SEISMIC SOIL–STRUCTURE INTERACTION : NEW EVIDENCE AND EMERGING ISSUES

by
George Gazetas [1] & George Mylonakis [2]

ABSTRACT

Key phenomena associated with the seismic interplay between soil, foundation, and structure are introduced and the available methods briefly explained. Emphasis is given to deep foundations (piles and caissons), but embedded foundations are also studied. Five special topics are selected for discussion, satisfying one or both of the following criteria : new field evidence on their significance has accumulated, or renewed practical interest has recently generated.

Specifically : (a) The role of dynamic soil-structure interaction (SSI) on the response of pile-supported structures is re-explored, and the failure of Hanshin Expressway Route 3 in Kobe (1995) is analyzed to show the detrimental role of SSI on the fatal behavior of the bridge. This and additional similar evidence contradict the currently prevailing view in structural engineering for a "beneficial" role of SSI. (b) For the kinematic distress of piles (under stable soil conditions) recent solutions are outlined, and then validated through two case histories involving actual measurements of bending strains. Among the unresolved issues : modeling large deformations of the surrounding soil due to lateral spreading or other instabilities. Caisson-type foundations have resisted successfully, and in fact restricted, such large deformations. (c) Pile-to-pile interaction under dynamic loading has a substantial effect on the stiffness of a large group of piles. The response of such groups can practically be obtained with the superposition method using simplified analytically-expressed interaction factors between two piles. The sensitivity of such factors to soil layering and soil nonlinearity complicates the task, as illustrated with examples from the recent literature. (d) Inclined piles had suffered disproportionately in earthquakes of the past and their seismic role has been widely viewed as detrimental; their use has been discouraged in seismic regulations. New theoretical work and some recent field evidence strongly suggest that the opposite may be the case. (e) The issue of structural yielding and inelastic bending of piles is being explored. Such action may be practically unavoidable (especially under bridge piers) and, in

---

1 National Techical University, Athens, Greece
2 The City College, CUNY, New York

fact, it may not be detrimental; but its consequences on the overall ductility demand imposed to the pier cannot presently be readily assessed.

The aim of the paper is *not* to provide complete solutions to these problems but only to highlight some of the issues, outline new results or field evidence, and raise questions of practical significance.

INTRODUCTION : THE MAIN QUESTIONS

This introductory chapter aims at providing background information on the effects of earthquake-induced vibrations on soil-foundation-structure systems, in general, and on piles and pile-supported structures, in particular. It illustrates the fundamental features of the problem and outlines some of the pertinent methods of analysis.

Seismic soil-structure interaction analysis must address the following questions:

(a) how does the interplay between soil, foundation, and structure influence the dynamic response of the *structure* and of any equipment in it?

(b) can the foundation transmit safely into the ground the inertial loads from the super-structure, i.e., without undergoing excessive settlement during and/or subsequently to earthquake shaking ?

(c) will the piles be subjected to unacceptably large bending or shear deformation due to the action of structural inertia forces applied at the pile head, or due to wave-induced ground displacements along their embedded length ?

(d) will the axial capacity (tension and compression) of the piles be compromised as a result of earthquake induced tractions and possible soil degradation along the shaft of the pile or at its tip?

FUNDAMENTAL PHENOMENA IN THE SEISMIC RESPONSE OF EMBEDDED-FOUNDATION AND PILE–SUPPORTED STRUCTURES

Figure 1 is a sketch illustrating the key features and introducing the basic terminology of the problem under study. A structure, modelled as a lumped-mass-and-stiffness cantilever, is supported on a rigid embedded raft, the load of which can be transmitted to the soil either directly from its base and side walls or through a number of piles penetrating several soil layers and bearing on stiff soil or rock.

In order to understand the dynamic response of this composite system, let us first consider the soil deposit alone. The soil layers overlying the rock are subjected to seismic excitation consisting of numerous incident waves: shear (S) waves, dilatational (P) waves, and surface (R or L) waves. Whereas seismological considerations dictate the wave composition, amplitude, and frequency content of the incoming seismic motion in the underlying rock, the geometric and stiffness characteristics of the layers of the soil deposit affect and modify (often profoundly) the ground motions experienced at the site. Indeed, multiple reflections and refractions at the soil layer interfaces and the free ground surface as well as *resonance* phenomena that may result from these, modify the *wave field* in these layers with respect to the wave field of the incident seismic waves (which prevails in nearby rock sites). In short, this is called *free-field* motion. This motion constitutes the micro-seismic environment that our *facility* must face. Defining this environment, in terms of free-field acceleration time-histories or design response spectra, is the necessary first step of the seismic analysis.

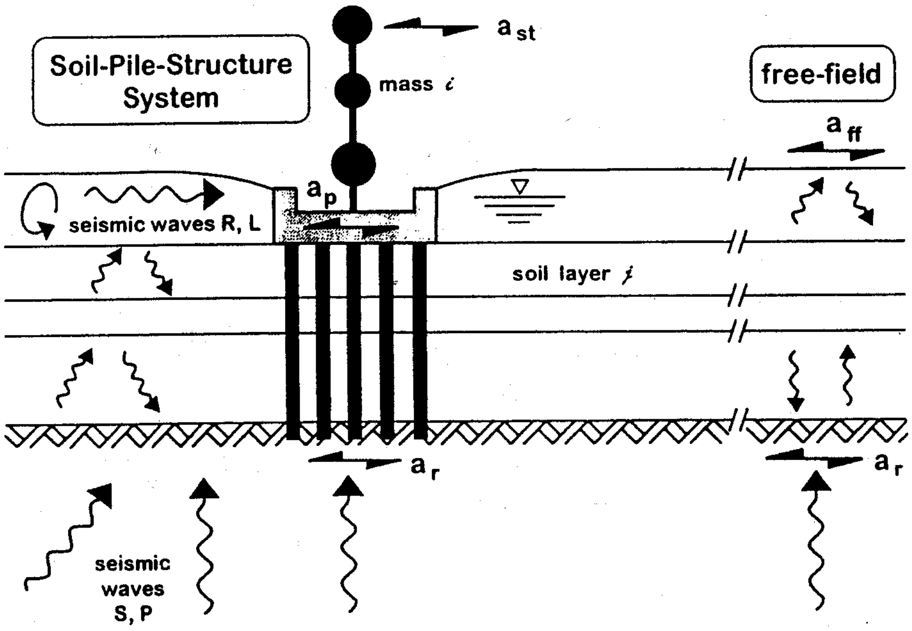


Figure 1 . Sketch of the soil-pile-structure interaction problem

Let us now consider the presence of a structure and its piled foundation in the soil. The seismically deforming soil will "carry" with it initially the piles and the embedded foundation, and subsequently the supported super-structure. Two important phenomena occur as a result of such imposed motion on the foundation and the structure.

First, the piles that are forced to follow the more-or-less wavy soil motion, tend to provide resistance, due to their flexural rigidity. The incident seismic waves are thus *reflected* and *scattered* while the piles are being *stressed* (developing curvatures and bending moments) and experience an oscillatory motion which may differ substantially from the free-field motion.

Then, this pile-foundation motion induces oscillations in the super-structure. The inertia forces that inevitably generate in the superstructure masses produce dynamic forces and overturning moments at its base which are subsequently transmitted onto the piles and eventually in the surrounding soil. Thus, the piles experience "new" dynamic movements, deflections and curvatures, while the footing and the super-structure undergo further dynamic displacements and accelerations.

The above two phenomena occur simultaneously with only a small time lag. However, it is convenient (conceptually and computationally) to separate them into two consecutive phenomena, as described above. The two phenomena are referred to *as* ***kinematic*** and ***inertial response.*** It will be shown that the response of the system can be obtained by proper superposition of the effects of *kinematic* and *inertial* response.

A complete understanding of the seismic behavior of a soil-pile-structure system can only be achieved when the following three interrelated analysis tasks are accomplished:

(a) Soil Response Analysis – to obtain a realistic estimate of the seismic environment to which the system will be subjected during the design earthquake. This task is often simplified by assuming that the incident seismic waves are exclusively vertical S waves. However, a distant source may also send inclined S waves as well as surface waves. Such waves must be taken into account in the analysis.
Successful completion of this task depends not only on the numerical tools of the analysis but also on the quality of both geotechnical and seismological data that will be input into the analyses. In addition to defining the seismic excitation, the analyses of this task will provide information for assessing the possibility for loss of strength due to pore-water pressure build-up in the saturated cohesionless layers.

(b) Kinematic Pile Response Analysis – to obtain the response of the piled foundation in the absence of inertia forces from the super-structure. What is mainly sought in this step are:

- the oscillatory motion of the raft (at the top of the piles) $a_p(t)$ which is in essence the *effective* "foundation input motion", i.e. the motion to which the (flexibly-supported) super-structure is subjected to,

- the distress of each pile, i.e. the bending moments and shear forces, which must be later superimposed on the moments and forces arising from the inertial loads developing in the superstructure.

(c) Inertial Soil-Structure Interaction Analysis, to obtain the dynamic response of the superstructure and the loads that this response imposes on the foundation. More specifically, this task determines internal dynamic forces, differential displacements,

and "floor response spectra" in the structure, and the additional pile motion and distress. The latter is likely to include a substantial vertical loading component, producing alternating tension and compression of the piles, even if the seismic excitation is exclusively horizontal.

The above decomposition of the problem into three tasks was done mainly for elucidating the phenomena that must be captured in the analysis. It does not necessarily imply that the steps must be performed separately, although this is most often the case in practice. *Complete* interaction analysis (frequently named *direct* analysis or *one-step* analysis) is, at least in principle, also possible. However, with foundations consisting of a group of piles, few computer codes would be available for such a *direct* analysis, especially in the reach of nonlinear soil behavior under strong seismic excitation. Moreover, the decomposition into (interrelated) subtasks has an additional major advantage over the one-step methods: accumulated empirical knowledge, (over many years), on particular aspects of the problem can be readily utilized. For instance, well-established methods of determining non-linear lateral pile deflection can be utilized in determining pile-head stiffness and damping (in the task of inertial interaction analysis). This would not only facilitate the analysis but would also increase its reliability. Engineering judgment could be brought into a greater play with a multistep analysis.

OVERVIEW OF METHODS OF SOIL–FOUNDATION–STRUCTURE INTERACTION ANALYSIS

The Superposition Theorem

The fundamental task of defining the "seismic environment", i.e., of the seismic excitation (in terms of the amplitude, frequency content, and spatial variation, of ground free-field motion), is not further discussed in this paper. Instead, we concentrate here on *methods of analyzing* the response of the system sketched in **Figure 2a**.

In addition to being useful for illustrating the main phenomena of the problem, the aforementioned decomposition into kinematic and inertial response is of great mathematical convenience. Its validity for linear material behavior (of soil, pile, and structure) stems from the so-called *superposition theorem* (Whitman 1973, Kausel & Roesset 1974, ASCE 1979). This theorem states that the seismic response of the system of **Figure 2a** to base rock acceleration a_r, can be computed either in a single step, by solving the following matrix differential equation :

$$[M]\ \{\ddot{u}\} + [K]\{u\} = -[M]\{I\}\, a_r. \tag{1}$$

where:

$\{u\}$ = relative displacement vector of points in the soil, the piles, or the structure with respect to the top of the "rock"

{I} = unit vector
[M] = mass matrix of the system
[K] = stiffness matrix of the system (in the general case this is a complex matrix, with the imaginary terms representing damping),

or in two steps, after expressing the relative displacement vector *(u)* as the sum of the following two components :

$$\{u\} = \{u_{kin}\} + \{u_{iner}\} \tag{2}$$

where: $\{u_{kin}\}$ = *kinematic* relative displacement and $\{u_{iner}\}$ = *inertial* relative displacement, by solving the following two coupled differential equations :

$$[M_{so}]\{\ddot{u}_{kin}\} + [K]\{u_{kin}\} = -[M_{so}]\{I\}\, a_r \tag{3a}$$

$$[M_{st}]\{\ddot{u}_{iner}\} + [K]\{u_{iner}\} = -[M_{st}]\,(\{\ddot{u}_{kin}\} + \{I\}\, a_r) \tag{3b}$$

where:

$[M_{so}]$ = Mass matrix assuming that only the soil and piles have mass (i.e. the mass of the superstructure is made zero)

$[M_{st}]$ = Mass matrix assuming that there is mass only in the superstructure (i.e. the mass of piles and soil are made zero).

Note that by definition: $[M] = [M_{so}] + [M_{st}]$.

The superposition of Eqns 3a and 3b results into Eqn 1. Eqns 1, 3a and 3b are illustrated graphically in **Figures 2a, 2b and 2c**, respectively. Eqns 3a and 3b, in particular, are the mathematical expression of the kinematic and inertial interaction effects, respectively. Hence, one proceeds by subdividing the analysis into:

(i) the kinematic interaction effect (sometimes also referred to as *"wave scattering"* effect), involving the response to base (rock) excitation of a hypothetical system which differs from the complete actual system of **Figure 2a** in that the mass of the superstructure is set equal to zero;

(ii) the inertial interaction effect, referring to the response of the complete soil-structure system to excitation by D'Alembert forces, $-M\, a_{kin} = -M(\ddot{u}_{kin} + a_r)$, associated with the acceleration $a_{kin} = \ddot{u}_{kin} + a_r$ of the superstructure due to the kinematic interaction.

Apparently, the *"superposition" theorem* is exact for linear soil, pile, and structure, if the analysis in both stages is performed rigorously. Nevertheless, as an engineering approximation, the superposition can be applied to moderately-nonlinear systems. This is because pile deformations due to lateral loading transmitted from the superstructure attenuate very rapidly with depth (they practically vanish below the

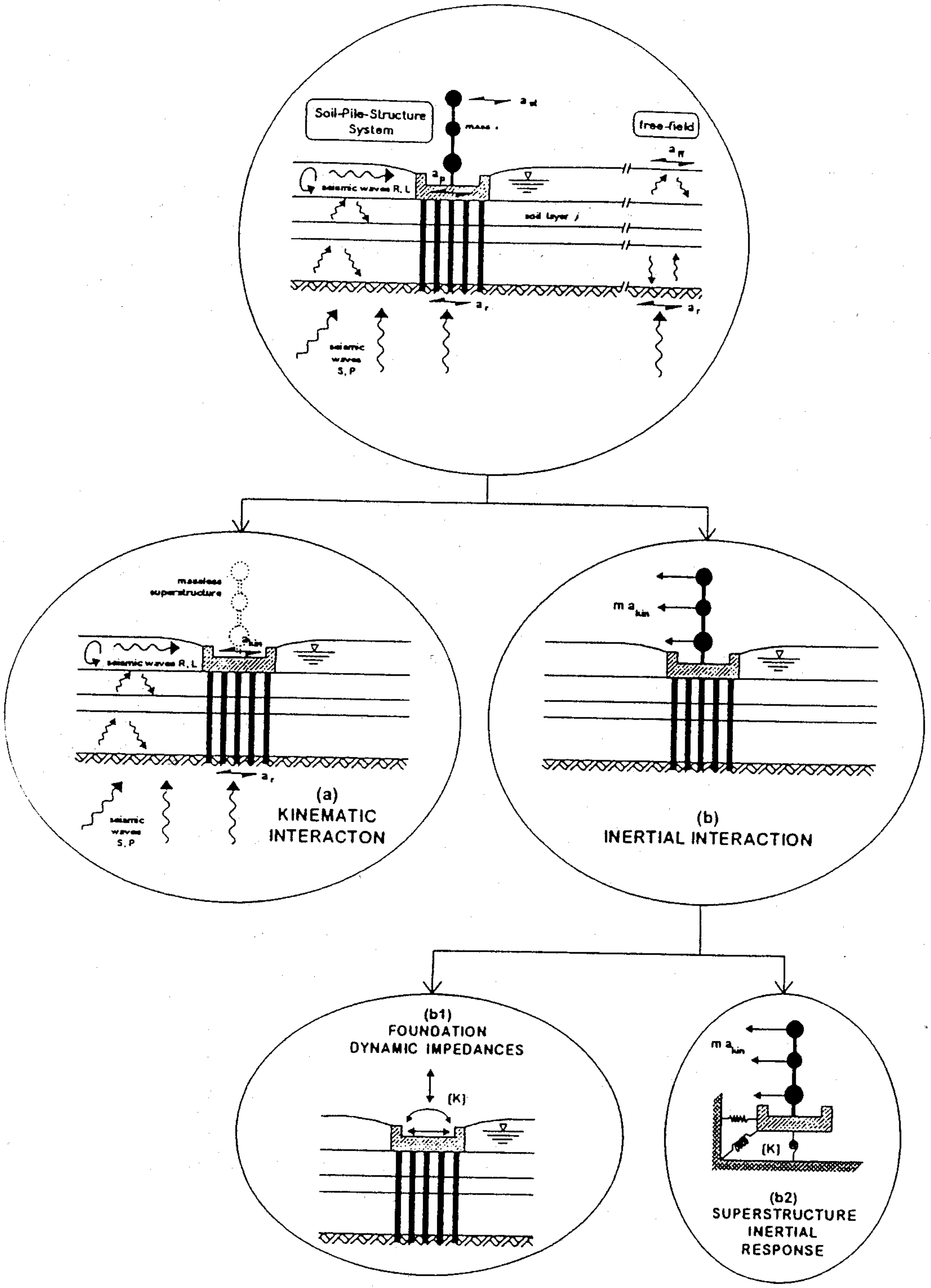


Figure 2 . Decomposition of seismic soil-pile-structure response into kinematic and inertial interaction. Analysis of the latter in two steps

"active" pile length, ℓ_a, which is typically of the order of 10 pile diameters below the ground surface). Therefore, shear strains induced in the soil due to inertial interaction may be significant only near the ground surface. By contrast, vertical S-waves induce displacements, curvatures and shear strains that are likely to be important only at relatively deep elevations. Thus, with soil strains controlled by inertial effects near the ground surface and by kinematic effects at greater depths, the superposition may be a reasonable approximation even when nonlinear soil behavior is expected.

For computational convenience, analysis of the inertial response of **Figure 2c** is further subdivided into two consecutive independent analysis steps, as follows:

(1) Computation of the dynamic impedances ("springs" and "dashpots") at the pile head or the pile-group cap, associated with the swaying (R_x and R_y), rocking (R_{ry} and R_{rx}) and cross-swaying-rocking ($R_{x,ry}$ and $R_{y,rx}$) motion of the foundation; and

(2) Analysis of the dynamic response of the superstructure supported on the "springs" and "dashpots" of step *(b1)*, subjected to the kinematic pile-head motion of step *"a"*. (The latter is also called *"Foundation Input Motion"*.)

For each of the above analysis steps, several alternative formulations have been developed and published in the literature, including finite-element, boundary-element, semi-analytical and analytical solutions, a variety of simplified methods, and semi-empirical methods. **Table 1** lists the most prominent developed methods, while the ensuing discussion highlights the key features of these methods and compares the results obtained from them. We begin with a general overview of the analysis methods for pile foundation under seismic loading.

GENERAL OVERVIEW OF METHODS OF ANALYSIS

Dynamic Finite Element Method

The finite element method is one type of numerical procedure from a class of finite methods; use of finite differences and the method of characteristics have also been proposed for analysis for soil-structure interaction. However, finite elements is most frequently used in practice at this time. The method is well suited for analyzing problems of complicated geometry (such as a pile in layered soil), not easily handled with analytical or semi-analytical formulations.

