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KINEMATIC BENDING MOMENTS IN PILES: DEVELOPMENTS FOR NEW-CODE REVISIONS

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ABSTRACT

The kinematically-induced bending moments on single free-head and fixed-head piles are studied in both the *frequency* and *time* domains. The pile is embedded in a two-layer deposit and the excitation consists of vertically-propagating waves producing a specified acceleration at the base of the deposit.

KEYWORDS

Piles; dynamics; waves; earthquake; bending moment; seismic codes

INTRODUCTION

Until recently, piles were designed to transmit safely ***only*** the inertial loads generated by the oscillation of the superstructure. Mizuno (1987) was the first to document a number of pile flexular failures at locations too deep to be influenced by loading from the pile top, and 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 strength and, particularly, stiffness discontinuities of the soil profile. The most likely cause is the relatively large curvatures imposed by the surrounding soil as it deforms while excited by upwards and downwards (after reflection) propagating seismic waves. This type of deformation is called *kinematic* to distinguish from the deformations generated by the inertial forces on the suprstructure.

ANALYSIS AND RESULTS

This mode of deformation has been studied extensively by Tazoh et al (1988), Gazetas *et al* (1992), Kavvadas & Gazetas (1993). Recently the authors have extended the Beam-on-Dynamic-Winkler-Foundation model of Makris & Gazetas (1992) and Kavvadas & Gazetas (1993) and have obtained a comprehensive set of parametric results. A very brief summary of the main findings of these studies are given herein, with the help of **Figures 1** and **2.** 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 pertinent conclusions that have emerged are as follows:

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

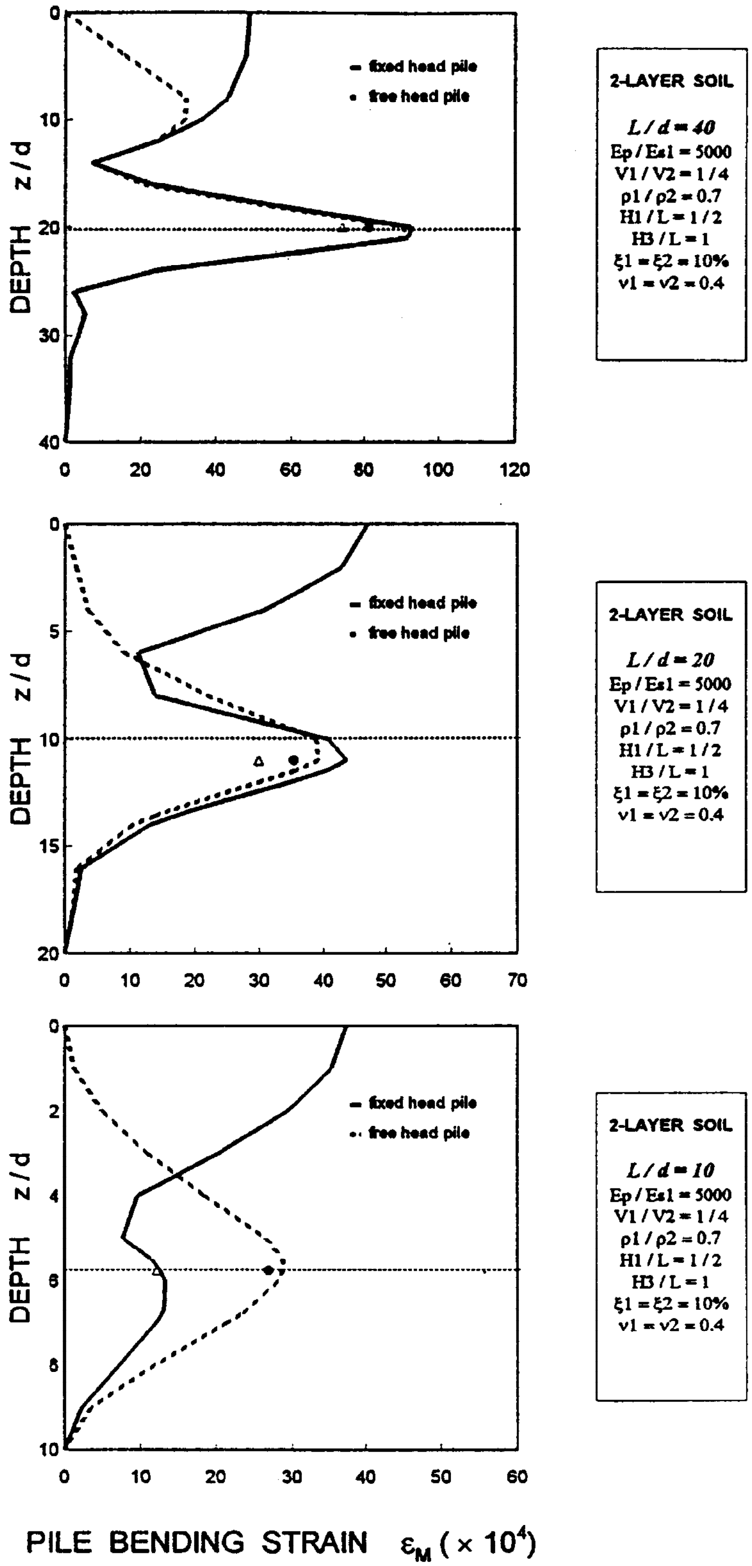


Fig. 1 : Envelopes of steady-state bending strain amplitudes for harmonic 1-m/s^2 excitation of piles with different slenderness ratios (L/d = 10, 20, 40)

- 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 their 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***

- 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_1 / 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: $\ell_a \approx 1.75\, d\, (E_p / E_{s1})^{0.25}$] 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 large bending moments at the pile head 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 decline when moving away from resonance. The ratio of the maximum pile moment at resonance (whether it occurs at the top of the soil deposit or at the layer interface) divided by the respective static moment [i .e., the ratio $\max M (T_1) / M(T)$] follows, more or less, the free-field amplification of acceleration (i.e., the ratio : a_{ff} / a_r). This shows the great influence of the first mode of vibration on the magnitude of the bending 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 ($V_1 / V_2 < 0.25$).

