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Chapter Number

Simplified Analyses of Dynamic Pile Response Subjected to Soil Liquefaction and Lateral Spread Effects

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1. Introduction

Pore water pressures greater than effective soil stress and subsequent liquefaction are known to occur in saturated sand deposits subjected to earthquake excitations. Liquefaction of soils can result in a reduction of soil strength and yields large settlement via lateral spreading. For superstructures supported on pile foundations embedded in such soils, these effects can be devastating. For example, the 1964 Niigata earthquake in Japan damaged the foundation piles under one of the piers of the 12 spans, 207 meter long Showa Bridge. After the earthquake, an excavation survey of damaged piles indicated that bending failure occurred due to the lateral spreading of river bed soils (Hamada, 1992). Similarly, in the 1994, when Northridge earthquake occurs, river bank areas between Santa Clarita and Fillmore, Highway 23 crosses over the Santa Clara River, where sand boils were observed near a bridge pier. Cracks induced by lateral spreading were found approximately 4.5 m away from the pier (Stephen et al., 2002). Afterward, in the 1995 Kobe Earthquake, quay walls along the coastline of Kobe moved up to several meters toward the sea as a result of lateral spreading (Tokimatsu and Asaka, 1998). Beside, some papers also discuss even that in long-span bridges under spatially-varying ground motions would suffer liquefaction-induced lateral spread (Abbas and Manohar 2002; Zerva and Zervas, 2002; Wang et al., 2004; Zerva 2009). More recent devastating earthquakes such as the March 2011 Tohoku earthquake in Japan and the January 2010 Haiti earthquake can be found and reported by EERI (Earthquake Engineering Research Institute) and USGS (United States Geological Survey).

Recent research has focused on understanding the transfer of forces between a pile and the surrounding layered soil during liquefaction (Hamada, 1992; Meyersohn, 1994; Tokimatsu, 2003, Bhatachaya et al., 2002、2004; Jefferies and Been, 2006). Excavation surveys by Hamada (1992) clearly showed that foundation piles are especially susceptible to damage at the interface between liquefied and non-liquefied layers. This observation was also verified by Meyersohn (1994) and Lin et al. (2005) with static numerical techniques.

In terms of static design for pile foundations, the current mechanism of failure assumes that the soil pushes the pile. The Japanese Road Association Code (JRA, 1996) has incorporated this concept. The code advises civil engineers that the non-liquefied layer acts passive

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pressure on the piles and liquefied layer offers thirty percents of overburden pressure when designing piles against bending failure due to lateral spread. Other codes such as USA code (NEHRP, 2000) and Eurocode 8, part 5 (1998) also have specifications about the problem (Bhattacharya et al., 2005). In the other hand, Tokimatsu (2003) investigated that the equivalent earth pressure acting on the pile during liquefaction in shaking table tests can be defined as the seismic passive pressures subtracting the seismic active pressures. This concept was also verified with the centrifuge tests by Haigh and Madabhushi (2005) and Madabhushi et al (2010). In a design process, engineers need the limit states to define the serviceability of members according to the safety of performances to structures (Priestley et al., 1996; Kramer and Algamal, 2001).

When piles are subjected to lateral spreading, lateral forces are exerted directly on the embedded depth of piles within liquefied layer. There are generally two methods to analyze this phenomenon. The first method is called the “Force-based method”. Using an explicit numerical procedure, earth pressure is applied onto the piles based on a viscous flow model (Chaudhuri et al. 1995; Hamada and Wakamatsu, 1998; Lin et al., 2010). In order to effectively use the force-based method, several soil parameters must be known. Also, the force-based method can account for the effect of soil topography. In the second method, known as the “Displacement-based method”, observed or computed lateral ground displacements are transmitted by theoretical soil springs on the whole pile system (Tokimastu and Asaka, 1998; Ishihara, 2003; Chang and Lin, 2003; Cubrinovski and Ishihara, 2004; Preitely et al., 2006). The second method has several advantages such as being able to choose a soil spring model that matches the complexity of the soil stratum. Also, nonlinear material effects can be considered.

This chapter investigates pile response to loading caused by liquefaction using the EQWEAP (Earthquake Wave Equation Analysis for Pile) numerical analysis procedure (Chang and Lin, 2003; Chang and Lin, 2006; Lin et al., 2010). Both a displacement and forced based form of EQWEAP are used. Methodology and case study comparisons with results of these two procedures are presented separately. The chapter ends with a final synthesis of observations and conclusions drawn from the two methods.

2. Methodology

2.1 Brief overview

The Winkler foundation model is often used in analyzing the deformation behaviors of the pile foundations. For solutions of the dynamic Winkler foundation model, or the so-called beam on dynamic Winkler foundation (BDWF) model, the wave equation analysis, initially proposed by Smith (1960), has been suggested for the driven piles. To make the wave equation analysis more accessible at the time-domain, the author (Chang and Yeh, 1999; Chang et al., 2000; Chang and Lin, 2003) has suggested a finite difference solution for the deformations of single piles under superficial loads. Such formulations can be extended for the case where the piles are subjected to seismic ground shaking. Prior to analysis of the pile system shown in Figure 1, the seismic induced free-field excitation behavior of the soil stratum needs to be obtained. A description of the soil stratum behavior during excitation provides a one-dimensional soil amplification solution for the site. For the site of interest, time dependent earthquake records are used with the modified M-O method to calculate the

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dynamic earth pressure coefficients (Zhang et al., 1998). Liquefaction potential at various depths of the site is evaluated for the limited pore water pressure ratio (Tokimatsu and Yoshimi, 1983). and numerical methods such as the finite element method or the mechanical model which models the discrete model of the pile system (Bathe, 1982) .

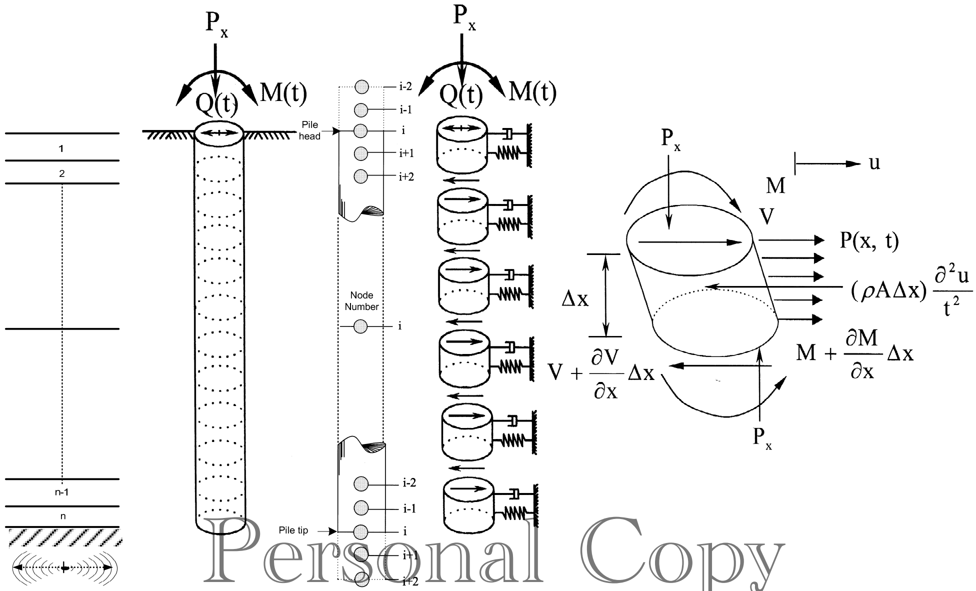

Fig. 1. Discrete system of the single pile

The earthquake motions can be decomposed into vertical and horizontal components. Pore water pressure effects are accounted for using an excess pore water pressure model. Soil deformation, seismic loading, resistance, damping and the inertia forces of the soil relative to time are applied to the pile segments and used to solve for the corresponding pile displacements. Figure 2 shows the layout of the described superposition procedure.