The use of finite elements in dynamic soil and foundation problems is different from other applications of finite elements in statics and dynamics in that soil strata of infinite extent in the horizontal (and sometimes in the vertical direction) must be

Table 1. Methods of Analysis of Problems Related to Seismic Soil Pile Foundation Structure interaction (Partial List)

1. ANALYSIS OF FREE-FIELD (SOIL) RESPONSE

One-dimensional elastic or inelastic wave propagation theories
Two- and three-dimensional elastic wave propagation theories
Empirical Knowledge

2. ANALYSIS OF KINEMATIC SEISMIC RESPONSE

(a) Single pile response
Beam-on-Dynamic Winkler-Foundation (BDWF) model
Extended-Tajimi formulation
Finite-element formulations
Semi-analytical and boundary-element formulations

(b) Pile group response
Simplified wave-transmission model
Extended-Tajimi formulation
Semi-analytical and boundary-element formulations

3. ANALYSIS OF PILE-HEAD IMPEDANCES

(a) Single pile
Simple closed-form solutions
Empirical nonlinear models ("p - y", "t - z")
BDWF model
Extended-Tajimi formulation
Novak's plane-strain formulation
Finite-element formulations
Semi-analytical and boundary-element formulations

(b) Pile group
Superposition method (using dynamic interaction factors)
Extended-Tajimi formulation
Finite-element formulation
Other simplified solutions
Semi-analytical and boundary-element formulations

4. ANALYSIS OF SUPERSTRUCTURE SEISMIC RESPONSE

Accounts for SSI through frequency-dependent foundation "springs" and "dashpots" from step 3 and uses the seismic response from step 2 as "foundation input motion" represented by a model of a finite size. Such a finite model creates a fictitious 'box' effect, trapping the energy of the system and distorting its dynamic characteristics. To avoid this problem, wave absorbing lateral boundaries are introduced to account for the radiation of wave-carried energy into the outer region that is not included in the model. Two main types of such boundaries are available:

- the *"viscous" boundary* proposed originally by Lysmer et al.,(1981) for shallow foundations. This boundary absorbs most (but not all) of the incident outward going waves, and hence it must be placed at some distance from the foundation. Its first application in calculating the dynamic response of a pile was published by Kuhlemeyer, (1979).

- the so-called *"consistent" boundary* developed by Waas and Kausel for surface and embedded mat foundations, is very effective in accurately reproducing the physical behavior of the system, and it also results in considerable economy by being placed directly at the edge of the foundation. This "consistent" boundary provides a dynamic stiffness matrix for the medium surrounding the plane or cylindrical vertical cavity which is assumed to occupy the central region under the strip or circular foundation. This matrix corresponds exactly to the boundary stiffness matrix that would be obtained from a continuum type formulation. Blaney at al., (1979) have applied this boundary to the dynamic analysis of a pile subjected either to lateral loading at its head or to base-rock-type of steady-state excitation. Thus, they obtained information pertaining to both inertial and kinematic pile-soil interaction.

Unfortunately, pile groups cannot be handled easily with finite elements. Consistent boundaries have been developed only for axisymmetric (cylindrical) and plane-strain geometries. No such boundary is available for truly three-dimensional [3D] geometries in cartesian coordinates (as are the problems of pile groups). Thus, to solve 3D problems a finite-element model must resort to "viscous" boundaries placed far away from the loaded area. In this way the fictitiously reflected waves are dissipated through hysteresis and friction (material damping) in the soil before they return to the foundation region. However, the effort required for such 3D analyses is very substantial. Hence, dynamic finite elements are used for (linear and nonlinear) analysis of single piles and are used rarely to estimate the seismic response of pile groups.

Boundary Element Type Methods

The methods of this class are in essence semi-analytical in that they utilize closed-form solutions to the pertinent wave equations for the soil domain and discretize only the boundaries and interfaces of the system. These closed-form solutions (called *"fundamental"* solutions or "Green's functions", depending on the particular method) have the ability to reproduce exactly the radiation of wave energy to infinity, without requiring the special lateral boundaries of the finite-element methods. On the other hand, they all involve discretization of the pile-shaft--soil interace into a number of cylindrical segments, while the base of the pile is considered as a circular disk.

Formulations of this type have been developed for both the single pile and the pile group under kinematic and inertial (static and dynamic) loading, Poulos 1968, 1971, Butterfield & Banerjee 1971, Kausel & Peek 1982, Kaynia 1982, Sen et al., 1985, Ahmad & Mamoon 1991. Evidently, this class of methods is the most versatile in treating a variety of incident wave fields (such as vertical and inclined body waves, Rayleigh and "travelling" waves, and so on). Usually, however, they cannot accommodate nonlinearities in the soil or the soil-pile interface; as such nonlinearities may be of dominant importance when soft soils are present and / or when high-amplitude oscillations are anticipated, considerable engineering judgment must be exercised in selecting the pertinent soil parameters and in performing such analyses. In current state of practice, it is preferable to use such sophisticated methods in conjunction with simpler methods that can better model the nonlinear response.

Winkler Models

Treating the pile as a beam or column supported on a "Winkler foundation", i.e. by a series of independent horizontal (lateral) or vertical (axial) springs distributed along its length, has been an old popular model for estimating pile-head deflection and settlement under *static* top loading (Matlock & Reese 1960). A major improvement of this model has been to make the Winkler springs nonlinear and then solve for the pile displacements using a finite difference or finite element discretization.

Although a variety of methods have been proposed over the years for "constructing" nonlinear springs, the empirical static approach of Matlock, (1970) and Reese, (1975), represents the current state of practice. Based on field experiments, their approach models the soil response at a particular in depth terms of a *p-y* curve for lateral loading or a *t-z* curve for axial loading; p and y denote the resultant lateral soil reaction per unit length of pile and the corresponding lateral pile deflection, at the particular depth; t and z denote the axial reaction per unit length and the vertical pile displacement.

The Winkler model has proved extremely versatile and has been applied to dynamic problems with success (Penzien 1970). To this end, the role of soil-pile interaction is played not by "simple" springs but by a set of continuously-distributed *springs and dashpots,* the parameters (k and c) of which are functions of the loading frequency. Several different methods are available for estimating $k = k(\omega)$ and $c = c(\omega)$, such as Novak's "thin layer" elastodynamic solution, (Novak et al 1978) in which a rigid cylindrical rod of unit thickness (representing a pile "slice") is surrounded by soil (which extends radially to infinity) and is subjected to horizontal or vertical oscillations. Hybrid methods, in which the "dashpot" is obtained from solving the aforementioned elastodynamic problem whereas the "spring" is obtained on the basis of finite-element analyses, have avoided some of the limitations of Novak's method (Gazetas et al., 1992, 1993; Makris & Gazetas 1992; Kavvadas & Gazetas 1993).

The dynamic Winkler model has been successfully extended to obtain not only the inertial but also the kinematic response of a pile. The developed formulation ("Beam-on-Dynamic-Winkler-Foundation" [BDWF] model in Table 1) uses the distributed springs and dashpots to connect the pile to the free-field soil; the wave-induced motion of the latter (computed in the soil response analysis [see Table 1] serves as the *support excitation* of the pile-soil system. **Figure 3** sketches this BDWF model.

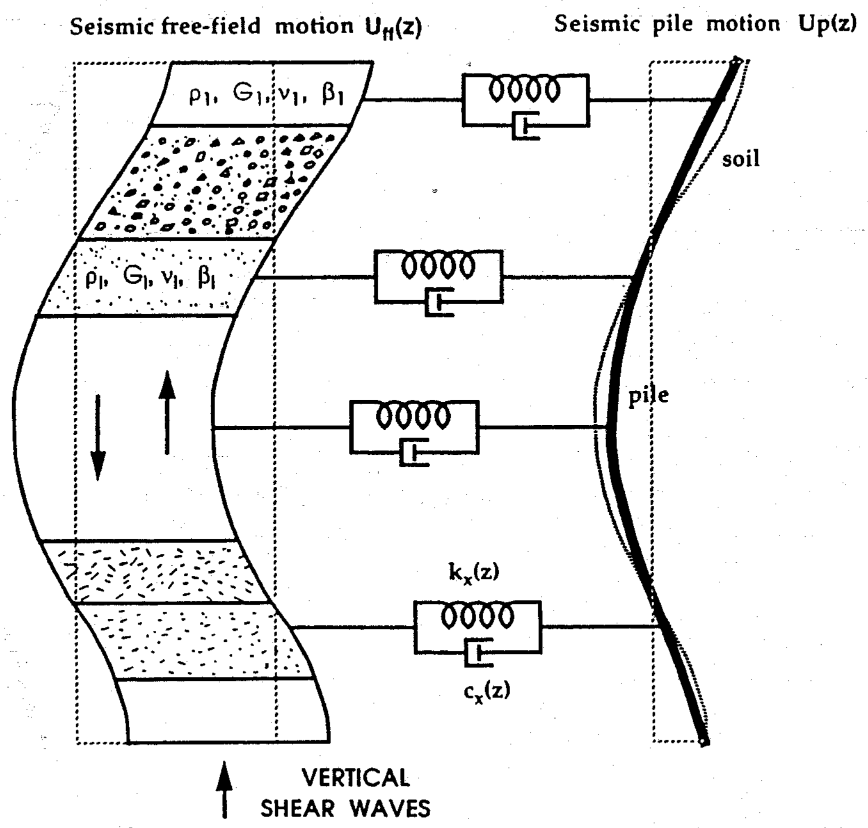


Figure 3 . Beam-on-Dynamic-Winkler-Foundation (BDWF) model for soil-pile interaction analysis in multi-layered soil

Finally, Winkler type models have been developed for computing the pile-to-pile interaction that takes place between the piles of a group (Nogami 1985, Makris & Gazetas 1992, Novak 1994, Mylonakis 1995, Ef-Naggar & Novak 1996, Mylonakis

& Gazetas 1998)). A host of other simplified methods have been developed for analyzing specific aspects of the seismic response of piles. **Table 1** lists only some of these methods; these will not be further discussed in this paper.

SOME NEW EVIDENCE AND EMERGING ISSUES

1. SOIL–STRUCTURE INTERACTION (SSI) CAN BE DETRIMENTAL

SSI and Seismic Code Spectra

The presence of (deformable) soil below and adjacent to a structure effects its seismic response in many different ways. First, a flexibly-supported structure has different natural vibration characteristics, most notably a longer fundamental period, $\tilde{T}$, than the period T_{st} of the same structure if this were rigidly supported. Second, part of the vibrational energy of the flexibly-supported structure is dissipated into the supporting and surrounding soil by radiation of waves and by hysteretic action—a phenomenon with no parallel in rigidly-supported structures, and expressed through an effective damping ratio $\tilde{\beta}$.

The seismic design of structures supported on deformable ground must properly account for such an increase in fundamental period and damping. Following early work by Veletsos & Meek (1994) and Jennings & Bielak (1973), the Applied Technology Council's provisions for the development of seismic regulations, known widely as ATC-3, proposed simple formulae for computing $\tilde{T}$ and $\tilde{\beta}$ of structures founded on mat foundations on a homogeneous halfspace. With these two fundamental variables the engineer would use the design response spectrum, appropriate for the seismicity of the region and the nature of the foundation soil, to derive the design *"base shear"* $\tilde{V}$.

Without exception, all codes today use (idealized, envelope) response spectra which attain constant acceleration values up to a certain period (of the order of 0.40 sec to 1.0 sec, at most) and thereafter *decrease monotonically with period* (e.g., as $T^{-2/3}$). As a consequence, soil-structure interaction (SSI) leads invariably to smaller accelerations and stresses in the structure, and thereby to smaller forces onto the foundation. This beneficial role of SSI has been essentially turned into a dogma for many structural engineers. Thus, frequently in practice dynamic analyses avoid the complication of accounting for SSI — a "conservative" simplification, leading (in the prevailing opinion : always) to improved safety margins. This beneficial effect is recognized in current seismic provisions. For example, the NEHRP-94 seismic code states:

"These [seismic] forces therefore can be evaluated conservatively without the adjustments recommended in Sec. 2.5 [i.e., for SSI effects]"

Since design spectra are derived on a conservative basis, the above statement may hold for a large class of structures and seismic environments. But not always. There is evidence, documented in numerous case histories, that the perceived beneficial role of SSI is an over-simplification that may lead to unsafe design for both the superstructure and the foundations.

To elucidate this, the ordinates of a "conventional" design spectrum typical of soft soil (type S3) are compared graphically in **Figure 4** against four selected response spectra: Bucharest 1977, Mexico City 1985 (SCT), Kobe 1995 (JMA, Fukiai, Takatori), presented in terms of spectral amplification. Notice that all the recorded response spectra increase with period and attain their maxima at periods exceeding 1.0 sec. It is therefore apparent that an increase in the fundamental period (due to SSI) will lead to increased rather than reduced response, in contrast with the expectation incited by the conventional design spectrum. It is important to note that all three earthquakes caused damage associated with SSI effects. Mexico City earthquake was particularly destructive to 10 to 12-story buildings (founded on the soft clay) whose period increased due to SSI from about 1.0-1.5 to nearly 2.0 seconds—resonance (Resendiz & Roesset 1985, Bazan-Zurita & Bielak 1994). The

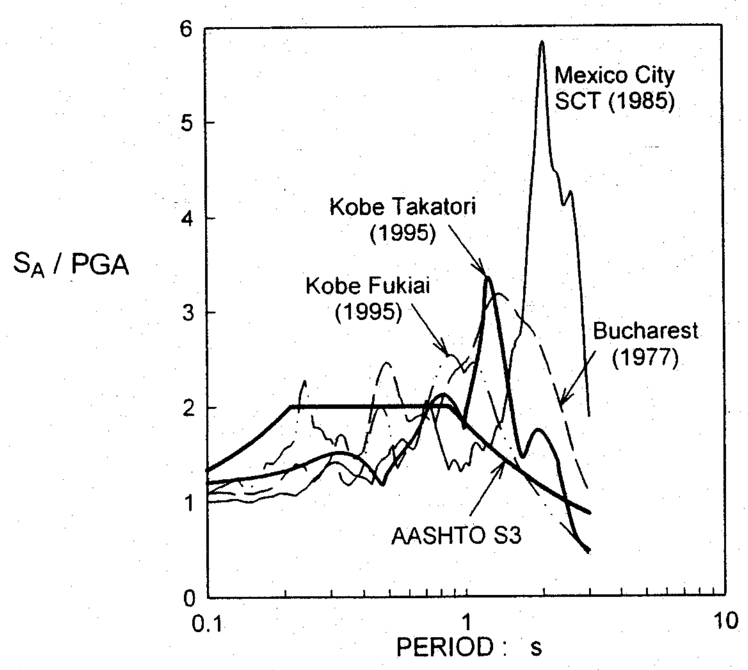


Figure 4 . Comparison of a typical seismic code design spectrum to the actual response spectra of several catastrophic earthquakes with strong long-period components (ξ = 5 %)

role of SSI on the failure of Hanshin Expressway in Kobe (subjected to shaking similar to that of the Fukiai and Takatori records) was also decisive and detrimental, and is discussed in more detail below.

Note that due to SSI large increases in the natural period of structures (i.e., of the order of 0.5 seconds or more) are not uncommon in relatively-tall yet rigid structures founded on soft soil (e.g., Mylonakis et al 1995; Gerolymos 1997). Evaluating the consequences of SSI on the seismic behavior of such structures may require careful assessment of both *seismic input and soil conditions;* use of conventional design spectra and generalized/simplified soil profiles in these cases may not reveal the danger of increased seismic demand on the structure.

Analysis of the Collapse of 630m of the Hanshin Expressway (Kobe 1995)

In the devastation caused by the 1995 Kobe Earthquake, the collapse and "overturning" of a 630 m length of the Hanshin Expressway in Higashi-nada was perhaps the most spectacular failure. This bridge was part of the elevated Hanshin Expressway Route No 3 that runs parallel to the shoreline. Built in 1969, it consisted of single circular columns (3.1 meters in diameter and, on the average, 11 meters in height). It was founded on groups of 17 piles. The columns were connected monolithically to the concrete deck. There were 18 spans in total, all of which failed. A cross-section of the typical pier with its foundation is shown in **Figure 5**. One of the causes of the failure relating to soil are outlined herein.

The First Role of Soil: Influence on the Pattern and Intensity of Ground Motion

Kobe is built along the shoreline, in the form of an elongated rectangle with length of about 30 km and width 2-3 km. The soil in the region consists primarily of sand and gravel of variable thickness (10-80 m), underlain by soft rock. The granitic bedrock (that outcrops in the mountainous outskirts of the city), dips steeply in the northwest–southeast direction; in the shoreline it lies at a depth of 1.0 to 1.5 km. Different soil thickness from one recording station to another may be responsible for the significant differences in the intensity and frequency content of the recorded motions. Seismic "directivity" was *one* of the phenomena that took place in Kobe: it undoubtedly led to the large differences between spectral values in directions normal and parallel to the fault rupture zone. Notwithstanding these effects of directivity, it is believed that *soil* further amplified the incoming seismic waves and produced variations in the characteristics of the records.

The Role of Soil–Pile–Structure Interaction

A rigorous treatment of the problem would require evaluation of the *inelastic* behavior of the superstructure. However, an attempt is made herein to extract

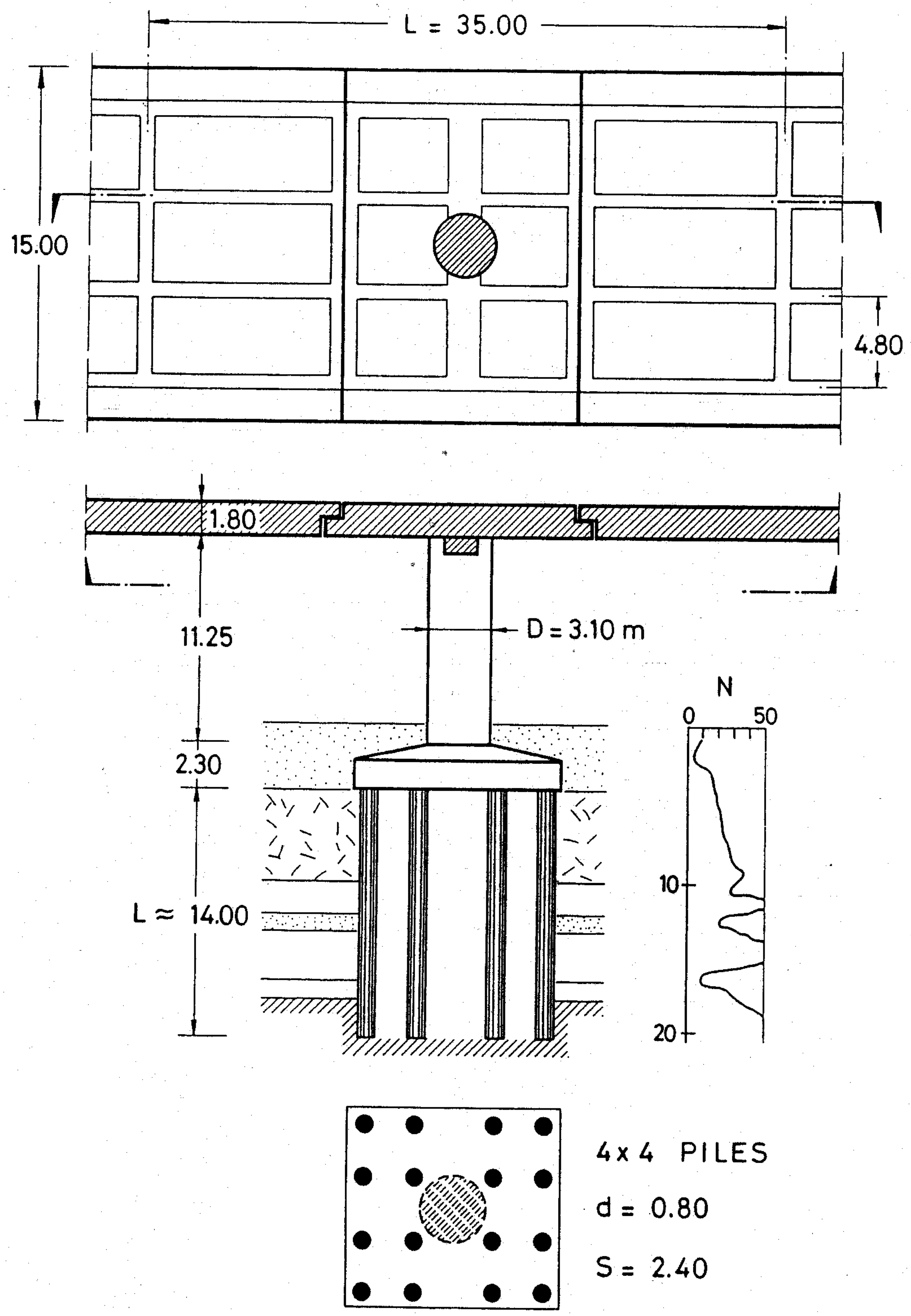


Figure 5 . Plan and cross-section of the failed Route 3 of the Hanshin-Expressway

information on the importance of SSI in the response of the Higashi-Nada bridge using the available elastic spectra and approximate analyses. Useful insight can be developed, even in the realm of preliminary calculations.