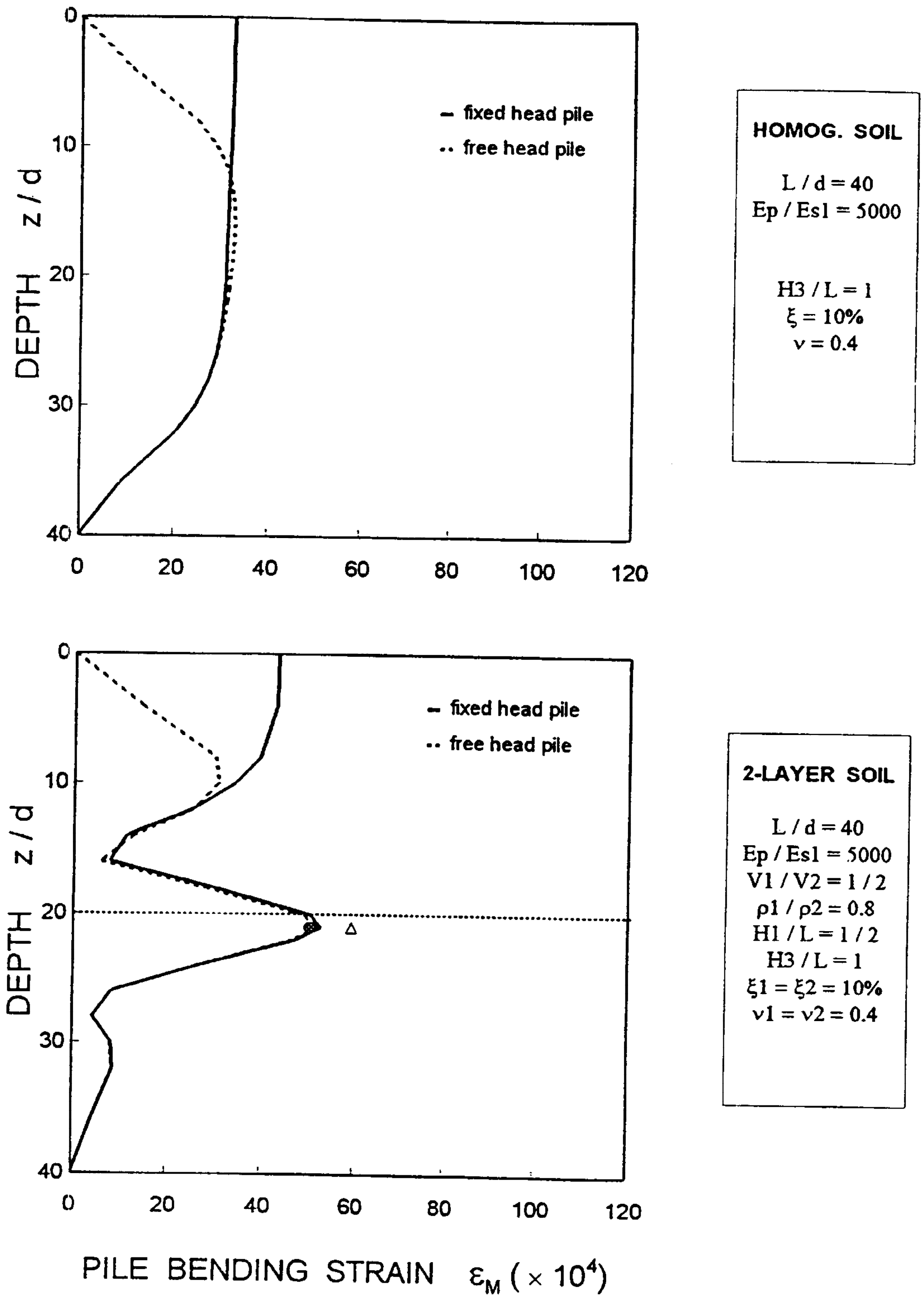
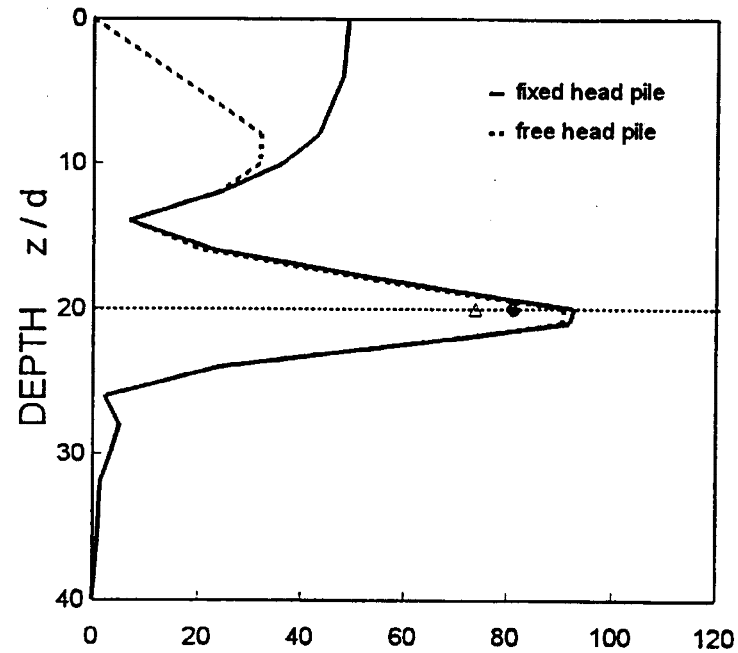
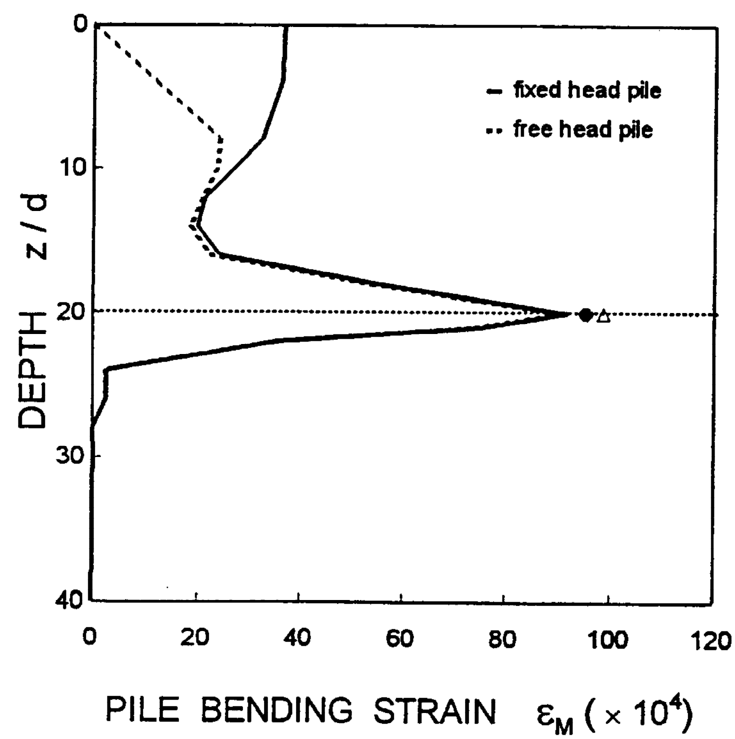