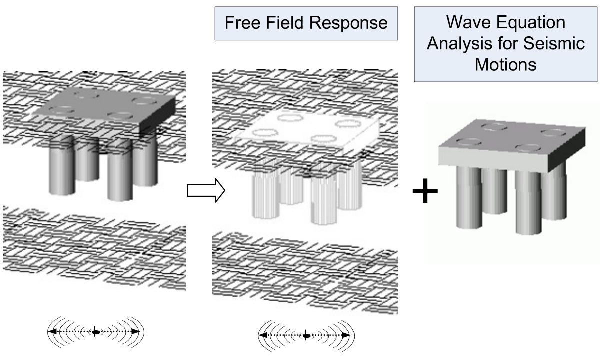

Fig. 2. Superposition of the free-field analysis and WEA

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Formulations can be derived from the wave equations of the piles. Analysis of the foundations can be performed assuming unloaded or time dependant sustained loading conditions. With proper boundary conditions at the pile head, interactions of the structural system can be modeled. The above procedure is known as EQWEAP, which mainly concerns the nonlinear behaviour of liquefied soil induced permanent ground displacement rather than piles. Figure 3 illustrates the flow chart for the EQWEAP

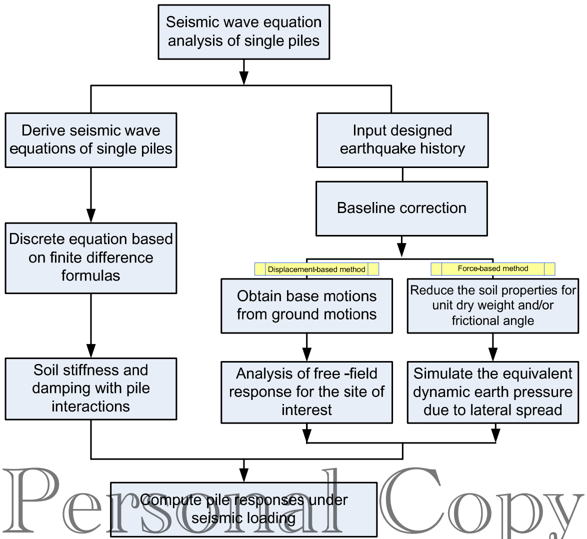

Fig. 3. Flowchart summarizing the numerical procedures of the analysis

2.2 EQWEAP: Displacement-based method

2.2.1 Wave equation of pile foundations concerning soil liquefaction

To make the wave equation analysis of the deformations of single piles under superficial loads more accessible in the time-domain, several authors (Chang and Yeh, 1999; Chang et al., 2000; Chang and Lin, 2003) have suggested a finite difference solution. Such formulations can be extended to the case where the piles are subjected to seismic ground shaking. Assuming force equilibrium, the governing differential equations of the pile segment exciting laterally can be written as:

$$E_p I_p \frac{\partial^4 u_p(x,t)}{\partial x^4} + \rho_p A_p \frac{\partial^2 u_p(x,t)}{\partial t^2} + P_x \frac{\partial^2 u_p(x,t)}{\partial x^2} + C_s \frac{\partial u(x,t)}{\partial t} + K_s u(x,t) = 0 \qquad (1)$$

where $u(=u_p - u_s)$ = relative pile displacements, u_p = absolute pile displacements, u_s =the absolute soil displacements, E_p =Young's modulus of the pile; I_p = moment of inertia of the pile, ρ_p = uniform density of the pile, A_p = cross-section area of the pile; P_x = superstructure loads, C_s and K_s = damping coefficient and stiffness of the soils along the pile, and x is ordinate variable, and t represents for time.. For earthquake loading transmitting from the soils, Eq. (1) can be expanded using the central difference formula as shown below:

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$$u_p(i,j+1)=\frac{1}{A+C}\begin{bmatrix}-u_p(i+2,j)+(4-B)u_p(i+1,j)\\ -(6-2A-2B+D)u_p(i,j)\\ +(4-B)u_p(i-1,j)-u_p(i-2,j)\\ -(A-C)u_p(i,j-1)+C[u_s(i,j+1)-u_s(i,j-1)]+Du_s(i,j)\end{bmatrix} \tag{2}$$

where $A=\dfrac{A_p\rho_p\Delta x^4}{E_pI_p\Delta t^2}$; $B=\dfrac{P_x\Delta x^2}{E_pI_p}$; $C=\dfrac{C_s\Delta x^4}{2\Delta t E_pI_p}$; $D=\dfrac{K_s\Delta x^4}{E_pI_p}$

Eq. (2) indicates that the absolute pile displacements under the earthquake excitations can be solved directly from the absolute displacements of the adjacent soil. A major advantage of this method is that the matrix analysis is not required in solving for the pile deformations. One can simply use a free-field analysis to obtain the liquefied soil displacements using the excess pore water pressure model(as described in Section 2.22) and then substitute the displacements into Eq. (2) to obtain the desired solutions. This is similar to those suggested in the multiple-step analysis of the soil-structure interaction problems. In addition, equations describing the lateral excitations of the highest and lowest elements of the pile should be modified using proper boundary conditions listed as follows.

Top of the pile:

a. Free head:

$$\frac{\partial^3 y_p(x,t)}{\partial x^3}=\frac{P_t}{E_pI_p};\frac{\partial^2 y_p(x,t)}{\partial x^2}=\frac{M_t}{E_pI_p} \tag{3}$$

b. Fixed head:

$$\frac{\partial^3 y_p(x,t)}{\partial x^3}=\frac{P_t}{E_pI_p};\frac{\partial y_p(x,t)}{\partial x}=0 \tag{4}$$

At the tip of the pile:

$$\frac{\partial^2 y_p(x,t)}{\partial x^2}=0\,;E_pI_p\frac{\partial^3 y_p(x,t)}{\partial x^3}+P_X\frac{\partial y_p(x,t)}{\partial x}=0 \tag{5}$$

where M_t and P_t are the external moment and load applied at the pile head. The discrete forms of these equations can be derived with the central difference schemes. Detailed derivations can be found in Lin (2006).

2.2.2 Soil stiffness and damping

For discrete models of the various soil types (sand, clay, etc.), both of the stress-displacement curves (t-z, q-z and p-y equations) and the Novak's dynamic impedance functions are used popularly in practice. The former, which is established empirically from the in situ pile load tests, can be used for substantial load applied slowly. The later, initially

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suggested by Novak (1972, 1974 and 1977) for the soils around the piles subjected to small steady-state vibrations, is able to capture the dynamic characteristics of the soil resistances and energy dissipations. Soil displacements close to a pile subjected to dynamic loading are nonlinear (Prakash and Puri, 1988; Nogami et al, 1992; Boulanger et al., 1999; El Naggar and Bently, 2000). El Naggar and Bently (2000) used a nonlinear soil model that incorporated a p-y curve approach to predict dynamic lateral response of piles to soil movement. The computed responses were found compatible with the results of the statnamic pile test. The nonlinear stiffness of the p-y equations is adapted in this investigation. The corresponding soil stiffness is described as below:

$$k_{NL} = \frac{8\pi G_m(1-v)(3-4v)\left[\left(\frac{r_0}{r_1}\right)^2+1\right]}{\left(\frac{r_0}{r_1}\right)^2+(3-4v)^2\left[\left(\frac{r_0}{r_1}\right)^2+1\right]\ln\left(\frac{r_1}{r_0}\right)-1} \tag{6}$$

where r_0 is the pile radius, r_1 is the outer radius of the inner zone, v is Poisson's ratio of the soil stratum, and G_m is the modified shear modulus of the soils. A parametric study shows that a ratio r_0 / r_1 of 1.1-2.0 yields the best agreement.

