FIFTH SHORT COURSE
SOIL DYNAMICS IN ENGINEERING PRACTICE

WHEELING, IL APRIL 29-30, 2013

PILE FOUNDATION UNDER SEISMIC LOADS (WITH APPLICATIONS)

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(updated March 2013)

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Figure 2., Failure of the Showa Bridge during the 1964 Niigata earthquake (NISEE[17]).

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Figure 3. Excavated piles of a building twenty years after the 1964 Niigata earthquake. The superstructure is three-storey RC building. The piles are precast RC piles 10 m long and 300 mm in diameter. (Photo courtesy: T. Tazon.)

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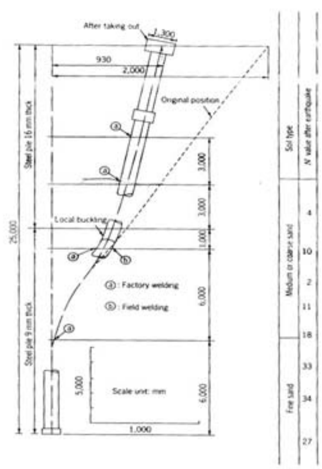

Figure 6. Pile pier no. 4, taken out from the ground after the Niigate earthquake (Fukuoka, 1966)

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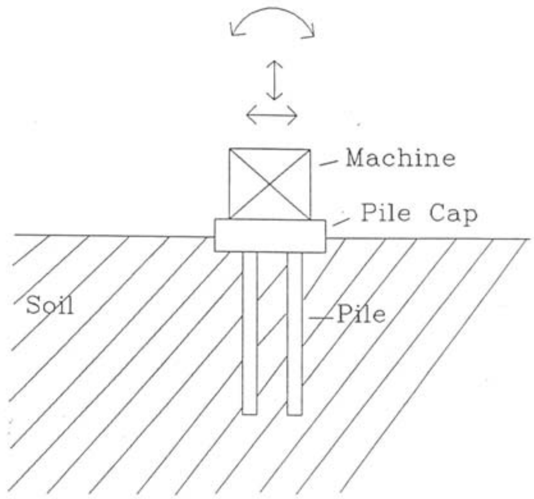

Figure 1. Machine Foundation Problem (Low Strain)

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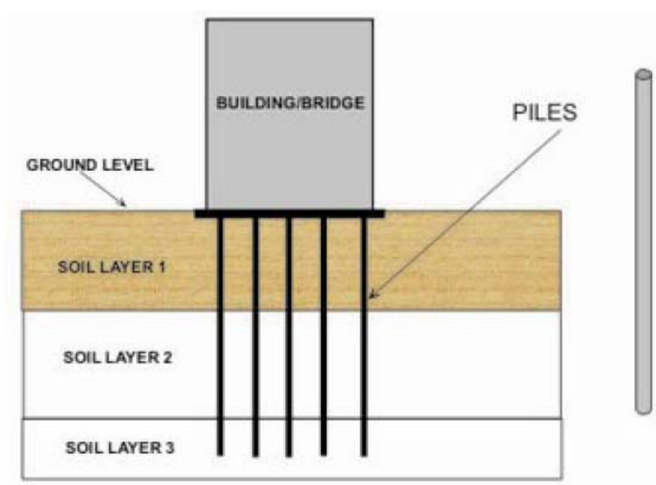

Figure 1. Schematic diagram of a pile-supported structure.

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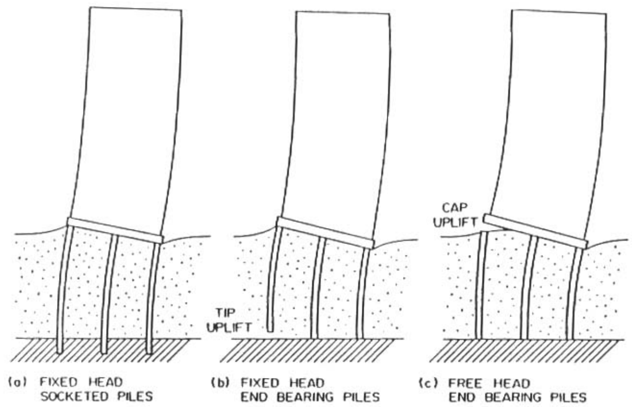

Figure 4. Uplift of pile cap and pile tip under seismic loading

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TOPICS

- **PILES IN NON-LIQUEFIABLE SOILS**
- **LOADING**
  - **Machine Foundations**
  - **Earthquake**
- **ANALYTICAL SOLUTIONS**
  - **Machine Foundations (Single pile & Pile Groups)**
  - **Earthquakes (Single pile & Pile Groups)**
- **GROUP INTERACTION FACTORS**
- **DESIGN PROCEDURES**
- **APPLICATION**
- **PILES IN LIQUEFIABLE SOILS**
- **CONCLUSIONS**

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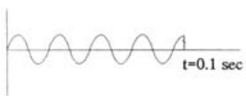
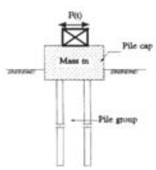
LOAD CHARACTERISTICS

Machine Loads



Periodicity



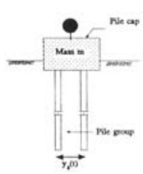
Load Transmission

structure

low	**Amplitude**
high (10-500 Hz)	**Frequency**
infinite cycles	**Duration**

LOAD CHARACTERISTICS

Earthquake Excitation



Earthquake Manitude	Number of Representative Cycles
8.5	26
7.5	15
6.75	10
6.0	5~6
5.25	2~3

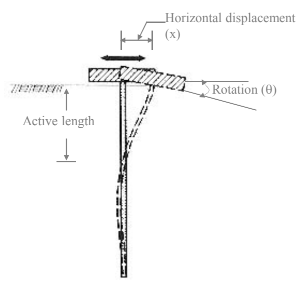
Amplitude	**high**
Frequency	**low (1-5 Hz)**
Duration	**few cycles of significant motion**

PILE BEHAVIOR

Machine Loads

Linear Response

Very small (permissible=0.02mm)



Effect of Frequency and Pile Spacing

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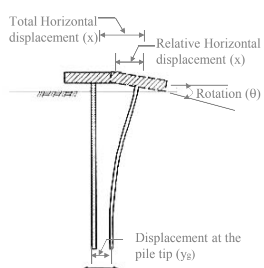
PILE BEHAVIOR

Earthquake Excitation

Non-linear response

Large Deformation



Effect of Frequency and Pile Spacing

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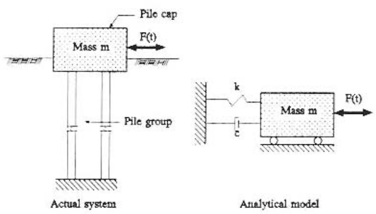
ANALYSIS

Machine Loads

Analytical Model



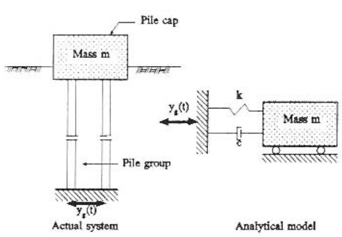
Equation of Motion

$$m\ddot{x} + c\dot{x} + kx = F_o \sin \omega t$$

Response

Closed from solution possible

- **Earthquake Excitation**



Equation of Motion

$$m\ddot{x} + c\dot{x} + kx = -m\ddot{y}_a(t)$$
$$x = z - y_g \text{ where z = disp. of mass}$$

Response

Use of numerical procedures is necessary

STEPS INVOLVED IN THE ANALYSIS

- CALCULATION OF STIFFNESS AND DAMPING OF SINGLE PILES
- CALCULATION OF STIFFNESS AND DAMPING OF PILE GROUPS
- CALCULATION OF RESPONSE

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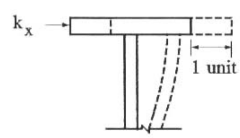
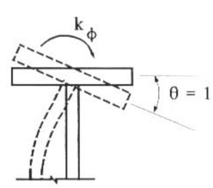
DEFINITIONS OF STIFFNESSES

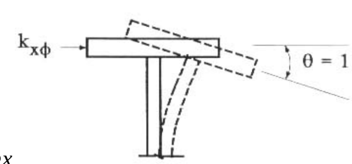
Lateral Stiffness



Rocking Stiffness



Cross-Rocking Stiffness



$$k_{x\Phi} = k_{\Phi x}$$

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PILE GROUP EFFECT

- Group Efficiency Factors

$$\frac{k_{group}}{n * k_{\sin gle}} = \varepsilon$$

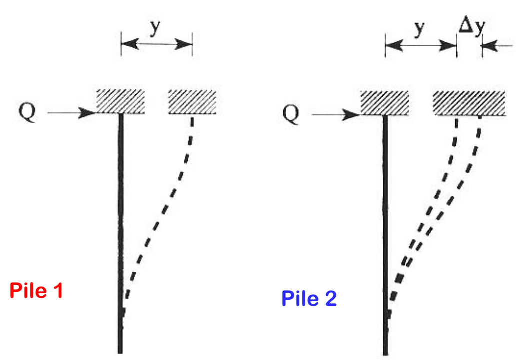
- Interaction Factor Superposition Approach)

$$\alpha(\omega) = \frac{\text{Displacement of pile 1 caused by pile 2}}{\text{Displacement of pile 1, considered individually}}$$