The foundation of the bridge consists of 17 reinforced concrete pile having length of about 15 m and diameter of 1m, connected through a rigid 11 m x 11 m cap. The soil surrounding the piles consists of medium dense sand with gravel. SPT values for the upper 20 meters of the soil can be seen in **Figure 5,** while shear wave velocity was found to be of the order of 200 to 300 m/s down to 30 m depth. The mass of the bridge during the earthquake was found to be equal to about 1100 Mg. Similarly, the horizontal stiffness of the pier (cracked conditions) was estimated to be of the order of 150 MN/m. From the above data, assuming the pier to be perfectly fixed at the base and considering the rotational inertia of the deck, the fundamental period of the bridge is estimated to be :

$$T_{st} \approx 0.65 \text{ sec} \tag{4}$$

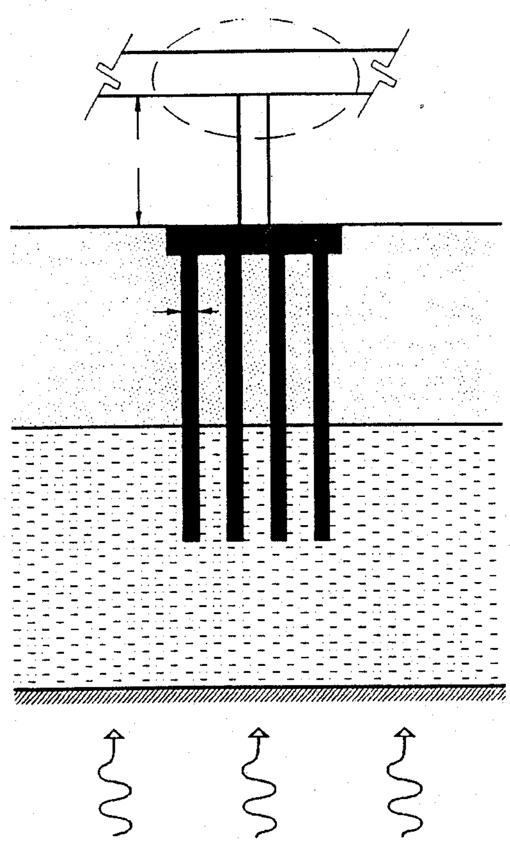


Figure 6. A bridge-pier supported on piles through a 2-layer soil

The dynamic interaction between soil, foundation, and superstructure increases substantially the natural period of this bridge system. An estimate of this period can be obtained from the approximate formula of ATC-3 (Veletsos 1977) once the stiffnesses of the pile group have been estimated. More appropriately, Mylonakis et al (1997) and Gerolymos (1997) have studied the system shown in **Figure 6** :a single-column bridge pier supported on 16 piles in 2-layered soil. Evidently, this system (**Figure** 5) models faithfully the failed piers and their foundation (**Figure 5**). The following expression has been derived for the fundamental period $\tilde{T}$ of this system, in terms of the fixed base period T_{st} (Gerolymos 1997):

$$\frac{\tilde{T}}{T_{st}} \approx 4.47\left[\left(\frac{V_1}{V_2}\right)^{1.00}\left(\frac{H_1}{\ell_a}\right)^{0.25}\left(\frac{H_{st}}{B}\right)^{1.30}\left(\frac{m_{st}}{\pi d^2 \rho_p \ell_a}\right)^{1.35}\left(\frac{\ell_a\sqrt{\rho_p}}{T_{st}\sqrt{E_p}}\right)^{3.1}\right]^{0.094} \qquad (5)$$

where ℓa = the active length of the pile (see discussion in the next section of the paper). Applying this expression, we obtain $\tilde{T} \approx 0.93$ sec.

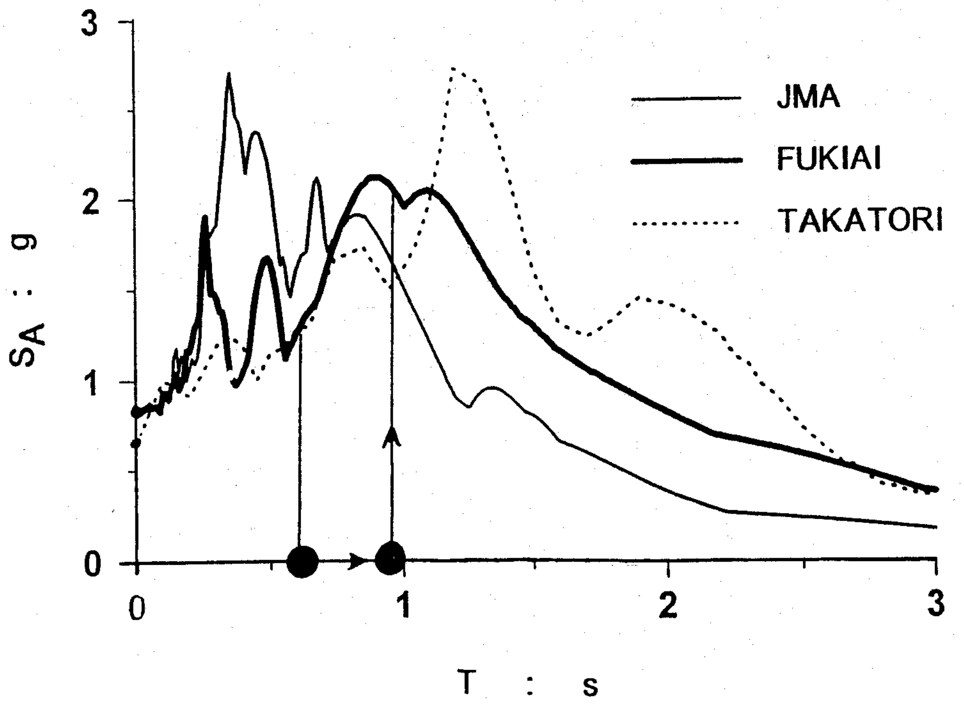


Figure 7 . Response spectra of the three accelerograms considered for the analysis of the Route 3 Section of the Hanshin Expressway

This preliminary result highlights the possible role of SSI, as it increases the "effective" period of the bridge by an appreciable 28%. The result of Eqn (5) was verified with a more comprehensive analysis (using the computer code SPIAB– Mylonakis et al 1997), from which we derived $\widetilde{T} \approx 0.90$ seconds.

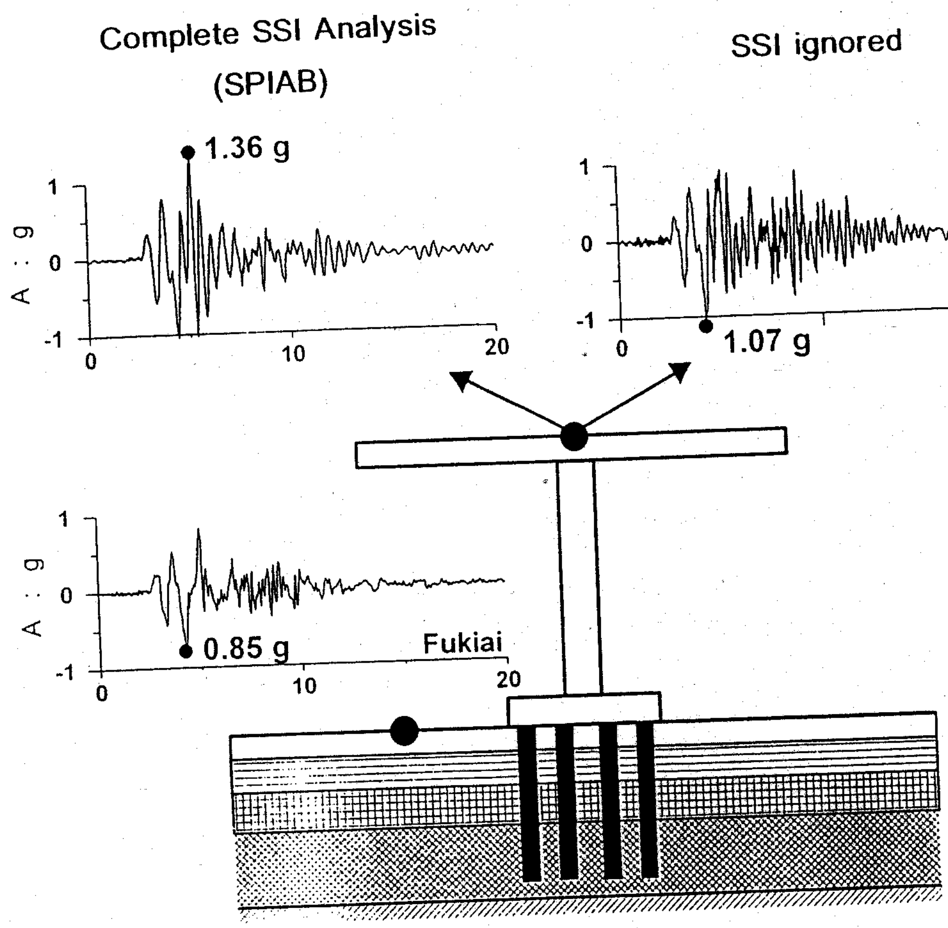


Figure 8 . Effect of dynamic SSI on the acceleration response of the failed Route 3 Section of the Hanshin Expressway

Three acceleration records (JMA: FUKIAI, and TAKATORI) with peak acceleration of values 0.85g, 0.85g, and 0.65g, respectively, and quite different frequency characteristics (**Figure** 7): are examined as representative of the (unknown) true excitation :

- The accelerogram JMA, with a peak value of 0.83g, was recorded on a relatively stiff soil formation (thickness of soil about 10-15 m)
- The accelerogram FUKIAI, with a peak value of about 0.85 g, was recorded on a softer and deeper deposit (thickness of alluvium about 70 m).
- The accelerogram at TAKATORI, with a peak value of 0.65 g, was recorded on a soft and deep deposit (thickness of alluvium about 80 m)

It should be noted that, as suggested by the distance of the bridge site from the mountain, soil conditions under the bridge seem to be closer to those of FUKIAI and TAKATORI, rather than of JMA.

From the spectra of **Figure** 7, the effect of SSI in the response of the bridge starts becoming apparent. If the actual excitation was similar to the JMA record, the increase in period due to SSI and the progressive cracking of the pier would tend to slightly reduce the response, as indicated by the decreasing trend of the spectrum beyond about 0.80 sec. In contrast, with either FUKIAI or TAKATORI as excitation, SSI would lead to progressively larger accelerations in excess of 1g (the exact value depending on damping). As a first approximation, for a best estimate of SA $\approx$ 1.40 g, the force reduction factor based on the calculated strength of the column of about 0.7g would be equal to 2. Taking the *equal displacement rule* as approximately valid, the ductility demand on the pier would be:

$$\mu_d \approx 2 \qquad (6)$$

which is probably larger than the corresponding ductility capacity, in view of the inadequate transverse reinforcement of the pier.

Detailed analyses were also performed to verify the results of the above simplified analysis. They included both equivalent-linear SSI analyses (using the computer code SPIAB) as well as elastoplastic analyses in which the foundation stiffnesses had been computed independently. A typical set of results of the SPIAB analyses (from Michaelides 1997) is shown in **Figure 8** for the FUKIAI record as excitation. The acceleration histories predicted for the deck with and without SSI exhibit different peaks and different frequency characteristics. Indeed the complete response (with SSI) is 25% higher than the response on fixed base — a perhaps crucial difference contributing to the failure of the bridge.

2. KINEMATIC RESPONSE OF EMBEDDED AND PILED FOUNDATIONS

Foundation Input Motion (FIM): Embedded Foundations

The discussion of the preceding section referred mainly to the *inertial* interaction between soil, foundation, and structure. Such an interaction is a direct consequence of *inertia forces* developing in the structure during its vibration; these forces transmitted to the foundation produce horizontal and rotational dynamic deformations which, in return, affect the structural response (by increasing the fundamental period and the effective damping of the system). This type of interaction is the only possible one with rigid surface foundations subjected to a synchronous and uniform horizontal translation (as the one arising from vertical S waves). In the more general cases, however, of embedded or piled foundations, another type of soil-foundation interaction would take place, even in the absence of a superstructure. Given the name *kinematic* interaction, it originates from the seismic free-field pattern of displacements that are imposed onto the foundation.

More specifically, the presence of a (more-or-less rigid) embedded foundation *diffracts* the 1D seismic waves, since its rigid body motion is generally incompatible with the free-field ("driving") motion. The wave field now becomes much more complicated, and the resulting motion of the foundation differs from the free-field motion, and includes both translational and rotational components. Since, according to **Figure 2**, this foundation motion is used as the *excitation* in the inertial interaction step of the whole seismic response analysis, it is named *Foundation Input Motion (FIM)*.

As an example, we refer to a rigid cylindrical massless foundation embedded in a homogeneous elastic halfspace and subjected to vertically incident harmonic S waves. The steady-state harmonic displacements of the free field under linear soil behavior vary with depth in the form

$$U_{ff}(z) = U_{ff}(0)\cos(\omega z / V_s) \qquad (7)$$

From this, the free-field horizontal translation and the free-field pseudo-rotation (which can be thought of as the two components of the "driving" motion) can be obtained :

$$U_{ff}(D) = U_{ff}(0)\cos(\omega D / V_s) \qquad (8)$$

$$\Phi_{ff}(D) = [U_{ff}(0) - U_{ff}(D)] / D = 2\,U_{ff}(0)\sin^2(\omega D/V_s) \qquad (9)$$

This spatial distribution of displacements is incompatible with the *linear* variation with depth imposed by the rigid vertical side-walls of the foundation — hence the scattering of incoming waves. The resulting foundation displacement at the center of the base-mat $U_B = U(D)$ differs from the free-field values of $U_{ff}(D)$ and $U_{ff}(0)$. Moreover, a rotation (rocking) $\Phi_B = \Phi(D)$ develops due to non-uniformly distributed tractions against the side-walls, but also differing from the free-field pseudo-rotation $\Phi_{ff}(D)$.

The following simple expressions [based on results by Luco (1969), Elsabee et al (1977), Tassoulas et al (1984), Harada et al (1981), and others] can be used for estimating the translational and rotational components of the *FIM*. For a foundation *embedded* at depth D, with or without sidewalls, and subjected to harmonic *vertical and oblique* waves:

$$U_B = U_{ff}(0)\ I_u(\omega) \tag{10}$$

$$I_u(\omega) \approx \begin{cases} \cos\left(\dfrac{\pi}{2}\dfrac{f}{f_D}\right) & \text{for} \quad f \le \dfrac{2}{3} f_D \\ 0.50 & \text{for} \quad f > \dfrac{2}{3} f_D \end{cases} \tag{11}$$

$$\Phi_B = [\ U_{ff}(0) / B\ ]\ I_\varphi(\omega) \tag{12}$$

$$I_\varphi(\omega) \approx \begin{cases} 0.20\ \left[1 - \cos\left(\dfrac{\pi}{2}\dfrac{f}{f_D}\right)\right] & \text{for} \quad f \le f_D \\ 0.20 & \text{for} \quad f > f_D \end{cases} \tag{13}$$

in which B = the foundation halfwidth or "equivalent" radius in the direction examined; ω = the (circular) frequency of the harmonic seismic waves; $f = \omega / 2\pi$ is the frequency in Hz; $f_D = \omega_D / 2\pi = V_S / 4D$ is the shear frequency in Hz of a (hypothetical) soil stratum of thickness D. Φ_B denotes rotation about a horizontal axis through the center of the foundation base.

Notice that the rotation is an integral and important part of the base motion for the massless foundation. Ignoring it and accounting only for the de-amplification of the translational component through the transfer function $I_u(\omega)$, may lead to errors on the unsafe side. These errors are perhaps negligible for determining the response of short squatty structures, especially very heavy ones. But they are substantial (of the order of 50%) for the *top* of tall slender structures. On the other hand, ignoring both the deamplifiction of the horizontal component [i.e., taking $I_u(\omega) = 1$] *and* the existence of the rotational component ($\Phi_B = 0$), usually leads to slightly conservative results; this is a simplification frequently followed in practice for non-critical structures.

Foundation Input Motion (FIM): Piled Foundations

While an extremely flexible pile might simply follow the seismic motion of the ground, real piles "resist" and, hence, modify soil deformations. As a result, the incident seismic waves are "scattered" and the seismic excitation to which the structure base is **effectively** subjected differs from the **free-field** motion, as was the case with the embedded foundation. In turn however, piles experience bending, axial and shearing stresses, in function of their overall rigidity relative to the soil. This wave-induced interplay between soil and piles is also affected by the kinematic constraints imposed at the head of the piles from the cap and the super-structure.

While, in real-life situations, rigorously modeling all the factors influencing the kinematic response is a formidable task (especially if non-vertical seismic waves are expected to impinge on the piles, or if substantial soil nonlinearities are likely to

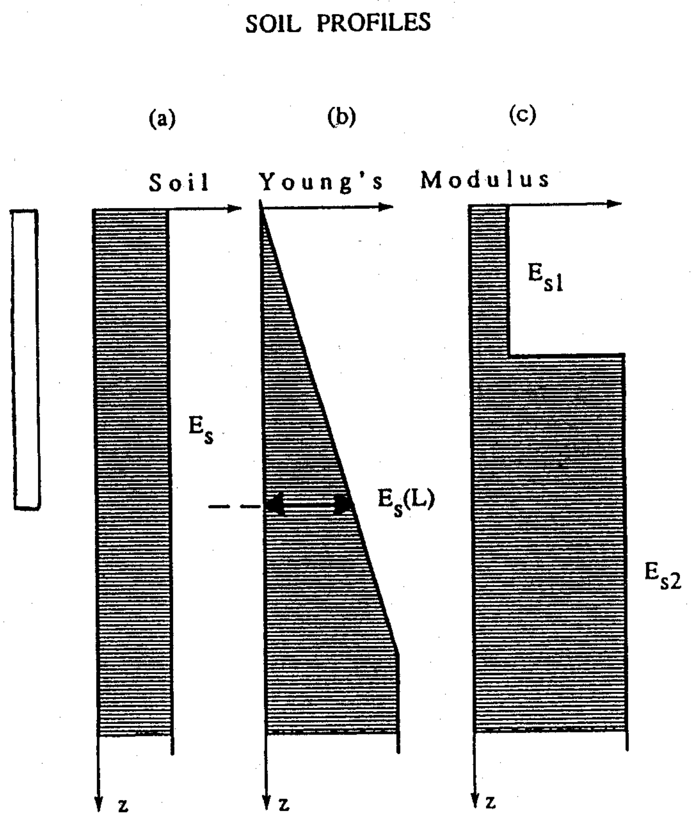


Figure 9. Sketch of 3 soil profiles for which kinematic response results are given

develop), practically useful results and valuable insight to the mechanics of soil-pile interaction have been obtained from linear analyses of simple idealized systems under vertical and inclined S-wave excitation. A few studies have also considered excitation by Raleigh waves. Such results have been reported in the last twenty years by a number of researchers, including Blaney et al (1976), Takemiya & Yamada (1981), Flores-Berrones & Whitman (1982), Wolf & Von Arx (1978), Kaynia & Kausel (1982), Gazetas (1984), Barhouthi (1984), and Tazoh et al (1988), Mamoon & Banerjee (1990), Fan et al (1991), Gazetas et al (1992, 1993), Kaynia & Novak (1992), Kavvadas & Gazetas (1993), Makris (1994), Nikolaou et al (1995), Tabesh & Poulos (1996).

