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ON FOUNDATIONS UNDER SEISMIC LOADS

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ABSTRACT

Shallow foundations may experience a reduction in bearing capacity and increase in settlement and tilt due to seismic loading. The reduction in bearing capacity depends on the nature and type of soil and ground acceleration parameters in non-liquefiable soils. In liquefying soils, the buildings on shallow footings may settle and tilt excessively. The soil-pile behavior under earthquake loading is generally non-linear, which is accounted for by defining soil- pile stiffness in terms of strain dependent soil modulus. Several behavioral and design aspects of shallow foundations and piles subjected to earthquakes have been critically reviewed, both in non-liquefiable and liquefiable soils.

INTRODUCTION

Structures subjected to earthquakes may be supported on shallow foundations or on piles which must be safe both for the static as well for the dynamic loads imposed by the earthquakes. Shallow foundations in non-liquefiable soils are commonly designed by the equivalent static approach. In liquefiable soils, pore pressure buildup and drainage conditions may result in decrease in strength and considerable damage due to tilting and settlement. Prasad et al (2004) made an experimental investigation of the seismic bearing capacity of sand.A practical method to account for reduction in bearing capacity due to earthquake loading was presented by Richards et al (1993).
For analysis and design of pile groups under seismic loads a simple approach to account for nonlinear soil-pile interaction in non-liquefiable soils, uses strain dependent soil modulus to define soil-pile stiffness and radiation damping. In liquefiable soils, piles may experience excessive bending moments, and movements due to lateral spreading of soils.

SHALLOW FOUNDATIONS IN NON-LIQUEFIABLE SOILS

The response of a footing to dynamic loads is affected by the (1) nature and magnitude of dynamic loads, (2) number of pulses and (3) the strain rate response of soil. Shallow foundations for seismic loads are usually designed by the equivalent static approach. The foundations are considered as eccentrically loaded and the ultimate bearing capacity is accordingly estimated. To account for the effect of dynamic nature of the load, the bearing capacity factors are determined by using dynamic angle of internal friction which is taken as 2-degrees less than its static value (Das, 1992).

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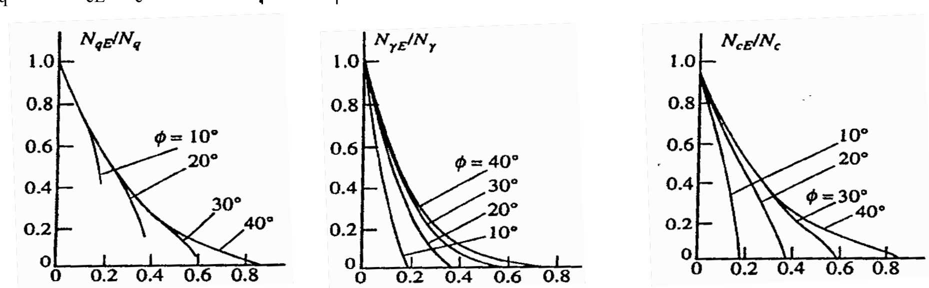
International Building Code generally permits an increase of 33 % in allowable bearing capacity when earthquake loads in addition to static loads are used in design of the foundation. This recommendation may be reasonable for dense granular soils, stiff to very stiff clays or hard bedrocks but is not applicable for friable rock, loose soils susceptible to liquefaction or pore water pressure increase, sensitive clays or clays likely to undergo plastic flow (Day, 2006).
Richards et al(1993) proposed a simplified approach to estimate the dynamic bearing capacity q_{ue} and seismic settlement S_{Eq} of a strip footing for assumed failure surfaces. The seismic bearing capacity (q_{uE}) is given by Eq. 1:

$$q_{uE} = cN_{cE} + qN_{qE} + \tfrac{1}{2}\,\gamma\, BN_{\gamma E} \tag{1}$$

where, γ = Unit weight of soil
$q = \gamma D_f$ and D_f = Depth of the foundation
N_{cE}, N_{qE}, and $N_{\gamma E}$ = Seismic bearing capacity factors which are functions of φ and $\tan\psi = k_h / (1-k_v)$
k_h and k_v are the horizontal and vertical coefficients of acceleration due to earthquake.
For static case, $k_h = k_v = 0$ and Eq. (1) becomes

$$q_u = cN_c + qN_q + \tfrac{1}{2}\,\gamma BN_\gamma \tag{2}$$

in which N_c, N_q and N_γ are the static bearing capacity factors. Figure 1 shows plots of $N_{\gamma E}/N_\gamma$, N_{qE}/N_q and N_{cE}/N_c with tan ψ and φ.



tan $\psi = k_h/(1-k_v)$, **tan $\psi = k_h/(1-k_v)$,** **tan $\psi = k_h/(1-k_v)$,**

Figure 1. Variation of N_{qE}/N_q, $N_{\gamma E}/N_\gamma$ and N_{cE}/N_c with φ and tan ψ (After Richards et al 1993)

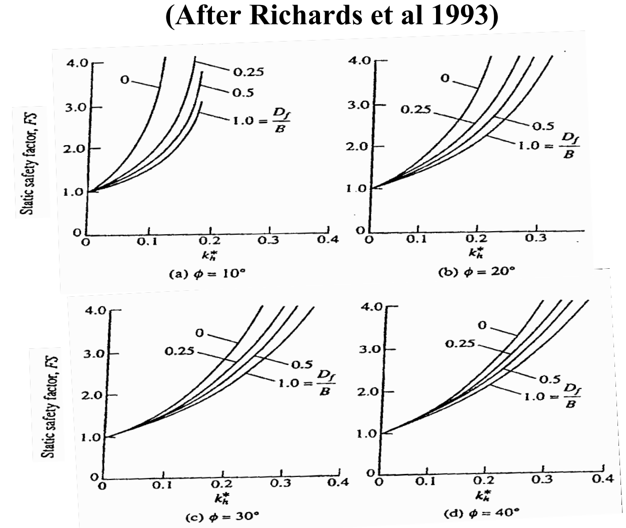


Figure 2. Critical acceleration k_h^* (After Richards et al, 1993)

Seismic Settlement of Foundations

Bearing capacity-related settlement takes place when the ratio $k_h/(1 - k_v)$ reaches a critical value $(k_h/1 - k_v)^*$. If $k_v = 0$, then $(k_h/1 - k_v)^*$ becomes equal to k_h^*. Figure 2 shows the variation of k_h^* (for $k_v = c = 0$; granular soil) with the static factor of safety (FS) applied to the ultimate bearing capacity (Eq. 2), for $\varphi = 10°, 20°, 30°$, and $40°$ and D_f/B of 0, 0.25, 0.5 and 1.0. The settlement (S_{Eq}) of a strip foundation due to an earthquake can be estimated (Richards et al, 1993) as

$$S_{Eq}(m) = 0.174 \frac{V^2}{Ag} \left| \frac{k_h *}{A} \right|^{-4} \tan \alpha_{AE} \quad (3)$$

where V = peak velocity for the design earthquake (m/sec),A = acceleration coefficient for the design earthquake, g = acceleration due to gravity (9.81 m/sec^2).

tan α_{AE} depends on φ and k_h*.In Figure 3, variation of tan tan α_{AE} with k_h* for φ of 15° - 40° is shown.(see also Puri and Prakash 2007)

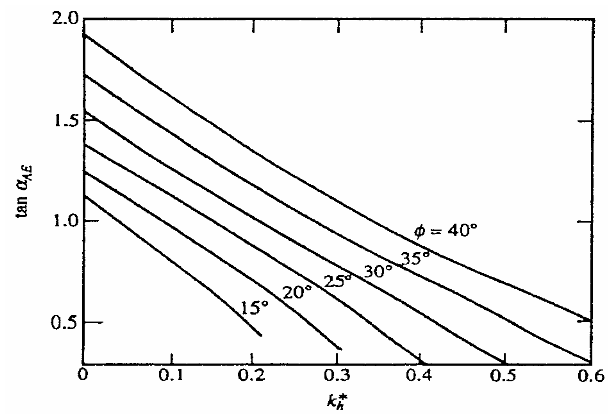


Figure 3. Variation of tan α_{AE} with k_h* and φ (After Richards et al 1993)

There is hardly any experimental verification of these theoretical solutions. There is a need for such validation.