Definition:

$$\frac{n * k_{\sin gle}}{k_{group}} = \sum_{1}^{n} \alpha(\omega)$$

GROUP EFFICIENCY FACTOR



The group efficiency factor and group interaction factor

$$\varepsilon = \frac{1}{\sum \alpha}$$

BEHAVIOUR OF PILES IN NON LIQUEFIABLE SOILS

1. Soil shear modulus degrades with increasing strain/displacement

2. Material damping increases with increasing strain/displacement

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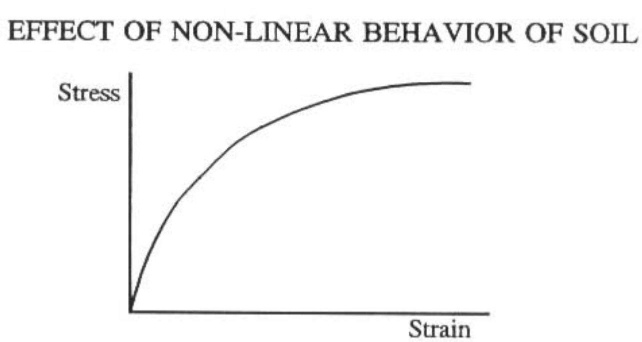
NONLINEAR SOIL PROPERTIES



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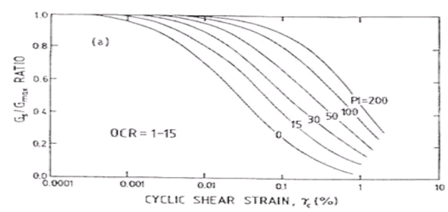
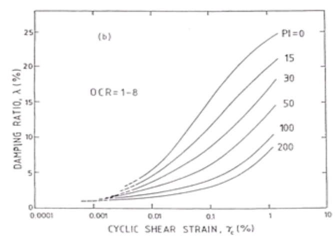
NONLINEAR SOIL PROPERTIES



Shear Modulus Degradation with Strain



Damping increases with Strain

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DESIGN PROCEDURE

- Pile under:
  - Vertical vibration
  - Horizontal vibration, and
  - Torsion

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SOIL PROPERTIES

- Shear modulus G_s and G_b
- Shear wave velocity V_s in soil
- Poisson's ratio n
- Weight γ for the soil both around the pile and below its tip respectively.

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PILE PROPERTIES AND GEOMETRY

- Pile length,
- Cross-section, and
- Spacing in the group,
- γ of pile and pile cap and
- Young's modulus of pile material.
- V_c compression wave velocity in pile

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PILE PROPERTIES AND GEOMETRY (cont)

- In practice, stiffness and damping of the soil below the pile cap are neglected.
- The stiffness and damping at the sides of the pile cap is also questionable.
- Cohesive soils may shrink and lose contact.
- Non-cohesive soils may not shrink, but settle and may provide some additional “k” and “c” at the sides and may reduce the response.

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Novak Model

- Novak (1974) model has been used with appropriate interaction factors.
- Main assumption:
  - The pile is circular and solid in cross section.
    For other then circular section an equivalent radius is determined in each mode of vibration.
  - The pile material is linear elastic
  - The pile is perfectly connected to the soil (i.e., there is no separation between soil and pile during vibration).

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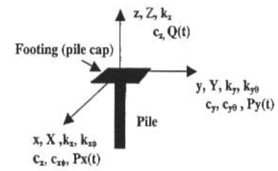
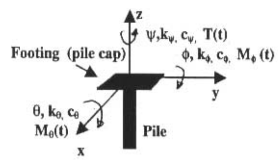
Sign Convention of Novak Model



a) Translational and coupled constants



b) Rotational constants

STIFFNESS AND DAMPING FACTORS OF SINGLE PILE

Vertical Stiffness and Damping Factor

$$k_z = (E_p \cdot A / r_o)\, f_{w1}$$

$$c_z = (E_p \cdot A / V_s)\, f_{w2}$$

Where

- E_p = modulus of elasticity of pile material
- A = cross section of single pile
- r_o = radius of a solid pile or equivalent pile radius
- V_s = shear wave velocity of soil along the floating pile

f_{w1} and f_{w2} are obtained from the following figure.

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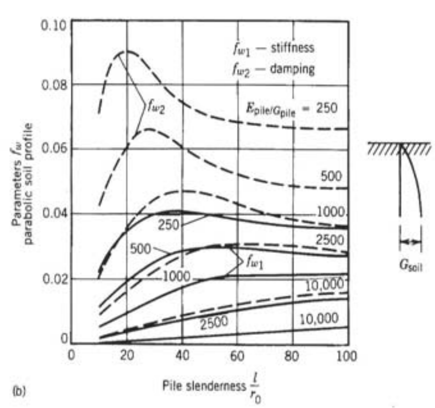
Vertical Stiffness and Damping Factor



Stiffness and damping parameter of vertical response of floating piles (Novak and El-shornouby, 1983)

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Sliding and Rocking Stiffness and Damping Factor

- **Sliding (k_x, c_x)**

$$k_x = (E_p \cdot I_p / r_o^3) f_{x1}$$

$$c_x = (E_p \cdot I_p / r_o^2 V_s) f_{x2}$$

- **Rocking (k_f, c_f) and (k_q, c_q)**

$$k_\phi = k_\theta = (E_p \cdot I_p / r_o) f_{\phi 1}$$

$$c_\phi = c_\theta = (E_p \cdot I_p / r_o^2 V_s) f_{\phi 2}$$

- **Cross-coupling (k_{xf}, c_{xf}) and (k_{yq}, c_{yq})**

$$k_{x\phi} = k_{y\theta} = (E_p \cdot I_p / r_o^2) f_{x\phi 1}$$

$$c_{x\phi} = c_{y\theta} = (E_p \cdot I_p / r_o V_s) f_{x\phi 2}$$

where:

I_P = moment of inertia of single pile about x or y axis
r_o = radius of a solid pile or equivalent pile radius
$f_{x1}, f_{x2}, f_{f1}, f_{f2}, f_{xf1}, f_{xf2}$ are Novak's coefficient and are obtained from the following table for parabolic soil profile, with appropriate interpolation and for n = 0.25

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Sliding and Rocking Stiffness and Damping Factor (contd.)

TABLE 7.5 Stiffness and Damping Parameters of Horizontal Response for Piles with $L/r_0 > 25$ for Homogeneous Soil Profile and $L/r_0 > 30$ for Parabolic Soil Profile

		Stiffness Parameters				Damping Parameters			
ν (1)	E_{pile}/G_{soil} (2)	$(f_{\phi 1})$ (3)	$f_{(x\phi 1)}$ (4)	(f^*_{x1}) (5)	(f^P_{x1}) (6)	$(f_{\phi 2})$ (7)	$f_{(x\phi 2)}$ (8)	(f^*_{x2}) (9)	(f^P_{x2}) (10)
				Homogeneous Soil Profile					
0.25	10,000	0.2135	−0.0217	0.0042	0.0021	0.1577	−0.0333	0.0107	0.0054
	2,500	0.2998	−0.0429	0.0119	0.0061	0.2152	−0.0646	0.0297	0.0154
	1,000	0.3741	−0.0668	0.0236	0.0123	0.2598	−0.0985	0.0579	0.0306
	500	0.4411	−0.0929	0.0395	0.0210	0.2953	−0.1337	0.0953	0.0514
	250	0.5186	−0.1281	0.0659	0.0358	0.3299	−0.1786	0.1556	0.0864
0.40	10,000	0.2207	−0.0232	0.0047	0.0024	0.1634	−0.0358	0.0119	0.0060
	2,500	0.3097	−0.0459	0.0132	0.0068	0.2224	−0.0692	0.0329	0.0171
	1,000	0.3860	−0.0714	0.0261	0.0136	0.2677	−0.1052	0.0641	0.0339
	500	0.4547	−0.0991	0.0436	0.0231	0.3034	−0.1425	0.1054	0.0570
	250	0.5336	−0.1365	0.0726	0.0394	0.3377	−0.1896	0.1717	0.0957
				Parabolic Soil Profile					
0.25	10,000	0.1800	−0.0144	0.0019	0.0008	0.1450	−0.0252	0.0060	0.0028
	2,500	0.2452	−0.0267	0.0047	0.0020	0.2025	−0.0484	0.0159	0.0076
	1,000	0.3000	−0.0400	0.0086	0.0037	0.2499	−0.0737	0.0303	0.0147
	500	0.3489	−0.0543	0.0136	0.0059	0.2910	−0.1008	0.0491	0.0241
	250	0.4049	−0.0734	0.0215	0.0094	0.3361	−0.1370	0.0793	0.0398
0.40	10,000	0.1857	−0.0153	0.0020	0.0009	0.1508	−0.0271	0.0067	0.0031
	2,500	0.2529	−0.0284	0.0051	0.0022	0.2101	−0.0519	0.0177	0.0084
	1,000	0.3094	−0.0426	0.0094	0.0041	0.2589	−0.0790	0.0336	0.0163
	500	0.3596	−0.0577	0.0149	0.0065	0.3009	−0.1079	0.0544	0.0269
	250	0.4170	−0.0780	0.0236	0.0103	0.3468	−0.1461	0.0880	0.0443

1 Prakash, S. and Sharma, H.D. (1990) "Pile Foundation in Engineering Practice", John Wiley & Sons, Inc.

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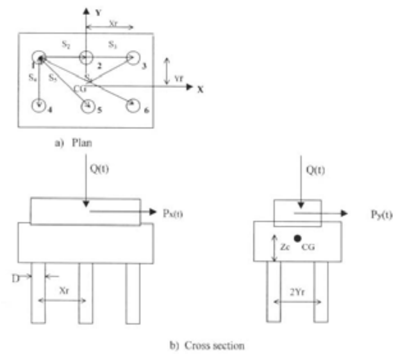
Group Interaction Factor

- To consider group effect, Poulos (1968) assume a pile in the group as reference pile.
- Pile No. 1 is assumed as a reference pile and distance “S” is measured from the center of other pile to center of the reference pile.