For the damper, a transformed damping model is used. Equivalent damping ratios, D, of the soils at steady-state excitations are first computed from the Novak's dynamic impedance functions, K* where

$$K^* = K_{real} + i\,K_{imag} = K(\omega) + i\,\omega C(\omega) \cong K(\omega)\,(1+2iD) \tag{7}$$

$$D \cong \frac{\omega C(\omega)}{(2K(\omega))} \tag{8}$$

In the above equations, $K(\omega)$ and $C(\omega)$are the frequency-dependent stiffness and damping coefficient of the impedance. For simplicity, the computed damping ratios are incorporated with the static stiffness Kst to model the kinematics of the soil. The revised damping coefficient $c(\omega)$can be written as:

$$c(\omega) \cong 2DK_{st} / \omega \tag{9}$$

Decomposing the actual load-time history into a series of small impulses, the damping coefficient $c(t)$ can be obtained by integrating a damping function c(t) to a set of unit impulses of the actual load-time history. Knowing that D=C(ω)ω/2K(ω), the associated geometric damping ratios can be computed. Modeling the values of $D(\omega)/\omega$ and assuming that they are symmetric with respect to the ordinate, a mathematical expression of the damping can be written as:

$$c(t) = AK_s t^{-B} \tag{10}$$

where A and B are the model parameters (Chang and Yeh, 1999; Chang and Lin, 2003).

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2.2.3 Modeling soil liquefaction

Soils affected by induced pore-water pressure reduce the lateral resistance of the piles. This study utilized an excess pore water empirical model to complete effective stress analysis (Martin et al., 1975; Finn et al, 1977; Finn and Thavaraj, 2001), and obtain free-field motions under liquefaction. Kim (2003) successfully predicted the excess pore-water pressure resulting in soils subjected to earthquake shaking by verifying results with laboratory tests. This model can be divided into undrained conditions and drained conditions as follows:

a. Undrained condition:

$$\Delta u_w = \frac{\Delta \varepsilon_{vd}}{\left(\dfrac{1}{\bar{E}_r} + \dfrac{n_p}{K_w}\right)} \tag{11}$$

where Δu_w = an increase in pore water pressure; $\Delta \varepsilon_{vd}$ = an increment in volumetric strain; $\bar{E}_r$ = one dimensional rebound modulus at an effective stress (σ'_v); n_p = porosity, and K_w = bulk modulus of water.

For saturated sand $K_w >> \bar{E}_r$ and therefore

$$\Delta u_w = \bar{E}_r \Delta \varepsilon_{vd} \tag{12}$$

According to simple shear test, the volumetric strain increment ($\Delta \varepsilon_{vd}$) is a function of the total accumulated volumetric strain (ε_{vd}) and the shear strain (γ). The relationship is given by

$$\Delta \varepsilon_{vd[i]} = C_1(\gamma - C_2 \varepsilon_{vd[i-1]}) + \frac{C_3 \varepsilon_{vd[i-1]}^2}{\gamma + C_4 \varepsilon_{vd[i-1]}} \tag{13}$$

$$\varepsilon_{vd[n]} = \sum_{i=1}^{n} \Delta \varepsilon_{vd[i]} \tag{14}$$

where [i] = ith time step or cycle; and C_1 , C_2 , C_3 , and C_4 are constants depending on the soil type and relative density. An analytical expression for rebound modulus ($\bar{E}_r$) at any effective stress level (σ'_v) is given by

$$\bar{E}_r = \frac{(\sigma'_v)^{1-m}}{mk_2} (\sigma'_{v0})^{m-n} \tag{15}$$

where σ'_{v0} is initial value of the effective stress; and k_2 , m and n are experimental constants for the given sand.

b. Drained condition:

If the saturated sand layer can drain during liquefaction, there will be simultaneous generation and dissipation of pore water pressure (Sneddon, 1957; Finn et al. 1977). Thus, the distribution of pore-water pressure at time (t) is given by

$$\frac{\partial u_w}{\partial t} = \overline{E}_r \frac{\partial}{\partial z}\left(\frac{k}{r_w}\frac{\partial u}{\partial z}\right) + \overline{E}_r \frac{\partial \varepsilon_{vd}}{\partial t} \tag{16}$$

where u = the pore-water pressure; z = the corresponding depth; and k = the permeability ; and r_w is the unit weight of water. Before conducting the free-field analysis, the adequate shear modulus (Seed and Idriss, 1970) may be determined from the following equation

$$G = 1000K_2(\sigma'_m)^{0.5} \tag{17}$$

where K_2 is a parameter that varies with shear strain and σ'_m is the mean effective stress. Pore water pressure will increase during shaking and leads to a decrease of effective stress. In some situations, pore-water pressure equals overburden stress in sand deposits and may liquefy. The initial shear modulus can be calculated from the initial effective stress. Then, G is modified due to the shear strain and pore water pressure under liquefaction. The modified value is substituted in place of the former one and convergence of solutions is obtained using an iterative manner.

In addition, to avoid over-predicting the excess pore water pressure and ensure compatibility with practical observations, it is suggested to use the pore water pressure ratio (r_u) to accurately control soil liquefaction levels (Lee and Albaisa, 1974; DeAlba et al., 1976; Tokimatsu and Yoshimi, 1983). The equation is given by

$$r_u = \frac{1}{2} + \frac{1}{\pi}\sin^{-1}\left(2F_L^{\frac{1}{\alpha\beta}} - 1\right) \tag{18}$$

where α, β are the experienced constants, and F_L is the safety factor of liquefaction. In order to use the above formulas, the liquefaction potential analysis of the site needs to be conducted prior to the analysis.

2.2.4 Free field analysis

The one-dimensional seismic excitations of soils onto the piles are computed from a free-field response analysis for the site of interest. Such an analysis can be conducted using the finite element technique, or be simply solved for using the 1-D wave propagation model and the lumped mass analysis. For simplicity, the lumped mass model is selected. To analyze the equations of motion of the soil layer under the earthquake excitations, the relative deformations of the structural system are obtained with the base accelerations induced by the earthquake. Base motions of the site are obtained by modifying the seismic accelerogram recorded at the ground surface of that site. This is done simply by obtaining the frequency-spectrum of the accelerogram, and then multiplying it with the analytical 'transfer function' represented for the ratios of the accelerations occurring at the base (bedrock) and those at the ground surface of that site (Roesset, 1977). This computation would complete a frequency-domain convolution and prepare a base-acceleration spectrum to solve for the corresponding accelerogram. To have consistent results for a specific site, one must be very cautionous about the wave velocities and the thickness of the soil layers used in the analyses. Crosschecks are required for vertical and horizontal excitations to ensure that the

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analytic parameters are rational. Notice that the discrete solutions of the wave equations are in terms of the displacements only. To obtain the time–displacement history of the soils, a baseline correction procedure (Kramer, 1996) is suggested to eliminate the integral offsets of the velocities and displacements appearing after the quake excitations. The responses of the free-field using the above procedure have been checked with the solutions of FEM as shown in Figure 4. Using this simplified model just be only computed one-way ground response depending on the inputted seismic motions. And, despite the simplicity of the geometry, an exact solution of the full model, and a detailed analysis of the phenomenon, have not perfectly been achieve (Schanz and Cheng, 2000).