Shallow Foundations in Liquefiable Soils

Gazezas et al (2004) studied tilting of buildings in 1999 Turkey earthquake. Detailed scrutiny of the "Adapazari failures" showed that significant tilting and toppling were observed only in relatively slender buildings (with aspect ratio: $H / B > 2$), provided they were laterally free from other buildings on one of their sides. Wider and/or contiguous buildings suffered small if any rotation. For the prevailing soil conditions and type of seismic shaking, most buildings with $H / B > 1.8$ overturned, whereas buildings with $H / B < 0.8$ essentially only settled vertically, with no visible tilting. Figure 4 shows a plot of H/B to tilt angle of building which were investigated in their studies.

Soil profiles based on three SPT and three CPT tests, performed in front of each building of interest, revealed the presence of a number of alternating sandy-silt and silty-sand layers, from the surface down to a depth of at least 15 m with values of point resistance $q_c \approx (0.4 – 5.0)$ MPa (Gazetas 2003). Seismic–cone measurements revealed shear wave velocities *Vs* less than 60 m/s for depths down to 15 m, indicative of extremely soft soil layers. Ground acceleration was not recorded in Tigcilar. Using in 1-D wave propagation analysis, the EW component of the Sakarya accelerogram (recorded on soft rock outcrop, in the hilly outskirts of the city) lead to acceleration values between 0.20 g -0.30 g, with several significant cycles of motion, with dominant period in excess of 2 seconds. Even such relatively small levels of acceleration would have liquefied at least the upper-most loose sandy silt layers of a total thickness 1–2 m, and would have produced excess pore-water pressures in the lower layers Gazetas et.al(2004).

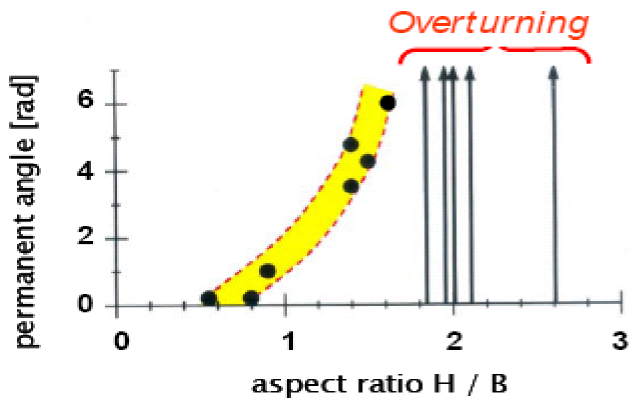


Figure 4. The angle of permanent tilting as a unique function of the slenderness ratio H/B(Gazetas et al 2004)

Gazetas et.al(2004) has studied tilting and settlements of the buildings during this earthquake in 2-ways:
First, toppling of the structural uplifting from the supporting soil is examined under the extreme assumption of linearly elastic soil.
Second, pseudostatic exceedance of the bearing capacity of the soil, to overturning is studied under the also extreme assumption of rigid−plastic M-C soil behavior and due consideration of pore pressure build-up is taken into account.
They estimated (pseudo static case) the critical over turning acceleration of 0.31, which is slightly higher than uplifting of 0.24. The FOS against tilting is then 0.6-0.9. It was concluded that of the several factors resulting in settlements and tilt of buildings, it is not possible to determine to what degrees each one is responsible. Analysis techniques need further refinement. For more details, see Gazetas et al 2003 and 2004.