Plan an Cross Section of Pile Group

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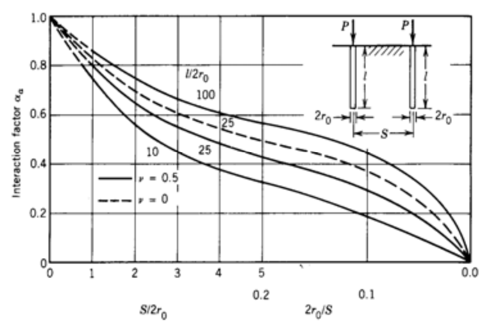
Group Interaction Factor

- For vertical direction use figure below to obtain α_A for each pile for appropriate $S/2r_o$ Values α_A's are function of length of the pile (L) and radius (r_o).



α_A as a function of pile length and spacing (Poulos, 1968)

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Group Stiffness and Damping Factor

Vertical group stiffness and damping factors:

$$k_z^g = \frac{\Sigma k_z}{\Sigma \alpha_L}$$

$$c_z^g = \frac{\Sigma c_z}{\Sigma \alpha_L}$$

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Group Interaction Factor (Contd.)

- For horizontal x-direction, considering departure angel β (degree), and use Figure (Poulos, 1972), to obtain a_L for each pile. a_L 's are function of L, r_o and flexibility K_R as defined in figure and departure angle (β). This procedure will also apply for the other horizontal direction.

- Based on calculated a_L for each pile, the group interaction factor (Σ a_L) is summation a_L for all the piles. Note that the group interaction factor in horizontal x-direction and y-direction may be different depending on number and spacing of piles in each direction.

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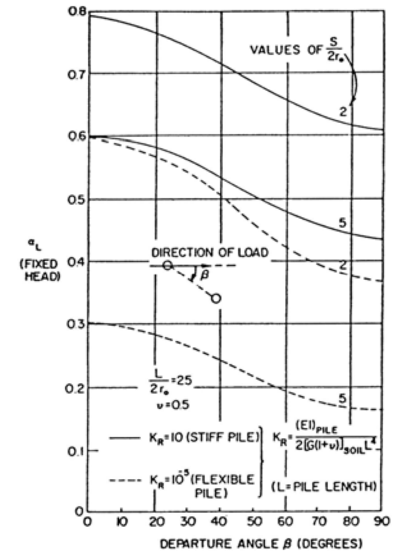
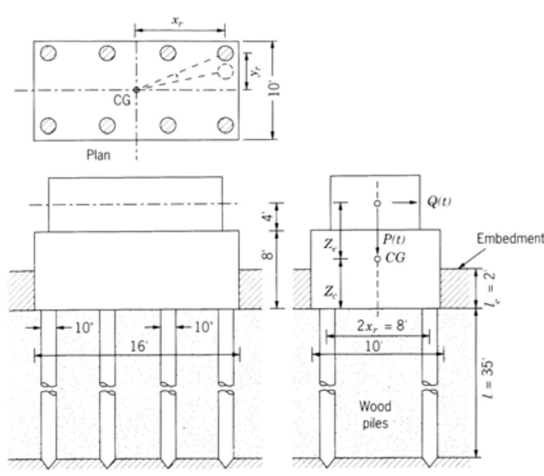
Graphical Solution for a_L (Poulos, 1972)



Dimensions of Pile Foundation



Group Stiffness and Damping Factor (Contd.)

Translation along x axis:

$$k_x^g = \frac{\Sigma\, k_x}{\Sigma\alpha_{Lx}}$$

$$c_x^g = \frac{\Sigma\, c_x}{\Sigma\alpha_{Lx}}$$

Translation along y axis:

$$k_y^g = \frac{\Sigma\, k_y}{\Sigma\alpha_{Ly}}$$

$$c_y^g = \frac{\Sigma\, c_x}{\Sigma\alpha_{Ly}}$$

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Group Stiffness and Damping Factor (Contd.)

Rocking about y axis:

$$k_\phi^g = \frac{1}{\Sigma\alpha_{Lx}} \Sigma\,(k_\phi + k_z x_r^2 + k_x z_c^2 - 2\, z_c\, k_{x\phi})$$

$$c_\phi^g = \frac{1}{\Sigma\alpha_{Lx}} \Sigma\,(c_\phi + c_z x_r^2 + c_x z_c^2 - 2\, z_c\, c_{x\phi})$$

Rocking about x axis:

$$k_\theta^g = \frac{1}{\Sigma\alpha_{Ly}} \Sigma\,(k_\theta + k_z x_r^2 + k_y z_c^2 - 2\, z_c\, k_{y\theta})$$

$$c_\theta^g = \frac{1}{\Sigma\alpha_{Ly}} \Sigma\,(c_\theta + c_z x_r^2 + c_x z_c^2 - 2\, z_c\, c_{y\theta})$$

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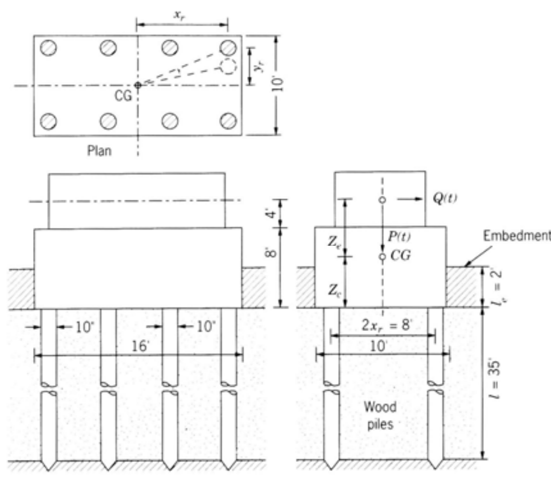
Dimensions of Pile Foundation



Group Stiffness and Damping Factor (Contd.)

Cross-coupling translation in x axis and rotation about y axis:

$$k_{x\phi}^{g} = \frac{1}{\Sigma\alpha_{Lx}} \Sigma\,(k_{x\phi} - k_{x}\, z_c)$$

$$c_{x\phi}^{g} = \frac{1}{\Sigma\alpha_{Lx}} \Sigma\,(c_{x\phi} - c_{x}\, z_c)$$

Cross-coupling translation in y axis and rotation about x axis:

$$k_{y\theta}^{g} = \frac{1}{\Sigma\alpha_{Ly}} \Sigma\,(k_{y\theta} - k_{y}\, z_c)$$

$$c_{y\theta}^{g} = \frac{1}{\Sigma\alpha_{Ly}} \Sigma\,(c_{y\theta} - c_{y}\, z_c)$$

GROUP ACTION

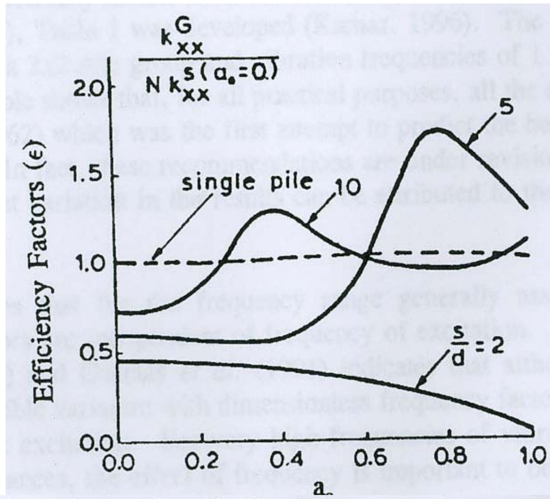


Fig. 2 Efficiency factors, ϵ (real part) as a function of dimensionless frequency factor, a_o for 2x2 pile group (Kaynia and Kausel, 1982)

EFFECT OF FREQUENCY

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Equation of Motion

- **Under dynamic loading, the equilibrium of forces is derived based on the Newton's second law of motion. This equilibrium in 2-dimensional analysis will give 4 equations of motion in one each in vertical and torsion, and two in two horizontal directions.**

Vertical Equation of Moti

$$m.\ddot{Z} + c_z{}^g.\,\dot{Z} + k_z{}^g\,Z = Q(t)$$

Torsional Equation of Motion:

$$m.\,\ddot{\psi} + c_\psi{}^g.\,\dot{\psi} + k_\psi{}^g.\,\psi = T(t)$$

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Strain-displacement Relationship

- Because evaluation of shear strain in the field is in many cases not clear, reasonable expressions must be assumed and used as the basis for evaluating the shear strain in each particular case.

- The shear strain and displacement relationship is not well defined in practical problems occuring in the field. However, the relationship has been recommended by Prakash and Puri (1981) as:

$$\gamma = \frac{\text{Amplitude of foundation vibration}}{\text{Average width of foundation}}$$

For vertical and horizontal vibration

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Strain-displacement Relationship (contd.)

