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# Design Prediction and Performance of Piles for Seismic Loads

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# DESIGN PREDICTION AND PERFORMANCE OF PILES FOR SEISMIC LOADS

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ABSTRACT

Pile foundations are regarded as a safe alternative for supporting structures in seismic areas. The performance of piles depends on soil profile, pile and earthquake parameters. The soils may also be prone to liquefaction. In non-liquefying soils the shear modulus degrades with increasing strain or displacements. Material damping increases with increasing strain or displacement. Stiffness of single pile and pile groups are needed for different modes of vibration; e.g., vertical vibrations, horizontal sliding in x or y direction, rotation about x or y axis and torsion. Group action is generally accounted for by including interaction factors. Pile response in any mode of vibration is determined from principles of structural dynamics. In liquefiable soils, the liquefaction may lead to substantial increases in pile cap displacements above those for the non-liquefied case. Down-drag due to liquefied soil may also pose problems. After liquefaction, if the residual strength of the soil is less than the static shear stresses caused by a sloping site such as a river bank, lateral spreading or down slope displacements may exert damaging pressures against the piles as observed during the 1964 Niigata and the 1995 Kobe earthquakes. The paper presents state of the art on analysis and design of piles subjected to seismic loading.

## INTRODUCTION

Piles are often the preferred choice of foundations in seismic areas. The seismic loading induces large displacements or strains in the soil. The shear modulus of the soil degrades and damping (material) increases with increasing strain. The stiffness of piles should be determined for these strain effects. The elastic solutions for determining response of piles subjected to dynamic loads have been presented by several investigators in the past (Kwaza and Kraft, 1980; Novak, 1974; Novak and El-Sharnouby, 1983; Novak and Howell, 1977; Poulos, 1971; Prakash and Puri, 1988; and Prakash and Sharma, 1991). Displacement dependent spring and damping factors for piles for vertical, horizontal and rotational vibrations have been presented by Munaf and Prakash (2002), Munaf et al. (2003) and Prakash and Puri (2008). For piles in non-liquefying soils the stiffness of the pile group is estimated from that of the single piles by using group interaction factors. The contribution of the pile cap, if any, is also included. The response of the single pile or pile groups may then be determined using principles of structural dynamics.

In liquefiable soils, progressive buildup of excess pore water pressure may result in loss of strength and stiffness resulting in large bending moments and shear forces in the pile. The mechanism of pile behavior in liquefying soil has been investigated by several investigators in the recent years (Liyanapathirana and Poulos, 2005).

The design of pile foundations subjected to earthquakes requires a reliable method of calculating the effects of earthquake shaking and post-liquefaction displacements on pile foundations. Keys to good design include

1. Reliable estimates of environmental loads.
2. Realistic assessments of pile head fixity.
3. A mathematical model which can adequately account for all significant factors that affect the response of the pile-soil-structure system to ground shaking and/or lateral spreading in a given situation.

## PILES IN NON-LIQUEFIABLE SOIL

The equivalent spring stiffness and damping for making the mathematical model of the soil-pile system for any mode of vibration are a function of Young's modulus of pile material ($E_p$), shear modulus of soil ($G_s$) , and geometry of the piles in the group. Shear modulus and hence spring and damping factors are strain or displacement dependent. There are six independent spring factors for a piles-cap system; ie; $k_x$ , $k_y$ , $k_z$ , in translation in *x*, *y* and *z* directions, respectively and $k_\theta$ , $k_\phi$ , $k_\psi$ rotational-springs about *x, y* and *z* directions respectively. There are two rotational cross-coupled springs; i.e.; $k_{x\phi}$ and $k_{y\theta}$ which include 2-components of displacement; i.e., translation and rotation about the appropriate axis. Also there are corresponding eight damping factors; i.e., $c_x$ , $c_y$, $c_z$ , $c_\theta$ , $c_\phi$ ,

$c_\psi$ , and. $c_{x\phi}$ and $c_{y\theta}$ . To develop displacement dependent relationships for the spring and damping factors, appropriate relationships between strain and displacement are needed. Also, modulus degradation with strain needs be built into these relationships. Appropriate non-linear relationships for spring and damping factors have been developed using Novak's solutions by Munaf and Prakash (2002) and Munaf et al. (2003), which have been used for analysis of bridge structures (Anderson and Prakash et al. , 2001 and Luna and Prakash et al. , 2001). Novak's (1974) model was used for the computation of stiffness and damping of single pile and pile groups, with appropriate interaction factors. Stiffness and damping in all the modes; i.e., vertical, horizontal, rocking and torsion and cross coupling in both the x and y direction have been evaluated (Munaf and Prakash, 2002). The sign convention is explained in Fig. 1. The main assumptions in Novak's model are:

1. The pile is a circular and solid in cross section. For non-circular sections, an equivalent radius $r_{o,}$ is determined in each mode of variation.
2. The pile material is linear elastic.
3. The pile is perfectly connected to the soil, i.e., there is no separation between soil and pile during vibrations. This assumption may not be valid and in practical situations separation between the pile and soil may occur during vibrations for some depth below the ground surface.

STIFNESS AND DAMPING FACTORS OF SINGLE PILE

Vertical Stiffness ($K_z$) and Damping Factors ($C_z$)

$$k_z = \left[\frac{E_p A}{r_o}\right] f_{w1} \qquad (1.a)$$

$$c_z = \left[\frac{E_p A}{V_s}\right] f_{w2} \qquad (1.b)$$

Where;
$E_p$ = modulus of elasticity of pile material
A = cross section of single pile
$r_o$ = radius of a circular pile or equivalent pile radius
$V_s$ = shear wave velocity of soil along the floating pile and $f_{w1}$ and $f_{w2}$ are obtained from Figure 2 for parabolic variation of shear modulus of soil '$G_S$' with depth.

Torsional Stiffness ($k_\psi$) and Damping Factors ($c_\psi$)

$$k_\psi = \left[\frac{G_p I_{p_p}}{r_o}\right] f_{T,1} \qquad (2.a)$$

$$c_\psi = \left[\frac{G_p I_{p_p}}{V_s}\right] f_{T,2} \qquad (2.b)$$

Where;
$G_p$ = shear modulus of elasticity of pile material
$Ip_p$ = Polar moment of inertia of single pile about z axis
$f_{T,1}$ and $f_{T,2}$ have been developed by Novak and Howell (1977) and are shown in figure 3.