PILES AND PILE GROUPS UNDER EARTHQUAKE LOADINGS

Non-Liquefiable Soils:
Prakash and Sharma (1990) have recommended the following procedures for design of single pile against earthquakes:
1 Estimate the dead load on the pile. The mass at the pile top which may be considered vibrating with the piles is only a fraction of this load.
2. Determine the natural frequency ω_{n1} and time period in first mode of vibrations.
3. For the above period, determine the spectral displacement S_d for the appropriate damping (radiation + material).
4. Using the pile displacements in (3) above, determine the bending moments and shear forces for structural design of the pile.

Analysis of Pile Groups in Non-Liquefiable Soils
The following steps are involved in analysis of a pile group (Prakash and Sharma 1990)
1. Determine the stiffness and radiation damping of the single pile
2. Determine the stiffness of the pile group by applying appropriate pile-soil-pile interaction factors.
3. Determine natural frequencies of the pile group.
4. Estimate the response of the pile group for the given input motion.
The pile cap displacements (translation and rotation) may be used as input parameters for superstructure analysis.

Stiffness and Radiation Damping of Single Piles

Stiffness of single piles in horizontal-translation and rotation may be determined as follows: Novak and El-Sharnouby (1983) and Gazetas (1991) have recommended expressions for horizontal sliding stiffness ($k_{xø}$, rocking stiffness ($k_ø$) and cross-coupling stiffness $k_{xø}$. Corresponding expressions for radiation damping coefficients C_x, $C_ø$ and $C_{xø}$, cross- damping respectively have been developed by both the investigators.

Stiffness and Radiation Damping of Pile-Group

Novak and El-Sharnouby(1983) and Gazetas (1991) have recommended pile-soil-pile interaction factors

Group Efficiency Factor (ε) is defined as:

$$\in = \frac{\text{Group stiffness } (k_{xg})}{\text{N X Single pile stiffness}(k_x)} \tag{4}$$

where s = Center to center spacing of piles

d = Diameter or width of pile

N = Number of piles in group

Similarly group radiation damping in sliding (C_{xg}) is given by Eq. 6.,

$$\frac{Group\ \ damping\ \ (C_{xg})}{N\ X\ Single\ \ pile\ damping\ \ (C_x)} \tag{5}$$

The above equation follows from the recommendations for pile group effects by Novak (1974).

Pile Group Natural Frequencies

The next step is to determine natural frequencies of the system, which may be determined from the following equation (Kumar and Prakash, 1995)

$$\left[\left\{ \omega^4 - \left(\frac{k_x}{m} + \frac{k_\phi}{M_m} + \frac{C_x C_\phi}{m M_m} - \frac{C_{x\phi}^2}{m M_m} \right) \omega^2 + \left(\frac{k_x k_\phi}{m M_m} - \frac{k_{x\phi}^2}{m M_m} \right) \right\}^2 + \left\{ \left(\frac{k_x C_\phi}{m M_m} + \frac{k_\phi k_x}{m M_m} - \frac{2 k_{x\phi} C_{x\phi}}{m M_m} \right) \omega - \left(\frac{C_\phi}{M_m} + \frac{C_x}{m} \right) \omega^3 \right\}^2 \right]^{1/2} \tag{6}$$

Pile Group Response

The response of the pile group may be determined by any standard structural dynamic method of analysis (Luna et al 2001). A detailed time history solution for a given ground motion may be obtained and the maximum response values be selected for design. The above response may be used as input motion for analysis of the superstructure. Alternatively, a spectral response method be used.

