Seismic response of buildings with pile uplift

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ABSTRACT: The effect of pile tip uplift and pile cap uplift on seismic forces on buildings and piles is examined and the need for pile socketing and rigid attachment of pile heads to the cap is evaluated. Various pile configurations are assumed and different ground acceleration time histories are considered.

1 INTRODUCTION

The seismic response of pile supported buildings depends on the type of attachment of the pile to the cap and also on pile socketing into the bearing stratum. These measures are costly and their actual usefulness is often questioned by consultants who have to implement them according to the requirements of some codes, e.g. The National Building Code of Canada. The paper examines the effect of pile socketing and their connection to the cap in a tension resisting fashion. The data presented in this paper complement a recent study by the authors (El Hifnawy and Novak, 1986) which was limited to narrow buildings and one earthquake ground motion. Broader buildings and a range of earthquake signals are considered in this paper.

2 BUILDINGS, FOUNDATIONS AND GROUND MOTIONS

2.1 The buildings

The situations considered are schematically depicted in Fig. 1 indicating (a) no uplift,

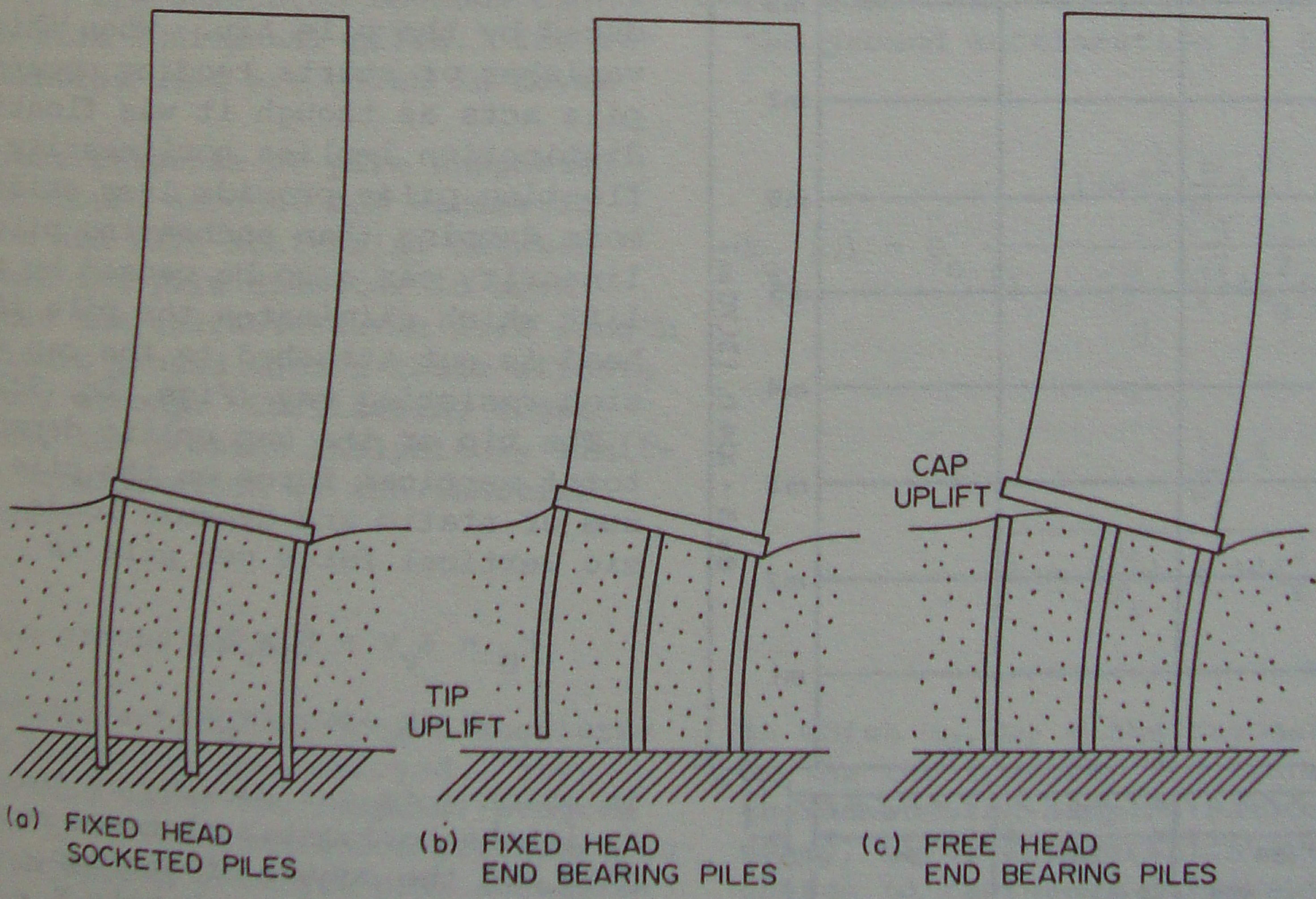


Fig. 1 Types of pile arrangement

(b) uplift of the pile tip and (c) uplift of the pile cap. The building has n floors and is either a shear building or a general building with a condensed stiffness matrix. With cap sliding and rotation (rocking), the building has n+2 degrees of freedom under horizontal excitation.