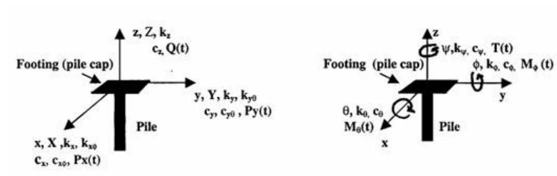


*Fig. 1. Sign Convention*

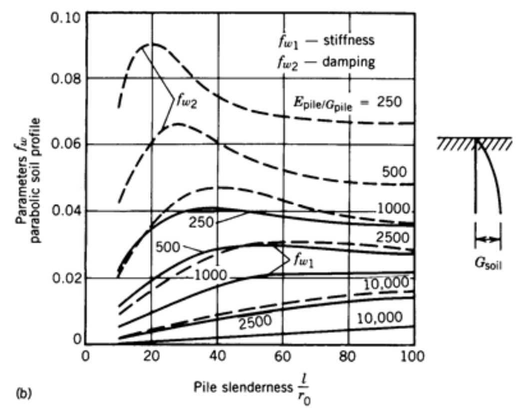


*Fig. 2. Stiffness and Damping Parameters for Vertical Response of Floating Piles (Novak and El-Sharnouby; 1983)*

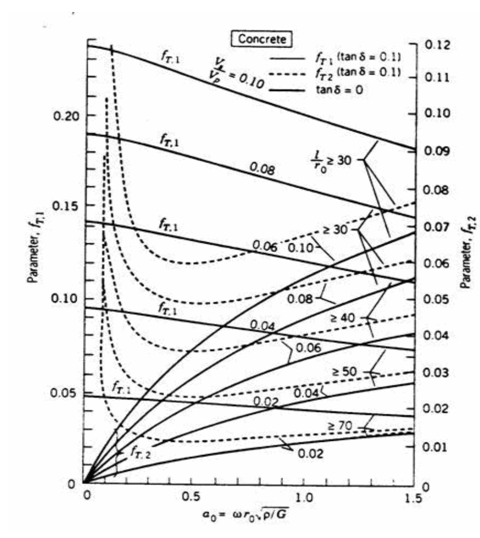


*Fig. 3. Torsional stiffness and damping parameters for Reinforced Concrete (Novak And Howell, 1977)*

Sliding and Rocking Stiffness and Damping Factors

Because, the pile is assumed to be cylindrical with a radius $r_{o,}$ its stiffness and damping factors in any horizontal direction are the same. However, in the pile group, the number of piles in the x and y directions may be different. Therefore the stiffness and damping factors of a pile group are dependent on the number of piles and their spacing in each direction.

Sliding ($k_x$, $c_x$)

$$K_x = \left[ \frac{E_p I_p}{r_0^3} \right] f_{x1} \tag{3.a}$$

$$C_x = \left[ \frac{E_p I_p}{r_0^2 V_s} \right] f_{x2} \tag{3.b}$$

Rocking ($k_\phi$, $c_\phi$) and ($k_\theta$, $c_\phi$)

$$K_\varphi = K_\theta = \left[ \frac{E_p I_p}{r_o^2} \right] f_{\phi 1} \tag{4.a}$$

$$C_\phi = C_\theta = \left[ \frac{E_p I_p}{r_o^2 V_s} \right] f_{\phi 2} \tag{4.b}$$

Cross-coupling ($k_{x\phi}$, $c_{x\phi}$) and ($k_{y\theta}$, $c_{y\phi}$)

$$K_{x\varphi} = K_{y\theta} = \left[ \frac{E_p I_p}{r_0^2} \right] f_{x\theta 1} \tag{5.a}$$

$$C_{x\phi} = C_{y\theta} = \left[ \frac{E_p I_p}{r_o V_s} \right] f_{x\phi 2} \tag{5.b}$$

Where;
$I_p$ = moment of inertia of single pile about x or y axis
$r_o$ = pile radius and $f_{x1}$, $f_{x2}$, $f_{\phi 1}$, $f_{\phi 2}$, $f_{x\phi 1}$, $f_{x\phi 2}$ Novak's coefficient obtained from Table 1 for parabolic soil profile, with appropriate interpolation and for $\nu = 0.25$

GROUP INTERACTION FACTOR

To consider group effect, Poulos (1968) assumed one of the piles in the group as a reference pile. In the illustration in Figure 4, pile No. 1 is assumed as a reference pile and distance 'S' is measured from the center of the reference pile to center of other pile. For vertical vibrations use Figure 5 to obtain $\alpha_A$ for each pile for appropriate $S/2r_o$ values. $\alpha_A$'s are function of length of the pile (L) and radius ($r_o$). Use Figure 6 (Poulos, 1971), to obtain $\alpha_L$ for each pile in the horizontal x-direction, considering departure angle $\beta$ (degrees). $\alpha_L$'s are a function of L, $r_o$ and flexibility $K_R$ as defined in Figure 6 and departure angle ($\beta$). This procedure will also apply for horizontal y-direction.

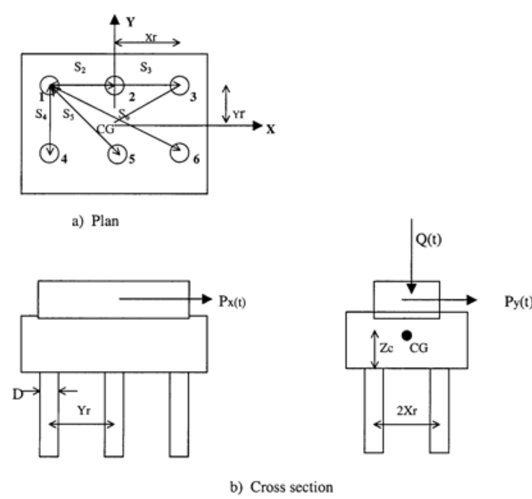


*Fig. 4. Plan and Cross Section of Pile Group for illustration.*

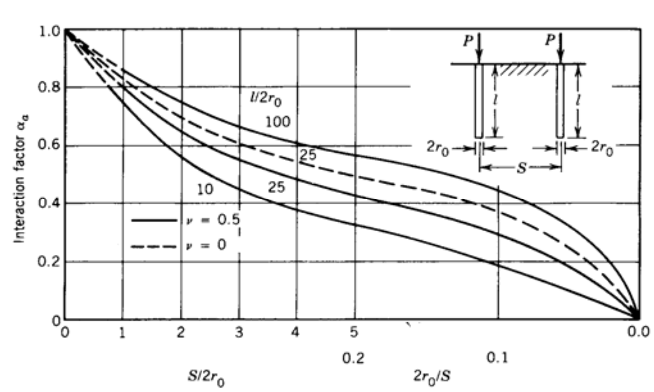


*Fig. 5. $\alpha_A$ as a Function of Pile Length and Spacing (Poulos, 1968)*

The group interaction factor ($\sum\alpha_L$) is the summation of $\alpha_L$ for all the piles. Note that the group interaction factor in x-direction and y-direction may be different depending on number and spacing of piles in each direction.