Most of the above studies have focused on the first effect of the interaction namely the modification of the free-field motion. This effect is usually quantified through the dimensionless ratios, I_u and I_φ, known as transfer functions or ***kinematic response factors,*** relating the absolute values of pile head amplitudes, U_p and Φ_p, to the free-field horizontal amplitudes at ground surface, U_{ff}, for each harmonic component of the motion (frequency ω).

The ratios are complex numbers (due to the generation of both radiation and hysteretic damping), and depend primarily on the type and stiffness of the soil profile, the relative frequency of the motion, the pile-cap or pile-head boundary conditions, the type of incident seismic waves, and the overall relative rigidity of the pile with respect to the soil. For the three simple soil profiles sketched in **Figure 9** (homogeneous, linearly-inhomogeneous, and a two-layered profile) the above important variables can be represented by the following dimensionless parameters:

- the frequency factor

$$\mathbf{a_0 = \omega\, d / V^*_S}$$

  where V^*_S is a characteristic value of the soil S-wave velocity profile, namely : $V^*_S = V_S$ for the homogeneous profile, $V^*_S = V_S(L)$ for the linearly inhomogeneous profile, and $V^*_S = V_{Sl}$ for the two-layered profile.

- the ratio of the effective pile modulus (E_p) to a characteristic soil Young's modulus E^*_S equals E_s for the homogeneous profile, $E_s(L)$, for the linearly inhomogeneous profile, and E_{S1} for the two-layered profile.

- the angle of wave incidence Θ .

Less significant parameters are: the pile slenderness ratio, L/d (length over diameter), Poisson's ratio(s), the ratio(s) fo mass densities, ρ_p / ρ_S, and for pile groups the number of piles and the relative pile spacing, s/d. From the results of the afore-mentioned studies the following trends have emerged (see Fan et al1991, for more details):

(1) The *general shape* of the kinematic displacement factor, $I_u = I_u(a_o)$, consists (for single piles and pile groups) of three distinct regions in the frequency range of greatest interest for earthquake loading ($a_o < 0.5$) :

- A low-frequency region ($0 < a_o < a_{o1}$) in which $I_u \approx 1$ – the end result of the pile(s) following closely the relatively-large-wavelength deformations of the ground.

- An intermediate-frequency region ($a_{o1} < a_o < a_{o2}$) characaterized by I_u declining rapidly with frequency — a direct consequence of the progressively-increasing incompatibility between the "wavy" pattern of soil movements and the flexurally-deforming pile(s).

- A relatively-high-frequency region $a_o > a_{o2}$ in which $I_u(a_o)$ fluctuates around an essentially-constant value of about 0.20 to 0.40 -- since at such frequencies the increasing "waviness" of the soil deformations is largely counterbalanced by the generally-decreasing amplitude of the ground surface motion. Indeed, $a_o > a_{o2}$ corresponds to seismic-excitation frequencies, f, being of the order of five-to-ten-times greater than the natural frequency, f_n, of the soil deposit in S-waves; and it is well known from the 1-Dimensional Soil Amplification Theory (Roesset 1976) that a linearly-hysteretic soil would experience at such frequencies U_{ff} values not very different from the values of the base-rock displacement.

(2) Whereas this general shape of $I_u = I_u(a_o)$ is approximately valid in all studied cases, four factors seem to significantly affect the transition frequencies a_{o1} and a_{o2} : *(i)* the type of soil profile; *(ii)* the relative rigidity of the pile; *(iii)* the pile-head fixity conditions; and *(iv)* the pile slenderness. Specifically:

The significant factor controlling the magnitude of a_{o1} and a_{o2}, and thereby the kinematic response of single piles and pile groups, is the nature of the soil profile as expressed by the variation of soil modulus, E_s, versus depth. In strongly non-homogeneous deposits, as the one having modulus proportional to depth, a_{o1} is very small -- of the order of merely 0.05 (see **Figure 10**), depending of course on the value of the other three factors. By contrast, in a homogeneous stratum or in a stratum with a thick homogeneous top layer a_{o1} may be as high as 0.20 - 0.30. (Thus, in terms of actual frequencies, ω, in deposits with the same **average** wave

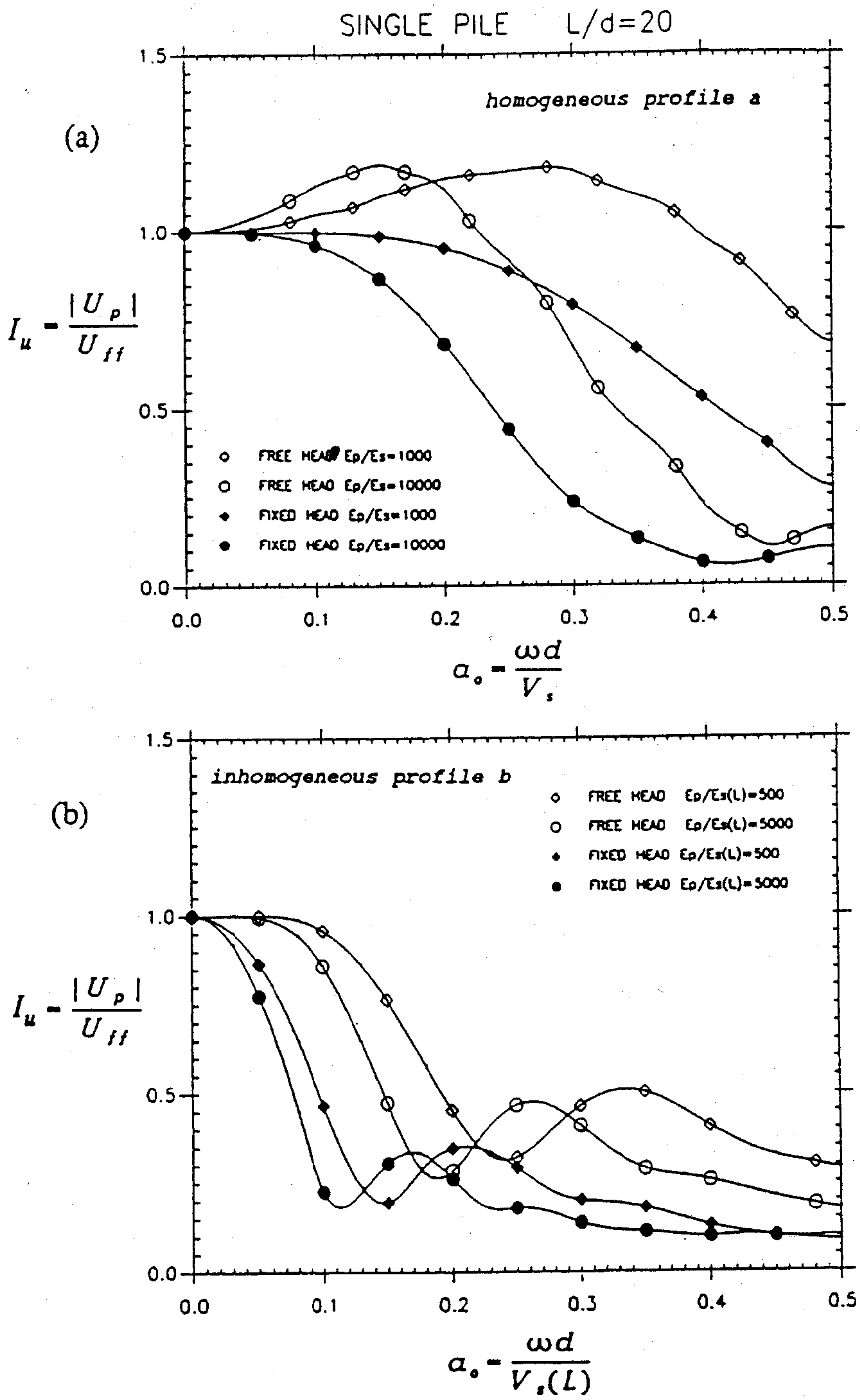


Figure 10. Influence of pile head fixity, stiffness ratio, and type of soil profile, on kinematic seismic response of single piles in : (a) homogeneous soil, and (b) non-homogeneous soil (L/ d =20) [from Fan et al, 1991]

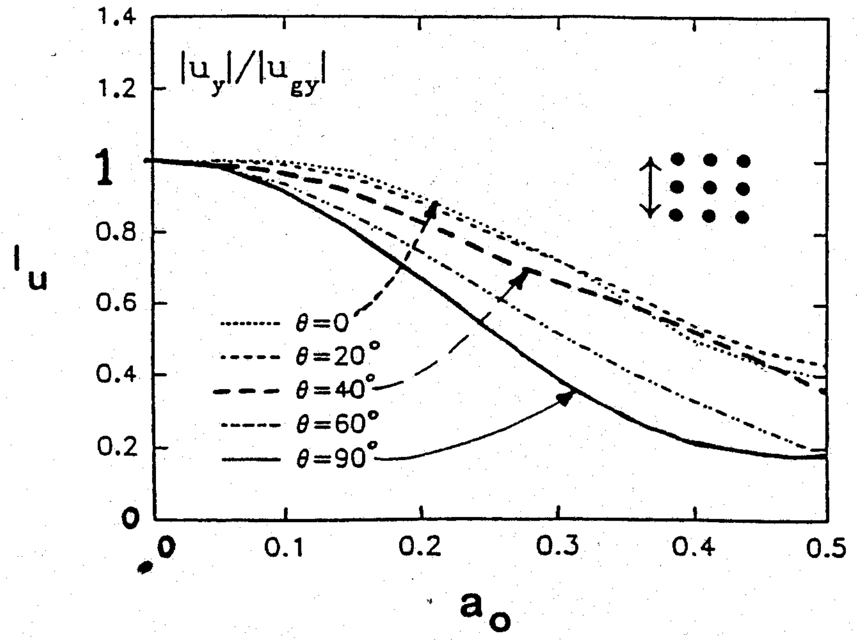
velocity the decaying branch of I_u will start earlier – by a factor of about 2 – in the non-homogeneous profile.) Similarly, a_{o2} is about 0.10 - 0.20 in the linear-modulus profile **"b"** compared to a_{o2} usually exceeding 0.40 in the two other profiles **"a"** and **"c"**. The practical implications of these differences are worthy of note: in non-homogeneous profiles, piles and pile groups will depress a much wider spectrum of the harmonic components of the incident seismic excitation (and thereby their heads will experience smaller "effective" horizontal input motions) than pile(s) in a homogeneous soil.

The relative rigidity of the pile(s), expressed through the aforementioned moduli ratios also affects a_{o1} and a_{o2} . As expected, the stiffer pile(s) are more effective in depressing a seismic soil movement and, hence, their kinematic response is characterized by smaller values of a_{o1} and a_{o2}, , compared with those of the softer piles.

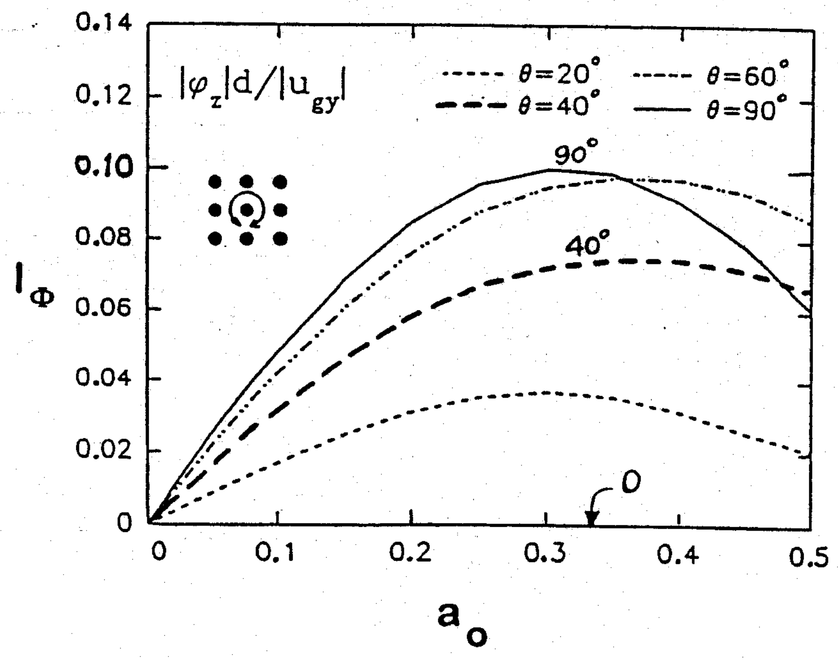
Increasing the degree of fixity at the pile-cap level (from "hinged"- or "free"-head to "fixed"-head piles) has an effect similar to the effect of increasing E_p / E_s : a_{o1} and a_{o2} tend to decrease and, hence, the "effective" pile-cap input motion in an earthquake excitation will tend to be less severe. An additional influence of pile-head fixity conditions has been observed with pile(s) embedded in homogeneous deposits and in "free"-head single piles and "hinged"-at-the-cap piles and pile groups embedded in such soils experience in the low to intermediate frequency range I_u values exceeding unity. This appears to be the only case where a small deviation from the aforescribed general shape of $I_u(a_o)$ has been observed. It impliles an "effective" pile top motion, $|U_p|$, greater than the free-field U_{ff}. With "fixed"-head piles this tendency for larger pile-top motion is completely suppressed.

(3) Pile group configuration ("row" versus "square"), number of piles in the group (1, 2, 3, 36), and pile-spacing ratio (s/d - 3, 5 and 10) make little difference on I_p , in the low ($a_o < a_{o1}$) and intermediate ($a_{o1} < a_o < a_{o2}$) frequency ranges. This remarkable conclusion is valid (within engineering accuracy, of course) for most studied profiles and relative pile rigidities. It implies that with seismic excitation there is little pile-to-pile interaction at this frequency range, even for close pile spacing. By contrast, with inertial excitation at the top, pile-to-pile interaction has been shown in the literature to play a dominant role in the response of pile groups.

(4) Increasing the angle of incidence Θ from 0° to 90° of the SH waves considered to excite the system, increases the "filtering" effect of kinematic interaction: at almost all frequencies I_u becomes smaller. At the same time the rotational components of motion at the pile cap increase. **Figure 11** from Kaynia & Novak 1992 illustrates this effect.



(a) Horizontal translation



(b) Torsion about vertical axis

Figure 11. Kinematic interaction factors for 3 x 3 pile group subjected to SH-waves (from Kaynia and Novak, 1992, for E_p / E_s = 1000, pile spacing s/d = 5, fixed head piles)

Exceptions to this general trend when dealing with multi-layered soil profiles are not impossible. One such exception is presented in **Figure 12** (Gazetas et al 1992) It refers to a soil deposit containing a *thin soft top layer* (h_a / d = 5 and V_a / V_b < 1/3). Plotted in this figure is the kinematic displacement factor I_u as a function of $a_o = \omega d / V_s$, for a number of V_a / V_b ratios. As V_a / V_b decreases, i.e. as the top layer becomes relatively softer, the kinematic displacement factor tends to fluctuate with frequency at an increasing rate. At certain frequencies, the pile head deflection may even be greater than the free-field surface displacement. To illustrate how this is possible, Gazetas et al plot the distribution of pile and soil displacements along the depth [normalized by the base ("rock") displacement amplitude] at a number of frequencies to show the considerable differences between soil and pile displacements in the upper soft layer when the velocity contrast between the two layers is large (e.g. $V_a / V_b = 1/6$).

Use of Kinematic Response Transfer Functions

Eqns 10 - 13 and the graphs shown in **Figures 10-12** give the kinematic response ("transfer") functions, relating the free-field horizontal ground surface motion to the effective "foundation input motion" (FIM) in the frequency domain. The mathematically correct but still approximate way of using these functions is as follows:

- obtain the Fourier Amplitude Spectrum (U_A) of the Design Motion at the free-field ground surface
- multiply U_{ff} by $I_u(\Omega)$ and by $I_\varphi(\Omega)$ / B to obtain the Fourier Amplitude Spectra functions (U_B and Φ_B) of the components of the FIM
- use these functions directly as excitation in the *inertial* interaction analysis, if the latter is done in the frequency domain, or through an inverse Fourier Transformation obtain the corresponding time histories to be used as excitation in a time domain inertial response analysis.

In practice, the most frequently used method involves a further vital simplification. It makes use of *Response Spectra* rather than Fourier Spectra, and is therefore particularly attractive whenever the design motion is specified in the form of a Design Response Spectrum $S_A(\omega)$ at the ground surface — a most usual case. The Response Spectrum of the *effective horizontal FIM* is approximated as the product of $S_A(\omega)$ $I_u(\omega)$ for the acceleration to be applied at the foundation mass and as the product $S_A(\omega)$ [$I_\varphi(\omega) + I_\varphi(\omega)$ h_c / B] for the acceleration to be applied at a structural mass located at a distance h_c from the base.

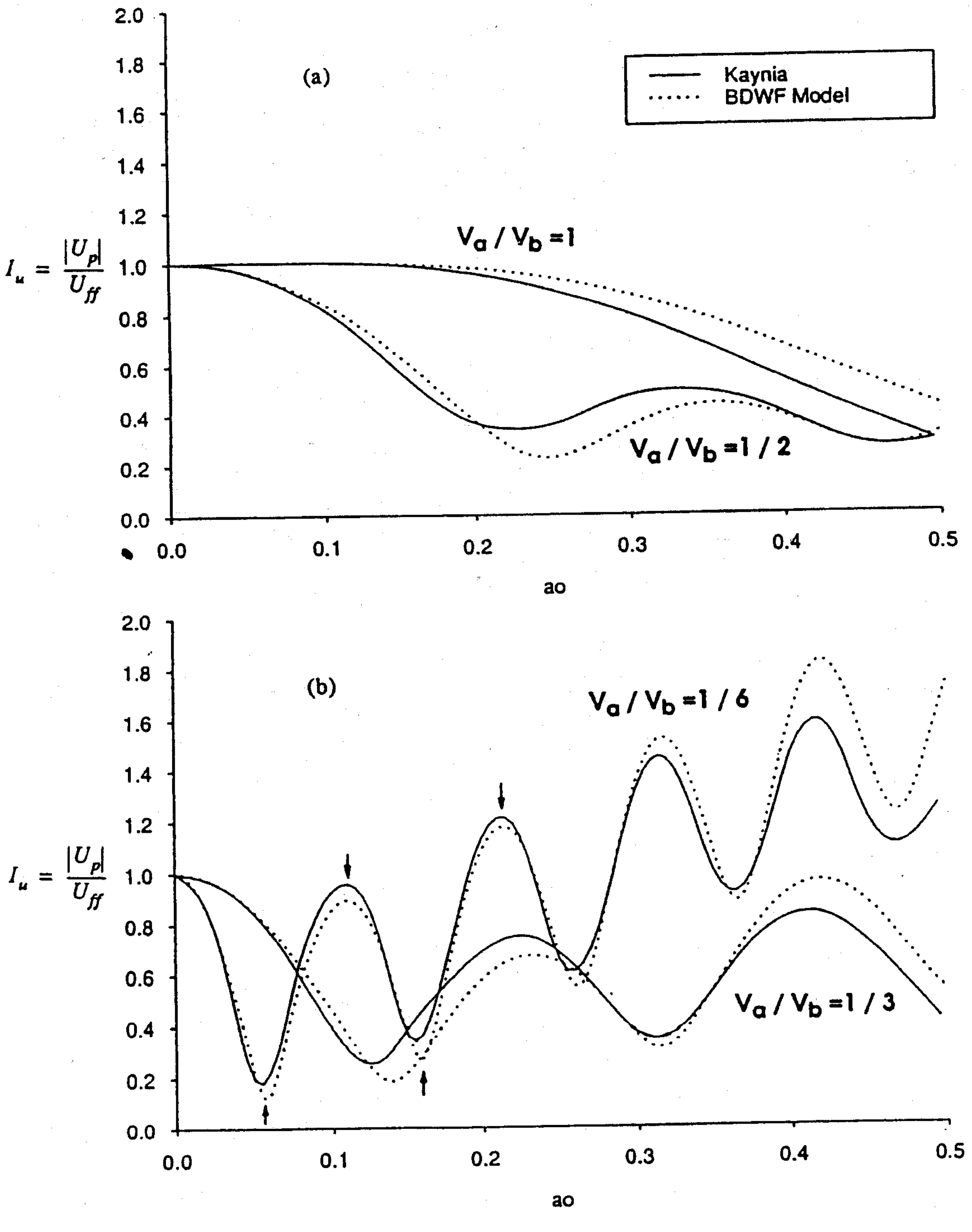


Figure 12 . Kinematic seismic response of fixed head single pile: (a) $V_a / V_b = 1$, 1/2 and (b) $V_a / V_b = 1/3$, 1/6 ($E_p / E_{sb} = 1000$, $h_a = 5d$, $L / d = 20$, $\nu_s = 0.4$, $\beta_s = 0.05$) [from Gazetas et al, 1992]

Kinematic Deformation and Bending of Piles

Until recently, piles were designed to transmit safely ***only*** the inertia loads from the oscillation of the superstructure. Mizuno in 1987 was the first to document a number of pile flexular failures at locations too deep to be caused by loading from the pile top, in soils that could not possibly have suffered a severe loss of strength (e.g. liquefaction). Damage was instead associated with the presence of strong discontinuities in strength and, especially, stiffness of the soil profile. The most likely cause is the relatively large curvatures imposed by the surrounding soil as it deforms while excited by up and down (after reflection) propagating seismic waves.

