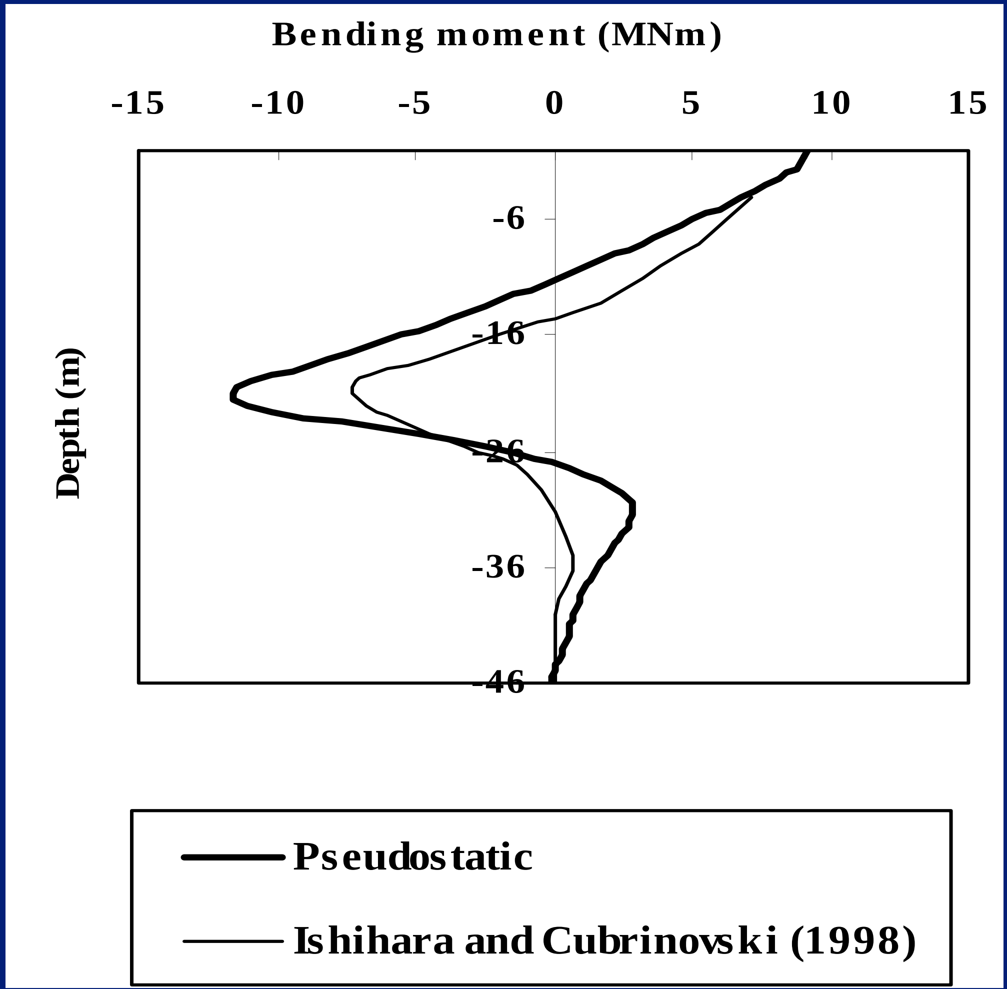


Figure 8. Cracks observed in the piles at bridge pier 211 in Uozakihama Island (Ishihara and Cubrinovski, 1998).

CALCULATED BENDING MOMENT vs DEPTH



SOME ASPECTS OF VERTICAL PILE RESPONSE

Generation of excess pore pressures during earthquake leads to:

- Ground settlement
- Negative friction on pile
- Additional pile settlement
- Additional axial, force in pile
- Temporary loss of pile capacity

ESTIMATION OF GENERATED PORE PRESSURES

- For sands, as per Seed et al approach
- For clays, use approach suggested by Matsuda & Ohara (1988, 1991)
- Pore pressures depend on:
  - Initial effective stress
  - Number of cycles
  - Dynamic strain amplitude
  - Soil characteristics