**Table 1** Stiffness and Damping Parameters of Horizontal Response for Pile with $L/R_o>25$ for Homogeneous Soil Profile and $L/R_o>30$ for Parabolic Soil Profile

| $\nu$ (1) | $E_{pile}/G_{soil}$ (2) | Stiffness Parameters $f_{\phi 1}$ (3) | $f_{x\phi 1}$ (4) | $f_{x1}$ (5) | $f^P_{x1}$ (6) | Damping Parameters $f_{\phi 2}$ (7) | $f_{x\phi 2}$ (8) | $f_{x2}$ (9) | $f^P_{x2}$ (10) |
|---|---|---|---|---|---|---|---|---|---|
| *(a) 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 |
| *(b) 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 |

*Source:* Novak and El-Sharnouby (1983). $f^P_{x1}$ and $f^P_{x2}$ are parameters for pinned end.

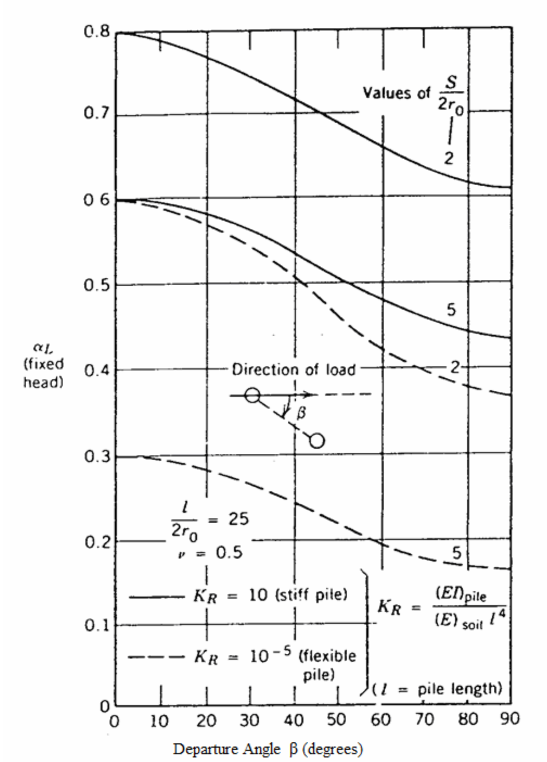


*Fig. 6. Graphical solution of $\alpha_L$ (Poulus, 1971)*

Group Stiffness and Damping

Figure 4 shows schematically the plan and cross sections of an arbitrary pile group. This figure will be used to explain the procedure for obtaining the stiffness and damping for a group of pile for all modes of vibration.

Vertical group stiffness ($k_z^g$) and damping factors ($c_z^g$)

$$k_z^g = \frac{\sum k_z}{\sum \alpha_A} \qquad (6.a)$$

$$c_z^g = \frac{\sum c_z}{\sum \alpha_A} \qquad (6.b)$$

Torsional group stiffness ($k_\psi^g$) and damping factors ($c_\psi^g$)

$$k_\psi^g = \frac{1}{\sum \alpha_A}\left[k_\psi + k_x\left(x_r^2 + y_r^2\right)\right] \qquad (7.a)$$

$$c_\psi^g = \frac{1}{\sum \alpha_A}\left[c_\psi + c_x\left(x_r^2 + y_r^2\right)\right] \qquad (7.b)$$

Sliding and Rocking and Cross Coupled Group Stiffness and Damping Factors

Translation along X-axis

$$k_x^g = \frac{\sum k_x}{\sum \alpha_{Lx}} \quad (8.a)$$

$$c_x^g = \frac{\sum c_x}{\sum \alpha_{Lx}} \quad (8.b)$$

Translation along Y Axis ($k_y^g$, $c_y^g$)

$$k_y^g = \frac{\sum k_y}{\sum \alpha_{LA}} \quad (9.a)$$

$$c_y^g = \frac{\sum c_y}{\sum \alpha_{Ly}} \quad (9.b)$$

Rocking About Y- Axis ($k_\phi^g$, $c_\phi^g$)

$$k_\phi^g = \frac{1}{\sum \alpha_{Lx}}\left[k_\phi + k_z x_r^2 + k_x z_c^2 - 2 z_c k_{x\phi}\right] \quad (10.a)$$

$$c_\phi^g = \frac{1}{\sum \alpha_{Lx}}\left[c_\phi + c_z x_r^2 + c_x z_c^2 - 2 z_c c_{x\phi}\right] \quad (10.b)$$

Rocking About X- Axis ($k_\theta^g$, $c_\theta^g$)

$$k_\theta^g = \frac{1}{\sum \alpha_{Ly}}\left[k_\theta + k_z y_r^2 + k_y z_c^2 - 2 z_c k_{y\theta}\right] \quad (11.a)$$

$$c_\theta^g = \frac{1}{\sum \alpha_{Ly}}\left[c_\theta + c_z y_r^2 + c_x z_c^2 - 2 z_c c_{y\theta}\right] \quad (11.b)$$

Cross-Coupling: Translation along X Axis and Rotation about Y Axis. ($k_{x\phi}^g$, $c_{x\phi}^g$)

$$k_{x\phi}^g = \frac{1}{\alpha_{Lx}}\sum\left(k_{x\phi} - k_x z_c\right) \quad (12.a)$$

$$c_{x\phi}^g = \frac{1}{\alpha_{Lx}}\sum\left(c_{x\phi} - c_x z_c\right) \quad (12.b)$$

Cross-Coupling: Translation along Y-Axis and Rotation about X Axis. ($k_{y\theta}^g$, $c_{y\theta}^g$)

$$k_{y\theta}^g = \frac{1}{\alpha_{Ly}}\sum\left(k_{y\theta} - k_y z_c\right) \quad (13.a)$$

$$c_{y\theta}^g = \frac{1}{\alpha_{Ly}}\sum\left(c_{y\theta} - c_y z_c\right) \quad (13.b)$$

Comparison of Computed and Predicted Pile Response

Using the above equations, the stiffness and damping for any vibration mode can be determined and the pile response can be calculated. Several researchers have attempted to make a comparison of the observed and predicted pile response. Small scale pile tests, centrifuge tests and full pile tests have been used for this purpose (Gle, 1981; Novak and ElSharnouby, 1984; Woods, 1984; and Poulos, 2007). Woods (1984) reported results of 55 horizontal vibration tests on 11 end bearing piles 15 - 48 m long. The outer diameter of piles was 35.56 cm and the wall thickness varied from 0.47 cm to 0.94 cm. typical amplitude –frequency plot for one of the piles in soft clay is shown in Fig. 7. It may be seen from this plot that the observed natural frequency decreases with an increase in the value of 'ɵ' (increase in 'ɵ' means an increase in dynamic force at the same frequency of vibrations) indicating non-linear behavior. Woods (1984) also compared the observed and computed response of the piles. The stiffness and damping values were obtained using computer program PILAY which uses continuum model accommodating soil layers and assumes homogeneous soil in the layer with elastic behavior. A typical comparison of the pile response so computed with the observed response is shown in Fig. 8. It may be observed from Fig. 8 that the calculated and computed responses do not match.