Non-Linear Analysis

The strains in soils during earthquakes may be of the order of 10^{-4} to 10^{-2}. Non-linear analysis may be performed by using step- by- step linear approach and repeating the process until convergence between predicted and assumed strains has been obtained

Strain-Displacement Relationships

Puri and Prakash (2007) have reviewed that shear strain and displacement relationship in many practical problems for use as the basis for evaluating the shear strain in each particular case; For vertically vibrating footings as:

$$\gamma = \frac{\text{Amplitude of foundation vibration}}{\text{Average width of foundation}} \qquad (7)$$

Shear strain (γ_x) and horizontal displacement (x);

$$\gamma_x = \frac{(1+\nu)X}{2.5D} \qquad (8)$$

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

γ_ϕ due to rocking ϕ can be reasonably determined as

$$\gamma_\phi = \frac{\phi}{3} \qquad (9)$$

Where, ϕ = rotation of foundation about y axis

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

$$\gamma = \frac{(1+\nu)X}{2.5D} + \frac{\phi}{3} \qquad (10)$$

Solution Technique for Displacement Dependent K's and C's

1. Obtain Unit weight, shear wave velocity, poisson's ratio, initial shear modulus; shear modulus degradation curve as function of soil shear strain
2. Obtain Pile length, pile diameter, elastic modulus of pile, shear wave velocity
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

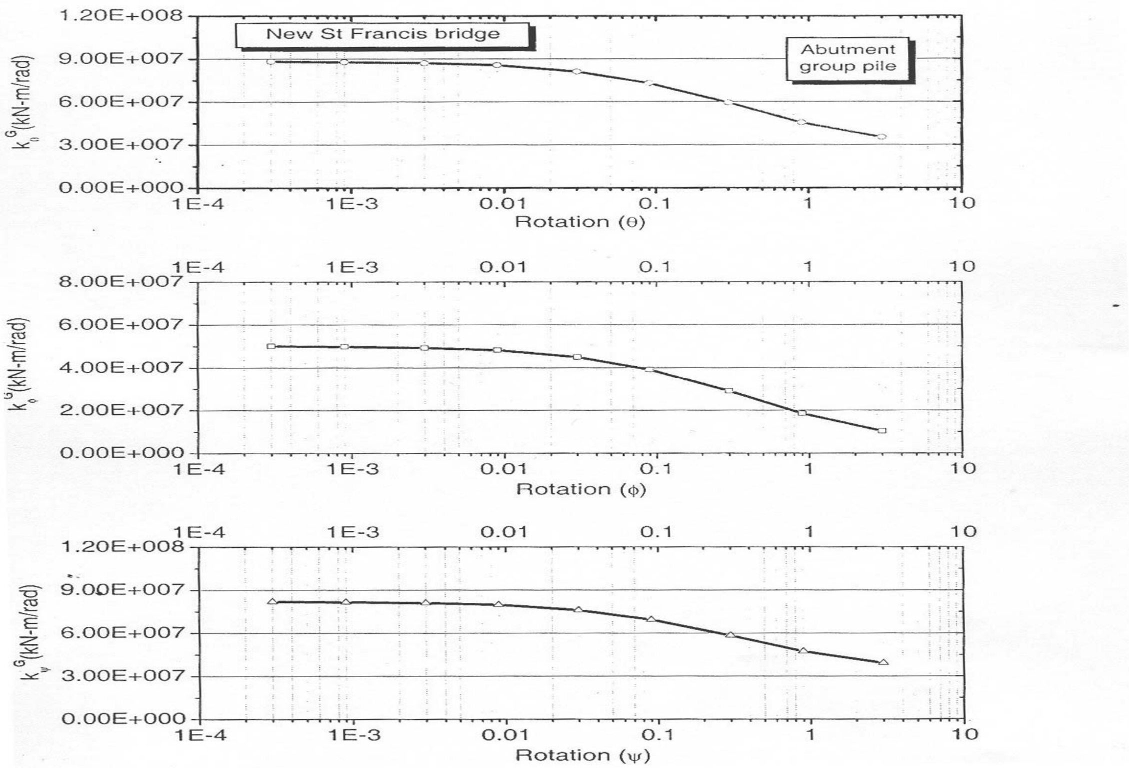


Figure. 5 k_x^g, k_y^g, k_z^g versus, non-dimensional displacement (δ/D)

Figure 5 shows a plot of k's versus non dimensional displacement of a typical pile group (Munaf and Prakash (2002)

These and similar relationships were used in the analysis of bridge structures at two locations (Anderson et al 2001, Luna et al 2001).