Structural damping is assumed to stem from interstorey hysteretic damping with material damping ratio β, and is approximated by equivalent viscous damping constants c_{ij} proportional to flexural stiffness k_{ij} such that $c_{ij} = 2\beta k_{ij}/\omega_1$, where ω_1 is the fundamental frequency of the building on a rigid foundation.

The specific building considered is shown in Fig. 2. The building is 30 m x 7.5 m in plan with 5 reinforced concrete columns of the cross-sectional area 0.8 m x 0.8 m. The numerical values of the building properties are given in Table 1. The structural deformation is assumed not to exceed the linear range.

The pile foundation. - The pile foundations comprise groups of endbearing piles whose number ranges from 18 to 44 piles. The piles are of reinforced concrete and are pinheaded. They are .75 ft (.23 m) in radius with a pile length to diameter ratio of 40. The soil is assumed to be homogeneous with shear wave velocity of 200 ft/s (81 m/s), Poisson's ratio $\nu = 0.25$, material damping ratio .025 and mass density $\rho = 3.1$ slug/ft^3 (1600 kg/m^3). Impedance functions of the piles K_i are formulated using the tables and charts given by Novak

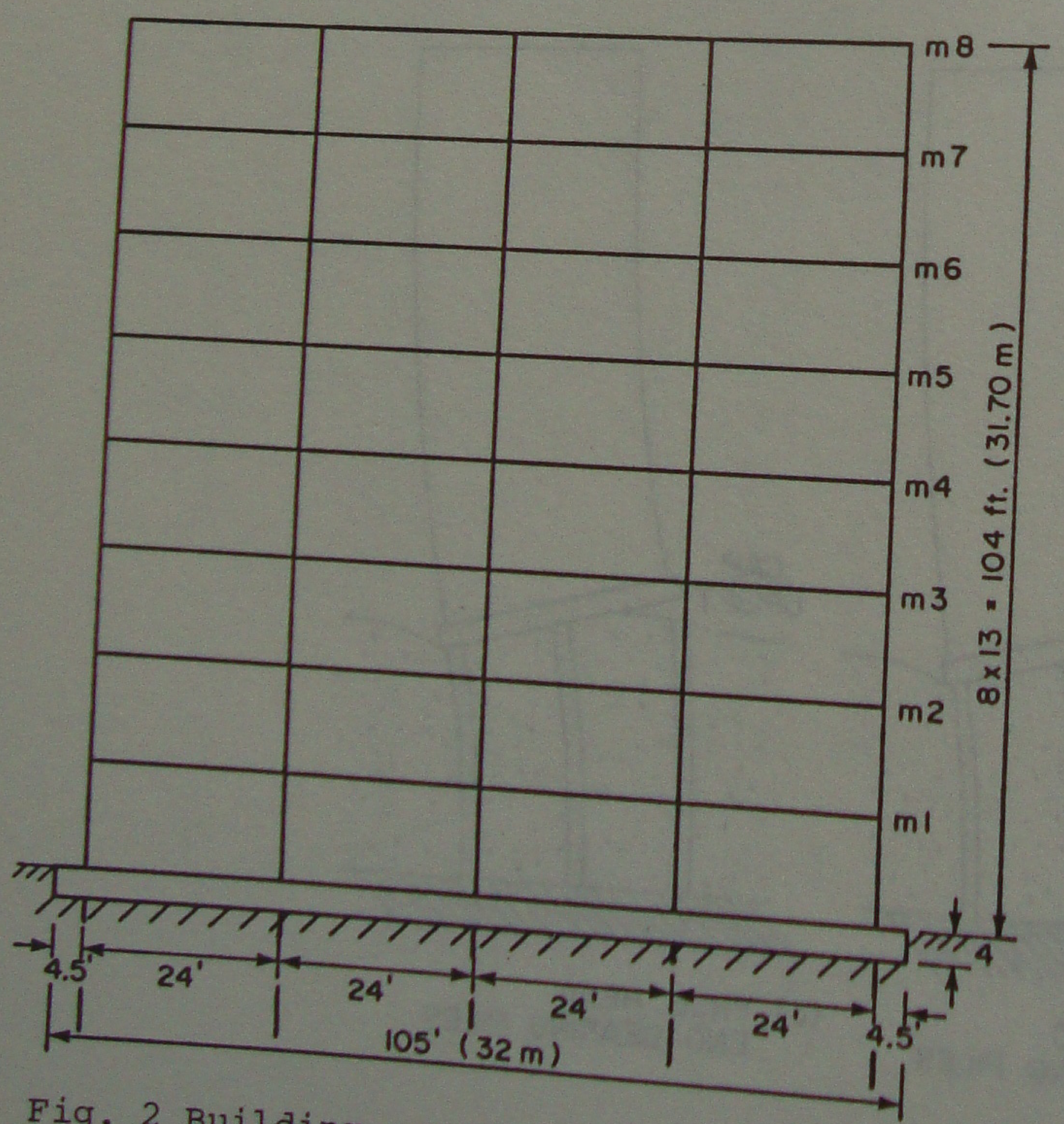


Fig. 2 Building assumed

Table 1. Building properties (one segment)

Number of storeys (n)	8
Floor mass (m_i)	$m_1 = 125 \times 10^3$ kg= 8540 slugs; $m_{2-8} = 120 \times 10^3$ kg= 8120 slugs
Base mass (m_b)	229.1×10^3 kg = 15652.17 slugs
Base moment of inertia (I_b)	19582353 kgm^2 = 14401303 slug. ft^2
Storey height	3.96m = 13 ft
Storey stiffness	988.06×10^6 N/m= 67.71×10^6 lb/ft