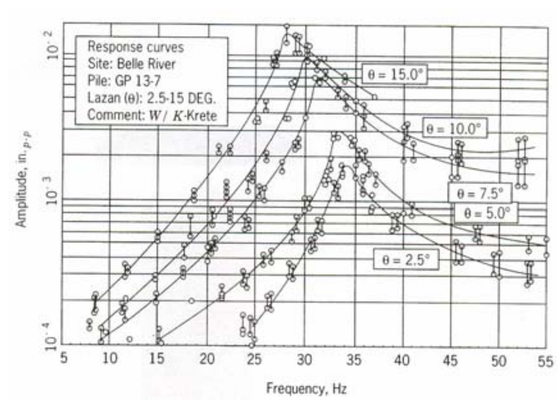


*Fig. 7. Response curves; a decrease in resonant frequency with increasing amplitudes. (Woods, 1984)*

Efforts were made to obtain a match between observed and predicted response by using reduced values of stiffness obtained from PILAY, which did not help much. A better match could, however, be obtained when a considerably softened or weakened zone was assumed surrounding the piles (program PILAY 2) simulating disturbance to soil during pile installation. A loss of contact of the soil with the pile for a short length close to the ground surface also improved the predicted response.

El-Sharnouby and Novak (1984) performed tests on 102 model pile groups using steel pipe piles. The response of the model pile groups was also computed by the following methods:

1. Using static interaction factors by Poulos (1971, 1975 and 1979) and Poulos and Davis (1980).
2. Using the concept of equivalent pier.
3. Using dynamic interaction factors given by Kaynia and Kausel91982)
4. Direct dynamic analysis of Wass and Hartmann (1981).

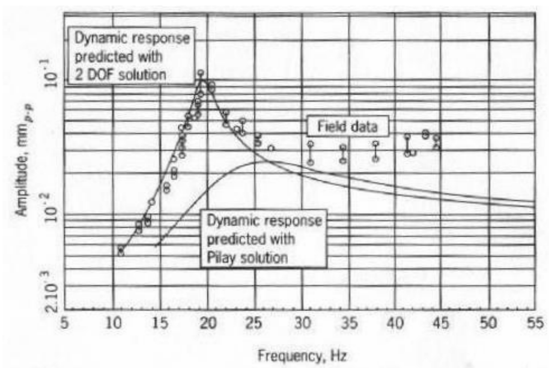


*Fig. 8. Typical response curves predicted by PILAY superimposed on measured pile response (Woods, 1984)*

A typical comparison of the theoretical and experimental horizontal response is shown in Fig.9. Plot a (Fig. 9) shows the theoretical group response without interaction effects. Response shown in plot 'b' was obtained by applying static interaction factors to stiffness only. Plot 'c' was obtained with arbitrary interaction factor of 2.85 applied to stiffness only.

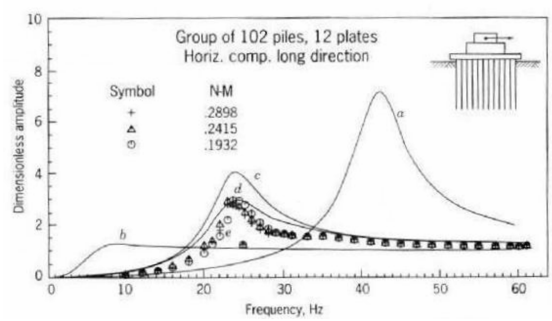


*Fig. 9. Experiment horizontal response curves and theoretical curves calculated with static interaction factors. (Novak and El-Sharnouby, 1984)*

Plot'd' was obtained by using an arbitrary interaction factor of 2.85 on stiffness and 1.8 on damping respectively. Plot 'e' shows the experimental data. The plot which shows an excellent match with experimental data was obtained by arbitrarily increasing the damping factor by 45%.

The concept of equivalent pier with some assumption and using PILAY2, gave a better match of the natural frequency but under-estimated the amplitude. The prediction improved when the damping in the calculation was reduced to 40% of the theoretical value (Novak and EL-Sharnouby, 1984). The comparison of theoretical response obtained by using dynamic interaction factors of Kaynia and Kausel (1982) is shown in plot 'a' in Fig.10. Plot 'b' in Fig. 10 shows the calculated data based on dynamic analysis of Wass and Hartmann (1981). The experimental data of Novak and EL-Sharnouby (1984) is shown by plot 'c' in the same figure. Novak and El-Sharnouby, (1984) also compared the observed response for vertical and torsional vibrations with the predicted response.

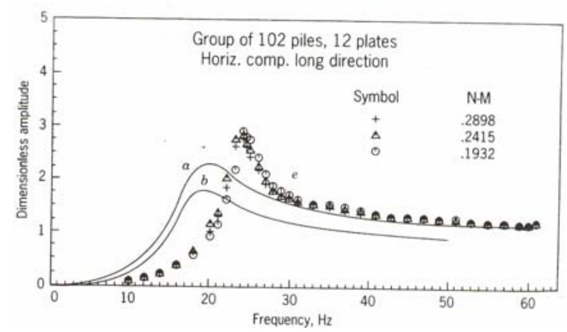


*Fig. 10. Experimental horizontal response curve and theoretical curves.*
*(a) Calculated with Kaynia and Kausel dynamic interaction factors*
*(b) Calculated with Wass and Hartmann impedances*
*(c) Experimental (Novak and El-Sharnouby, 1984).*

It was observed by El-Sharnouby and Novak (1984), Novak (1991) and Prakash and Sharma (1990) that the observed and predicted response for horizontal vibrations shows better agreement when a softened zone surrounding the piles and separation between pile and soil near the ground surface are accounted for in calculations.

El Marasafawi et al (1990) conducted horizontal vibration tests on a 0.32 m diameter, 7.5 long piles and compared with the calculated theoretical response after accounting for the weak zone surrounding the piles, Fig. 11. Similar data for a six pile group is shown in Fig. 12.