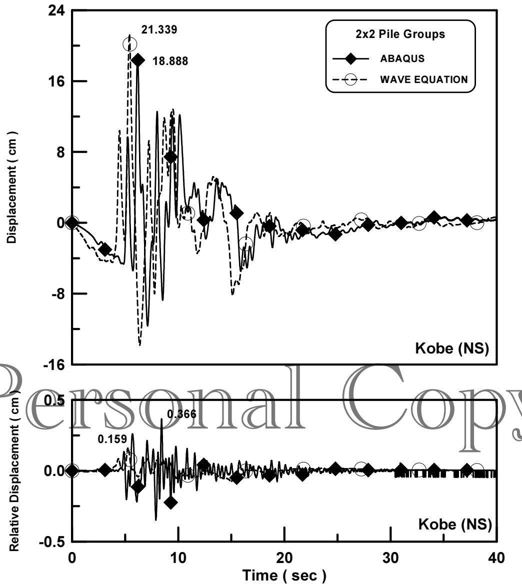

Fig. 4. Comparison of numerical results from WEA and FEM

2.3 EQWEAP: Force-based method

2.3.1 Wave equation of pile foundations concerning lateral spread

The wave equation describing a single pile under lateral loads can be derived based on a force equilibrium of the pile segments shown in Figure 1 as follows,

$$EI\frac{\partial^4 u(x,t)}{\partial x^4}+\rho A\frac{\partial^2 u(x,t)}{\partial t^2}+P_x\cdot\frac{\partial^2 u(x,t)}{\partial x^2}=P(x,t) \tag{19}$$

where u is the lateral pile displacement relative to the soil, E is the Young's modulus of the pile, I is the moment inertia of the pile, ρ is mass density of the pile, A is the cross-sectional area of the pile, P_x are the superstructure loads, $P(x,t)$ is the time-dependent loading due to laterally spreading at various depths, x is ordinate variable, and t represents for time. Using explicit finite difference schemes, the discrete form of Eq. (19) can be written as

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$$u(i,j+1)=\frac{1}{A_1}\begin{bmatrix}-u(i+2,j)+(4-B_1)\cdot u(i+1,j)\\ +(2A_1+2B_1-6)\cdot u(i,j)\\ +(4-B_1)\cdot u(i-1,j)-u(i-2,j)\\ -A_1\cdot u(i,j-1)+C_1\end{bmatrix} \tag{20}$$

where $A_1=\dfrac{\rho A\Delta x^4}{EI\Delta t^2}$; $B_1=\dfrac{P_x\Delta x^2}{EI}$; $\dfrac{P(x,t)\Delta x^4}{EI}$.

For the initial condition, $u(i,j)$ and $u(i,j-1)$ are set to zero. Equation 20 can only calculate the responses of piles under lateral loads along the lenfth of the pile. The head and tip can not be solved for. With proper boundary conditions (see Eq. 3~4), the other equations can then be derived. While the liquefaction-induced dynamic earthquake pressures are computed, the pile responses at various depths can be solved through the above formulations.

2.3.2 Dynamic earth pressure

Since Okabe (1926) and Mononobe and Matsuo (1929) introduced the concept of dynamic lateral pressure, many reports and practical works have been conducted in this manner (Ishibashi and Fang, 1987; Richard et al., 1990; Ishibahi et al., 1994; Budhu and Al-karni, 1993; Richard et al., 1993; Soubra and Regenass, 2000). Tokimatsu (1999, 2003) and Uchida and Tokimatsu (2005) determened several factors that affect the response of a pile in saturated sand by using a shaking table tests. They suggested that the total earth pressure acting on the foundation, when neglecting the friction between foundation and soil (see Fig. 5), is define as:

$$P_E=P_{EP}-P_{EA}=Q-F \tag{21}$$

where P_E is total earth pressure, P_{EP} and P_{EA} are earth pressures on the active and passive sides, Q is shear force at the pile head, and F is total inertial force from the superstructure

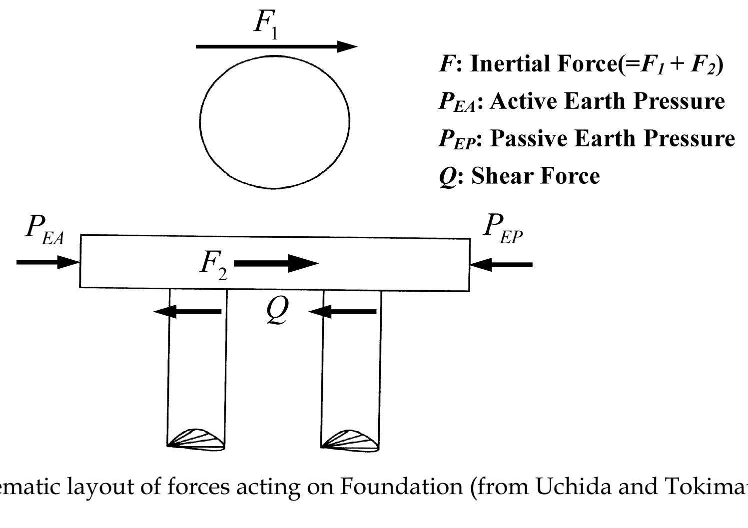

Fig. 5. Schematic layout of forces acting on Foundation (from Uchida and Tokimatsu, 2005)

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and foundation. In addition, Haigh and Madabhushi (2005) have verified that the adjacent stresses of single piles subjected to lateral spreading forces would range between the active state and the passive state through centrifuge modeling.

Based on the Mononobe-Okabe method, Zhang et al. (1998) successfully derivates the time-dependent coefficients of earth pressure under active and passive states that involves the motions of soils and foundations. One can also modify the plane strain model of soil wedge to extend it to be three dimensional analysis. The descriptions and formulations of the coefficients of active and passive earth pressures are referred to Zhang et al. (1998).

2.3.3 Modeling lateral spread

For lateral spread induced by liquefaction, the soil properties such as the unit weights and the friction angles of the soils could be corrected based on the calculated pore water pressure ratios. There are two ways depicting the weakness of soils during liquefaction (Matsuzawa et al. 1985; Ebeling and Morrison, 1993). Those equations are given by

$$\gamma'_s = \gamma_s(1 - r_u) \tag{22}$$

$$\phi'_{eff} = \tan^{-1}[(1 - r_u)\tan\phi'] \tag{23}$$

where γ'_s is the unit weight of the soil, γ_s is the effective unit weight of the soil, ϕ' is the friction angle of the soil, and ϕ'_{eff} is the effective friction angle of the soil.

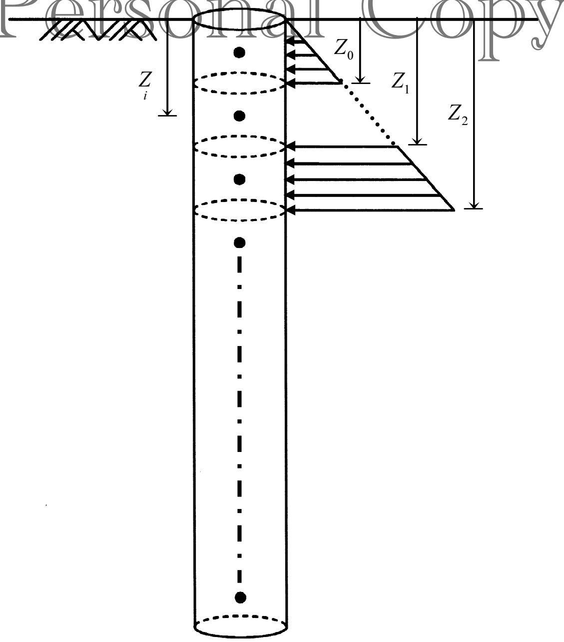

Fig. 6. Distribution of earth pressure along a pile

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Figure 6 illustrates the distributions of earth pressures along the pile with the discrete blocks and nodes. According to the geometry of pile (see figure 7) and Eq. (24), the lateral forces at various depths are determined by

$$P_E = (\gamma'_s \underset{i}{Z} K_E) \cdot (B) \tag{24}$$

where $\underset{i}{Z}$ is the corresponding depth of node, K_E is the equivalent dynamic coefficients of earth pressure (i.e. $K_E = K_{EP} - K_{EA}$), and B is the loaded width of the pile body ($= \pi\, d/2$, where d is the pile diameter).