PILES IN LIQUEFIABLE SOILS

Excess pore pressure during seismic motion may cause lateral spreading resulting in excess moments in the piles and settlements and tilt of the pile caps and the superstructure. Excessive lateral pressure may lead to failure of the piles which were experienced in 1964 Niigata and the 1995 Kobe earthquake (Finn and Fujita 2004). Damage to a pile under a building in Niigata caused by about 1m of ground displacement is shown in Figure 6.

Figure.6 Damage to pile by 2m of lateral ground displacement during 1964 Niigata earthquake

Displacement of Quay wall and damage to piles supporting tank TA72 during 1995 Kobe earthquake has been reported by Ishihara (2004)

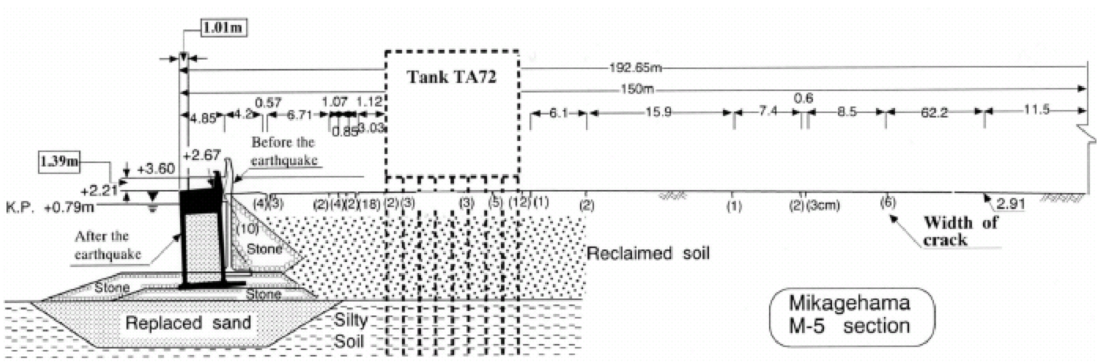


Figure.7 Detailed profile of the quay wall movement and ground distortion in the backfills at Section M-5

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 Figure.8, the permanent lateral ground displacement corresponding to the location of Tank TA72 is seen somewhere between 35 and 55 cm (Ishihara and cuvrenovsky 2004). To inspect the damage to the piles of an 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. The damaged section of Pile No.2 and 9 are shown in figures 9 and 10 respectively.

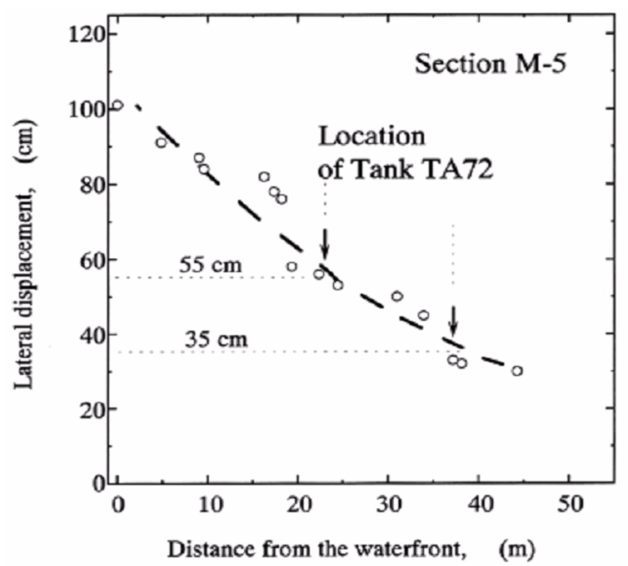


Figure.8 Lateral ground displacement versus distance from the waterfront along Section M-5(Kobe EQ 1995)