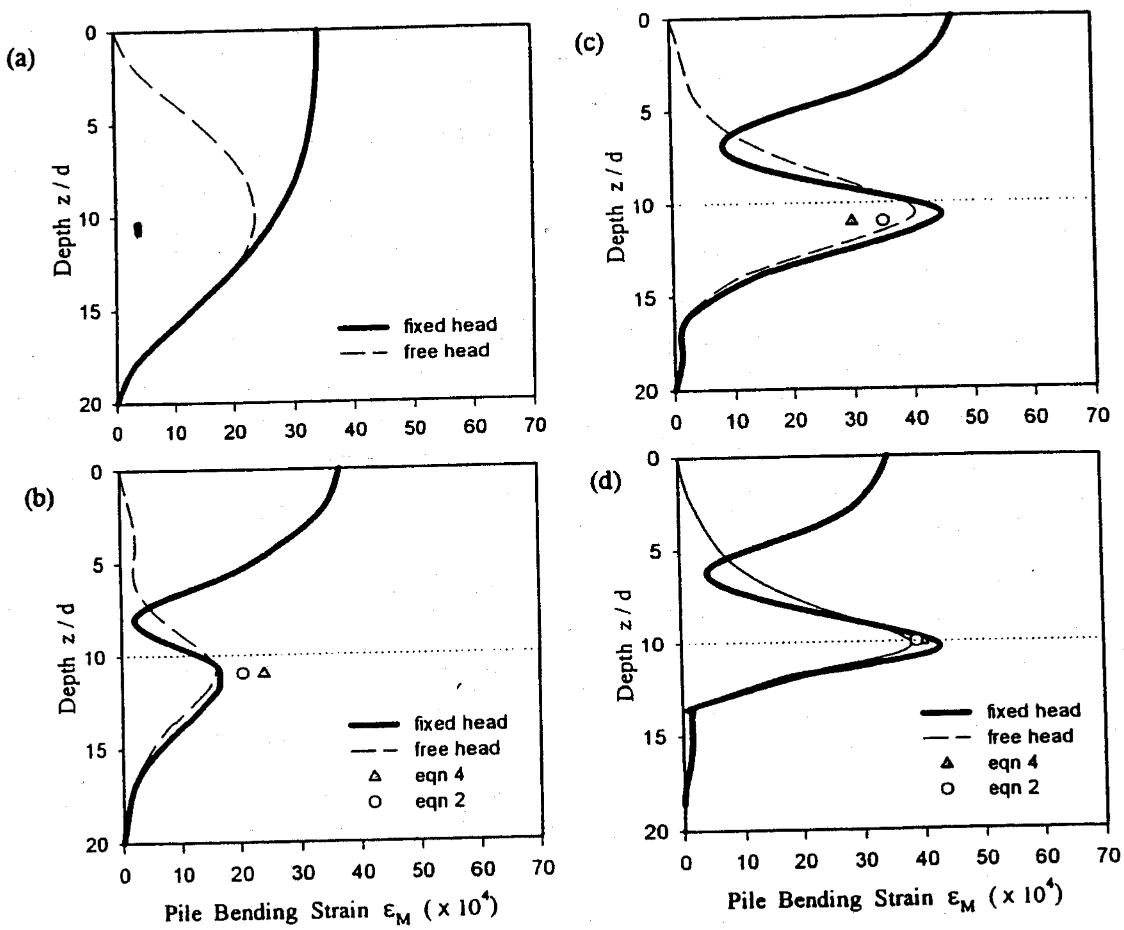


Figure 13. Steady-state bending moments for a fixed and free head pile with L/d = 20 and E_p / E_{s1} = 5000 (a) in a homogeneous profile, (b) in a 2-layer profile with V_1/V_2 = 1/2, (c) in a 2-layer profile with V_1/V_2 = 1/4, (d) in a 2-layer profile with V_1/V_2 = 1/10 (from Nicolaou & Gazetas, 1997)

This mode of deformation and potential failure has been studied extensively by Tazoh et al (1988), Ahmad & Mamoon (1991), Gazetas et al (1992), Kavvadas & Gazetas (1993), Poulos & Tabesh (1996), Nikolaou and Gazetas (1997). **Figures 13-14**.and the discussion that follows summarize the main findings of these studies. They refer to a single pile in a two-layer soil profile, subjected to harmonic steady-state excitation, which consists exclusively of vertically-propagating S waves. The most important conclusions that have emerged are as follows:

(1) For a given excitation, the kinematic bending moments depend mainly on:

- the stiffness contrast between any two consecutive soil layers in the deposit; for the examined profile this contrast can be measured with the ratio V_1 / V_2 of the respective S-wave velocities.

- the boundary conditions at the head of the pile or the pile cap; the results presented herein consider only the two extreme cases, i.e. of *fixed-head piles with no cap rotation* and of *free-head* piles with no rotational constraint at their top

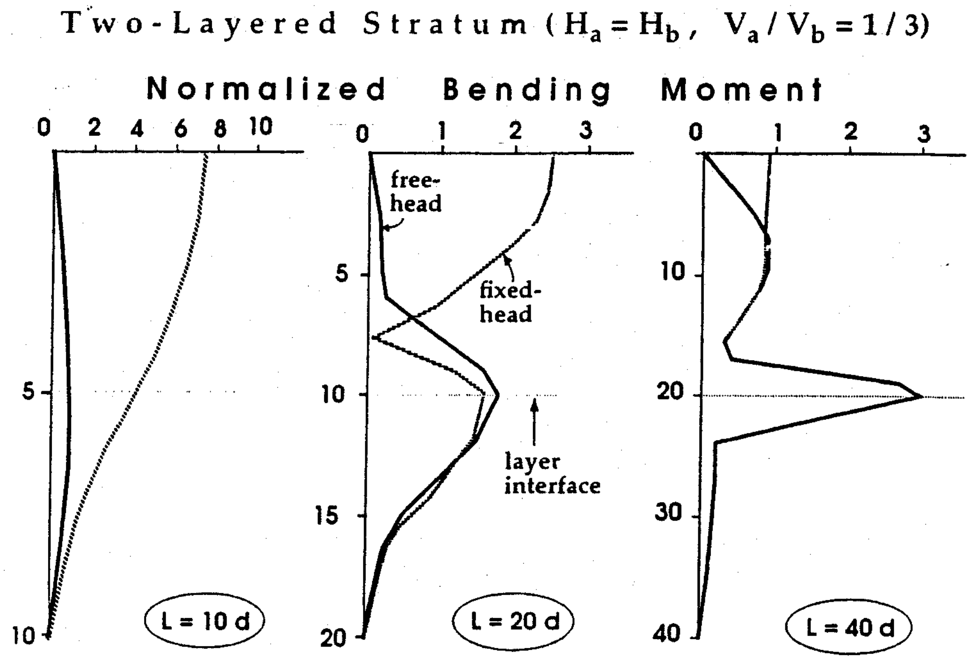


Figure 14. Steady-state bending moments for a fixed and a free head pile with L/d = 10, 20 and 40 in a 2-layer profile with $V_a / V_b = 1/3$

- the proximity of the excitation frequency ω to the fundamental (first) natural frequency ω_1 of the soil deposit and, to a lesser degree, to the second natural frequency ω_2 of the deposit

- the relative depth H_l / ℓ_a measured from the top of the pile down to the interface of the layers with the sharpest stiffness contrast normalized with respect to the active length ℓ_a of the pile.

(2) The bending moments are largest either at the pile head, or in the vicinity of the interface of soil layers with the sharpest stiffness (one pile diameter away from the interface). The moments at the interface for free and fixed head piles are almost identical, except when the pile is "short" and "rigid" (meaning, when $h_1 < \ell_a$). Atop the fixed-head piles the moment is generally of the same order of magnitude as, or smaller than, the moment created at the interface of the two layers. In some cases though, the moment at the top may be much higher than at the interface. These are the cases for which the ***active pile length*** ℓ_a well-known from the inertial interaction as:

$$\ell_a \approx 1.5\, d\, (E_p / E_{s1})^{0.25} \qquad (14)$$

is larger than the height of the first soil layer. Apparently, these will be the cases where relative *stiff* piles (e.g. $E_p / E_{s1} > 5000$) with relative *small depth* to the interface ($H_1 / L < 1/2$) are forced to develop high pile head bending moment in order to satisfy the quite severe no-rotation top boundary condition.

(3) In most cases, the maximum steady-state bending moment occurs at the fundamental natural period of the soil deposit. The pile moment transfer functions display a very rapid reduction of the moment when moving away from resonance. The ratio of the maximum pile moment at resonance (whether it occurs at the top or at the interface) divided by the respective static moment [i.e., the ratio $\max M(T_1)$ / M(static: T)] follows, more or less, the free-field amplification of acceleration: a_{surf} / a_{rock}. This shows the great influence of the first mode of vibration on the magnitude of the developed moment and contradicts some earlier statements in the literature that higher modes would produce larger kinematic moments. Indeed, while higher frequencies do tend to develop "wavy" shapes of deflection and thus have the potential for inducing relatively large curvatures at the soil interface, the actual curvature is also affected by the overall drift between the top and the bottom of the pile. This drift becomes maximum at the first natural mode and consequently produces the largest moments at the first resonance.

(4) Pile moment in the vicinity of the interface is also influenced by the pile and soil characteristics as well as the differences between the two layers. The relative pile-

soil stiffness is of great importance : the stiffer the pile with respect to the soil, the larger the developing moment. The stiffness contrast between the two layers is also very significant : high values of kinematic moment occur mainly when the stiffness changes sharply across the interface ($V_1 / V_2 < 0.25$).

(5) A closed-form expression has been developed for approximately computing the maximum steady-state bending moment at the interface between the two layers Nikolaou & Gazetas (1997), is based on the shear stress τ_{interf} that is likely to

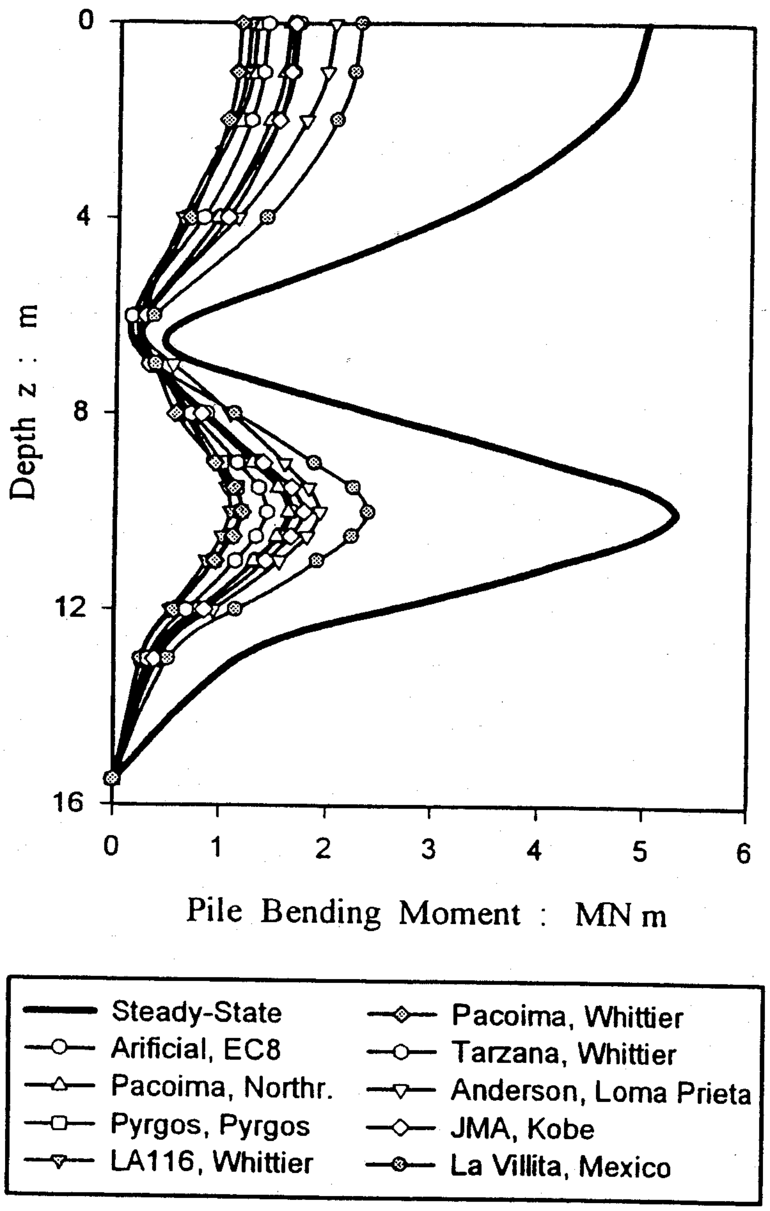


Figure 15 . Distribution with depth of peak steady-state moments and envelope of time-domain moments using 9 records (from Nicolaou & Gazetas, 1997)

develop at that interface, as a function of the free-field acceleration at the surface, $a_{surface}$:

$$\tau_{interf} \approx a_{surface} \, \rho_1 \, H_1 \qquad (15)$$

The fitted formula is written :

$$M_{max} = 0.042 \, \tau_{interf} \, d^3 \, (L/d)^{0.30} \, (E_p / E_{s1})^{0.65} \, (V_1/V_2)^{-0.50} \qquad (16)$$

Under transient seismic excitation, described through actual and artificial accelerograms applied at the base of the deposit, Nikolaou et al (1995) have shown that the above conclusions are still valid, but with one exception : the peak values of the transient bending moments are smaller than the steady-state amplitudes. An example is given in **Figure 15** , for a relatively rigid fixed-head concrete pile (diameter 1.3 m, length 15.5 m) penetrating a 9.5 m thick top layer of soft clay and socketed 6m into a deep layer of dense sand. Eight actual accelerograms and one artificial motion, all scaled to a 1 m/s^2 (0.10 g) peak acceleration, were used as excitation at the rock level. It is evident from this figure that the envelope of peak moments (in the "time domain") has a distribution with depth which is of the same shape as the distribution of steady-state amplitudes (in the "frequency domain"). But the values of the latter are about 3 - 5 times larger than the former, depending on the excitation.

Nikolaou et al (1995) have proposed that a reduction factor η be applied to the maxM(ω), the maximum steady-state pile bending moment, in the frequency domain, to arrive at : maxM(t), the maximum pile bending moment in the time domain. This factor is a function of the number of cycles, N_{cycles}, the ratio T_p / T_1, and the effective damping in the soil, β_{eff}. Quantification of the variables N_{cycles}, and T_p can not be realized without pertinent seismological input.

Adoption in Seismic Codes and Field Evidence

The kinematic interplay between soil and pile may be quite important in the seismic performance of a piled foundation. The effects on these bending moments from an even partial "loss" of strength of the overlying saturated sandy soils should not be dismissed without study.

It is perhaps worthy of note that the importance of kinematic loading has been recognized in the recently published Seismic Code of the European Union (Eurocode 8), dealing with the seismic design of civil structures. Part 5 of that Eurocode, devoted to foundations and geotechnical engineering, states:

"Piles shall be designed for the following two loading conditions :
(a) inertia forces from the superstructure . . .
(b) soil deformations arising from the passage of seismic waves which impose curvatures and thereby lateral strain on the piles along their whole length . . . Such kilnematic loading may be particularly large at interfaces of soil layers with sharply differing shear modulil. The design must ensure that no 'plastic hinge' develops at such locations . . . "

A comprehensive set of field records presented by Tazoh et al (1988) and analyzed by Gazetas et al (1993) confirm the significance of kinematic loading on piles and validate the theoretical methods described in chapter 3 for computing its consequences. Two of the sixty four piles of a bridge pier were equiped with strain meters at four depths along their length; the deepest location (at about -22m) coincided with the interface of the very soft clay deposit through which the 0.60 m-diameter piles penetrate and the underlying hard clay into which the piles are founded (see **Figure 16**). Both axial and bending strains could be recovered. A significant earthquake of magnitude 6, 42km from the bridge site, triggered the strain meters as well as numerous accelerometers paced at depth in the hard clay ["base"]

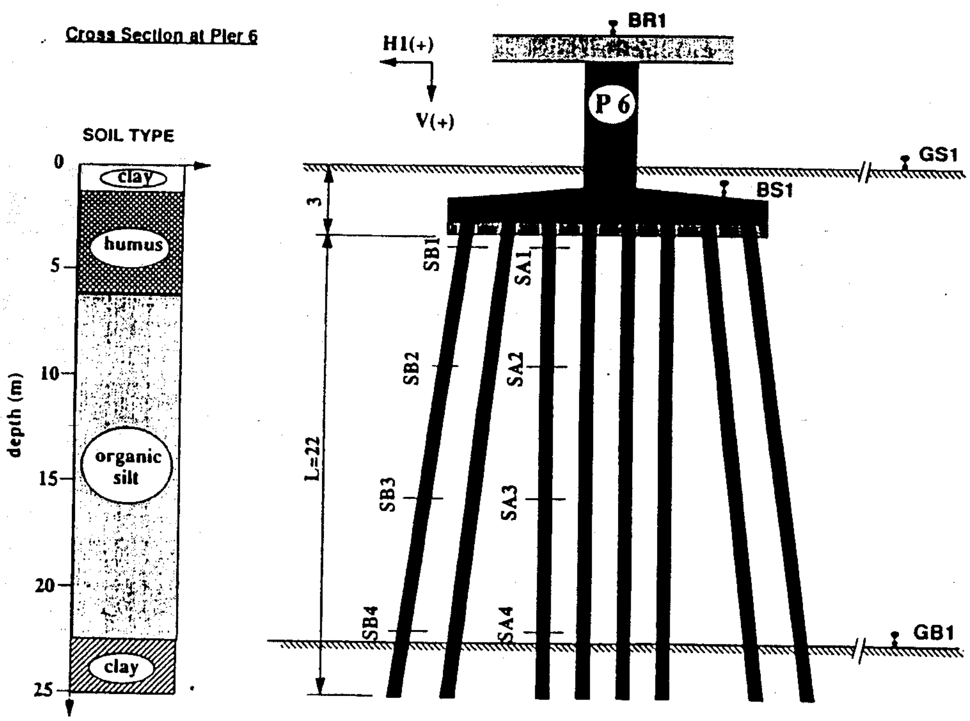


Figure 16 Ohba-Ohashi in Japan. Setup of strain meters on the piles under pier 6 (from Tazoh et al, 1984 and Gazetas and al, 1993)

and on the ground surface). Peak accelerations reached about 0.03g in the base and about 0.12g on the surface. The recorded four bending-strain histories, their Fourier Amplitude Spectra (FAS), and the distribution with depth of the maximum FAS values are depicted in **Figure 17**. Also plotted in this figure for comparison are the FAS of the bending strains computed with the afore-mentioned Beam-on-Dynamic-Winkler-Foundation (BDWF) method, considering only kinematic response, i.e. ignoring the inertial loading. Thus, the role of kinematic response can be seen more clearly.