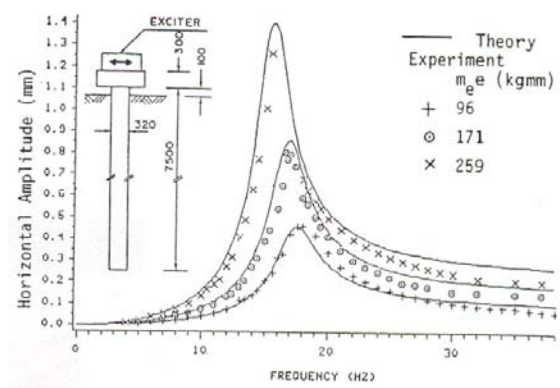


*Fig. 11: Theoretical and experimental horizontal response of concrete pile for three levels of harmonic excitation (El Marsafawi et al., 1990)*

STRAIN DEPENDENT SPRING AND DAMPING VALUES

Vucetic and Dobry (1991) have developed modulus degradation and damping relationships with shear strain.

These and similar other relationships can be been used to develop non-linear spring and damping constants.

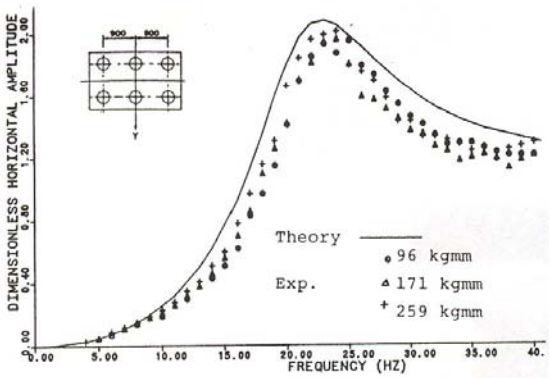


*Fig. 12. Horizontal theoretical and experimental response in Y-direction for group of six concrete piles 7.50 m long, 0.32 m in diameter (El Marsafawi et al., 1990)*

## STRAIN-DISPLACEMENT RELATIONSHIPS

Shear strain and displacement relationships are not well defined in many practical problems. Reasonable expressions are assumed and used as the basis for evaluating the shear strain in each particular case. One such relationship has been recommended by Prakash and Puri (1988) for vertically vibrating footings as:

$$\gamma = \frac{\text{Amplitude of foundation vibration}}{\text{Average width of foundation}} \qquad (14)$$

Kagawa and Kraft (1980) used the following relationship between shear strain ( $\gamma_x$ ) and horizontal displacement (x);

$$\gamma_x = \frac{(1+\nu)X}{2.5D} \qquad (15)$$

Where,
ν = Poisson's ratio
X = horizontal displacement in x-direction
D = diameter of pile

Rafnsson (1991) recommended that the shear strain $\gamma_\phi$ due to rocking ϕ can be reasonably determined as

$$\gamma_\phi = \frac{\phi}{3} \qquad (16)$$

Where,
ϕ = rotation of foundation about y axis

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

$$\gamma = \frac{(1+\nu)X}{2.5D} + \frac{\phi}{3} \qquad (17)$$

Note that, equations 15, 16, and 17 may be adopted for other directions as well.

## SOLUTION TECHNIQUE FOR DISPLACEMENT DEPENDENT K'S AND C'S

1. OBTAIN unit weight, shear wave velocity, Poison's ratio, and initial shear modulus; shear modulus degradation curve as function of soil shear strain.
2. OBTAIN pile length, pile diameter, elastic modulus of pile, and shear wave velocity in the pile.
3. SELECT relationship for half space stiffness and damping parameters as function of soil parameters, pile dimensions, and piles arrangements.
4. DETERMINE strain-displacement relationship.
5. DETERMINE stiffness and damping factor for single pile at selected displacements
6. CALCULATE group efficiency factor
7. CALCULATE group piles stiffness and damping factors
8. REPEAT Steps 5-7 for all desired displacements and plot stiffness (k) and damping (c) parameters versus displacement functions

## EXAMPLE

A bridge abutment was supported on 6 piles shown in Fig. 13. The pile length was 7.01 m, pile diameter D = 0.406 m, $x_r$=0.407 m, $y_r$ =0.914 m, $z_c$ = 0.407 m and pile elastic modulus Ep = 2.15x$10^7$ kN/$m^2$ Figure 14 and 15 show the calculated stiffness and damping factors, plotted against displacement functions.

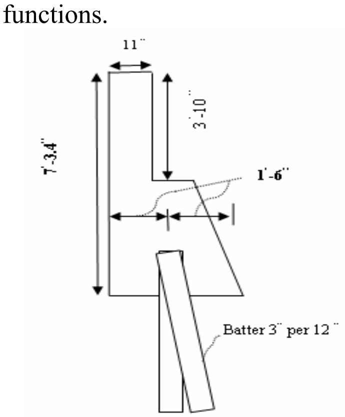


*Fig. 13. Schematic Section of abutment of Old Wahite Bridge.*

## PRACTICAL APPLICATION OF DISPLACEMENT-DEPENDENT STIFFNESS AND DAMPING

The spring and damping values shown in Fig. 14 and 15 were used to determine the displacement response of an existing bridge abutment for magnitude 6.4 and 7 earthquakes.

A typical computed displacement versus time response is shown in Fig. 16 (Andesen et al., 2001).

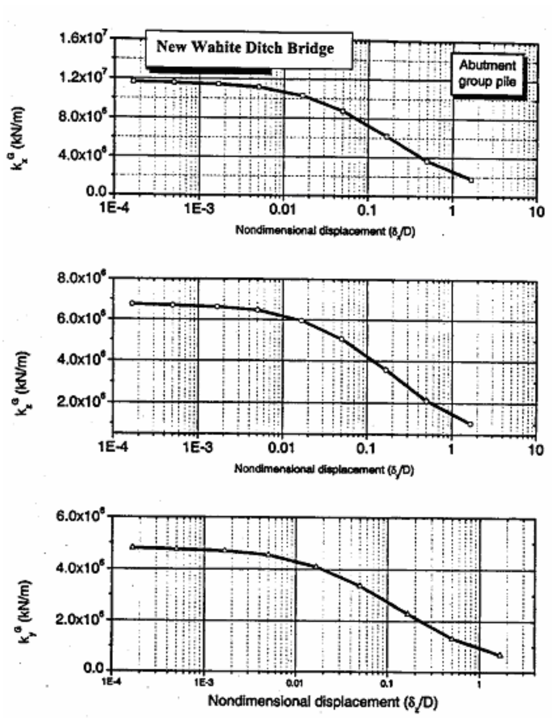


*Fig. 14. Spring Stiffness for different Modes of Vibration (Munaf and Prakash 2002)*