Fundamental frequency (ω_1) for fixed base structure	16.847 rad/sec

and El Sharnouby (1983).

The situations at the pile head or tip considered in the analysis are schematically depicted in Fig. 1. According to these situations, the vertical impedance functions of piles may undergo changes indicated diagrammatically in Fig. 3. The symbols k_v and c_v indicate respectively the vertical stiffness and damping constants and v is butt displacement. Only for socketed piles with tension resisting connection to the cap are the stiffness and damping in compression the same as in tension (Figs. 1a and 3a). For endbearing piles with a rigid connection to the cap, the pile tip may uplift due to the effect of overturning moment (Fig. 3b). Thus each pile is treated as an endbearing pile as long as there is a downward end force produced by the pile tip. When this force vanishes or starts tending upwards, the pile acts as though it was floating. This distinction implies nonlinearity because floating piles provide less stiffness but more damping than endbearing piles. Nonlinearity can also be caused by the cap uplift which eliminates the pile if the pile head is not attached to the cap in a tension resisting way (Figs. 1c, 3c).

The tip or the cap uplift depend on the total vertical force on the pile being the sum of static and dynamic loads. The dynamic vertical force per pile is

$$F_D = K_v v = K_v x_r \psi$$
$$= (k_v + i\omega c_v) x_r \psi \quad (1)$$

in which ψ is the rocking of the cap and x_r is the horizontal distance between the pile and the centre of the cap. Then the total average force per pile F_t, becomes