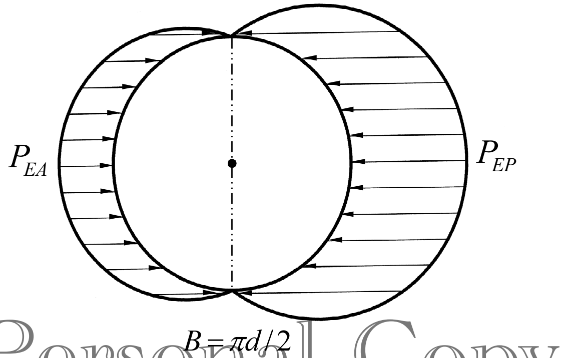

Fig. 7. The loaded width of the pile body due to lateral spreading

3. Practical simulation

In the following section, two case studies are presented, one of which focuses on pile foundation damagess caused by the Niigata earthquake in Japan (Hamada, 1992) and the other which focuses on foundation pile cases damaged during the 1995 Kobe earthquake. The Niigata earthquake case study utilize the displacement-based EQWEAP method, in which the free-field and the wave equation analysis are both performed to calculate the dynamic responses of piles under liquefaction. In The Kobe earthquake case studies, the force-based EQWEAP method is utilized to assume lateral flow induced forces on the piles. Dynamic earth pressures caused by lateral spreading of the liquefied layers are first generated and used to model forces exerted on the piles where the deformations of piles occur. These results show the pile failure pattern validate the applied methodology.

3.1 Case study: Pile damages due to soil liquefaction

The Niigata Family Court House was a four-story building located on the left bank of the Shinano River. The building was supported on a concrete pile foundation (Figure 8) each pile of the foundation having a a diameter of 35 cm and length of 6 to 9m. During the earthquake, the pile foundations were damaged by liquefaction-induced ground displacement. Excavation surveys showed that two piles (No.1 pile and No.2 pile) had severe cracks (Figure 9). They were conjecturally crushed by excessive bending moments at

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the interface between liquefied and non-liquefied layers as shown in Figure 9. According to aerial photographs of the area, the permanent ground displacement in the vicinity of building moved approximately 1.1m and the maximum displacement of No.1 pile and No.2 pile were respectively 50 cm and 70cm. For simplification, the entire soil system could be assumed as an upper layer and a lower layer. The upper layer from the ground surface to the depth of 8m is classified as medium-dense sand. The lower layer from the depth of 8 to 11m is classified as dense sand. The time history of earthquake record adopted the NS-component of the 1964 Niigata Earthquake as illustrated in Figure 10.

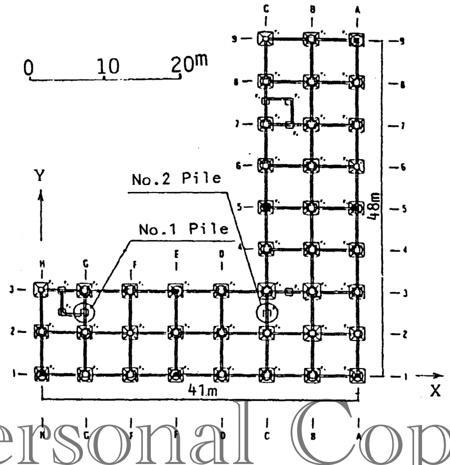

Fig. 8. Footing and foundation beams of Niigata Family Court House (from Hamada, 1992)

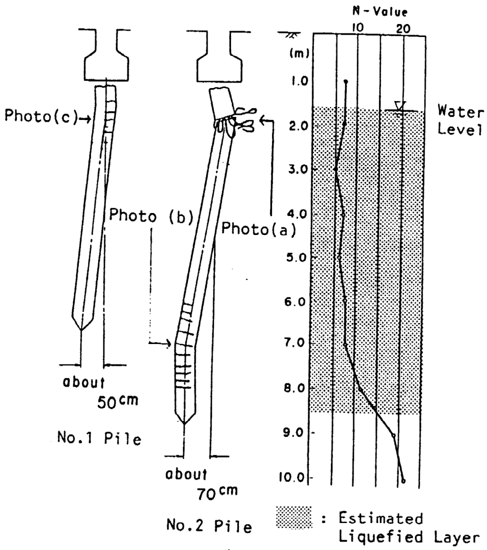

Fig. 9. Damage to piles and SPT-N values in sit u (from Hamada, 1992)

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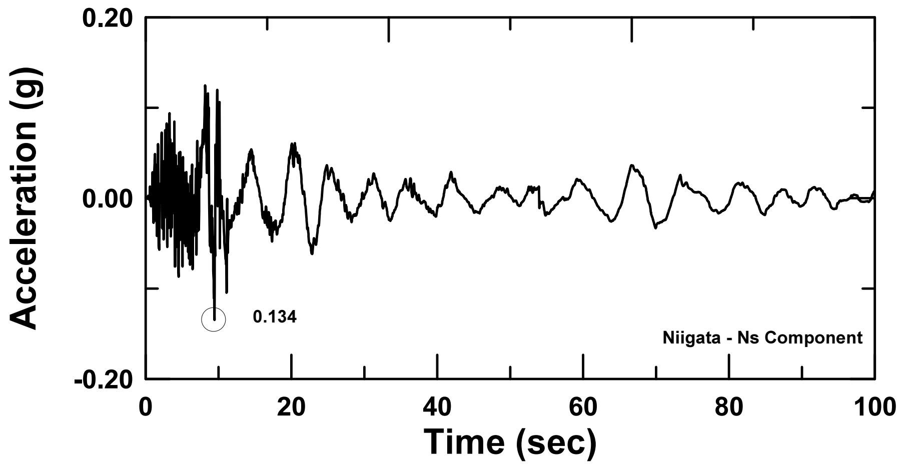

Fig. 10. Time history of Niigata Earthquake (NS Component)

The initial shear modulus of the soils at the any depth can be calculated by Eq. (17). The distribution of shear modulus is similar to the hyperbolic form observed in gibson soils and increases with the depth. The determination of pore water ratio pressure (r_u) and reduction factors (D_E) versus depth can be estimated by the liquefaction potential method suggested by Tokimatsu and Yoshimi (1983) with Eq. (18) for various levels of liquefaction. Moreover, one can conduct EQWEAP analysis to obtain the liquefied free-field response considering the effect of pore water. The excess pore pressure ratios at different depths are shown in Figure 11. It was found that the soil layer reached a liquefied state gradually after 2.8 seconds.

Figure 12 shows the time histories of ground motions. The maximum displacement of the ground which takes place at the surface is 47.3 cm at about 10 seconds. The liquefied layer ($r_u = 100\%$) ranging between the depths of 2 m to 8m displaces by 30 cm to 45 cm (see figure 12). The displacements reduce to about 3 cm below the liquefied layer for $r_u = 14 \sim 45\%$ as shown in Figure 12. Figure 13 indicate the maximum displacements of piles at various depths from wave equation analysis. Based on the results form Figure 13, the peak value occured at the pile head and the relative displacements between the pile head and pile tip are 50 cm and 69 cm. The maximum bending moments of piles are shown in Figure 14 and those peak values would also occur approximately at the interface between liquefied and non-liquefied layers. Comparing the numerical results by Meryersohn (1994), the computed values are nearly consistent with the ones reported. In the meantime, the peak shear forces of piles also occur at this zone. Therefore, the excessive bending moment and shear zone of the pile is again revealed in this study using the suggested procedures.