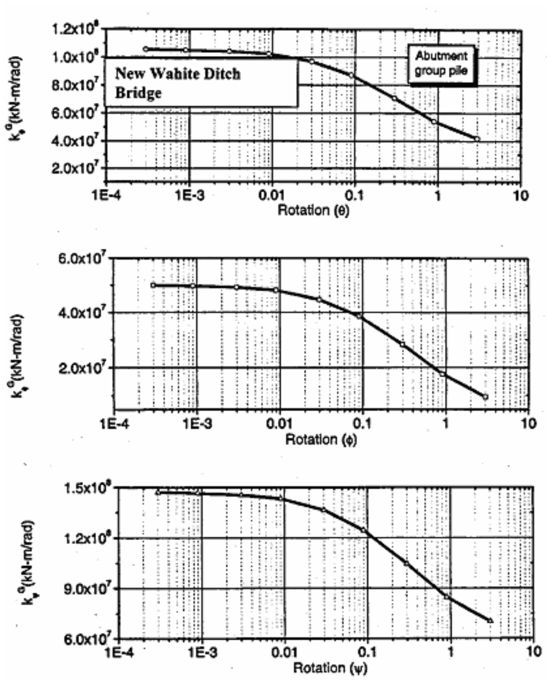


*Fig. 14. Continued*

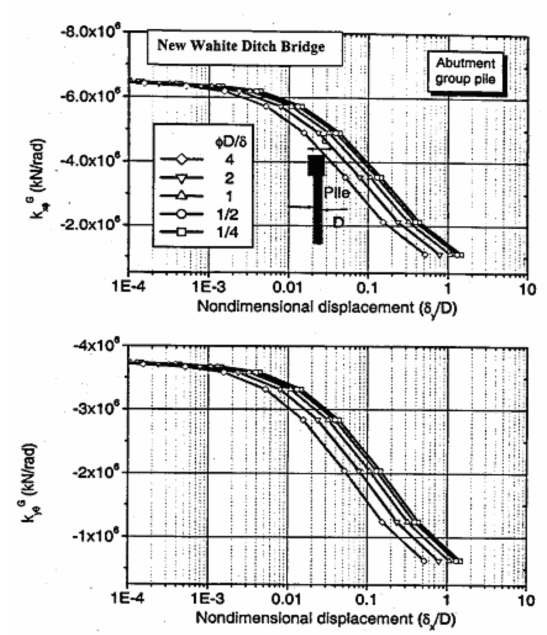


*Fig. 14. Continued*

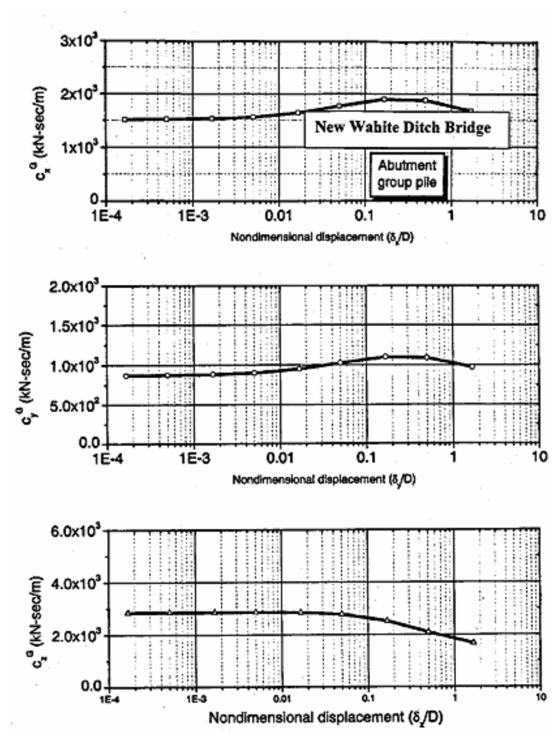


*Fig. 15. Damping for Different Modes of Vibration (Munaf and Prakash 2002)*

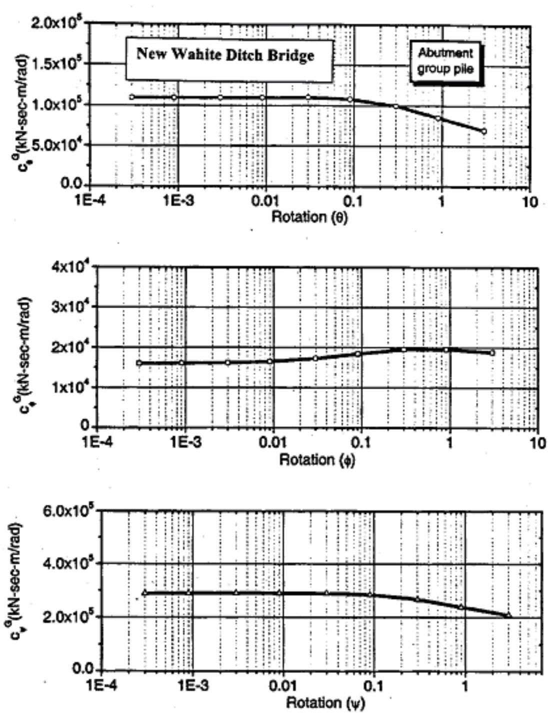


*Fig. 15. Continued*

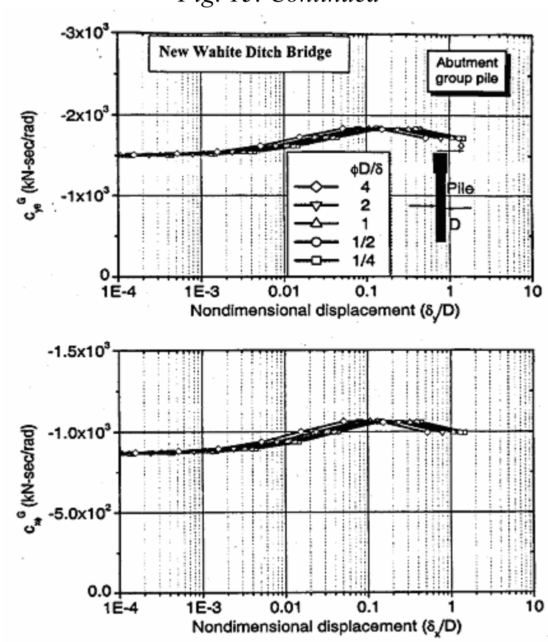


*Fig. 15. Continued*

PILES IN SOILS SUCEPTIBLE TO LIQUEFACTION

Excess pore pressures during seismic motion may cause lateral spreading resulting in large moments in the piles and settlements and tilt of the pile cap and the superstructure. Excessive lateral pressure may lead to failure of the piles which was experienced in the 1964 Niigata and the 1995 Kobe earthquakes (Finn and Fujita, 2004).