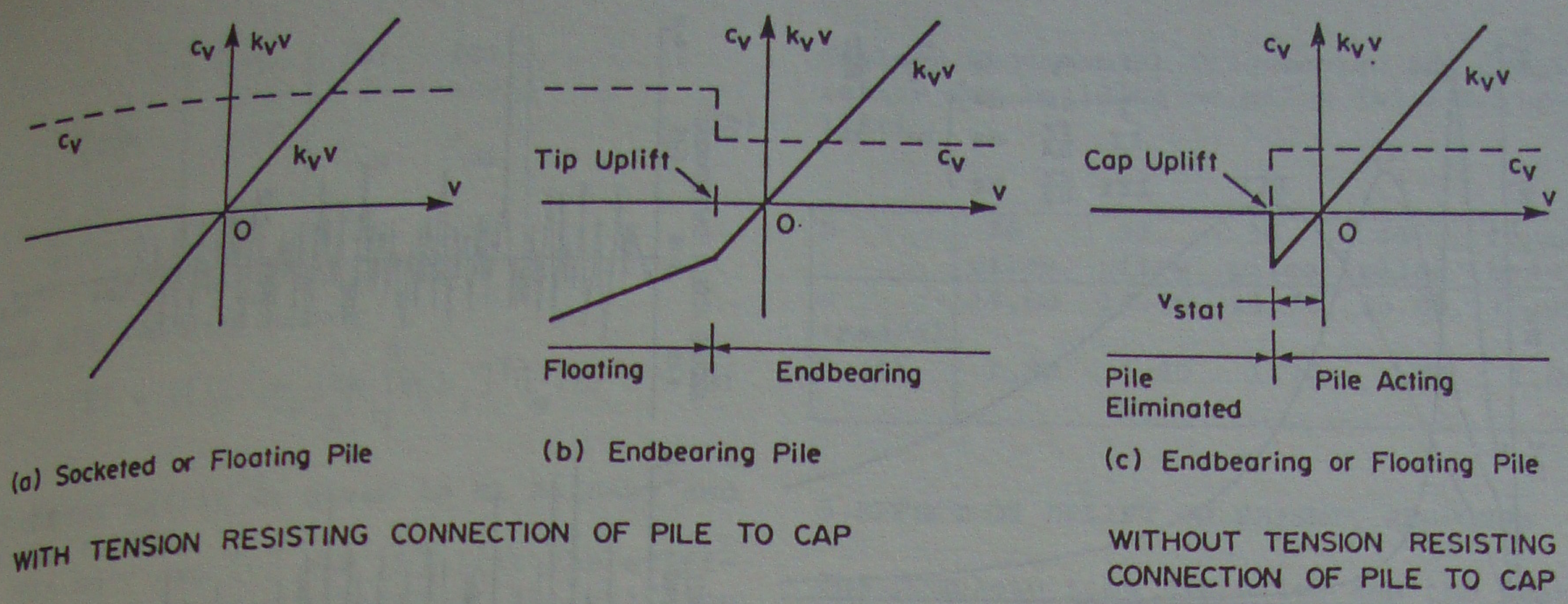


Fig. 3 Schematic of pile vertical force $k_v v$ and damping coefficient c_v vs. vertical displacement v corresponding to the situations shown in Fig. 1

$$F_t = F_D + (m_b + \sum_{i=1}^{N} m_i)\ g/N \qquad (2)$$

where g is gravity acceleration and N is the number of piles. When the total force in the pile vanishes or tends to become tensile, the pile is treated as floating for the case of endbearing piles with a rigid connection to the cap, or the pile is eliminated for the case of piles without a rigid connection to the cap.

The evaluation of the impedance functions of the whole group of piles should incorporate the effect of pile-soil-pile interaction but this is neglected. Then, the group stiffness constants k_{ij} and damping constants c_{ij} are evaluated as sums of contributions from individual piles. The resulting formulae used are given in Novak (1974).

The seismic response of pile supported structures should account for kinematic interaction and wave scattering between piles. When the pile diameter is much smaller than the characteristic wave length of the seismic ground motion these effects are not very strong. Considering these assumptions, the present analysis is limited to the consideration of the inertial interaction .

2.2 Seismic ground motion

In this study, artificial ground acceleration time histories are employed. They were generated using the computer program SIMQKE (1976). The simulation procedure in the program allows a target peak acceleration and a spectral density function to be specified. The method used by the program for artificial motion generation is based on the superposition of sinusoids having random phase angles and amplitudes derived from a stationary power spectral density function of the motion. To simulate the transient character of a real earthquake, the steady state motions are multiplied by a deterministic envelope function. The final simulated motion is, then, stationary in frequency content with a peak acceleration close to the target peak acceleration.

One form of the power spectrum of ground acceleration was suggested by Clough and Penzien (1975). These authors extended the formula due to Kanai (1957) by passing a white-noise bedrock acceleration, whose spectrum is S_o, through two filters (transfer functions) to obtain the spectrum of the ground acceleration in the form

$$S_{\ddot{u}_g}(\omega) = S_o \frac{[1+4\xi_g^2(\frac{\omega}{\omega_g})^2]}{[1-(\frac{\omega}{\omega_g})^2]^2+4\xi_g^2(\frac{\omega}{\omega_g})^2} \cdot \frac{(\frac{\omega}{\omega_f})^4}{[1-(\frac{\omega}{\omega_f})^2]^2+4\xi_f^2(\frac{\omega}{\omega_f})^2} \qquad (3)$$

in which ω_g, ω_f = the resonant frequencies of the two transfer functions and ξ_g, ξ_f = the associated damping ratios. For this study, three acceleration spectra are chosen to represent different earthquakes. They are shown in Fig. 4 in which their parameters are also given. (These