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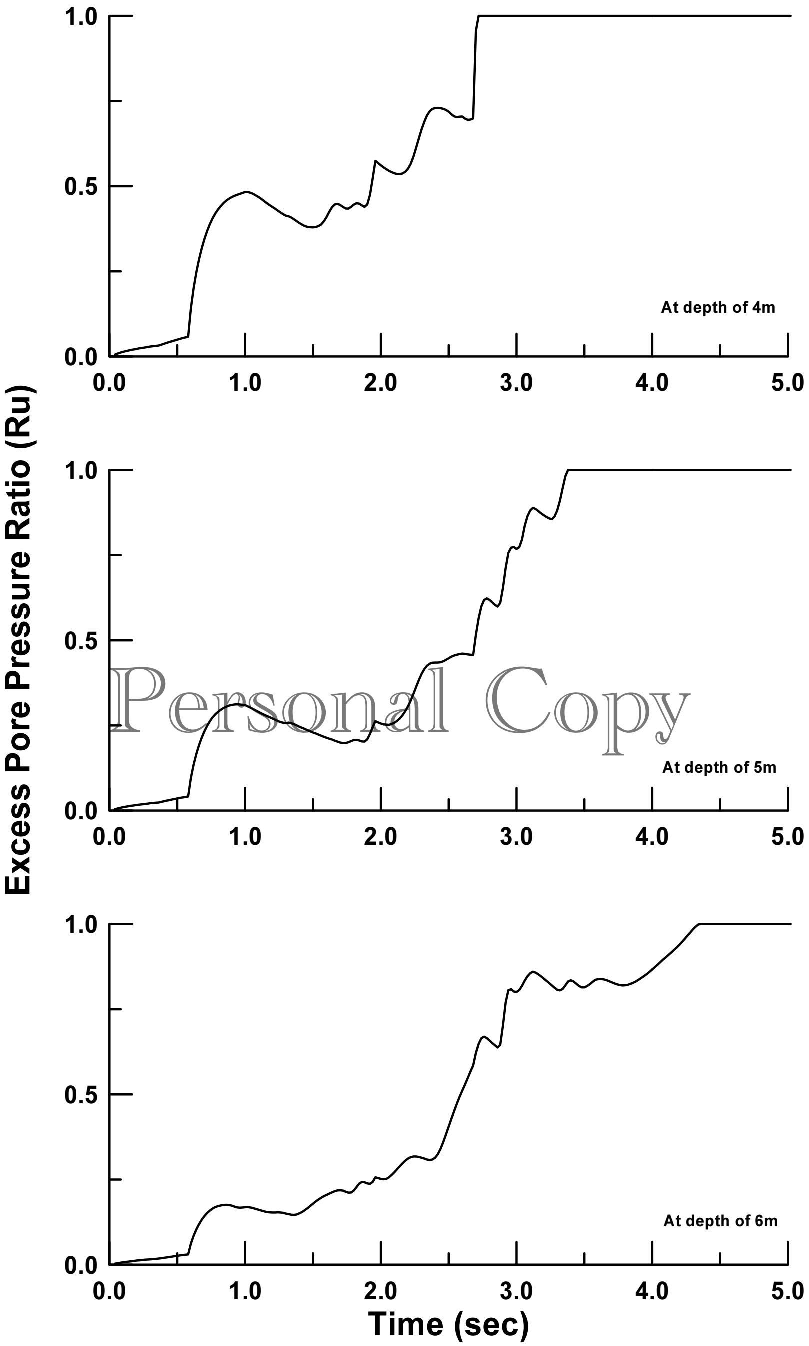

Fig. 11. The time history of excess pore pressure ratios at different depths

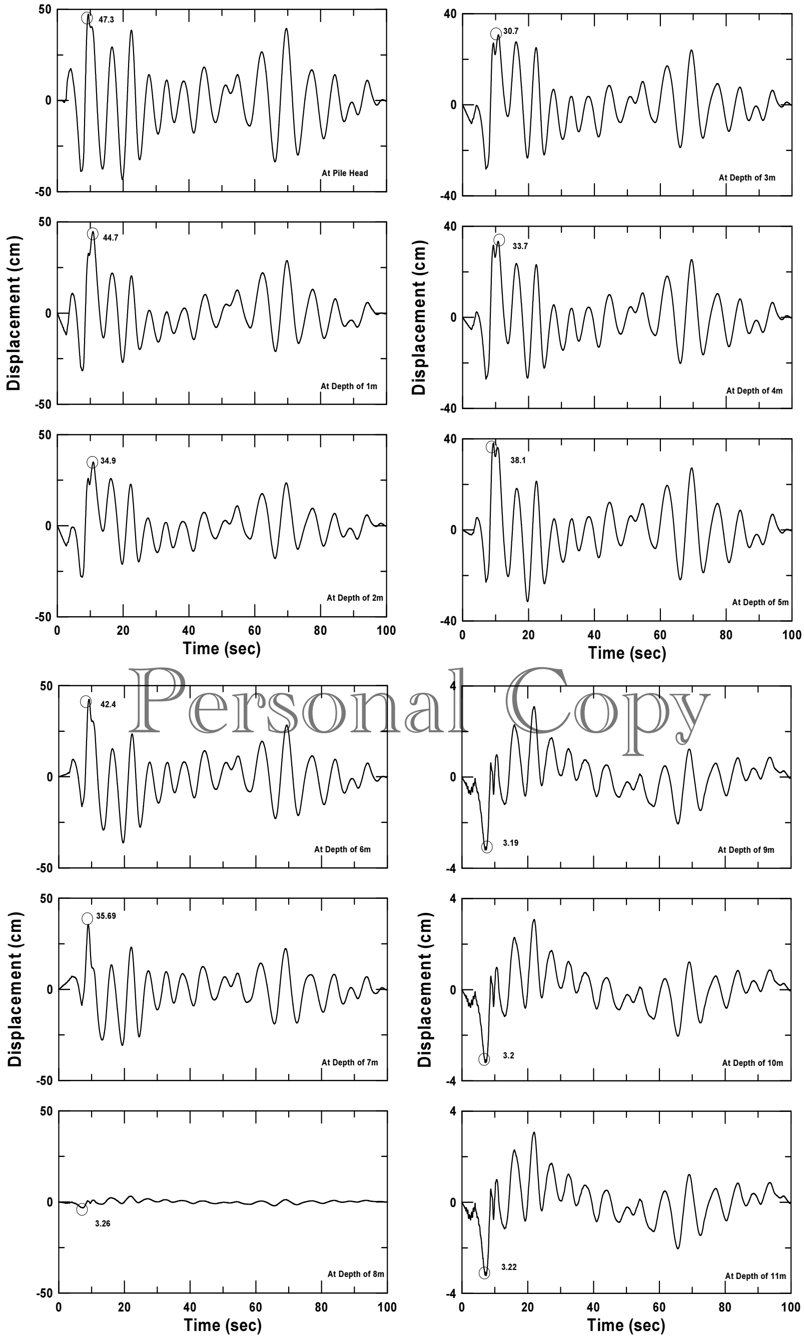

Fig. 12. Time histories of ground motions at different depths

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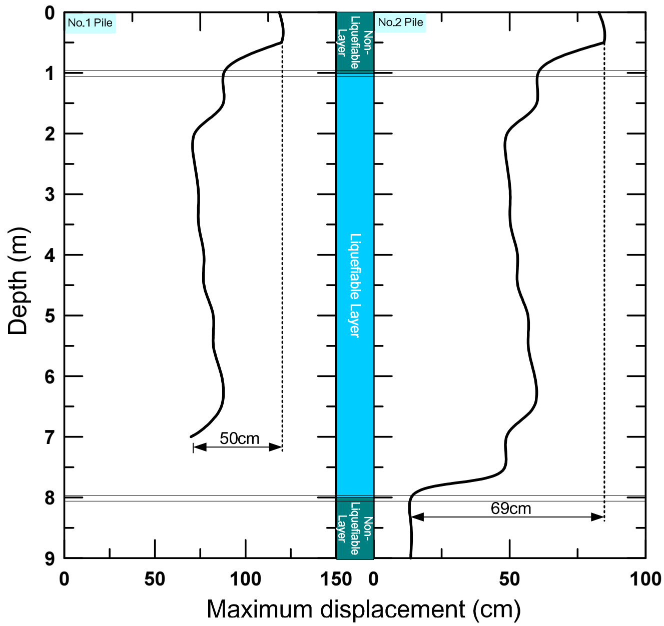