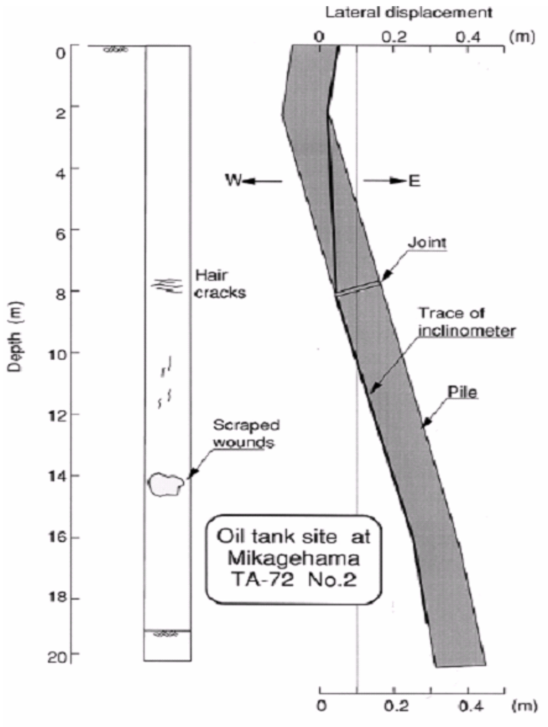


Figure.9 Lateral displacement and observed cracks on the inside wall of Pile No.2 Kobe EQ 1995

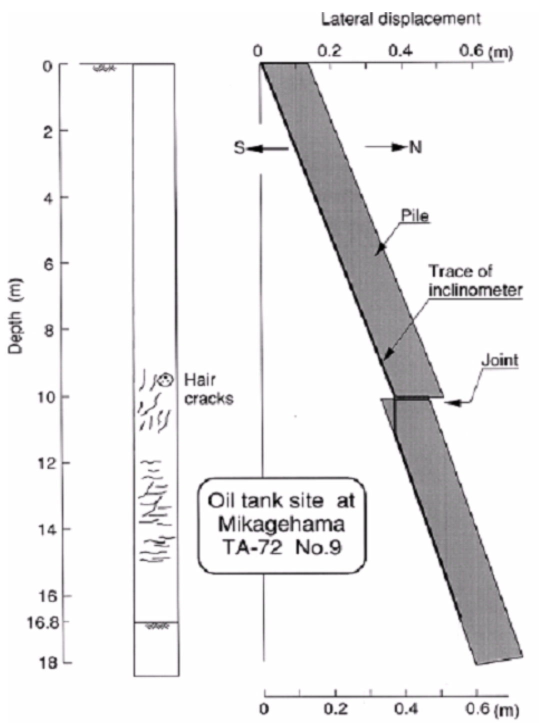


Figure .10 Lateral displacement and observed cracks on the inside wall of Pile No.9 Kobe EQ 1995

DESIGN OF PILE FOUNDATIONS IN LIQUEFIABLE SOILS

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).

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.

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. Japanese practice is more comprehensive (Finn and Fujita 2004).

CONCLUSIONS

1. A method to estimate seismic bearing capacity and settlements of strip foundations has been reviewed. Analytical solution need validation on model, full scale and/or centrifuge tests.
2. Further research on model and field tests on settlement and tilt of shallow foundations and their analysis are needed to develop reasonable design procedures for earthquake resistant design.
3. A procedure to determine non-linear response of soil-pile systems in non-liquefiable soils, using stiffness based on strain- dependent soil modulus and damping is presented.
4. In Liquefiable soils, piles may suffer both structural damage and excessive displacements. Methods of computation and design of piles in liquefiable soils need much more work before it is used in practice as a routine design.

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