Fig. 2a : Envelopes of pile bending strain amplitudes for harmonic steady-state 1-m/s^2 excitation Effect of soil profile



2-LAYER SOIL

$L / d = 40$
$Ep / Es1 = 5000$
$V1 / V2 = 1 / 4$
$\rho 1 / \rho 2 = 0.7$
$H1 / L = 1 / 2$
$H3 / L = 1$
$\xi 1 = \xi 2 = 10\%$
$\nu 1 = \nu 2 = 0.4$



2-LAYER SOIL

$L / d = 40$
$Ep / Es1 = 5000$
$V1 / V2 = 1 / 10$
$\rho 1 / \rho 2 = 0.6$
$H1 / L = 1 / 2$
$H3 / L = 1$
$\xi 1 = \xi 2 = 10\%$
$\nu 1 = \nu 2 = 0.4$

Fig. 2b : Envelopes of pile bending strain amplitudes for harmonic steady-state 1-m/s^2 excitation Effect of soil profile

Time versus Frequency-Domain Bending Moments

Under transient seismic excitation, described through actual and artificial accelerograms applied at the base of the deposit, the authors have found that the above conclusions are still valid, with one exception : the peak values of the transient bending moments are smaller than the steady-state amplitudes. An example is given in **Figure 3** , 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 6 m 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.

A correlation can beproposed between the largest values of bending moments, in the time and frequency domains:

$$\max M(t) = \eta \cdot \max M(\omega) \tag{1}$$

where : *max M(t)* is the maximum pile bending moment in the time domain and *maxM(ù)* is the maximum steady-state pile bending moment, which usually corresponds to a frequency equal to the funamental frequency of the deposit. The frequency-to-time reduction η takes values between about *0.15* and *0.50*, depending on:

- o the duration of the accelerogram in terms of the number of strong motion cycles N_{cycles}
- o the relative frequency characteristics between the excitation "signal" and the pile moment transfer function; as a first approximation, this relation is expressed in terms of the ratio T_p / T_s (i.e. "predominant" earthquake period / fundamental period of the soil deposit), and
- o the sharpness of the pile-moment transfer function, expressed by the ratio ***maxM(T) / M (T)*** which is directly related to the effective damping ratio ξ_{eff} of the pile-soil system.

Thus,

$$\eta = \eta\left(N_{cycles},\ T_p / T_s,\ \xi_{eff} \right) \tag{2}$$

Quantification of the variables N_{cycles} and T_p requires pertinent seismological input.

CONCLUSION

The kinematic interplay between soil and pile may be quite important in the seismic performance of a piled foundation. It is perhaps worthy of note that the importance of kinematic loading has been recognized in the recently published Eurocode 8, dealing with the seismic design of civil structures. Part 5 of that Eurocode states:

> *"Piles shall be designed for the following two loading conditions:*
>
> *(a) inertia forces from the superstructure . . .*