Indeed, it is seen that the largest maximum spectral amplitudes, in both the recorded and computed FAS, occur at the fundamental natural period ($T = 1.4$ s) of the soil deposit, at the top and bottom locations. Evidently, bending strains can be quite large at great depths wherever interfaces of soil layers with sharply differing striffnesses exist, as at $z = 22$ m in Ohba-Ohashi. Such bending deformations are not affected by the inertial load transmitted from the superstructure onto the head of the piles; hence, a solely-kinematic analysis can predict them very well. By contrast, the inertia-induced bending strains are significant only near the top of the piles, arising both from the horizontal inertia force and from the restraining and overturning moment atop the pile.

The findings of theoretical studies on pile distress during earthquakes are largely confirmed by these measurements.

Lateral–Spreading Effects on Piles

The *kinematic deformation* and stressing of piles studied in the preceding paragraphs is of a vibratory nature. It is "driven" by the oscillatory motion of the ground — a result of (primarily shear) waves, propagating up and down through the soil. Piles however can also be subjected to a different type of *kinematic loading* if the surrounding ground becomes unstable and moves laterally and one-directionally on its base. Such is the case when *"lateral spreading"* of the top soil layers is caused by liquefaction of soil layers and subsequent movement downslope or towards an unsupported ground depression or excavation.

The phenomenon had already been described (e.g., Youd & Perkins 1987; Hamada & O'Rourke, eds, 1992; Youd 1993; Benuzka, ed., 1990) when the 1995 Kobe earthquake occurred and the interest in the subject skyrocketed. Liquefaction of gravelly-silty-sands was widespread over vast areas of reclaimed lands in the harbor wharves and islands of Kobe and led to *lateral spreading* with seaward movement of the harbor quaywalls of about 3 to 4 meters. Horizontal displacements of the ground remained higher than 0.50 m for distances of 50 m - 100 m from the quaywalls. (Hamada et al 1996, Ishihara et al 1997).

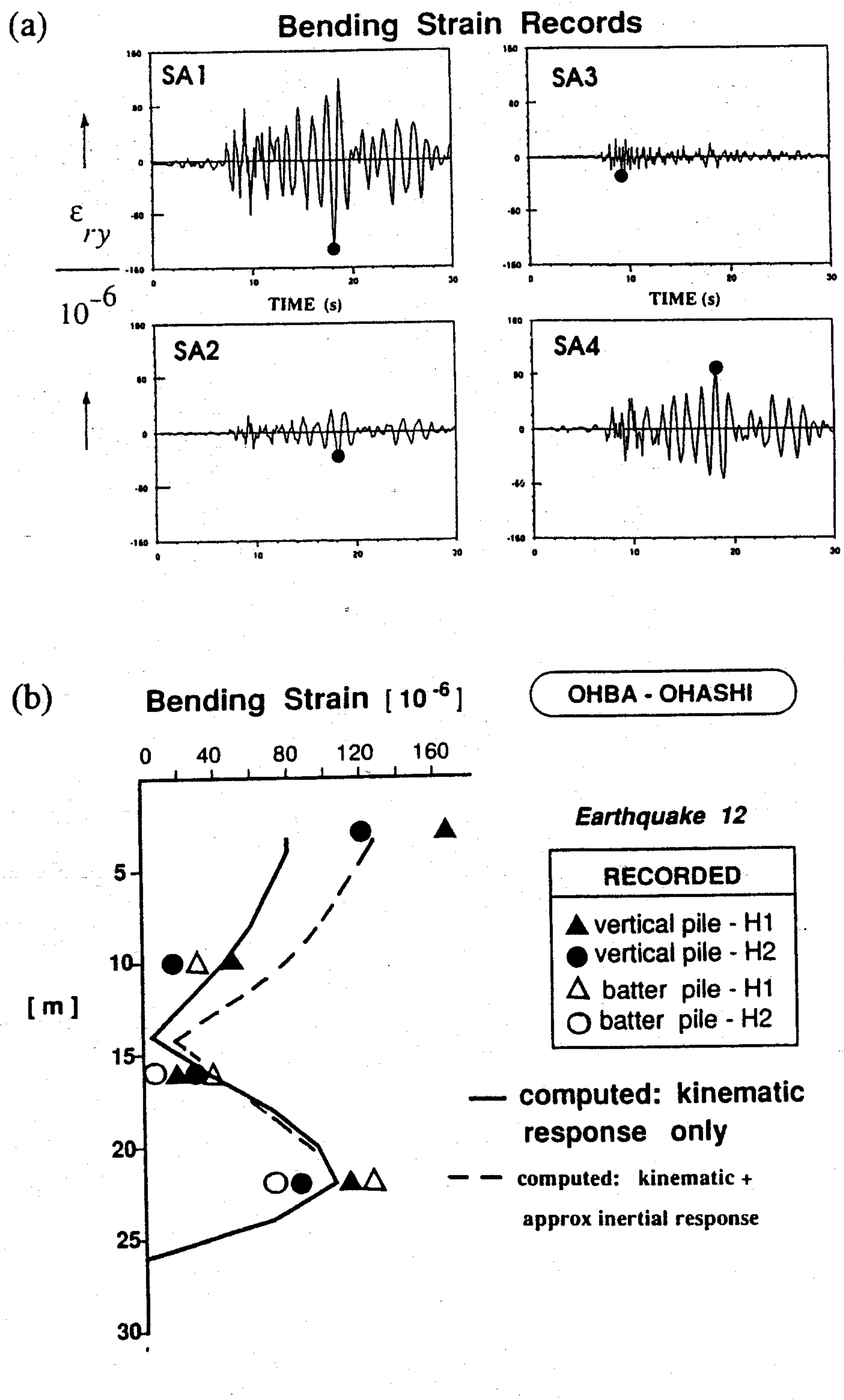


Figure 17. (a) Ohba Ohashi in Japan. Recorded bending strain time histories at four depths of the vertical pile (Tazoh et al, 1988), (b) Comparison of the recorded and computed maximum spectral bending strains along the vertical and batter pile (Gazetas et al 1993)

Numerous water-crossing bridge piers were founded in this region, most of them belonging to Route 5 (Harbor Highway). Severe industrial tanks and some buildings were also located in the region. The pile foundation of these structures were subjected to large drag forces from the "flowing" (laterally–spreading) liquefied soil and possibly by the stiffer soil crust that was "carried along" on the underlying liquefied soil. Post- earthquake reconnaissance has revealed severe cracking at the pile head, i.e immediately below the pile cap, and at the boundaries between liquefied and nonliquefied soil layers.

Many in-situ surveys documenting pile damage due to lateral spreading have been reported in the Special Issue (January 1996) of Soils & Foundations (e.g., Hamada et al 1996 , Ishihara et al 1996, Inagaki et al 1996, Matsui & Oda 1996, Tokimatsu et al 1996 and in the recent Terzaghi Oration by Ishihara 1997).

It is recognized that *a* key parameter (if not *the* key parameter) controlling the pile bending distress from such kinematic loading *is the permanent horizontal ground displacement* in the free-field near the pile. Both the displacement at the ground surface and its distribution with depth are of importance. It was noted, however, in observations that pile heads undergo smaller displacement than the surrounding ground.

To analyze the phenomenon one must first predict the permanent ground deformation (both magnitude and spatial distribution) caused by lateral spreading. This is a formidable task for which no general engineering method is yet universally accepted — in contrast with the prediction of whether liquefaction will occur or not. Simplified analytical methods numerical F.E. analyses and empirical expressions and charts have been developed for predicting lateral-spreading displacements(Dobry & Baziar 1991, Iai 1996, Hamada et al 1986, Bartlett & Youd 1992, Ishihara et al 1997). Centrifuge model tests have been increasingly used to evaluate specific case histories and calibrate computer programs (Dobry 1996).

Once determined or estimated, this permanent ground displacement profile (the "driving" deformation) is imposed statically on the piles, through a series of Winkler nonlinear "springs", attached to sliders. The model is fully compatible with the aforementioned dynamic beam-on-Winkler-Foundation model used for kinematic response analysis of piles in stable ground. Extensive results of such pseudo-static analyses on the effects of lateral spreading have been presented, among others, by Miura & O' Rourke 1991 and Ishihara 1997. They invariably show smaller deflection of the pile than the "driving" permanent displacement of the free-field.

However, pile head deflection is strongly affected by the kinematic restriction imposed from the superstructure. In general, *there is* an interaction between piled foundation and superstructure which *must* be taken into account. In many cases, however, the superstructure is not free to *"move along with the piles"* — its displacement is restricted by "forces" external to the interacting system. Two examples from Kobe :

- Miyakawa Ohashi — the piles of each of the abutments–piers of this simply supported 45 m long bridge were displaced 0.50 m at their top, while the surrounding ground experienced lateral spreading of about 2.0 m. The piles were found cracked at several depths, but it was *not* their own rigidity that prevented them from displacing to the extent of the surrounding ground ; it was rather the axial stifness of the steel deck restricting the motion at the top of the abutment wing-walls (Tazoh & Gazetas 1996).

- Port Island Ohashi — the caissons of this 320m-long steel arch bridge did not exactly follow the laterally-spreading ground, which experienced about 2.0 m of horizontal permanent displacement in each bank. The two huge caissons (D = 15 m, and L = 33 m), founded on moderately dense silty gravelly sand, displaced only 0.40 m (Kobe side) and 0.20 m (Port Island side) and saved the bridge. The latter was supported through a sliding roller on the Kobe-side caisson; this roller indeed displaced $\approx$ 0.20 m + 0.40 m = 0.60 m. In this case the interaction between soil and caisson seems to have played the dominant role, but the constraint from the superstructure could have had a secondary but measurable effect as well.

The above two examples are only meant to illustrate the complexity of the interaction between : the "flowing" ground, the piles or caissons embedded in it, and the superstructure.

While in the above presentation it was tacitly assumed that the phenomenon is driven by soil displacements, an interesting case history from New Zealand's Landing Road bridge (Berrill et al 1997, Keenan 1996). It was found out that during the 1987 M_L 6.3 Edgecumbe earthquake, the greatest load imposed on the bridge foundation came from the 1.5m-thick non-liquefied crust that was "carried along" on the underlying spreading sand layer towards the river channel. This load far exceeded the drag forces from the "flowing" sandy layer, and caused passive failure of the crust. As mentioned in a later section herein, the bridge was saved partly due to the restraining effect of the pile group which included several inclined (raked) piles.

Clearly the issues of SSI in a laterally–spreading environment have begun to emerge.

3. INERTIAL RESPONSE OF PILE GROUPS: PILE–TO–PILE INTERACTION

Basic Features of Dynamic Group Effect; Results for Linear and Homogeneous Soil

The dynamic impedance of a group of piles in any mode of vibration cannot be computed by simply adding the stiffnessses of the individual piles, since each pile is affected not only by its own load, but also by the load and deflection of the neighbouring piles. Similarly, the seismic response of a pile group may differ from the response of each individual pile taken alone, because of additional deformations transmitted from the adjacent piles. This *pile-to-pile interaction* is frequency-dependent, a result of waves emitted from the periphery of each pile and propagating to "strike" the neighboring piles. A variety of numerical and analytical methods have been developed to compute the dynamic response of pile groups accounting for pile-to-pile interaction. Kaynia (1982) and Makris & Gazetas (1992) have shown that, this *group effect* is of profound importance *only for head-loaded piles.* For kinematic seismic loading the group effect could in many practical cases be neglected. Therefore, only inertial type loading is considered herein.

Under static loads, pile-to-pile interaction increases the settlement of the group, redistributes the loads on individual piles, and reduces the bearing capacity (unless this reduction is counteracted by densification of the soil within the group due to pile driving in cohesionless soils). The investigation of static group effects was put on a rational continuum-mechanics basis by Poulos (1968, 1971) and Butterfield and Banerjee (1971). The static data are useful even for dynamics, because at low frequencies and particularly below the fundamental frequency of a stratum the dynamic stiffness is usually quite close to the static stiffness.

The first theoretical dynamic analysis of pile-soil-pile interaction was conducted by Wolf & Von Arx (1978) who employed an axisymmetric finite element formulation to establish the dynamic displacement field due to ring loads. Waas and Hartmann (1981, 1984) formulated an efficient semi-analytical method which uses ring loads and is well suited for layered media, properly accounting for the far field; the layers ought to be thin. Kaynia (1982) further improved the accuracy by combining the cylindrical loads, actually a boundary element formulation, with the consistent stiffness matrix of layered media to account for the far field. A very similar approach is employed in the paper by Kobori et al., (1991) who use the cylindrical loads for the pile and disk loads for the base. Boundary element solutions, employing Green's functions of generally layered media, were formulated by Banerjee & Sen (1987), Banerjee et al (1987).

Pile-to-pile interaction may affect the response of a group so substantially that the group dynamic impedance (stiffness and damping) may have no resemblance to the sum of the impedances of the individual piles. As an example, for a square group of 2 x 2 rigidly-capped piles embedded in a deep homogeneous stratum, **Figure 18** portrays the variation with frequency of the vertical and horizontal ***dynamic group stiffness and damping factors,*** defined as the ratios of the group dynamic stiffness and dashpot coefficients, respectively, to the sum of the static stiffness of the individuall solitary piles. At zero frequency, the stiffness group factors reduce to the respective static group factors (also called "efficiency" factors) which are invariable smaller than unity. The continuous curves in **Figure 18**, adopted from the rigorous solution of Kaynia and Kausel (1982), reveal that, as a result of dynamic pile-to-pile interaction, the dynamic stiffness group factors achieve values that may far exceed the static efficiency factors, and may even surpass unity. Both stiffness and damping factors exhibit undulations associated with wave interferences, which are not observed in the single-pile response. Specifically, *the peaks of the curves occur*

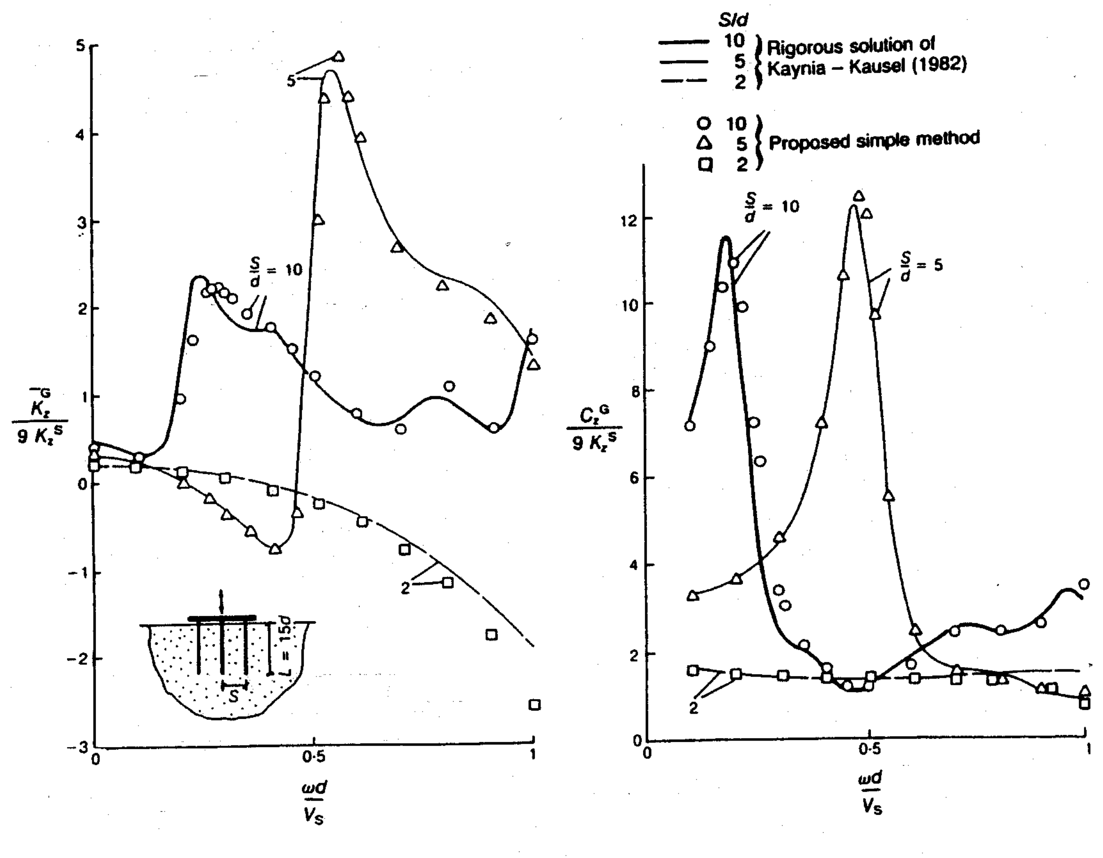


Figure 18 . Vertical dynamic stiffness and damping group factors as a function of frequency : comparison of the Dobry & Gazetas (1988) simple method with rigorous solution of Kaynia (1982) for a group of 3 x 3 fixed-head piles in a homogeneous halfspace

whenever waves originating with a certain phase from one pile arrive at the adjacent pile in exactly opposite phase, thereby inducing an upward displacement at a moment that the displacement due to this pile's own load is downward. Thus, a larger force must be applied onto this pile to enforce a certain displacement amplitude, resulting in a larger overall stiffness of the group, compared to the sum of the individual pile stiffnesses.

Also depicted in **Figure 18** as points, are the results of a *very simple* analytical method proposed by Dobry & Gazetas (1988) and later developed by Makris & Gazetas (1989), and Mylonakis (1995). Despite a number of physically-motivated approximations that the method introduces, its results are remarkably close to the rigorous curves for the three considered pile separation distances of two, five and ten pile diameters. Even some detailed trends in the group response seem to be adequately captured by the simple solution.

The fundamental idea of the method is that the displacement field along the sidewall of an oscillating pile (in any mode of vibration) propagates and affects the response of neighboring piles. The most crucial of the simplifying assumptions is that *the waves created by an oscillating pile emanate simultaneously* from all perimetric points along the pile length, and hence, for a homogeneous stratum, they form cylindrically-expanding waves. To determine the group impedances, the method adopts Poulos' superposition approach, which has been extended to dynamic loading by Kaynia (1982) and Sanchez-Salinero (1984). Development of frequency-dependent *interaction factors,* i.e., displacement ratios expressing the effect of one pile onto another, is an important intermediate step in the analysis. For two vertically oscillating piles floating in a homogeneous halfspace the method leads to the following simple expression for the axial interaction factor:

$$\alpha_v = (r_o / s)^{1/2} \exp[-(i + \beta)\, \omega\, S / V_S] \tag{17}$$

where $i = \sqrt{(-1)}$ and β = soil hysteretic damping ratio. Similarly-simple expressions have been obtained for the interaction factors in lateral vibration. Evidently, dynamic interaction factors are complex numbers, oscillatory functions of frequency, i.e. achieving negative as well as positive values. Negative values of the imaginary part indicate a possible increase in group damping characterized by group efficiency greater than unity. A complete set of interaction factors is available for floating piles in a homogeneous and a linearly inhomogeneous soil (Gazetas et al., 1991, El-Marsafawi et al., 1992).

The Main Limitations : Layering and Non-linearity

The values of the interaction factors computed with the above outlined methods are large. Even at spacings of $s > 10d$ the interaction factors still attain "appreciable" values at certain frequencies such that if a foundation contains a large number, n, of piles the effect of pile-to pile interaction could be very large, with a very significant (often dramatic) effect on the pile-group stiffness. This stems from two facts : *(a)* that the total number of pile pairs whose interaction must be considered is: n (n - 1) ; and (b) the majority of these pairs would likely be spaced at distances in excess of 10 to 15 diameters. As an example, in a foundation of a major power plant consisting of 230 piles at a (closest) spacing $s \approx 2d$, approximately 65% of the pile pairs (i.e. about 34,000 pairs) are spaced at distances exceeding 15d !