EXAMPLE PROBLEM

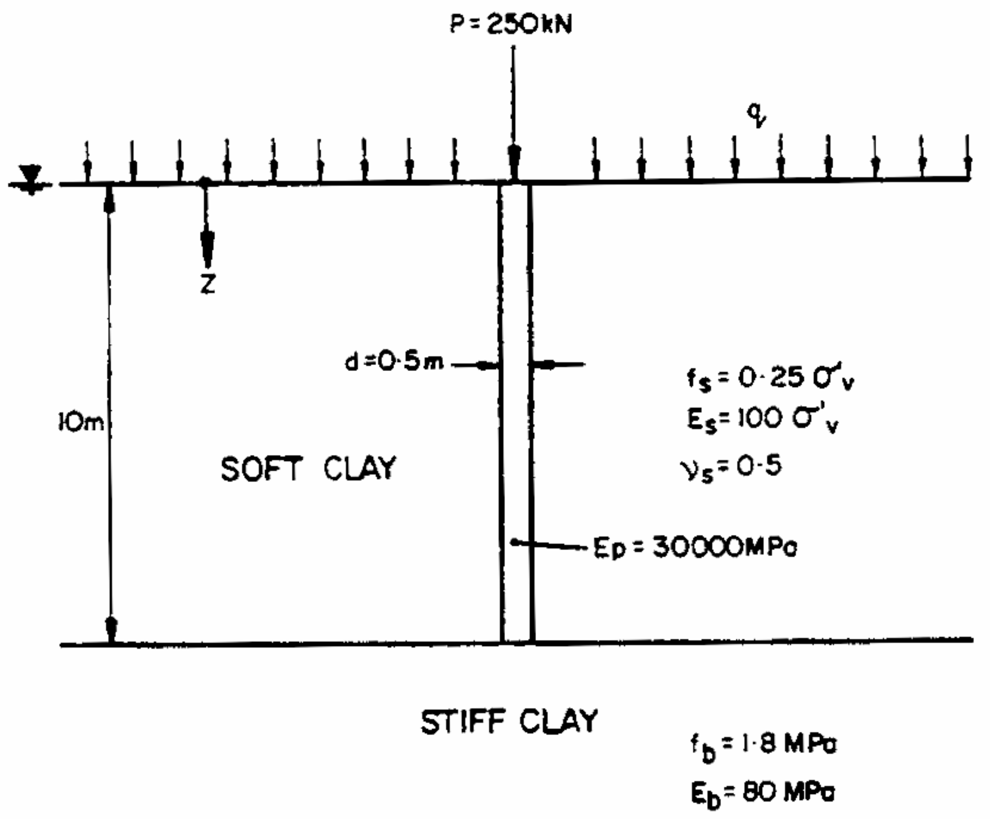


FIGURE 6. Pile Problem Analysed

COMPUTED EXCESS PORE PRESSURES

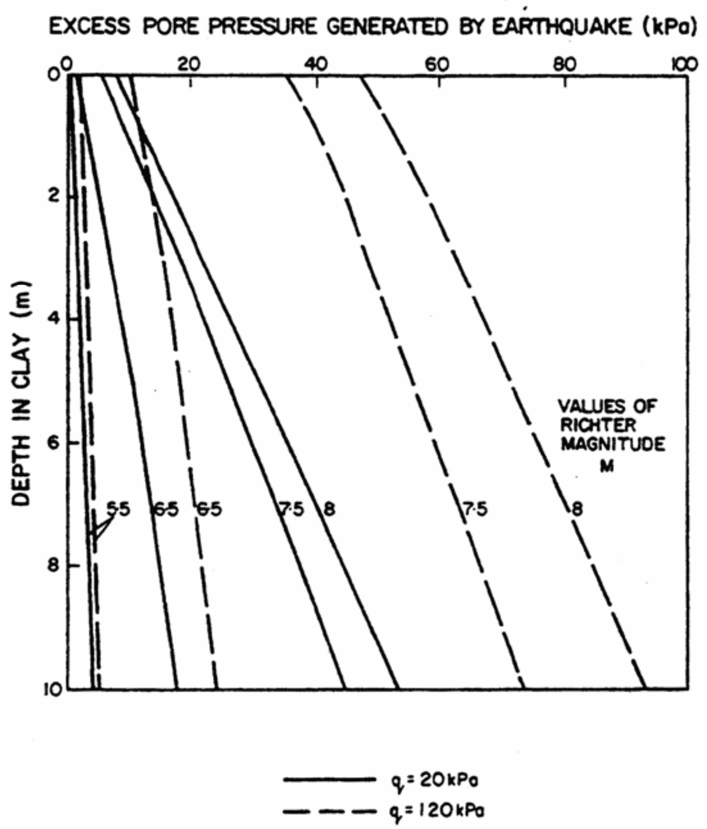


FIGURE 3. Excess Pore Pressures Generated by Earthquake

Excess pore pressures increase with:

•Increasing earthquake magnitude

•Increasing surcharge pressure

COMPUTED RATE OF SETTLEMENT AFTER EARTHQUAKE

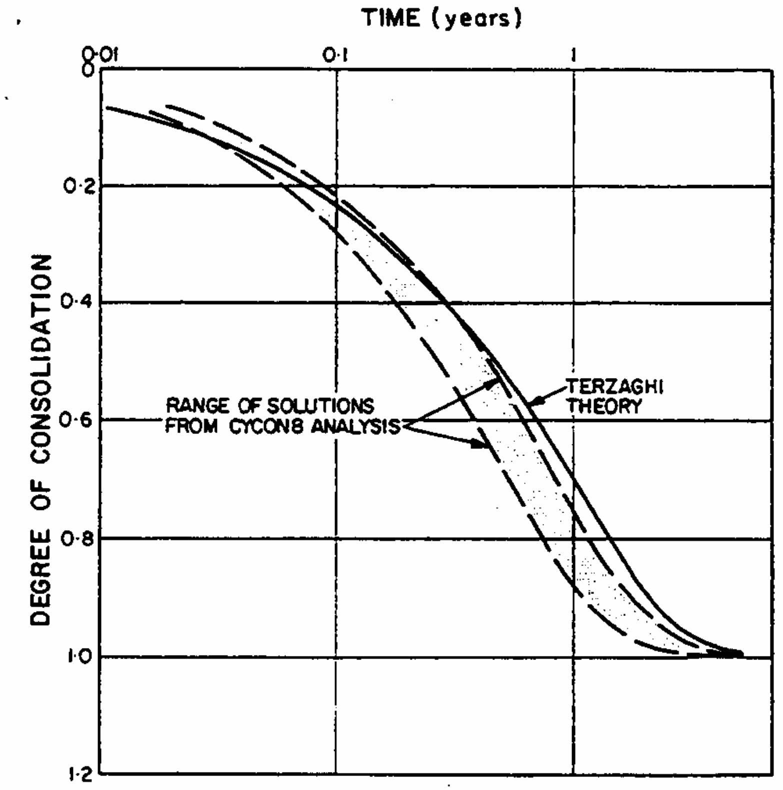


FIGURE 5. Solutions for Rate of Consolidation

Rate of settlement is similar to conventional Terzaghi 1-D analysis

COMPUTED FINAL SETTLEMENT vs EARTHQUAKE MAGNITUDE

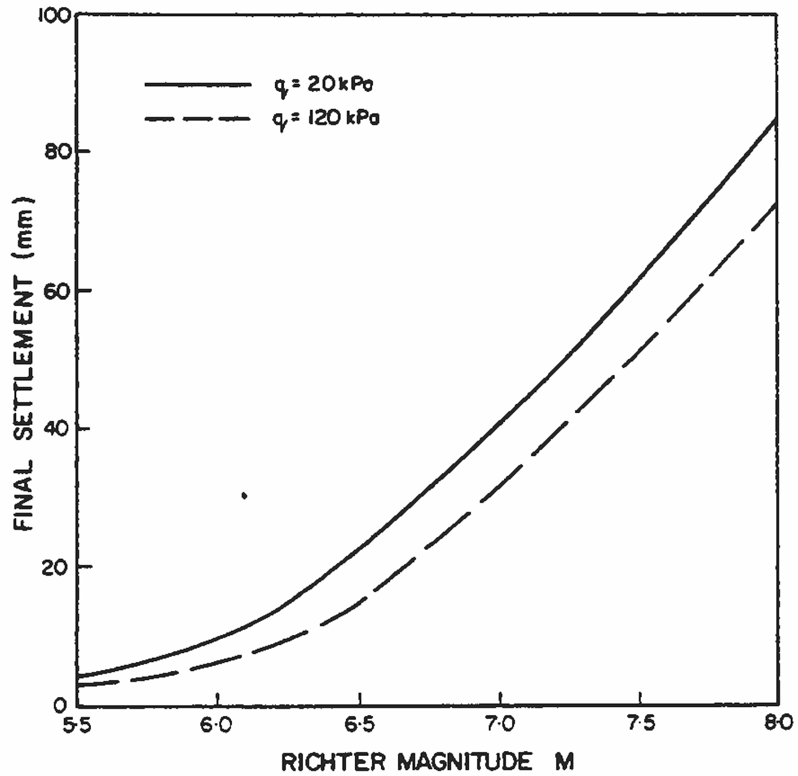


FIGURE 4. Computed Final Settlements Due to Earthquake Excitation

Final settlement increases with increasing earthquake magnitude and surcharge pressure