- Kagawa and Kraft (1980) used following relationship for horizontal displacement in front of a pile:

$$\gamma_x = \frac{(1+\upsilon)X}{2.5D}$$

Where, n = poisson's ratio

X = horizontal displacement in x-direction

D = diameter of pile

- Rafnsson (1992) recommended that, the shear strain due to rocking can be reasonably determined as:

$$r_\phi = \phi / 3$$

Where, F = rotation of foundation about x or y axis

- Shear strain-displacement relationship for coupled sliding and rocking can be determined as:

$$\gamma_x = \frac{(1+v)X}{2.5D} + \frac{\phi}{3}$$

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Non –linear Solution

- Strain (Displacement) Dependent Springs and Damping constants
- Appropriate Numerical Technique of Iteration Solution
- Convergence of Solution

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Non –linear Spring and Damping Constants

EIGHT SPRING CONSTANTS

k_x, k_y, k_z	TRANSLATION
k_θ, k_ϕ, k_ψ	ROTATION
$k_{x\phi}, k_{y\theta}$	CROSS-COUPLING

EIGHT DAMPING CONSTANTS

c_x, c_y, c_z	TRANSLATION
c_θ, c_ϕ, c_ψ	ROTATION
$c_{x\theta}, c_{y\theta}$	CROSS-COUPLING

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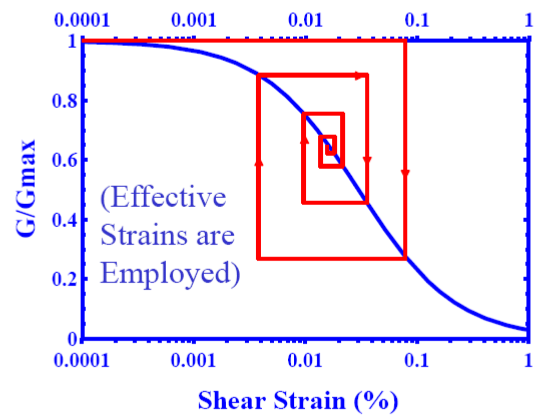
Non-Linear Iterative Solution Technique

Equivalent Linear Concept



A. Elgamal

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Non –linear Alternate Analysis (Iteration Procedure)

1. Assume G_1 of soil for any instant of time (if t=0, assume $G_1 = G_{max}$)
2. Obtain all k's and c's
3. Solve equation of motion for displacement at that instant of time
4. Estimate shear strain in the soil. Appropriate displacement (X, Y or Z) and shear strain (γ) relationships are used
5. Estimate G_2 for strain calculated in (4) above
6. If G_1 and G_2 are within acceptable range, the solution is OK and go to step 7, otherwise assume a new value of G_1' in (1) above as $(G_1+G_2)/2$ and repeat step 2 and 6
7. Repeat step 1-6 at other time with G_1 in (6) above to complete the time domain solution

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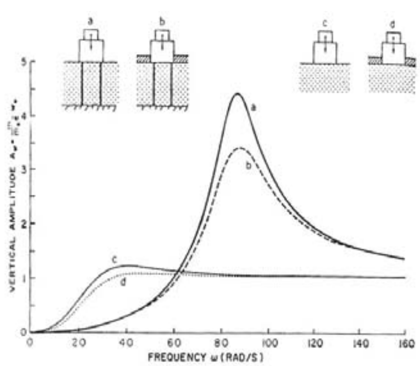
APPLICATIONS

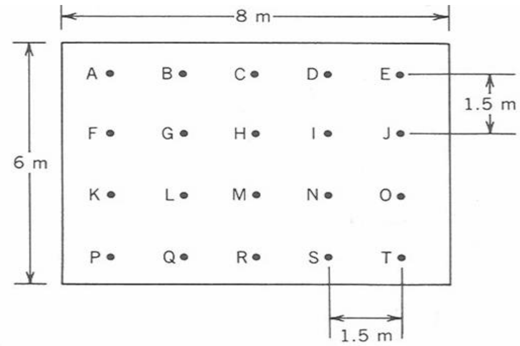
Vertical Response



a. The foundation supported on pile with no pile cap embedment
b. The pile cap embedded in a soil layer
c. The foundation is supported on elastic half space
d. The foundation embedded in a soil layer

PILE GROUPS

Effect of pile cap contact on k_z and c_z



Arrangement of 5 × 4 pile group for Example 12.5.1.

$$k_w^g = 138{,}087 \ t/m$$

$$c_w^g = 363 \quad t/(m/\sec)$$

$$k_w^f = 21{,}600 \ t/m$$

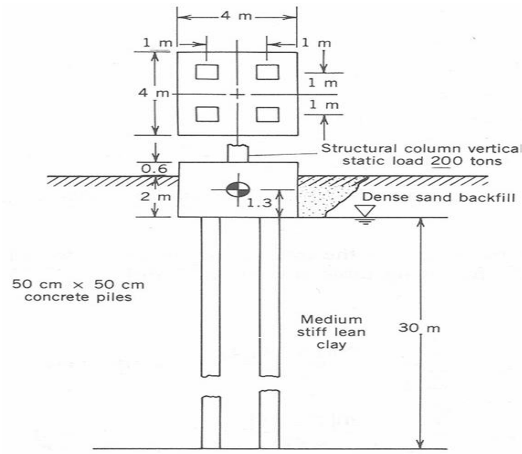
$$c_w^f = 1330 \ t/(m/\sec)$$

Total Stiffness and Damping Values

Total $k_w^g = 138{,}087 + 21{,}600 = 159{,}687 \ t/m$

Total $c_w^g = 363 + 1330 = 1694 \, t/(m/\sec)$

Effect of pile cap contact on k_ϕ^f and c_ϕ^f



Pile foundation Example 12.5.2.

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Stiffness and damping

Pile Group (only)

$$k_\phi^g = 469 \times 10^8 kg\ cm/rad$$

$$c_\phi^g = 1.90 \times 10^8 kg\ cm\ \sec/rad$$

Pile Cap (only)

$$k_\phi^f = 115.6 \times 10^8 kg\ cm/rad$$

$$c_\phi^f = 1.36 \times 10^8 kg\ cm\ \sec/rad$$

Total stiffness and damping values are:

Total stiffness $k_\phi^g = (469.1 + 115.6) \times 10^8 = 584.7 \times 10^8 kg\ cm/rad$

Total damping $c_\phi^g = (1.90 + 1.36) \times 10^8 = 3.26 \times 10^8 kg\ cm\ \sec/rad$

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PREDICTIONS AND PERFORMANCE

- PILES UNDER DYNAMIC LOADS -

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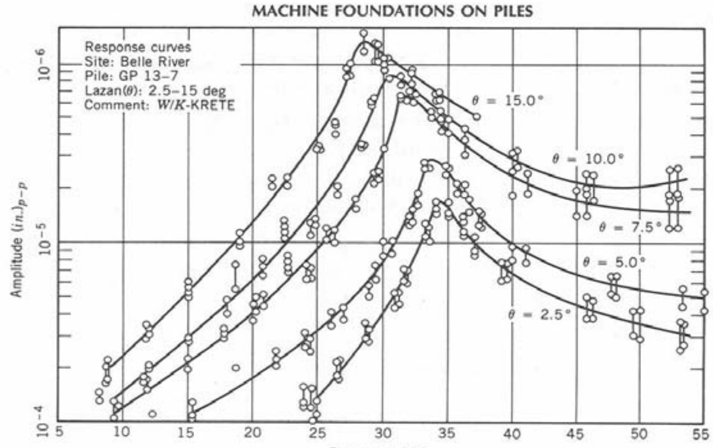
MACHINE FOUNDATIONS ON PILES



Response curves show a decrease in resonant frequency with increasing amplitudes in horizontal vibrations. (From R. D. Woods, Lateral interaction between soil and pile, in D. E. Beskos, Theodore Kranthammer, and I. Vardoulakis, eds., "Dynamic Soil-Structure Interaction," 1984, A. A. Balkema, Rotterdam.)

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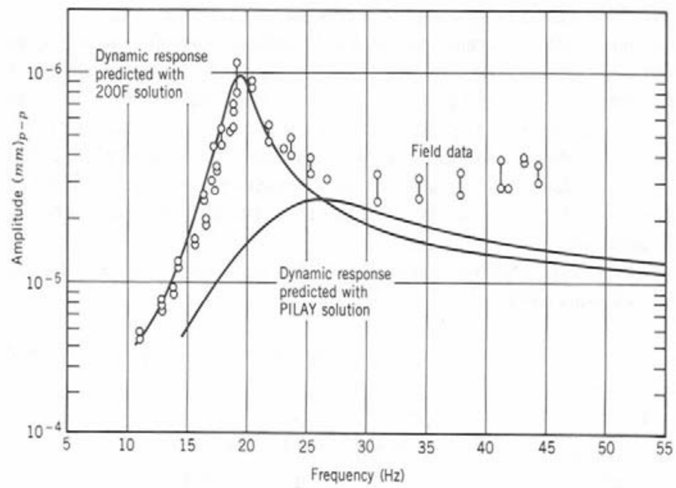
COMPARISON OF MEASURED AND PREDICTED PILE RESPONSE



Typical response curves predicted by PILAY superimposed on measured pile response. (After Gle and Woods, 1984.)

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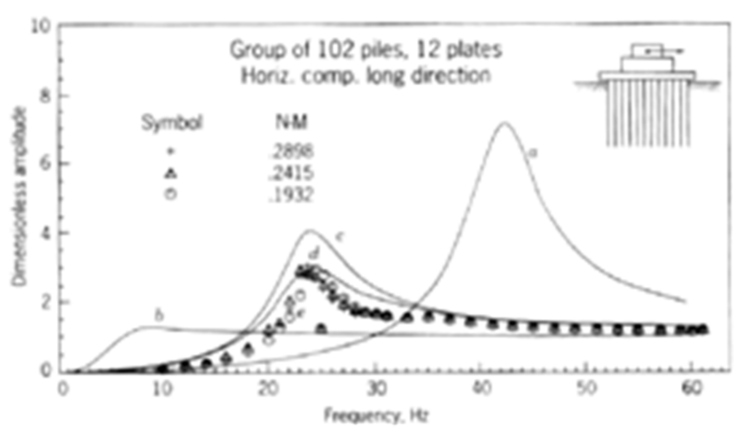

Experiment horizontal response curves and theoretical curves calculated with static interaction factors. (Novak and El-Sharnouby, 1984)