Fig. 13. Maximum pile displacements for No.1 Pile and No.2 Pile

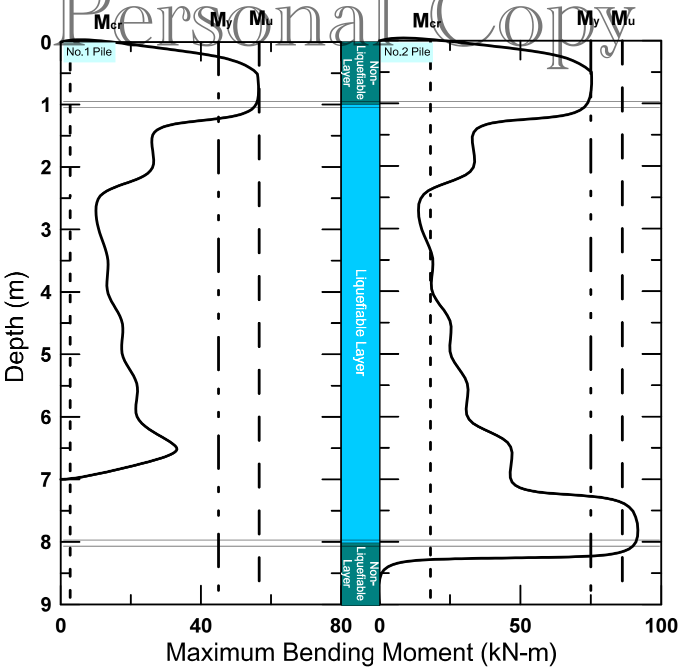

Fig. 14. Maximum pile bending moments for No.1 Pile and No.2 Pile

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3.2 Case study: Pile damages due to lateral spread

Mikagehama is a man-made island in the port area of Kobe home to a number of liquefied propane gas (LPG) and oil tanks. During the 1995 Kobe Earthquake, the soils underlying the foundatinos of tanks liquefied. A quay wall moved seaward and lateral spreading of the backfill soils damage the piles supporting the tanks. Oil-storage tank TA72 is chosen to be a target, which is located in the west part of the island about 20m from the waterfront. Figure 15 illustrates the cross sectional view of tank and underlying pile foundations. The tank has a diameter of 14.95 m and its storage capacity is about 2450 kl. It is supported on 69 precast concrete piles each with the length of 23 to 24 m and diameter of 45 cm. The water table is estimated at the depth of 2 to 3 m. Sand compaction piles were conducted to increase the SPT-N values of the Masado layer around the outside of Tank TA72.

According the relation between the bending moment (M) and curvature (φ) where D_0 is the diameter of pile and N is axial load on pile head, one can know that the cracking bending moment (M_{cr}), the yield bending moment (M_y) and the ultimate bending moment (M_u) are 105 kN-m, 200 kN-m, and 234 kN-m respectively. The ultimate shear strength is 232 kN with regards to ACI (1998). Ishihara and Cubrinovski (2004) have utilized bore-hole cameras and inclinometers to inspect the damages of the piles. Their results for pile No. 2 are shown in Figure 15. The main failure field was located at depth of 8 to 14 m where the piles were found to have developed many cracks. Moreover, pile No. 2 has scraped wounds due to large deformations where lateral spreading of liquefied soils develops along the weak interface. To quantifying damage in structures under earthquakes in terms of Park and Ang damage indices, which provides a measure on the structure damage level, and making a decision on necessary repair possible, the value approached to be 0.8 to represent collapse state (Park and Ang, 1985; Moustafa, 2011).

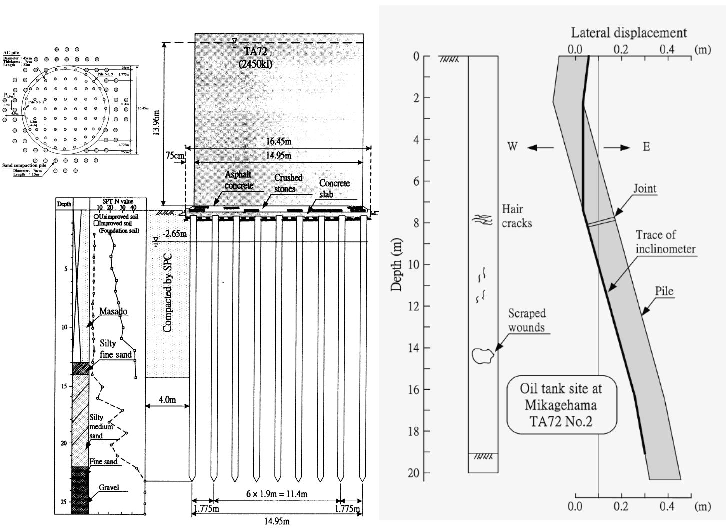

Fig. 15. Cross sectional view of Tank TA 72 and its foundation (from Ishihara and Cubrinovski, 2004)

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In this study, the length of pile is assumed to be 24 m with a diameter of 45 cm. Seismic record of the NS-component of 1995 Kobe Earthquake is adopted. According to the field data, distributions of pore water ratio pressure (r_u) versus the depths can be estimated by evaluating the liquefaction potential of that site. With all the required data and incorporating with the modified M-O model (Zhang et al, 1998), the dynamic coefficients of earth pressure are computed as shown in Figure 16. Also, the unit weight of the soil is reduced by r_u (refer to Eq. 22). When obtaining those dynamic earth forces to insert and execute the wave equation analysis, the time histories of displacements along the pile can be illustrated as shown in Figure 17. The displacement of the pile head oscillates significantly with time, but the peak value is smallest. As the depth increases, the peak displacement of pile becomes larger. Those peak displacements along the pile are shown in Figure 18(a). The maximum value among them occurs at the pile tip about 52.7 cm and the maximum relative displacement between the pile top and the pile bottom is estimated about 44.7 cm. The deformed shape of the pile is similar to pile No. 2. It can be found that the maximum bending moments which exceed the ultimate bending moment at depths of 2 to 23 m and that this zone is the mose dangerous zone.. With regards to the shear failure, the weak interface exists at a depth of 11 m, in which the maximum shear force is close to the ultimate (Figure 18b~18c). The above observations are agreeable to field investigations reported by Ishihara and Cubrinovski (2004).