Damage to a pile under a building in Niigata caused by about 1 m of ground displacement is shown Figure 17 (Yasuda et al., 1990). Displacement of Quay wall and damage to piles supporting tank TA72 (Fig. 18, 19 and 20) during 1995 Kobe earthquake has been reported by Ishihara and Cubrinovski, (2004)

The quay wall moved approximately 1 m 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 21 the permanent lateral ground displacement corresponding to the location of Tank TA72 is seen somewhere between 35 and 55 cm (Ishihara and Cubrinovski, 2004).

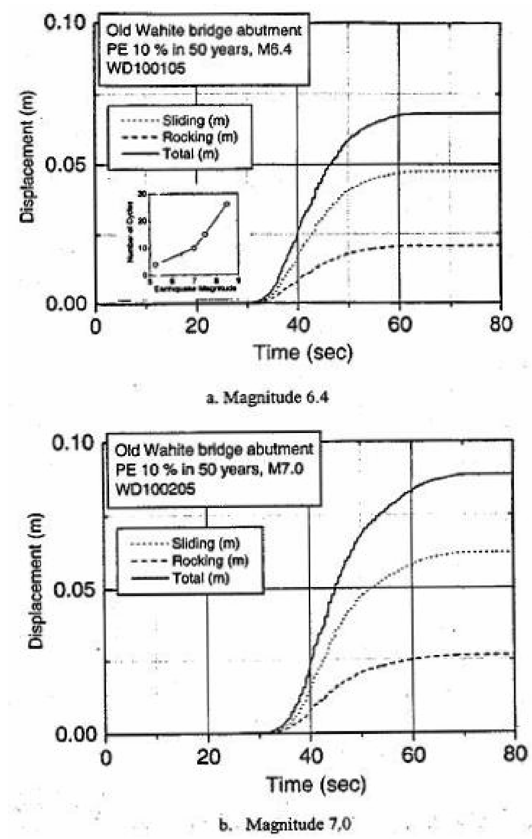


*Fig. 16. Time histories of sliding, rocking and total permanent Displacement of the Old Wahite Ditch Bridge Abutment PE 10% in 50 years, (a) Magnitudes 6.4 and (b) Magnitude 7.0*

To inspect the damage to the piles supporting the oil tank site after Kobe (1995) event, 70 cm wide and 1 m deep trenches were excavated at 4 sections and the upper portions of two piles 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. It was observed that the piles had suffered multiple cracks and the largest damage was at the interface between the liquefying fill layer and the underlying silty soil layer.

## DESIGN OF PILES IN LIQUEFYING SOIL

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 and Fujita, 2004). The methods currently in use for design of piles in liquefying soil are:

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

*Fig. 17. Damage to pile by 2m of lateral ground displacement during 1964 Niigata earthquake (Yosuda et al.1999)*

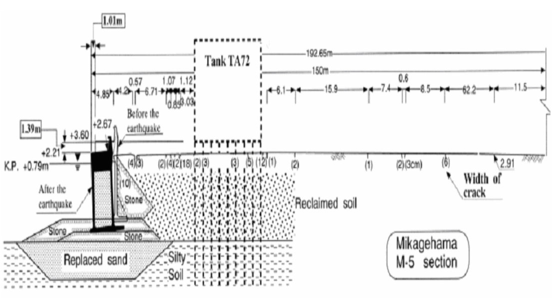


*Fig. 18. Detailed profiles of the quay wall movement and ground distortion in the backfills at Section M-5 (Ishihara and Cubrinovski, 2004)*

The Force or Limit Equilibrium Analysis

This method of analysis is recommended in several Japanese design codes for analysis of pile foundations in liquefied soils undergoing lateral spreading (JWWA, 1997; JRA, 1996). The method involves estimation of lateral soil pressures on pile and then evaluating the pile response.

A schematic sketch showing lateral pressures due to a non-liquefied and liquefied soil layers is shown in Figure 22. The non-liquefied top layer is assumed to exert passive pressure on the pile. The liquefied layer is assumed to apply a pressure which is about 30% of the total overburden pressure. 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 lateral pressure is assumed to occur at the interface between the liquefied and non-liquefied soil layers.

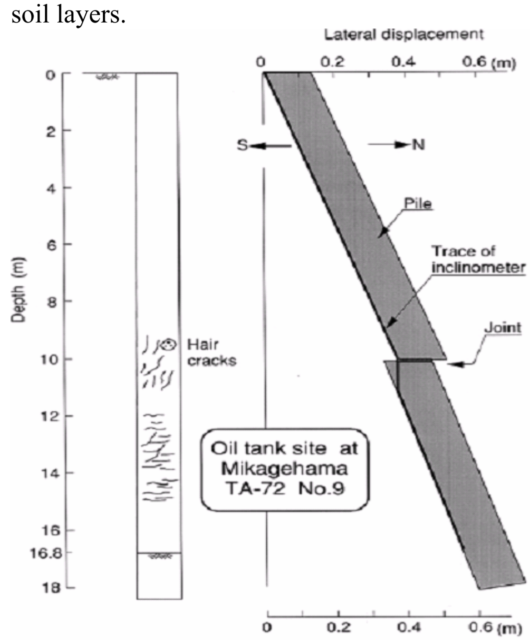


*Fig. 19. Lateral displacement and observed cracks on the inside wall of Pile No. 9 Kobe 1995 EQ (Ishihara and Cubrinovski, 2004)*

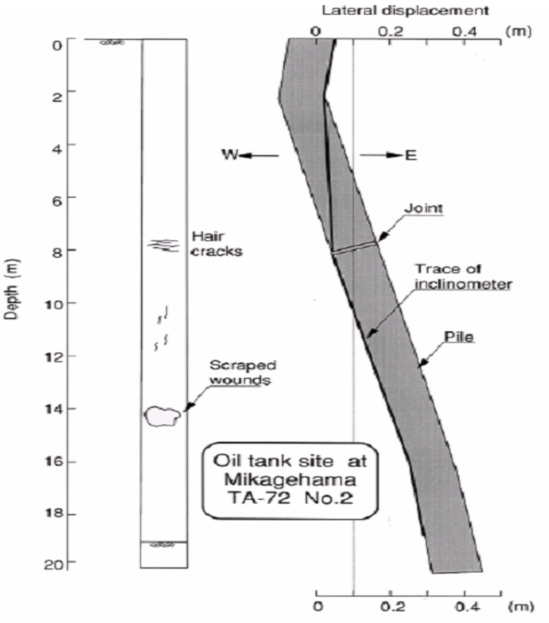


*Fig. 20. Lateral displacement and observed cracks on the inside wall of Pile No. 2 Kobe 1995 EQ (Ishihara and Cubrinovski, 2004)*

Displacement or p-y Analysis

This method involves making Winkler -type spring mass model shown schematically in Figure 23. 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 as shown in Fig. 23 (Finn and Thavaraj, 2001).