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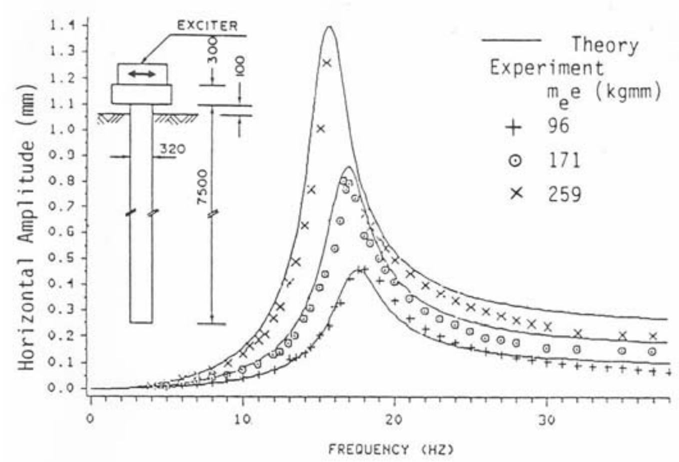
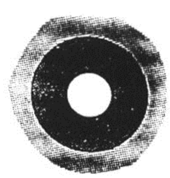
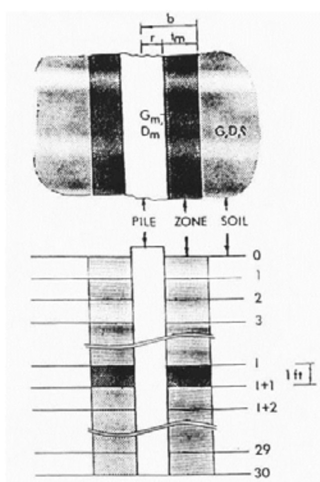


Theoretical and experimental horizontal response of concrete pile for three levels of harmonic excitation (El Marsafawi et al., 1990)

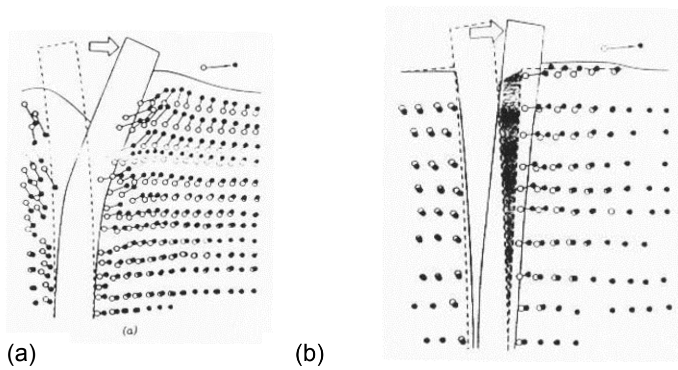
Concept of Softened Zone





Concept of Softened Zone Surrounding Pile for Pilay 2 Analysis (Novak et al, 1981)

Movements of Soils



(a) (b)

Movements of Soils: (a) Sand, (b) Clay

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Shear Modulus

- G-Value is over estimated at $\gamma = 10^{-6}$, which gives higher computed frequency

- Damping is also over-estimated, which gives smaller response at resonance

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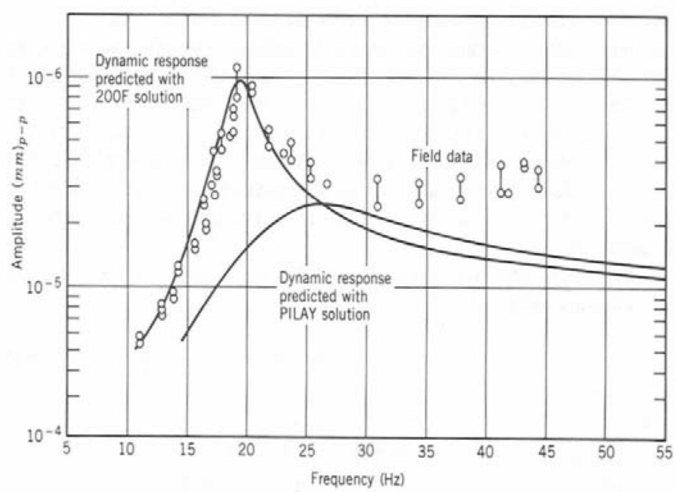
COMPARISON OF MEASURED AND PREDICTED PILE RESPONSE



Typical response curves predicted by PILAY superimposed on measured pile response. (After Gle and Woods, 1984.)

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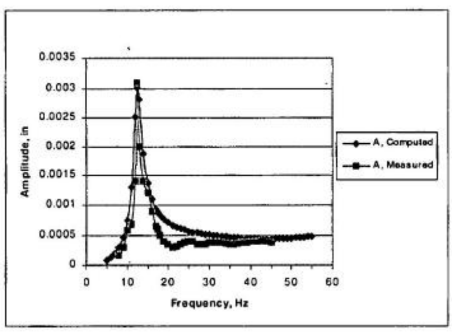

Figure 6. Comparison of Observed and Computed data

- Prakash and Jadi (2001) reanalyzed the reported pile test data of Gle (1981) for the lateral dynamic proposed reduction factors for the stiffness and radiation damping obtained by using the approach of Novak and El-Sharnouby (1983). The suggested equations for the reduction factors are:
- $\lambda G = -353500\ \gamma 2 - 0.00775\ \gamma + 0.3244$ (1)
- $\lambda c = 217600\ \gamma 2 - 1905.56\ \gamma + 0.6$ (2)
- where, λG and λc are the reduction factors for shear modulus and damping.

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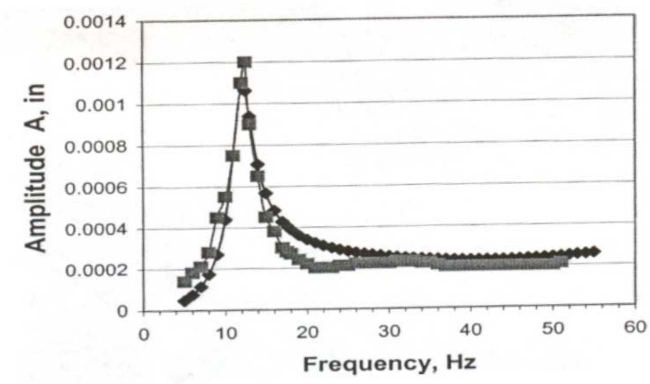

Fig. 6. Measured and Reduced predicted lateral dynamic response for pile for lateral dynamic load test for pile L1810(θ=2.5°), Belle River site (Jadi, 1999)

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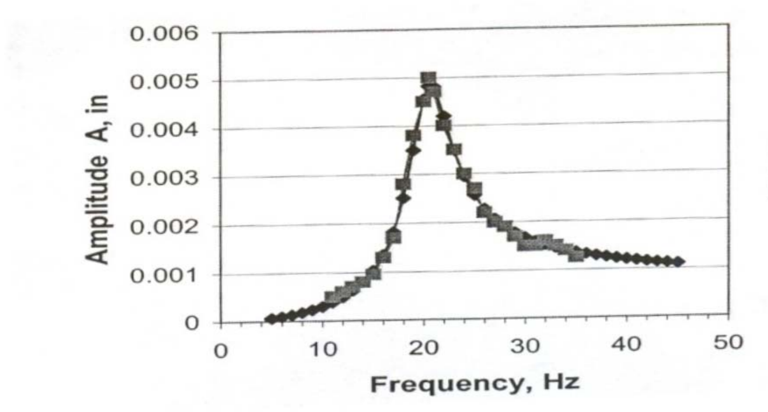

Fig. 7. Measured and arbitrarily reduced predicted lateral dynamic response for pile LF6 (θ=10°), St. Clair site. (Jadi, 1999)

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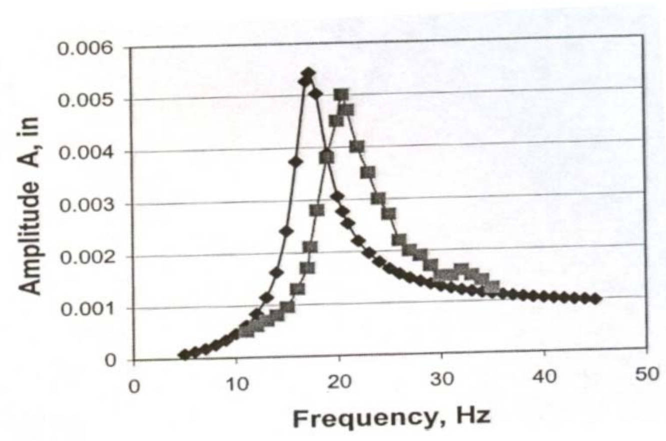

Fig. 8. Measured and Reduced predicted lateral dynamic response for pile LF16 (θ=10°), St. Clair site
λ_G=0.321, λ_c=0.4 (Jadi, 1999)

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JADI's ANALYSIS

The method of analysis used in this study is as follows (Jadi (1999) and Prakash and Jadi (2001)):

Step 1. Field data obtained from lateral dynamic tests performed by Gle (1981) on full-scale single piles embedded in clayey soils, were collected.

Step 2. Theoretical dynamic response was computed for the test piles, using Novak and El-Sharnouby's (1983) analytical solution for stiffness and damping constants, with no corrections.

Step 3. The soil's shear modulus and radiation damping used for the response calculations were arbitrarily reduced, such that measured and predicted natural frequencies and resonant amplitude matched.

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JADI's ANALYSIS, Cont.

Step 4. The reduction factors obtained from step 3 were plotted versus shear strain at resonance without corrected G and 'c'. Two quadratic equations were developed to determine the shear modulus reduction factors (λG) versus shear strain, (γ) and the radiation damping reduction factor (λC) versus shear strain (γ).

Step 5. For all the pile tests considered in this study, the empirical equations determined in step 4 were used to calculate shear modulus and radiation damping reduction factors. Predicted responses before and after applying the proposed reduction factors were then compared to the measured response.

Step 6. To validate this approach, the proposed equations were used to calculate shear modulus and radiation damping reduction factors for different sets of field pile tests. The new predicted response was then compared to the measured response, both for Gle (1981) tests and two other cases.

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COMPARISON OF COMPUTED AND PREDICTED PILE RESPONSE IN NON LIQUEFYING SOILS

Jadi (1999) and Prakash and Jadi (2001) reanalyzed the reported pile test data of Gle (1981) for the lateral dynamic tests on single piles and proposed reduction factors for the stiffness and radiation damping obtained by using the approach of Novak and El-Sharnouby (1983) as:

$$\lambda_G = -353500\ \gamma^2 - 0.00775\ \gamma + 0.3244$$

$$\lambda_c = 217600\ \gamma^2 - 1905.56\ \gamma + 0.6$$

where, λ_G and λ_C are the reduction factors for shear modulus and damping and γ is shear strain at computed peak amplitude, without any correction.