An overestimation of the interaction factors at such distances may lead to unrealistically reduced foundation stiffnesses, if other factors do not accidentally counteract. No wonder why many knowledgeable practitioners are reluctant to use the interaction factor method in their pile analyses ; instead, they simply introduce empirical reduction factors on the basis of the size of the group.

In fact, soil layering, and soil non-linearity are two significant factors which when properly taken into account tend to reduce the interaction factors. There is evidence showing that realistic group stiffnesses are obtained under such conditions.

Soil layering, has been found to affect substantially interaction factors. For example, in the presence of a stiff soil layer (or of rock) at the pile base (tip) the vertical interaction factors (which control both vertical and rocking oscillations) would decrease to only a small fraction of the values computed for piles "floating" in homogeneous halfspace. *Lateral* interaction factors are also affected from layering, and even from the presence of a stiff underlying layer (e.g. rock). Computation of interaction factors in such cases can be done either with rigorous boundary-element type methods (Kaynia 1982), or with a variety of simplified methods, such as the *"layered wave-interference"* method of Mylonakis et al (1995). The latter is an extension of the simple method of Dobry and Gazetas (1988) to layered soils,.

An example is offered in the attached **Figure 19.** Note that the open circles (corresponding to a homogeneous profile for which the rigorous Boundary Element Method [BEM] gives nearly identical results with the Dobry-Gazetas method) are much lower than the solid circles for a two-layered deposit, in which the topmost layer is much softer than the lower one ($V_{s1} / V_{s2} = 1/4$). However, the effect of layering is not as substantial for lateral loading.

Nonlinear soil behavior (*and nonlinearities such as gapping and sliding*) also play an important role in pile-to-pile interaction. Specifically, the stronger the nonlinearity (inelasticity) of the soil, the smaller the value of the interaction factors. As an

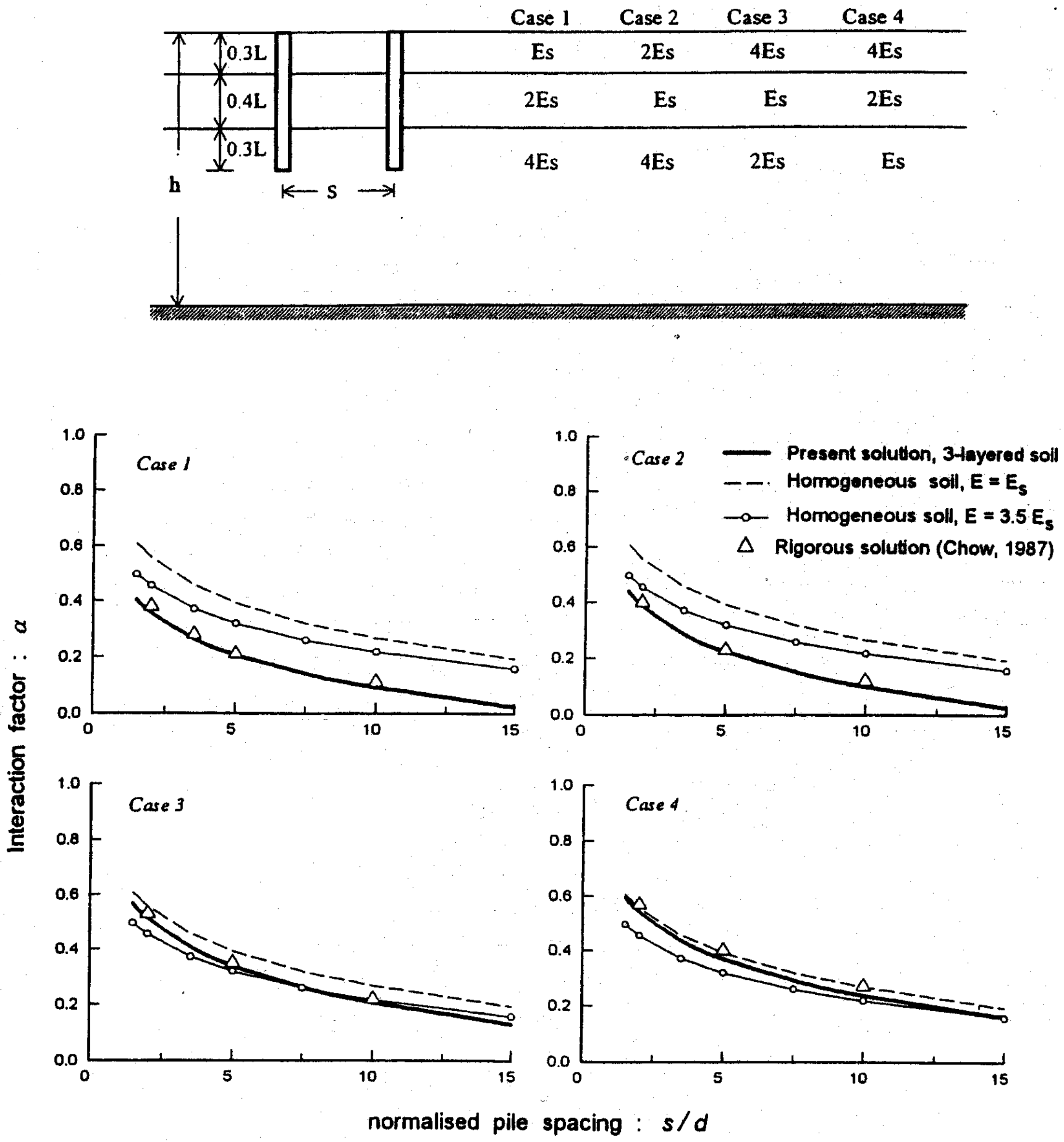


Figure 19 . Interaction factors for piles in layered soil. Comparison with the numerical solution of Chow (1987) for 4 different soil profiles (Cases 1-4) resting on rigid bedrock located at h = 2L. E_p / E_{s1} = 1000, L/d = 25, ν_s = 0.30 (from Mylonakis, 1995)

example, Michaelides (1997) computed the ratio λ_v of the interaction factor, α_V, in vertical oscillation for nonlinear ($\Lambda_S > 0$) and linear ($\Lambda = 0$) soil, as

$$\lambda_v = \alpha_V(\Lambda_S) / \alpha_V(0) \approx (1 - 0.2\,\Lambda_S^{1/3})\,(s/d)^{-5a\Lambda_S^2} \qquad (18)$$

where Λ_S is a *loading intensity factor* which is proportional to the ratio of the imposed shear-stress amplitude, τ_{co}, at the pile shaft periphery to the frictional capacity of the pile-soil interface, f_s. Evidently, strong soil nonlinearity tends to reduce pile-to-pile interaction — especially at large distances (say $s > 5\,d$).

This effect, however, may not be as significant for lateral loading. The ratio

$$\lambda_H = \alpha_H(\Lambda_X) / \alpha_H(0) \qquad (19)$$

i.e., the reduction of the horizontal interaction factor due to nonlinearity, is smaller than in the vertical case. Λ_X = the lateral *"loading intensity"* factor (Michaelides 1998).

4. SEISMIC BEHAVIOR of INCLINED (RAKED) PILES: IS THEIR PRESENCE DETRIMENTAL or BENEFICIAL ?

For years, the seismic role of *inclined* piles (also called *raked* or *battered* piles) has been considered detrimental, and the codes required that such piles be avoided. For instance the French Seismic Code (AFPS 90) states flatly that ***"inclined piles should not be used to resist seismic loads"***. The aforementioned seismic eurocode EC8 in clause 5.4.2 is a little less restrictive:

> " *It is recommended that no inclined piles be used for transmitting lateral loads to the soil. If, in any case, such piles are used, they must be designed to carry safely axial as well as bending loading.*"

The main arguments against the use of inclined piles have been the following:

(a) such piles are subjected to ***"parasitic" bending stresses*** due to soil densification (following an earthquake) and/or soil consolidation under static conditions (if the pile penetrates soft clay layers on which external loads are placed).

(b) such piles may induce large ***forces (of alternating sign) onto the pile cap***, which may thereby experience significant distress.

(c) when the inclination of the piles is not symmetric (i.e., when all piles are inclined in one direction, or when some piles are inclined and some are vertical), ***permanent rotation*** may develop due to different stiffness of the pile group in each direction of loading.

(d) it was an old tradition to design a group of inclined piles as a simple statically-determinate system, ignoring the effect of the soil. Inclined piles would then undertake only axial forces, whereas in reality they attracted significant bending moments; their failure in the next seismic event was then in essence the result of a design error.

However, in recent years evidence has been accumulating that (the above arguments notwithstanding) inclined piles may, at least in certain cases, be ***beneficial rather than detrimental*** both for the structure they support and the piles themselves. Here is part of a growing body of evidence.

Theoretical Evidence

- Recent research by Banerjee & coworkers at SUNY Buffalo has shown that the response of a bridge type structure improves in many respects when supported by inclined piles. As an example, Guin (1997) has presented results such as those of **Figure 20** for a typical bridge pier supported on a group of 2 x 3 piles, of which only the two in the center are vertical. For angles of range $\psi = 20^o$ to 30^o, a substantial improvement is achieved in the response of the bridge deck (accelerations reduced to only 50% of the value developing in the case of only vertical piles: $\psi = 0^o$). The shear force on the pile cap also decreases to about 70% of the "$\psi = 0^o$ " value. But the *individual* inclined piles will certainly receive an appreciable shear force and bending moment at their top.

- Lam et al 1987 have shown that in a liquefied soil both cap displacements and pile bending moments may be reduced dramatically thanks to the stiffening effect of the inclined piles. The Edgecumbe 1987 case history (outlined below) offers some support (although indirect) to such findings.

Field Evidence

- *KOBE 1995*: One of the few quay-walls that survived this disaster in the harbor of Kobe was a composite wall (in MAYA Wharf) relying on inclined piles . The nearby (also in MAYA Wharf) wall with only vertical supporting piles was devastated (completely disappeared). Indiscriminately disallowing inclined piles is clearly not always in the interest of safety.

- EDGECUMBE (NZ) 1987: Berrill et al (1997) and Keenan 1997) investigated the near-failure response of the foundation of the Loading Road Bridge in New Zealand. Lateral spreading "carried" a top cohesive soil layer (crust) on the

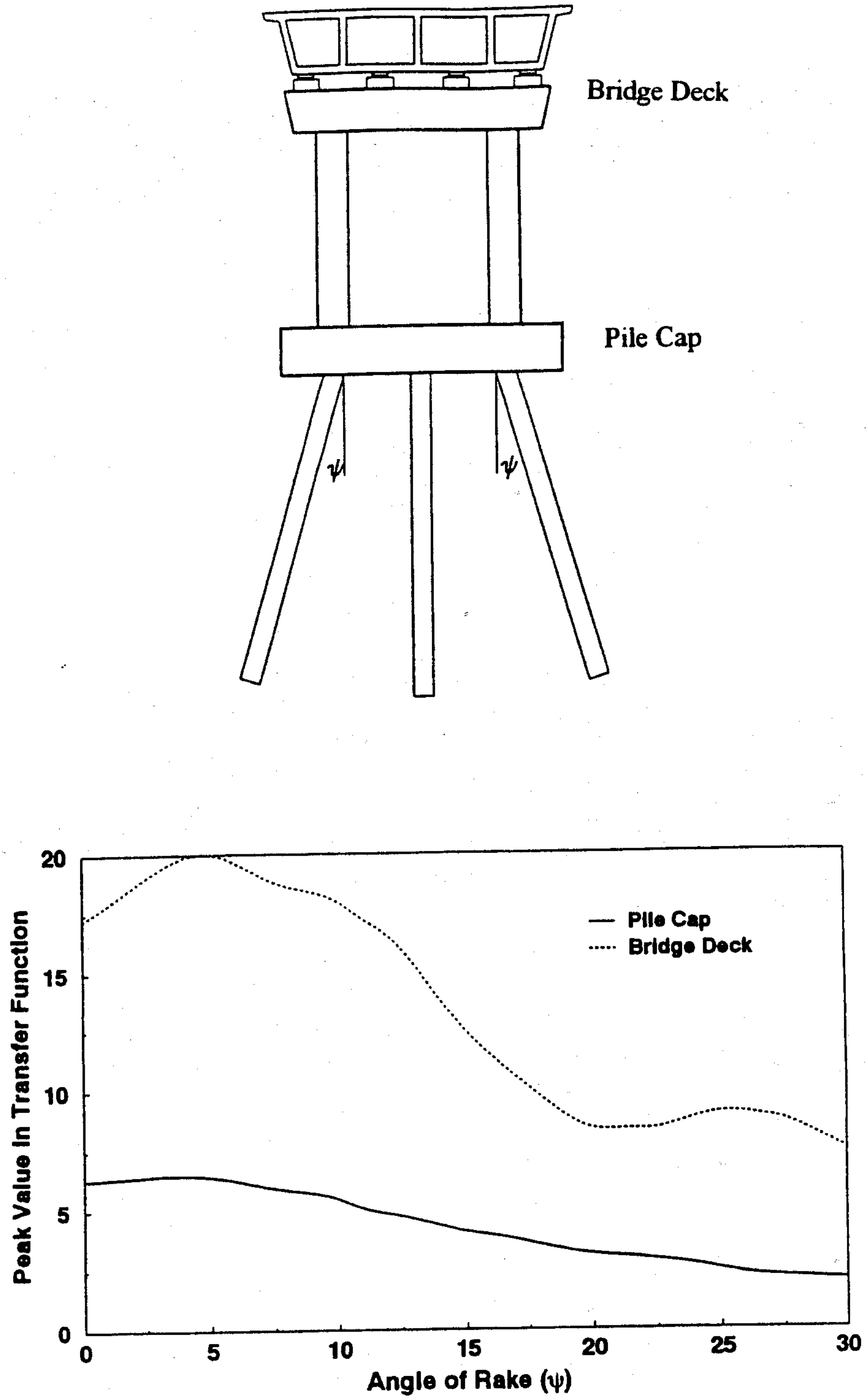


Figure 20. An example of a bridge founded on raked piles. Variation with rake of peak values in the response transfer function (Guin, 1997)

underlying liquefied sand ; the loads imposed by this crust on the pile cap far exceeded the drag forces from the "flowing" sand. Yet, "the motion towards the river was impeded by the buried raked-pile foundations which resisted the lateral spreading of the upper 6 m of soil toward the river channel." (Berrill et al 1997).

It is anticipated that the question of the role of inclined (raked) piles (in stable and in unstable ground) will receive much greater attention than in years past.

5. INELASTIC STRUCTURAL DEFORMATION of PILES: CONSEQUENCES to SUPERSTRUCTURE and the PILE LOAD–CARRYING CAPACITY

Piles in soft soils supporting building or bridge structures can be subjected to large horizontal displacements when subjected to severe earthquakes. The resulting deformations can result in significant curvatures in some region of the piles. One region of high curvature is at the pile-pile cap interface, due to the end fixity of the pile in the pile cap. In addition, for piles embedded in layers of soil, with the soil modulus varying down the depth, the pile curvature may be particularly high at the interface between hard and soft layers of soil, as explained in a previous section of the paper. Severe damage to piles has been observed during some earthquakes.

Because of difficulties with identification and repair of piles after an earthquake, the general design philosophy has been to ensure that piles as structural members remain in the elastic range. By contrast inelastic deformations (plastic hinging) is provided usually at the base of the column, above the foundations, or else seismic energy is dissipated through mechanical and base isolation devices.

However, in some cases it is not economically-possible to avoid inelastic action in the pile, as in the case of a single column-pile bent. The aforementioned kinematic response of a pile, in which large moments develop at the depth where there is an interface of a soft and a hard soil layers, is another case where inelastic actions may be unavoidable.

The currently prevailing view (Joen & Park 1990) is that such regions of a pile (where high curvatures occur) must be designed to possess adequate ductility, where ductility may be defined as the ability to undergo large amplitude cyclic deformations in the post-elastic range without significant reduction in strength. Because complete loss of flexural strength of the pile is invariably accompanied by loss of vertical load carrying capacity.

However, uncertainties exist with regard to soil-structure interaction and the resulting actual pile behavior during a severe earthquake. It would appear to be essential to detail piles of building and bridge structures so as to be capable of a reasonable degree of ductile behavior. Also, in wharf type structures it is desirable to design the piles (which also constitute the columns of the structure) to possess ductile plastic hinge behavior.

Some of the questions that must be raised are as follows:

- Should inelastic action in piles be allowed and under what conditions ?
- If yes, how does one compute the response of an (elastic or inelastic) super-structure supported by a (structurally) inelastic foundation ?
- How is the overall du ctility factor affected by the ductility of the pile ? [μ versus μ_{pile}]
- What are the allowable levels of pile ductility ?

Developing an understanding and obtaining practical solution to these issues will require cooperation between structural and geotechnical engineers. The computational aspects of the problem are not the easiest in view of the double nonlinearity — in the soil and the pile. Some first attempts in that direction have already been reported (Badoni. & Makris 1997).

REFERENCES

Badoni, D. & Makris, N.(1997), "Analysis of the Nonlinear Response of Structures Supported on Pile Foundations", University of California, Berkeley, Earthquake Engineering Research Center.

Barghouthi, A.F. (1984), "Pile response to seismic waves", Ph.D. thesis, Dept. of Civil Engrg., Univ. of Wisconsin-Madison.

Banerjee, P.K. & Butterfield, R. (Eds.) (1987), "Dynamic Behaviour of Foundations and Buried Structures", Elsevier Appl. Science, NY

Banerjee, P.K. & Sen, R. (1987), "Dynamic Behavior of Axially and Laterally Loaded Piles and Pile Groups", Chapter 3, in Dynamic Behavior of Foundations and Buried Structures, Elsevier App. Sc., pp.95-133.

Banerjee, P.K., Sen, R. & Davies, T.G. (1987), "Static and Dynamic Analyses of Axially and Laterally Loaded Piles and Pile Groups", Chapter 7 in Geotechnical Modelling and Applications, pp.322.

Bartlett, S.F., & Youd, T.L., (1992), "Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spreads", NCEER Report 92-0021, Buffalo

Benuzka, L., editor (1990), "Loma Prieta Earthquake Reconnaissance Report", Supplement to Vol.6 of Earthquake Spectra.

Berrill, J.B., Christensen, S.A., Keenan, R.J., & Okada W.(1997), " Lateral-Spreading Loads on a Piled Bridge Foundation" , Seismic Behaviour of Ground and Geotechnical Structures, P.Seco e Pinto, Ed.. Balkema, pp. 173-183.

Blaney, G.W., Kausel, E. & Roesset, J.M. (1976), "Dynamic stiffness of piles", Proc. 2nd Int. Conf. on Numerical Methods in Geomechanics, ASCE, Vol.2,pp. 1001-1012.

Blaney, G. W. and O' Neill, M. W. (1986 b), " Analysis of Dynamic Laterally Loaded Pile in Clay", Jnl. Geotech. Eng. , Proc. ASCE, Vol. 112, No 9, pp. 827-840.

Butterfield, R. & Banerjee, P.K. (1971), "The Elastic Analysis of Compressible Piles and Pile Groups", Geotechnique, Vol. 21, pp.43-60.

Chow, Y.K. (1987), "Axial and Lateral Response of Pile Groups Embedded in Nonhomogeneous Soils:, Int. Jnl. For Numerical and Analytical Methods in Geomechanics, Vol.11, 621-638.

Dobry, R., & Gazetas, G. (1988), "Simple Method for Dynamic Stiffness and Damping of Floating Pile Groups", Geotechnique, Vol. 38, No.4, pp.557-574.

Dobry, R. and Baziar, M.H. (1990), "Permanent Ground Deformations due too Lateral Spreading During Earthquakes", Third Japan-U.S. Workshop on Liquefaction, NCEER Report, Buffalo, pp. 209-223.