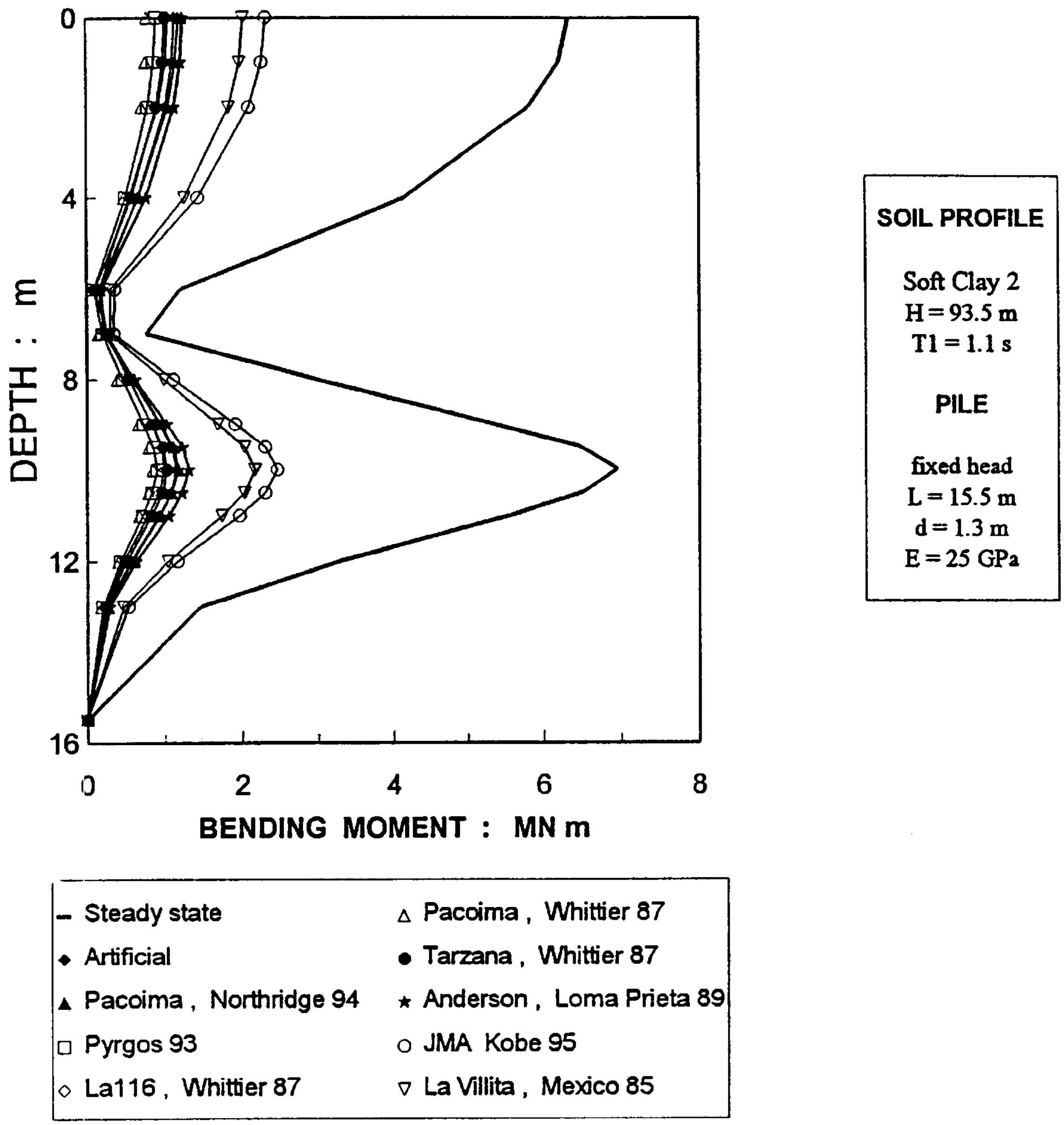


Fig. 3 : Soft-Clay Profile : Comparisons of envelopes of pile peak bending moments for various earthquake excitations versus the maximum steady-state harmonic response curve

(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 kinematic loading may be particularly large at interfaces of soil layers with sharply differing shear moduli. The design must ensure that no "plastic hinge" develops at such locations . . . . "***

These findings from theoretical studies on pile distress during earthquakes have been largely confirmed by full-scale measurements in Japan during earthquakes (Tazoh et al , 1988 a and b; Gazetas et al , 1993).

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