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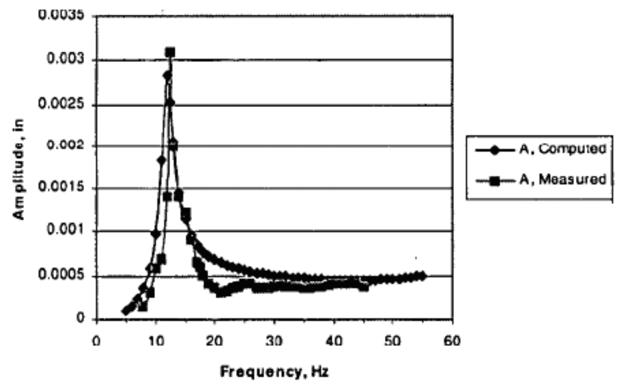

Fig. 12. Measured vs Predicted Lateral Dynamic Response with Proposed Reduction factors for Pile K16-7 ($\theta = 5^o$)

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CHECK WITH DIFFERENT TEST DATA

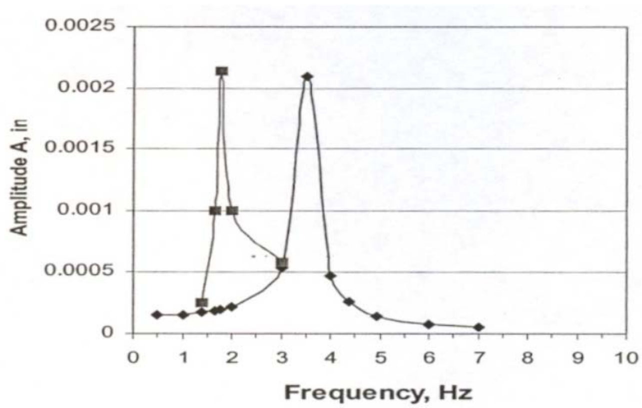

Fig. 15. Measured vs reduced predicted lateral dynamic response for pile 1 using proposed reduction factors, FHWA vibrator, λG = 0.32, λc= 0.54 (Jadi, 1999)

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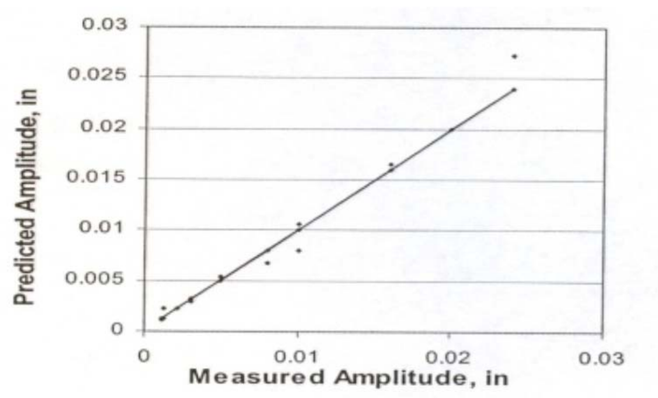

Fig. 10. Measured resonant amplitude vs predicted resonant amplitude computed with proposed radiation damping reduction factor (Jadi, 1999)

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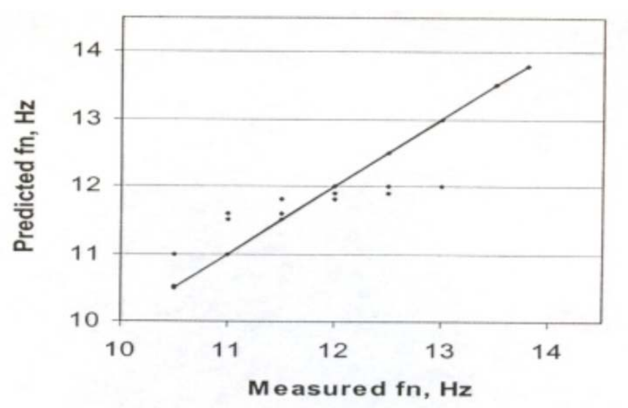

Fig. 9. Measured natural frequency vs predicted natural frequency computed with proposed shear modulus reduction factor (Jadi, 1999)

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CHECK WITH DIFFERENT TEST DATA

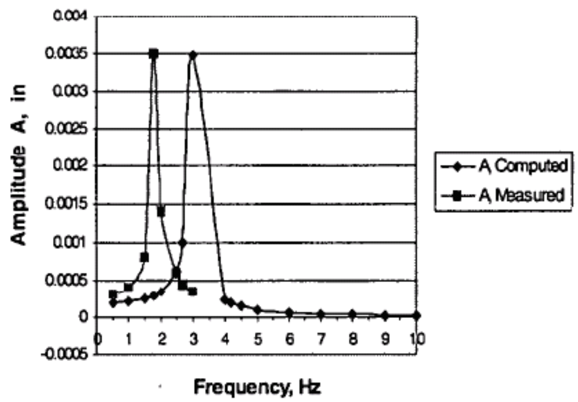

Fig. 14. Measured vs Reduced Predicted Lateral Dynamic Response of Pile I -WES Vibrator (λ_G = 0.31, λ_C =0.5)

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CHECK WITH DIFFERENT TEST DATA

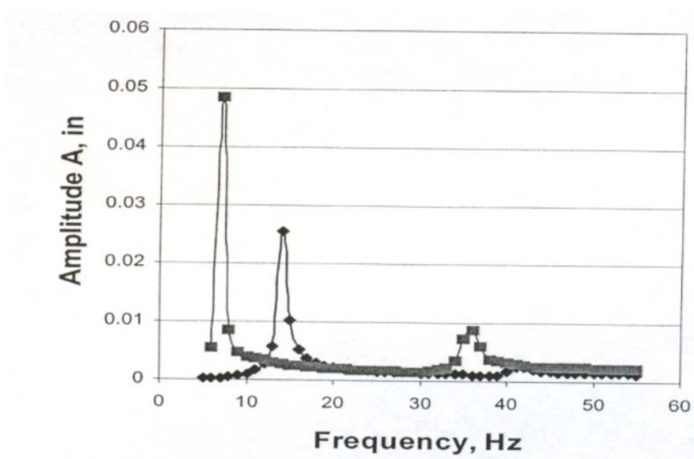

Fig. 16. Measured vs predicted lateral dynamic response for the 2.4” pile tested by Novak and Grigg, 1976 without correction factors.

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CHECK WITH DIFFERENT TEST DATA

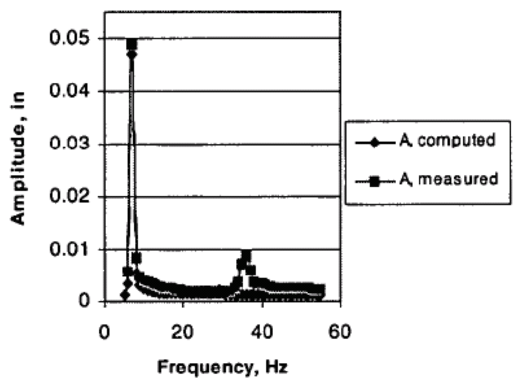

Fig. 18. Measured vs Reduced Predicted Lateral Dynamic Response of the 2.4" Pile tested by Novak and Grigg,1976 (($\lambda_G = 0.044$, $\lambda_C = 0.34$)

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COMMENTS ON PREDICTIONS

Novak and El Sharnouby (1984) have attempted to match the observed with predicted response by adjusting, arbitrarily, the group stiffness and damping values. No guidelines were developed to modify these values.

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COMMENTS ON PREDICTIONS

Woods (1984) used Pilay program with modified stiffness to match prediction and performance.

COMMENTS ON PREDICTIONS

Jadi (1999) developed rational correction factors to both stiffness and damping to match the computed and predicted responses. She was reasonably successful in her efforts. Her approach is more scientific but based on a limited data. More studies are needed to develop relationships for the reduction factors for different modes of vibration, and different soils.

CAMBIO (2012) Model

1. Cambio (2012) analyzed the existing available pile test using DYNA5 and proposed an equivalent linear model to predict response of piles.
2. The model incorporates frequency dependent parameters and the effects of soil non-linearity by using strain dependent values of shear modulus .
3. To improve upon the computed response a set of reduction factors on soil shear modulus and total damping were determined
4. The predicted and the measured amplitudes and frequencies match .
5. Empirical equations relating the reducing factors with soil shear strain, elastic properties of soils and piles, and pile geometry are given below:

$$\lambda_G = 0.912385 + 0.00165\, L/r_o - 0.0001334 \,.\, Ep/Gmax - 1.407 \times 10^{-9} . Emax + 43.246 \Upsilon_s$$

$$\lambda_c = 0.573217 - 119.542 \Upsilon_s - 0.01182 fmax$$

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Where,

λ_G =Reduction factor for shear modulus of soil.

λ_c =Reduction factor for total damping in soil.

L=Pile Strength

r_o =Radius of pile or equivalent radius for a non circular pile.

Ep =Young's modulus of pile material.

Gmax =Maximum Shear Modulus of soil

Emax = Maximum value of Young's Modulus of soil.

Fmax =Maximum value of natural Frequency.

Υ_s = Shear strain in soil.