Dobry, R., Taboada, V., and Liu, L. (1995), "Centrifuge Modeling of Liquefaction Effects during Earthquakes", Keynote Lecture, Proc. First Int. Conference on Earthquake Geotechnical Engineering, Tokyo, Vol. 3.

Dobry, R., Abdoun, T., and O' Rourke, T.D. (1996), "Evaluation of Pile Response due to Liquefaction-Induced Lateral Spreading of the Ground", Proc. Fourth CALTRANS Seismic Research Workshop, Sacramento, California.

Dobry, R. (1996), "Lateral Spreading Effects on Pile Foundations due to Soil Liquefaction", 11th Word Conference on Earthquake Engineering, Acapulco.

El-Marsafawi, H., Han, Y.C. & Novak, M. (1992), "Dynamic Experiments on two Pile Groups", Jnl. Geotech. Eng., Proc. ASCE, Vol. 118, No.6, pp.839-855.

El-Naggar, M.H. & Novak, M.(1996), " Nonlinear Analysis for Dynamic Lateral Pile Response", Jnl Soil Dynamics and Earthquake Engineering, Vol.15, No.4, pp. 233-244.

Elsabee, F., Morray, J.P. & Roesset, J.R.M. (1977), "Dynamic Behaviour of Embedded Foundations" Research Report R77 - 33, MIT.

Eurocode EC8 (1994), Structures in Seismic Regions, Part 5: Foundations, Retaining Structures, and Geotechnical Aspects

Fan, K., Gazetas, G., Kaynia, A., Kausel, E. & Ahmad, S. (1991), "Kinematic Seismic Response of Simple Piles and Pile Groups", Jnl. of Geotech. Engrg, ASCE, Vol. 117, No. 12, pp. 1531-1548.

Flores-Berrones, R. & Whitman, R.V. (1982), "Seismic Response of End-Bearing Piles", Jnl. Geotech. Engng Div., ASCE, 108, No.4, pp.554-569.

Gazetas, G. & Dobry, R. (1984), "Horizontal Response of Piles in Layered Soils", Jnl. Geotech. Engng., ASCE, Vol. 110(1), 20-40.

Gazetas, G. (1987), "Soil Dynamics : an Overview", Dynamics of Foundations and Buried Structures, Elsevier Applied Science, Chapter I, pp.1-43.

Gazetas, G. (1987), "Simple Physical Methods for Foundation Impedances", Dynamics of Foundations and Buried Structures, Elsevier Applied Science, Chapter II, pp.45-93

Gazetas, G. and Makris, N. (1991), " Dynamic Pile-Soil-Pile Interaction, I : Analysis of Axial Vibration", Jnl. Earthq. Engng. & Stuct. Dyn. , Vol. 20. No. 2.

Gazetas, G., Fan, K., Kaynia, A., & Kausel, E. (1991), " Dynamic Interaction Factors for Floating Pile Groups", Jnl. of Geotech. Engrg, ASCE, Vol. 117, No. 10, pp. 1860-1879.

Gazetas, G., Fan, K., Tazoh, T., Shimizu,K., Kavvadas, M., & Makris, N., (1992), "Seismic Pile-Group-Structure Interaction", Piles under Dynamic Loads, ASCE, S. Prakash, ed., pp. 56-93.

Gazetas, G., Tazoh, T., Shimizu, K., Fan, K. (1993), "Seismic Response of the Pile Foundation of Ohba-Ohasi Bridge" Proc. 3rd Int. Conf. on Case Histories in Geotech. Engrg.,pp. 1803-1809.

Gazetas. G., Fan, K.,& Kaynia A. (1993), "Dynamic Response of Pile Groups with Different Configurations", Soil Dynamics & Earthq. Engng., 12, pp. 239-257.

Gazetas. G., Mylonakis G., & Nikolaou A. (1995), " Simple Methods for the Seismic Response of Piles, Applied to Pile-Bridge Interaction", Proc. 3rd International Confer. on Recent Advances in Geotech. Earthq. Engrg. and Soil Dynamics, St. Louis, Vol. III.

Gerolymos .N., (1997), "Effective Fundamental Period and Damping of Pile-Supported Bridge Piers", Diploma Thesis, National Technical University, Athens.

Guin. J., (1997), " Advances in Soil-Pile-Structure Interaction and Nonlinear Pile Behavior", PhD Thesis, State University of New York, Buffalo

Gohl. B. (1993), "Response of Pile Foundations to Earthquake Shaking General Aspects of Behaviour and Design Methodologies", Seismic-Soil Structure Interaction Seminar, Vancouver, B.C.

Hamada, M., Isoyama, R., and Wakamatsu, K. (1996), "Liquefaction-Induced Ground Displacement and its Related Damage to Lifeline Facilities", Soils and Foundations, Special Issue on Geotechnical Aspects of the January 17, 1995 Hyogo-Ken Nambu Earthquake, pp. 81-97.

Harada, T., Kubo, K. & Katayama, T. (1981). "Dynamic Soil-Structure Interaction by Continuum Formulation Method", Report No. 190, Inst. of Industrial Science, University of Tokyo.

Hatanaka, M., Uchida, A., and Ohara, J. (1997), "Liquefaction Characteristics of a Gravelly Fill Liquefied during the 1995 Hyogo-ken Nambu Earthquake", Soils and foundations, Vol. 37, No.3, pp. 107-115.

Ishihara, K. Yoshida, K., and Kato, M. (1997), "Characteristics of Lateral Spreading in Liquefied Deposits during the 1995 Hanshin-Awaji Earthquake", Journal of Earthquake Engineering, Vol. 1, No.1, pp. 23-55, Imperial College Press.

Ishihara, K. (1997), "Geotechnical Aspects of the 1995 Kobe Earthquake", Terzaghi Oration 14th ICSMFE, Hamburg.

Inagaki, H., Iai, S., Sugano, T., Yamazaki, H., and Inatori, T. (1996), "Performance of Caisson Type Quaywalls at Kobe Port" Soils and Foundations, Special Issue on Geotechnical Aspects of the January 17, 1995 Hyogo-Ken Nambu Earthquake, pp. 119-136.

Iwasaki, T. (1996), "Perspectives of Seismic Design Criteria for Highway Bridges in Japan", Special Theme Session on Seismic Design of Bridges, 11th Word Conference on Earthquake Engineering, Acapulco, Mexico.

Jennings, P.C. and Bielak J., (1973), "Dynamics of Building-Soil Interaction," Bull. Seism. Soc. Am., Vol.63, No.1, pp.9-48.

Joen H.P. & Park R. (1990), "Simulated Seismic Load Tests on Prestressed Concrete Piles and Pile-Pile Cap Connections", PCI Journal.

Kausel, E. (1974), "Forced Vibrations of Circular Foundations on Layered Media", Res. Rept. R74-11, MIT., Cambridge, MA.

Kausel, E. & Roesset, J.M. (1974), "Soil- Structure Interaction for Nuclear Containment Structures", Proc. ASCE, Power Division Specialty Conference, Boulder, Colorado.

Kausel. E. & Roesset, J.M. (1975), "Dynamic Stiffness of Cylindrical Footings", ASCE, Jnl. Engng Mechanics Div. 101(EM6), pp. 771-785.

Kausel. E. and Peek, R. (1982), "Boundary Integral method for Stratified Soils", Res. Re. R82-50, MIT, Cambridge, MA.

Kavvadas. M. & Gazetas, G. (1993), "Kinematic Seismic Response and Bending of Free-Head Piles in Layered Soil", Geotechnique, Vol. 43, No. 2, pp.207-222.

Kaynia, A.M. & Kausel, E. (1982), "Dynamic Behavior of Pile Groups", Proc. 2nd Int. Conf. on Numerical Methods in Offshore Piling,Texas Univ., Austin,pp.509-532.

Kaynia, A.M. and Novak,M. (1992),"Response of Pile Foundations to Rayleigh Waves and Obliquely Incident Body Waves", Jnl. Earthq. Eng. and Struct. Dyn., Vol.21, No. 4.

Keenan P.R., (1996), " Foundation Loads due to Lateral Spreading at the Landing Road Bridge, Whakatane", Department of Civil Engineering , University of Canterbury , Christchurch New Zealand.

Korobi, T., Nakasawa, M., Hijikata, K., Kobayashi, Y., Miura, K., Miyamoto, Y. and Moroi, T. (1991), " Study on Dynamic Characteristics of a Pile Group Foundations", Proc. 2nd Int. Conf. on Recent Advances in Geotech. Earthq. Engrg. and Soil Dynam., St. Louis.

Kuhlemeyer, R.L. (1979), "Static and Dynamic Lterally Loaded Floating Piles", Jnl. Geotech. Eng., Proc. ASCE, Vol. 105, GT2, pp.289-304.

Lam Po I. (1995), "Design Procedure for Drilled Shafts", in Seismic Vulnerability of New Highway Construction, NCEER Research Report, Buffalo, New York.

Lysmer, J., Tabatabaie, M., Tajirian, F., Vahdani, S., & Ostadan, F. (1981), "SASSI, a System for Analysis of Soil-Structure Interaction", UCB/GT/81-02, University of California, Berkeley.

Luco, J.E. (1969), "Dynamic Interaction of Shear Wall with the Soil", Jnl. Engrg. Mech. Div. ASCE, Vol. 95, pp. 333-346.

Luco, J.E. (1974), "Impedance Functions for a Rigid Foundation on a Layered Medium", Nuclear Engrg. and Design, Vol. 31, 204 - 217.

Makris, N. & Gazetas, G. (1992), "Dynamic Pile-Soil-Pile Interaction. Part II: Lateral and Seismic Response", Earthquake Engineering and Structural Dynamics, Vol. 21, pp.145-162.

Mamoon, S.M., & Banerjee, P.K. (1990), "Response of Piles and Pile Groups to Travelling SH-waves", Earthquake Engineering and Structrular Dynamics, Vol. 19l, No.4.

Matlock, H. & Reese, L.C. (1960)," Generalized Solutions for Laterally Loaded Piles", ASCE, Jnl. Soil Mech. and Found. Div.,Vol. 86 SM5, pp.63-91.

Matlock, H. (1970), "Correlations for Design of Laterally Loaded Piles", ASCE, Jnl. Soil Mech. and Found. Div., SM5, pp. 63-91.

Matlock, H., Foo, H.C. & Bryant, L.M. (1978),. "Simulation of Lateral Pile Behaviour Under Earthquake Motion", Proc. ASCE Specialty Conf. on Earthq. Eng. & Soil Dyn., Vol. II, pp. 600-619.

Matsui, T. and Oda, K. (1996), "Foundation Damage of Structures", Soils and Foundations, Special Issue on Geotechnical Aspects of the January 17, 1995 Hyogo-ken Nambu Earthquake, pp. 189-200

Michaelides, O., Gazetas, G., Bouckovalas, G., & Chrysikou, E. (1998), "Approximate Nonlinear Dynamic Axial Response of Piles", Geotechnique, 48, pp. 33-53.

Michaelides, O.,& Gazetas, G. (1995), "Impedance Functions of Piles in Inhomogeneous Media", Jnl. of Geotech. Engng. ASCE, Vol. 121, No. 2, pp. 234-236.

Mizuno, H. (1987), " Pile Damage During Earthquake in Japan (1923-1983)", Dynamic Response of Pile Foundations, ASCE, Geotech. Special Publ. No. 11 (ed. by Nogami),pp. 53-78.

Mylonakis, G. (1995), "Contributions to Static and Seismic Analysis of Piles and Pile-Supported Bridge Piers", Ph.D. Thesis, State University of New York at Buffalo.

Mylonakis, G., Nikolaou, A., & Gazetas, G. (1995), "Soil-Pile-Bridge Seismic Interaction: Kinematic and Inertial Effects, Part I: Soft Soil", Earthquake Engineering & Struc. Dynamics, Vol. 26, 337-360 Technical Report, Buffalo, NY.

Nikolaou, A., Mylonakis, G., & Gazetas, G. (1995), "Kinematic Bending Moments in Seismically Stressed Piles", NCEER Technical Report, Buffalo, NY.

Nikolaou, A., & Gazetas, G. (1997), "Seismic Design Procedure for Kinematically Stressed Piles", Seismic Behaviour of Ground and Geotechnical Structures, Seco e Pinto (ed.), Balkema.

Nogami, T. (1985),"Flexural Responses of Grouped Piles Under Dynamic Loading", Earthquake Engineering & Structtural Dynamics, vol. 13, pp.321-336.

Nogami, T. (Ed.) (1987), Dynamic Response of Pile Foundations- Experiment, Analysis and Observation, ASCE, Geotech. Special Publication, No. 11, pp.186.

Novak, M. (1977), "Vertical Vibration of Floating Piles", Jnl. Eng. Mech. Div. ASCE, Vol. 103, EM1,pp. 153-168.

Novak, M. & Nogami, T. (1977), "Soil Pile Interaction in Horizontal Vibration", Earthq. Engng. Struct. Dyn., vol.5, pp.263-282.

Novak, M. & Nogami, T. & Aboul-Ella, F.(1978), "Dynamic Reactions for Plane Strain Case" Jnl. Engineering Mech. Div. ASCE, 104, 953-959

Novak, M. & Sheta, M. (1980), "Approximate Approach to Contact Problems of Piles", ASCE: Dynamic Response of Pile Foundations: Analytical Aspects, pp.53-79.

Oh-oka, H., Onishi, K., Nanba, S., Mori, T., Ishikawa, K., Koyama, S. and Shimizu, S. (1997), "Liquefaction-Induced Failure of Piles in 1995 Kobe Earthquake", Proc. 3rd Kansai Forum on Comparative Geotechnical Engineering (KIG-Forum'97), pp. 265-274.

Pender, M. (1993), "A seismic Pile Foundation Design Analysis", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol 26, No. 1, pp.49-160.

Penzien, J. (1970),"Soil-Pile Foundation Interaction", Earthquake Engineering, Ch. 14, Prentice-Hall, pp.349-381.

Poulos, H.G. (1968), "Analysis of the Settlement of Pile Groups", Geotechnique 18, No. 4, 449-471.

Poulos, H.G. (1971), "Behaviour of Laterally-Loaded Piles, II: Pile Groups", Jnl. Soil Mech. & Found. Engrs, ASCE 97, SM5, pp. 773-751.

Poulos, H.G. & Tabesh A. (1996), "Seismic Response of Pile Foundations-some Important Factors", 11th World Conference on Earthquake Engineering, Acapulco, No.2085.

Prakash, Shamsher (Editor) (1992), "Piles Under Dynamic Loads". Geotech. Special Publication, No. 34., NY.

Reese, L.C., Cox, W.R. & Koop, F.D. (1974)," Analysis of Laterally Loaded Piles in Sand". Proc. 6th Offshore Technology Conference, Houston, Texas, Paper 2080, pp.473-483.

Roesset, J.M. (1977). "Soil Amplification of Earthquakes", Numerical Methods in Geotechnical Engineering, Mc Graw Hill, C.S. Desai & J.T. Christian, eds, Ch. 19, 639 - 682.

Sanchez-Salinero,I. (1983), "Dynamic Stiffness of Pile Groups: Approximate Solutions", Geotech. Engng. Rep. GR83-5, Univ. Texas, Austin.

Sen, R., Davies,T.G. and Banerjee, P.K. (1985), "Dynamic Analysis of Piles and Pile Groups Embedded in Homogeneous Soils", Earthquake Engineering & Structyral Dynamics, Vol. 13, pp. 53-65.

Sen, R., Kausel, E. and Banerjee, P.K. (1985), "Dynamic Analysis of Piles and Pile Groups in Non- Homogeneous Soils", Int. Jnl. Numerical and Analytical Methods in Geomechanics, Vol. 9, pp. 507-590.

Tabesh A & Poulos H.G. (1997), "A review of Dynamic Analysis of Pile Foundations", Report R757 Univ. Syndney.

Takemiya, H. and Yamada, Y. (1981), "Layered Soil-Pile-Structure Dynamic Interaction" Earthquake Engineering and Structural Dynamics , Vol. 9, pp. 437-458.

Tazimi, H. (1966-1969), "Earthquake Response of Foundation Structures", Rep. of Fac. Sci. Engng., Nihon Univ. 1966.3, 1.1-3.5 (in Japanese)

Tazoh, T., Wakahara, T., Shimuzu, K. & Matsuzaki, M. (1988a), "Effective Motion of Group Pile Foundations", Proc. 9th, WCEQE, Tokyo-Kyoto, Vol. III, pp.587-592.

Tazoh, T., Shimuzu, K., & Wakahara, T. (1988b), "Seismic Observations and Analysis of Grouped Piles", Dynamic Response of Pile Foundations-Experiment, Analysis and Observation, Geotechnical Special Publication, No. 11, ASCE. pp.1-20.

Tazoh, T., & Gazetas, G. (1996), "Pile Foundations Subjected to Large Ground Deformations : Lessons from Kobe and Research Needs", 11th World Conference on Earthquake Engineering, Acapulco No.2081.

Tokimatsu, K., Oh-oka, H., Samoto, Y., Nakazawa, A., and Asaka, Y. (1997), "Failure and Deformation Modes of Piles Caused by Liquefaction-Induced Lateral Spreading in 1995 Hyogo-ken Nambu Earthquake", Proc. 3rd Kansai International Forum on Comparative Geotechnical Engineering (KIG-Forum'97), pp. 239-248

Tassoulas, J.L. (1984). "An Investigation of the Effect of Rigid Sidewalls on the Response of Embedded Circular Foundations to Obliquely-Incident SV and P waves", Dynamic Soil-Str. Interaction, A.A. Balkema, pp. 55 - 63.

Veletsos, A.S. and Meek J.W., (1974), "Dynamic Behavior of Building-Foundation Systems", Int. J. Earthquake Eng. Struct. Dynamics, Vol. 3, No.2, pp.121-138.

Veletsos, A.S. and Doston, K.W. (1986), "Impedances of Soil Layer with Disturbed Boundary Zone", Jnl. Geotech. Engng , ASCE, Vol. 112, No.3.

Velez, A.., Gazetas, G. and Krishnan, R. (1983), "Lateral Dynamic Response of Constrained-Head Piles", Jnl. Geotech. Engng., ASCE, Vol. 109, No. Gt8, pp. 1063-1081.

Waas, G., and Lysmer, J. (1972), "Vibrations of Footings Embedded in Layered Media", Applications of the Finite Element Method in Geotechnical Engineering, Desai, C.S.(ed), U.S. Army Enineer, Waterways Experiment Station, Vicksburg, MS, Vol. II, pp. 581-604.

Waas, G. and Hartmann, H.G. (1981), " Pile Foundations Subjected to Dynamic Horizontal Loads", European Simulation Meeting on Modelling and Simulation of Large Scale Structural Systems, Capri, Italy, Sept., pp.17.

Waas, G. and Hartmann, H.G. (1984), "Seismic Analysis of Pile Foundations Including Pile-Soil-Pile Interaction", Proc. 8th WCEE, San Francisco, Vol. 5, pp.55-62.

Whitman, R.V.(1972), "Analysis of Soil-Structure Interaction : State-of-the-Art Review", Experimental and Structural Dynamics, Inst. of Sound and Vibration, Southampton.

Wolf, J. P. & Von Arx G.A. (1978), "Impedance Function of a Group of Vertical Piles", Proc. of the Specialty Conf. on Earthq. Engng. and Soil Dynamics, ASCE, Pasadena, Calif. Vol. 2, pp.1024-1041.

Wolf, J.P. (1995), "Simple Physical Models for Foundation Dynamics", State-of-the-art paper, 3rd Int. Conf. on Recent Advances in Geotech. Earthq. Engrg., St. Louis, Missouri, Vol. 2, pp.943-984.

Youd, T.L., and Benett, M.J. (1983), "Liquefaction Sites, Imperial Valley, California," Journal of Geotechnical Engineering, Vol. 109, No.3, pp. 440-457.

Youd, T.L. (1993), "Earthquake-Induced Ground Failure Hazards", Transportation Research Record, published by the Transportation Research Board and the National Research Council, Washington, D.C., No. 1411, pp.35-41