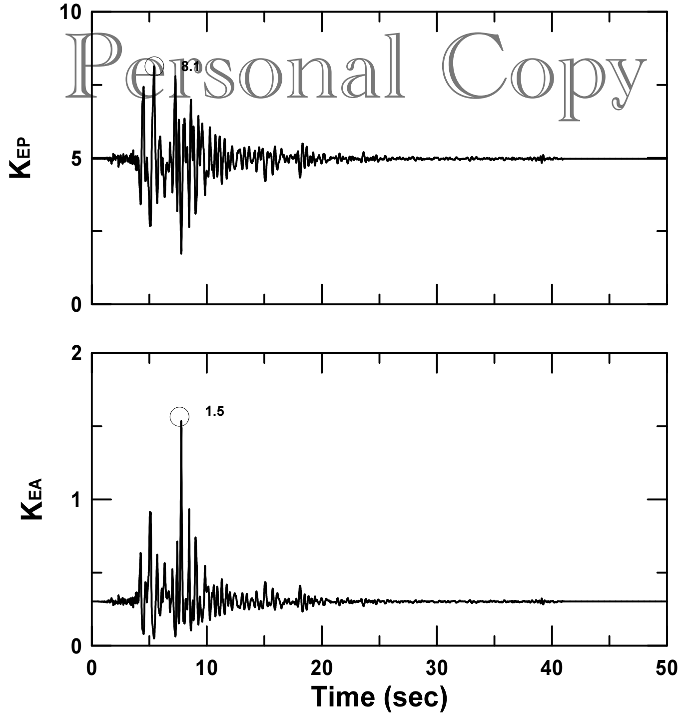
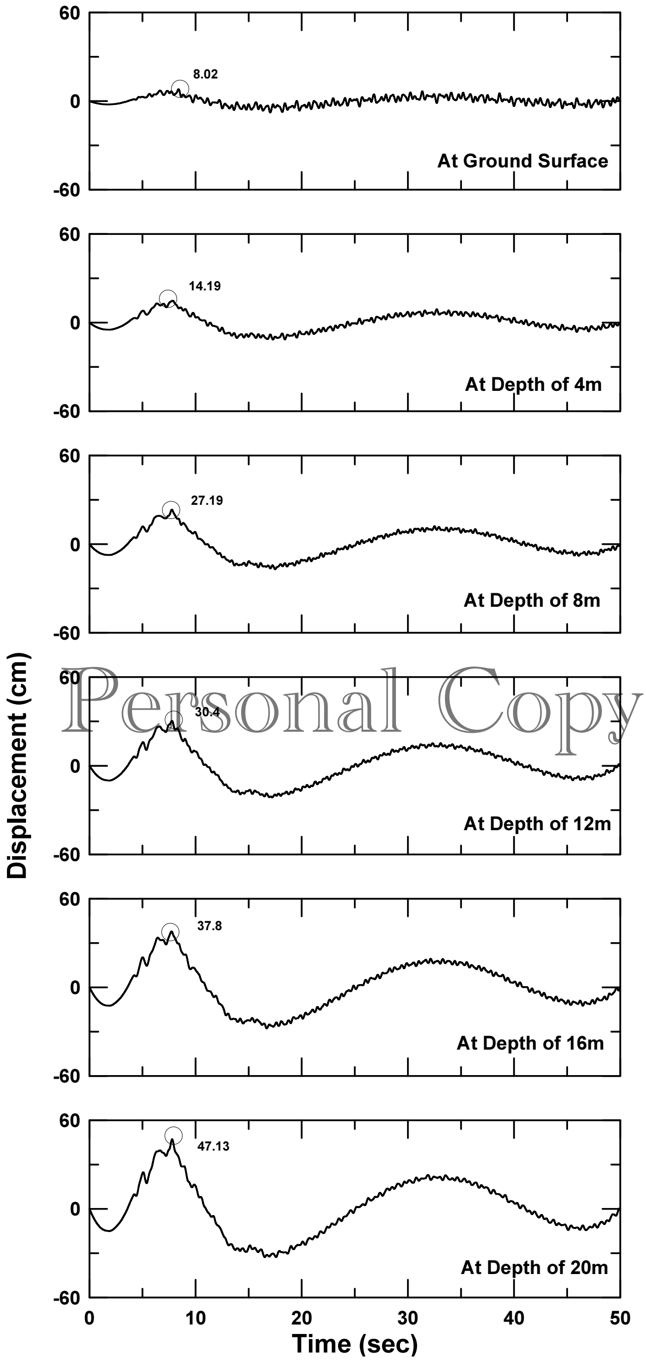


Fig. 16. Dynamic coefficients of earth pressure



1

2 Fig. 17. Time histories of lateral displacement along the Tank TA72 No.2 pile

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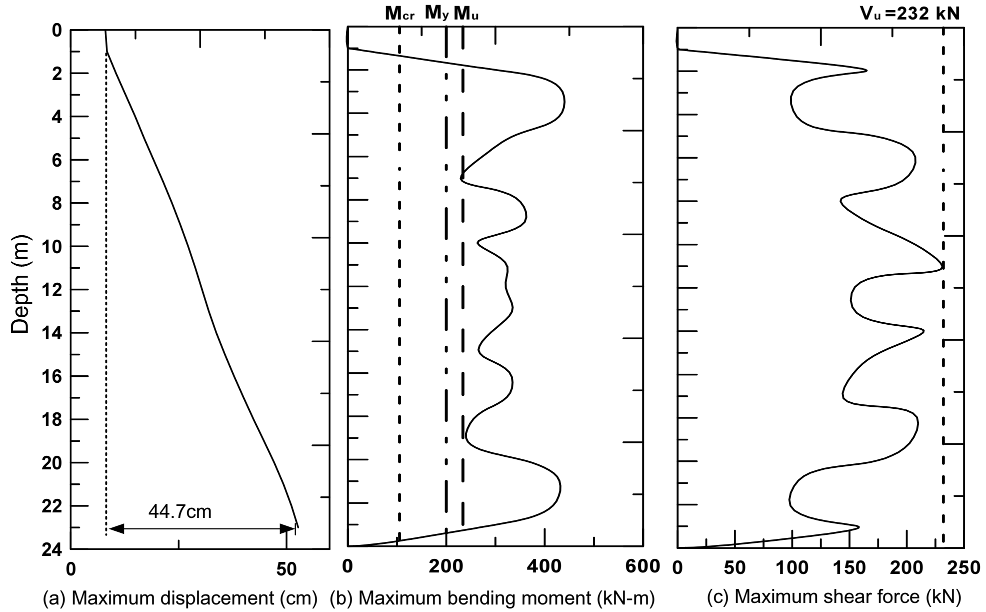

Fig. 18. Maximum structural response along the Tank TA72 No.2 pile
(a) Maximum displacement; (b) Maximum bending moment; (c) Maximum shear force

4. Conclusions

EQWEAP is a simplified but effective procedur to analyze the dynamic pile-soil interaction under the earthquake. In the analysis, the pile deformations are obtained solving the discrete wave equations of the pile, where the seismic ground motions are pre-calculated from one-dimensional lumped mass model assuming a free-field condition or dynamic earth pressure are directly exerted onto the pile. This chapter presented both displacement- and forced-based form of the EQWEAP analysis method along with two comparative case studies: Using wave equation analysis and the EQWEAP method, pile response to liquefaction has been computed and compared to the case histories of the Niigata earthquake records. Case histories of the Kobe earthquake show that the lateral spreading can be a major cause to damage the piles. Specifically conclusions for the displacement and forced based EQWEAP methods can be summarized as follows:

1. Based on the suggested numerical procedure using EQWEAP (Chang and Lin, 2003; Lin et al., 2011), one can evaluate the motions of the soil stratum and the pile foundations at various depths to estimate the occurrence of pile damages and patterns of failure. This procedure provides a simplified but rational dynamic analysis to the pile foundation design work.
2. The use of the empirical excess pore pressure model for liquefaction can be applicable to soils underneath the liquefiable layers using a minimum pore pressure ratio. The pore pressure ratio should be calculated using the empirical formula suggested by Tokimatsu and Yoshimi (1983) providing that the factors of safety against liquefaction are known.

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3. Not only the interfaces between the liquefied and non-liquefied layers can exert excessive bending moments and shear stress, but also the layer contrast of the soils can yield similar effects. Engineers need to be more careful in designing pile shafts that are susceptible to fail due the liquefaction resulting from earthquakes and the layer contrast.
4. The wave equation analysis can be used to model the pile responses under lateral spread due to earthquake. The modified M-O model (Zhang et al., 1998) incorporating reduction methods for soil parameters were successfully used to represent the dynamic earth pressures of the lateral spread. The numerically determined pile deformations were similar to deformations discovered at piles actually affected by lateral spread. In advance, if nonlinear behaviour of pile such as the moment-curvature relationship and complexity of pile geometries can also be considered simultaneously in this method, the results would be enhanced to capture detailed mechanism and definite performance of piles foundations.

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