LOSS OF SHAFT CAPACITY DURING EARTHQUAKE

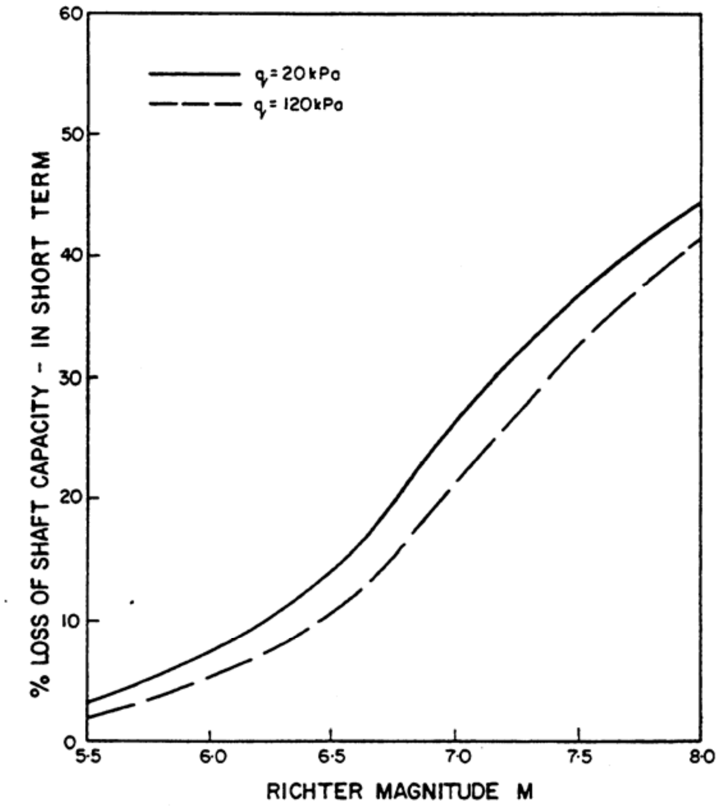


FIGURE 7. Relative Loss of Shaft Capacity Immediately After Earthquake

Loss of capacity is relatively small unless earthquake magnitude is very large

INDUCED AXIAL FORCE & SETTLEMENT OF PILE

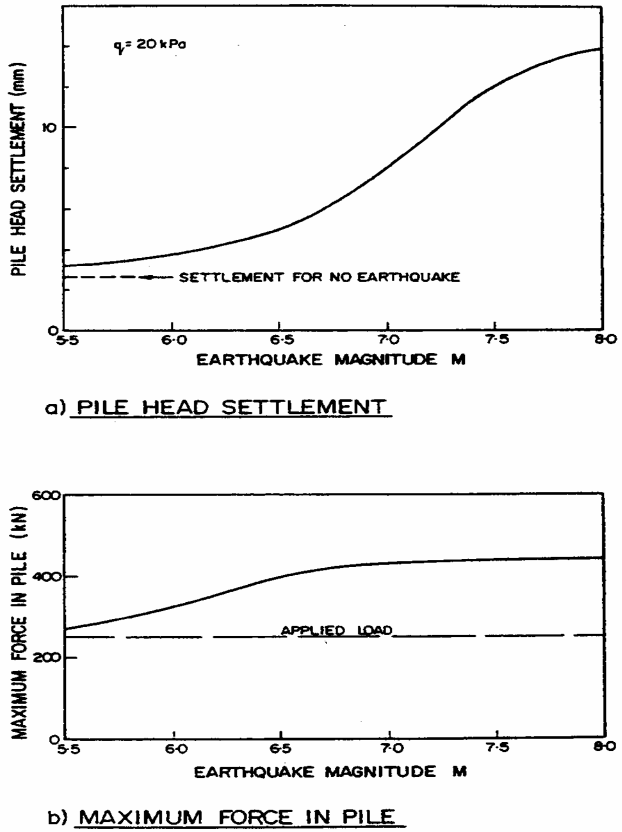


FIGURE 8. Influence of Earthquake on Long-Term Behaviour of Pile

SUMMARY (1)

- Ground movements due to earthquakes can induce large moments and forces in piles.
- Key factors influencing pile response are:
  - Pile length
  - Pile diameter
  - Soil modulus (and layer depth)
  - Cap mass
  - Nature of earthquake

SUMMARY (2)

- Pile response to earthquake-induced motions can be analyzed via pseudo-static analyses
- Combine:
  - Maximum ground movements from 1-D ground response analysis
  - Inertial force at pile head = cap mass* max. ground acceleration
- Carry out conventional pile-soil interaction analysis.

SUMMARY (3)

- Liquefaction can be important
  - Can exacerbate effects of ground motions
- Pseudo-static approach can be extended to this case
- Allow for reduction in soil modulus and pile-soil pressure due to excess pore pressures
- Calculations show encouraging agreement with results of centrifuge tests

SUMMARY (4)

- Axial effects can also be important
- Additional pile axial load and settlement (negative friction problem)
- Temporary loss of load capacity is a matter of potential concern
- Can be assessed via relatively simple means