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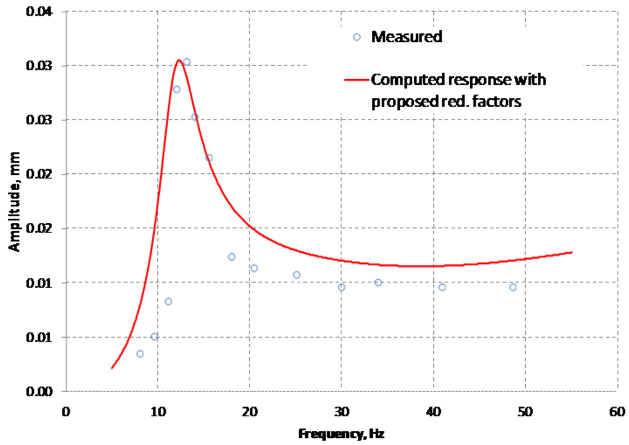
Figure 5.7



Measured and reduced predicted lateral dynamic response on pile L1810_θ = 5° (Gle, 1981), using proposed equations.

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Figure 5.10

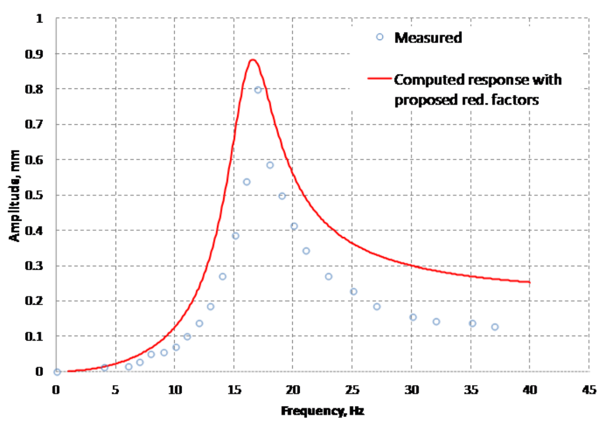

Fig 5.10 Measured and reduced predicted lateral dynamic response on pile 2-exitation kgmm 171 (Marsafawi et al., (1992), using proposed equations

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Figure 5.28

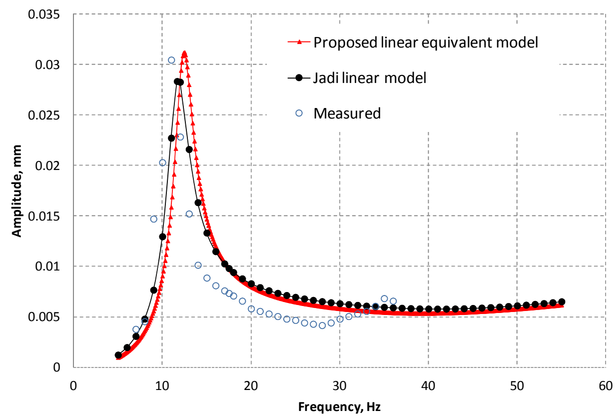

Fig 5.28 Measured and reduced predicted lateral dynamic response on pile GP 13-7_θ = 2.5° (Gle, 1981), from Jadi's work (1999).

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Figure 5.29

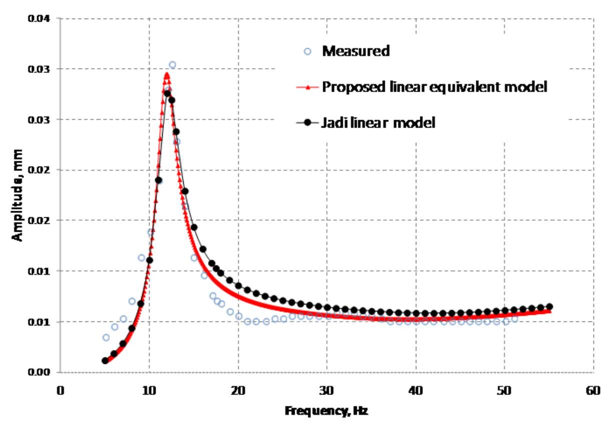

Fig 5.29 Measured and reduced predicted lateral dynamic response on pile L1810_θ = 2.5° (Gle, 1981), from Jadi's work (1999).

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Figure 5.30

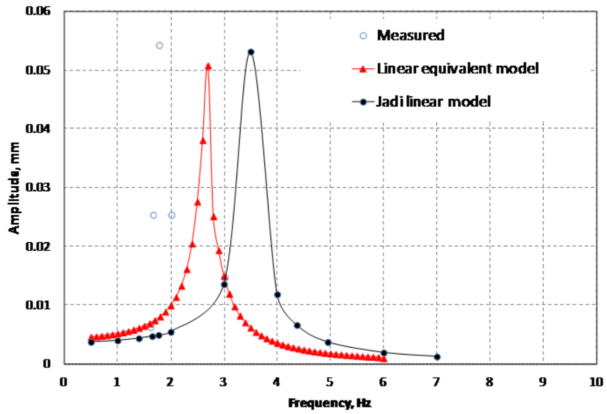

Fig 5.30 Measured and reduced predicted lateral dynamic response on pile FHWA (Blaney, 1983), from Jadi's work (1999).

Figure 5.34

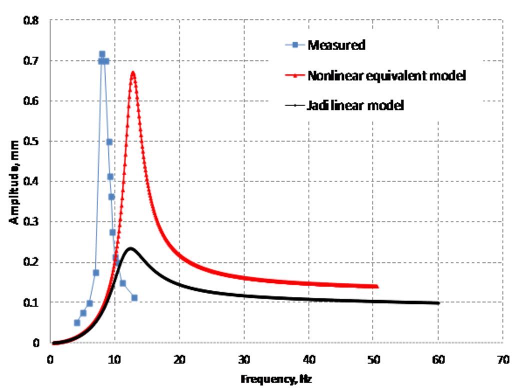

Fig 5.34 Measured and reduced predicted lateral dynamic response on pile 1-2 (Sa'don et al. 2010) using Jadi's model (1999).

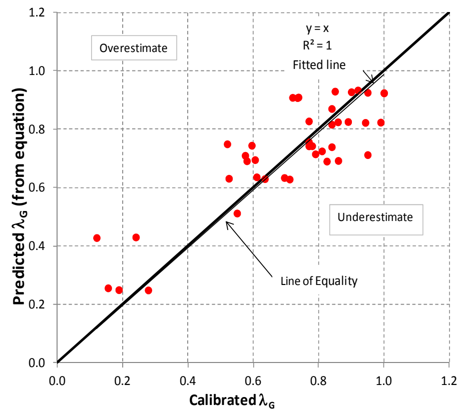

Figure 5.5 Predicted versus calibrated λ_G reduction factor

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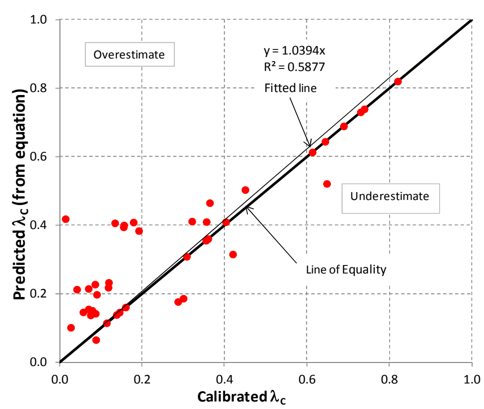

Figure 5.6 Predicted versus calibrated reduction factor. λ_C.

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Final Comments

- Cambio's analysis is a bit more general than Jadi's.
- However considerable more work is needed for credible prediction.

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CONCLUSIONS

PILES IN NON-LIQUEFIABLE SOILS

1. Soil-pile behavior is strongly strain dependent
2. Simple frequency independent stiffness and damping equations of Novak give reasonably good results.
3. Group interaction factors are also frequency independent, since predominant excitation frequencies may not exceed 6-10 Hz in soft soils.
4. The proposed concept of reduction factors for shear modulus and damping by Jadi (1999) appears reasonable but more research is needed before this method can used in practice with confidence.

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BEHAVIOUR OF PILES IN LIQUEFIABLE SOILS

- Lateral spreading of liquefied ground
- Strong shaking accompanied by the development of high pore water pressures or liquefaction

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LIQUEFACTION

Liquefaction may lead also to substantial increases in pile cap displacements above those for non-liquefied case

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LIQUEFACTION

After liquefaction, if the residual strength of the soil is less than the static shear stresses caused by a sloping site or a free surface such as a river bank, significant lateral spreading or down slope displacements may occur.

The moving soil can exert damaging pressures against the piles, leading to failure. Such failures were prevalent during the 1964 Niigata and the 1995 Kobe earthquakes (Finn, 2004).

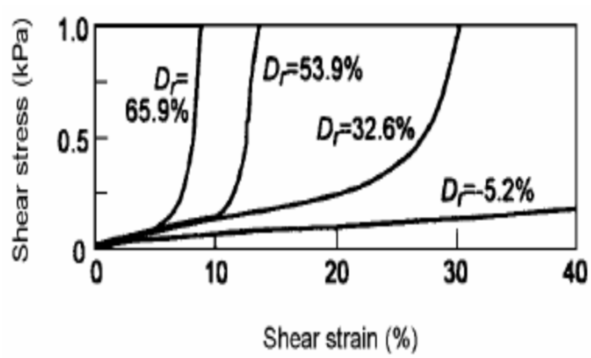

Fig.16. Post-liquefaction undrained stress-strain behavior of sand (Yasuda et al 1999).

$$D_\gamma = 65.9 - 53.9\%$$

Dilatancy effects are predominant

Damage to a pile under a building in Niigata caused by about 1m of ground displacement is shown in Fig 2 (Yasuda et al 1990)

Fig.2. Damage to pile by 2m of lateral ground displacement during 1964 Niigata earthquake (Yoshida et al. 1990).