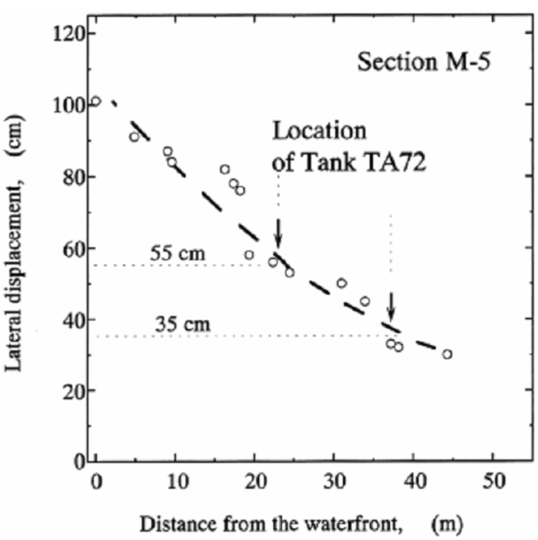


*Fig. 21. Lateral ground displacement versus distance from the waterfront along Section M-5, Kobe 1995 EQ (Ishihara and Cubrinovski, 2004)*

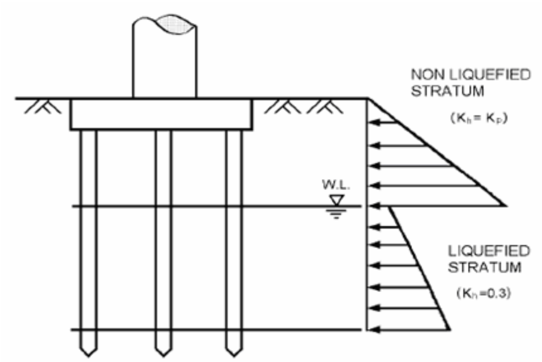


*Fig. 22. Schematic sketch showing pressure distribution against the piles due to lateral soil flow associated with liquefaction (JWWA, 1997)*

Degraded p-y curves may be used for this kind of analysis. In the Japanese practice the springs are assumed to be linearly elastic-plastic and can be determined from the elastic modulus of soil using semi-empirical formulas (Finn and Fujita, 2004). The soil modulus can be evaluated from plate load tests or standard penetration tests. Reduction in spring stiffness is recommended by JRA (1996) to account for the effect of liquefaction. Such reduction is based on ‘$F'_L$’ the factor of safety against liquefaction. These reduction factors are shown in Table 2.

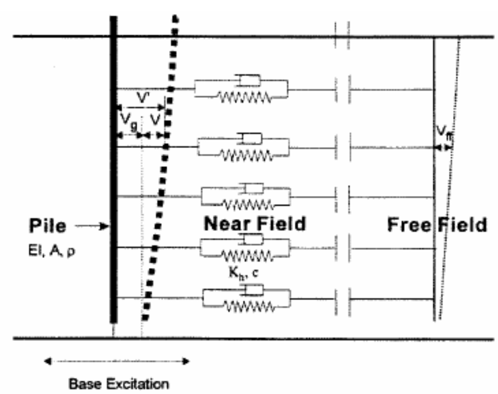


*Fig. 23. Schematic sketch for Winkler spring Model for pile foundation analysis (Finn and Thavaraj, 2001)*

Ashford and Juirnarongrit (2004) compared the force-based analysis and the displacement-based analysis for the case of single piles subjected to lateral spreading. They observed that the force- based analysis reasonably estimated the pile moments but underestimated the pile displacements. The displacement-based analysis was found to make a relatively better prediction of both the pile moments and the pile displacements compared to the force-based analysis.

**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: $R \leqq 0.3$ | Dynamic Shear Strength Ratio R: $0.3 < Ra$ |
|---|---|---|---|
| $F_L \leqq 1/3$ | $0 \leqq x \leqq 10$ | 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 |

The 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. The values ‘p’ seems to decrease with pore water pressure increase (Dobry et al., 1995) and become 0.1 when the excess pore water pressure is 100%. Wilson et al. (1999) suggested that the value of ‘p’ for a fully liquefied soil also depends on the initial relative density Dr. The values of ‘p’ range from 0.1 to 0.2 for sand at about 35% relative density and from 0.25 to 0.35 for a relative density of 55%. It was found that the resistance of the loose sand did not pick up even at substantial strains in the denser sand, after an initial strain range in which it showed little strength, picked up strength with increasing strain Fig. 24. This finding suggests that the good performance of the degraded p-y curves which did not include an initial range of low or zero strength, must be test specific and the p-multiplier may be expected to vary from one design situation to another. Dilatancy effects may reduce the initial p-y response of the dense sands (Yasuda et al. 1999).

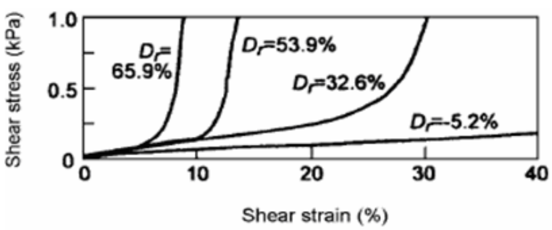


*Fig. 24. Post-liquefaction un-drained stress-strain behavior of sand (Yasuda et al 1999)*

Liyanapathirana and Poulos (2005) developed a numerical model for simulating the pile performance in liquefying soil. They also studied the effect of earthquake characteristics on pile performance and observed that the ‘Arias intensity’ and the natural frequency of the earthquake strongly influence performance of the pile in liquefying soil. Bhattacharya (2006) re-examined the damage to piles during 1964 Niigta and 1995 Kobe earthquakes and noted that pile failure in liquefying soil can be better explained as buckling type failures.

The force based and displacement based design procedure are based on limited number of observations. More research is needed to arrive at realistic design procedures for pile in liquefied soil.

## CONCLUSIONS

(a) 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

(b) 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 earthquakes)
3. Japanese and North American design practices may not give identical solutions.
4. Considerably more research is needed to refine design methods

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