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TANK TA72

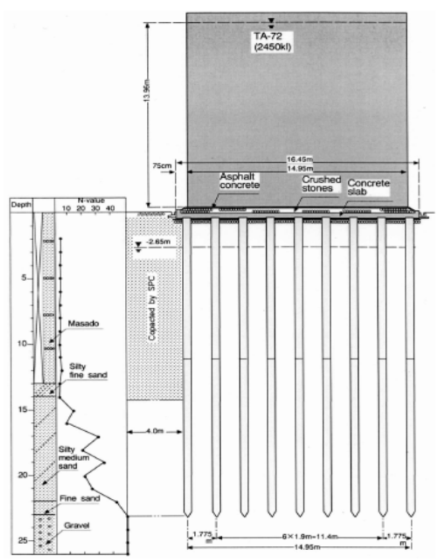

Fig. 3. Cross sectional view of Tank TA72 and its foundation

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DISPLACEMENT OF QUAY WALL

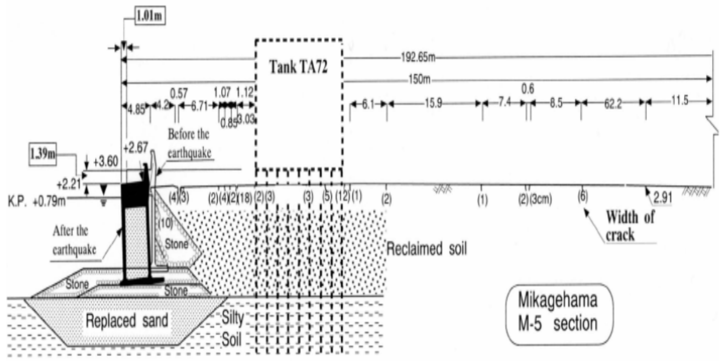

Fig. 8. Detailed profile of the quay wall movement and ground distortion in the backfills at Section M-5

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The quay wall moved approximately 1m towards the sea. The seaward movement of the quay wall was accompanied by lateral spreading of the backfill soils resulting in a number of cracks on the ground inland from the waterfront. The lateral ground displacement was plotted as a function of the distance from the waterfront. As indicated in the Fig. 9, the permanent lateral ground displacement corresponding to the location of Tank TA72 is seen somewhere between 35 and 55 cm (Ishihara 2004).

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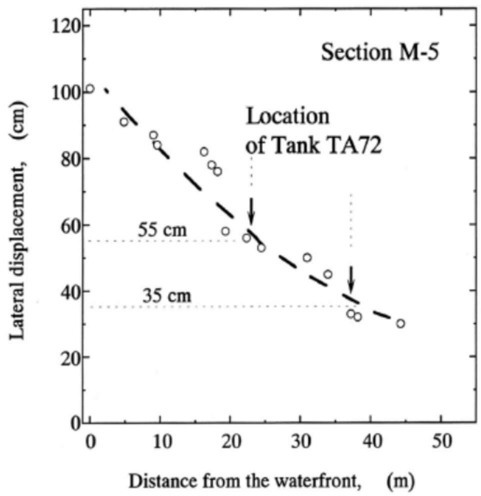

Fig. 9. Lateral ground displacement versus distance from the waterfront along Section M-5

(Kobe 1995 EQ)

DAMAGE TO PILES

To inspect the damage to the piles of the oil tank site after Kobe (1995) event, 70cm wide and 1m deep trenches were excavated at 4 sections and the upper portion of the pile was exposed. The wall of the cylindrical piles was cut to open a window about 30cm long and 15cm wide. From this window, a bore-hole camera was lowered through the interior hole of the hollow cylindrical piles to examine the damage to the piles throughout the depth (Ishihara, 2004).

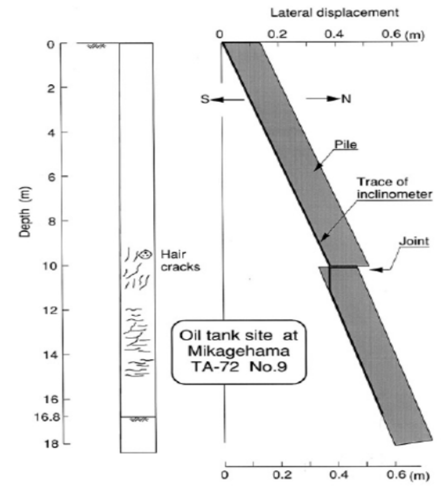

Fig. 12. Lateral displacement and observed cracks on the inside wall of Pile No. 9

Kobe 1995 EQ

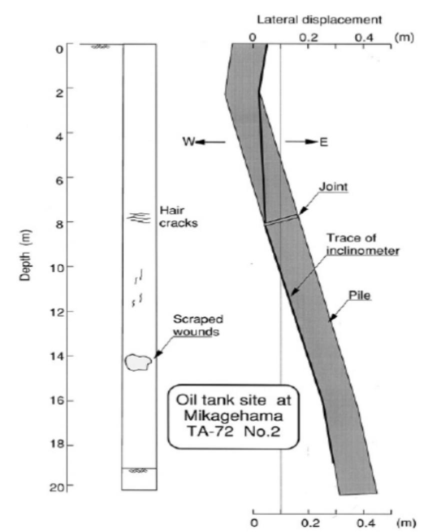

Fig. 11. Lateral displacement and observed cracks on the inside wall of Pile No. 2

Kobe 1995 EQ

DESIGN

The design of pile foundations in liquefied soils requires a reliable method of calculating the effects of earthquake shaking and post liquefaction displacements on pile Foundations (Finn 2004)

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KEYS TO GOOD DESIGN

1. **Reliable estimates of environmental loads**
2. **Realistic assessments of pile head fixity**
3. **The use of methods of analysis that can take into account adequately all the factors that control significantly the response of the pile-soil-structure system to strong shaking and/or lateral spreading in a specific design situation**

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DESIGN (Contd.)

1. The force or limit equilibrium analysis and
2. The displacement or p-y analysis
3. Dynamic analysis

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The Force or Limit Equilibrium Analysis

- Estimation of lateral soil pressures on pile for evaluating the pile response.

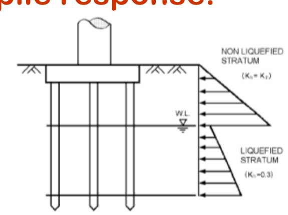

Fig. 7. . Schematic Sketch Showing Pressure Distribution Against the Piles due to Lateral Soil Flow associated with Liquefaction (JWWA, 1997)

The non-liquefied top layer is assumed to expert passive pressure on the pile. The liquefied layer is assumed to apply a pressure which is about 30% of the total overburden pressure

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This estimation of pressure is based on back calculation of case histories of performance of pile foundations during the Kobe earthquake(Ashford and Juirnarongrit, 2004 and Finn and Fujita, 2004).The maximum is assumed to occur at the interface between the liquefied and non-liquefied soil layer.

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Displacement or p-y Analysis

This method involves making Winkler type spring mass model shown schematically in Fig.8.The empirically estimated post liquefaction free field displacements are calculated. These displacements are assumed to vary linearly and applied to the springs of the soil-pile system.
Degraded p-y curves may be used for this kind of analysis.

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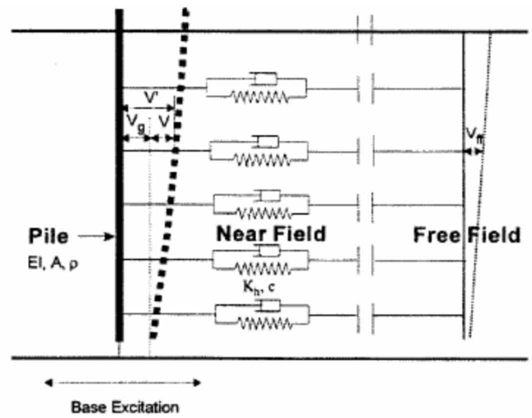

Fig.8. A Schematic Sketch for Winkler Spring Model for Pile Foundation Analysis (Finn and Thavaraj, 2001)

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DESIGN

Table 2. Reduction coefficients for soil constants due to liquefaction (JRA 1996)

Range of F_L	Depth from the Present Ground Surface x (m)	Dynamic Shear Strength Ratio R	
		$R \leqq 0.3$	$0.3 < Ra$
$F_L \leqq 1/3$	$0 \leqq x \leqq 10$	0	1/3
	$10 < x \leqq 20$	1/3	1/3
$1/3 < F_L \leqq 2/3$	$0 \leqq x \leqq 10$	1/3	2/3
	$10 < x \leqq 20$	2/3	2/3
$2/3 < F_L \leqq 1$	$0 \leqq x \leqq 10$	1/3	1
	$10 < x \leqq 20$	1	1

$F_L - FOS\ \ against\ \ Liquefaction$

$R = c_w R_L \qquad c_w = 1\ or\ 1-0.2\ depending\ \ upon\ \ the\ \ type\ I\ \ or\ \ type\ II\ \ motions$

(See code for details)

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DESIGN

North American Practice is to multiply the p-y curves, by a uniform degradation
factor p, called the p-multiplier, which ranges in values from 0.3 - 0.1

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Discussion

1. The force based method is based on the observation of pile damage during Niigata and Kobe earthquakes and the pile performance may be influenced by
   a. earthquake parameters
   b. variation in the soil profile and
   c. the pile geometry

How far are these factors accounted for in this method?

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Discussion (contd.)

2. The displacement method requires the prediction of surface displacements estimated empirically and the development of the p-y curves for generating the post-liquefaction behavior

This introduces certain amount of uncertainty

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CONCLUSIONS (Contd.)

PILES IN LIQUEFIABLE SOILS

1. Liquefaction may result in large pile group displacements.
2. Lateral spreading of soils may cause large bending moments and shears on the pile, which may result in failure of piles below the ground level (as in Niigata and Kobe earth quake).
3. Japanese and North American design practices may not give identical solutions because of the uncertainties and questions described above.
4. Considerably more research is needed to refine design methods.

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THANK YOU

NOT AN EASY PROBLEM

PLEASE ASK ONLY SIMPLE QUESTIONS

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