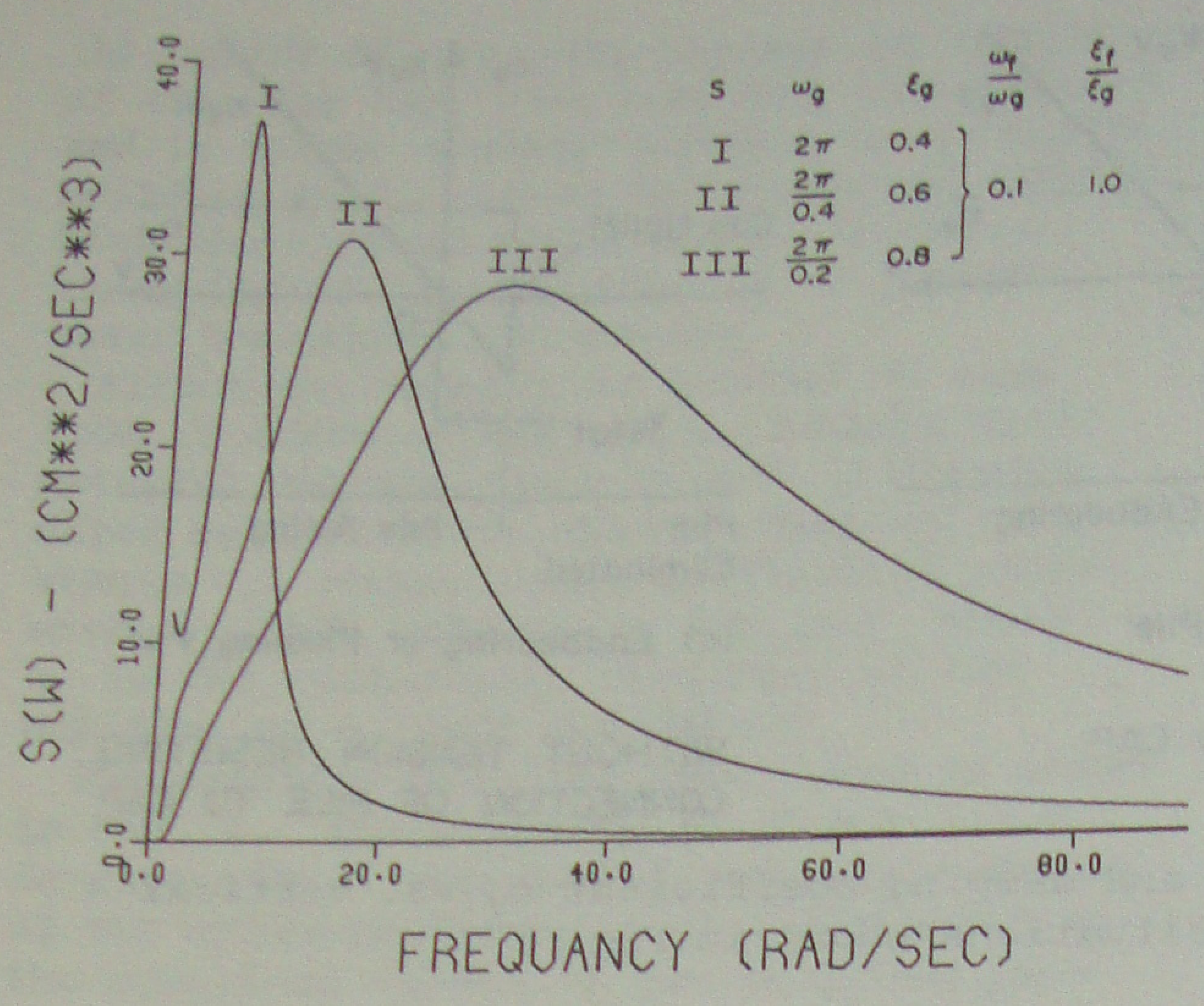


Fig. 4 Power spectra of ground acceleration

parameters were established by Hindy and Novak (1980).) The ratios $\omega_f/\omega_g = 0.1$ and $\xi_g/\xi_f = 1.0$ are common. The curves shown in Fig. 4 are normalized by S_o.

The time histories of the simulated accelerations are shown for a peak acceleration of 0.1g in Fig. 5. The accelerations used in the parametric study are scaled to different levels in order to examine the variation of the building response with ground motion peak acceleration and power. The latter is characterized by the variance of the ground motion acceleration

$$\sigma^2_{\ddot{u}_g} = \int_0^\infty S_{\ddot{u}_g}(\omega)\,d\omega \tag{4}$$

3 EQUATIONS OF MOTION AND THEIR SOLUTION

With the assumptions outlined above, the governing equations of seismic response of a shear building on piles can be written in terms of relative displacements u_i, base displacement u_b and rotation ψ as

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\} = \{P\} \tag{5}$$

in which the displacement vector

$$\{U\} = \begin{bmatrix} \{u\} \\ \hline u_b \\ \psi \end{bmatrix} \tag{6}$$

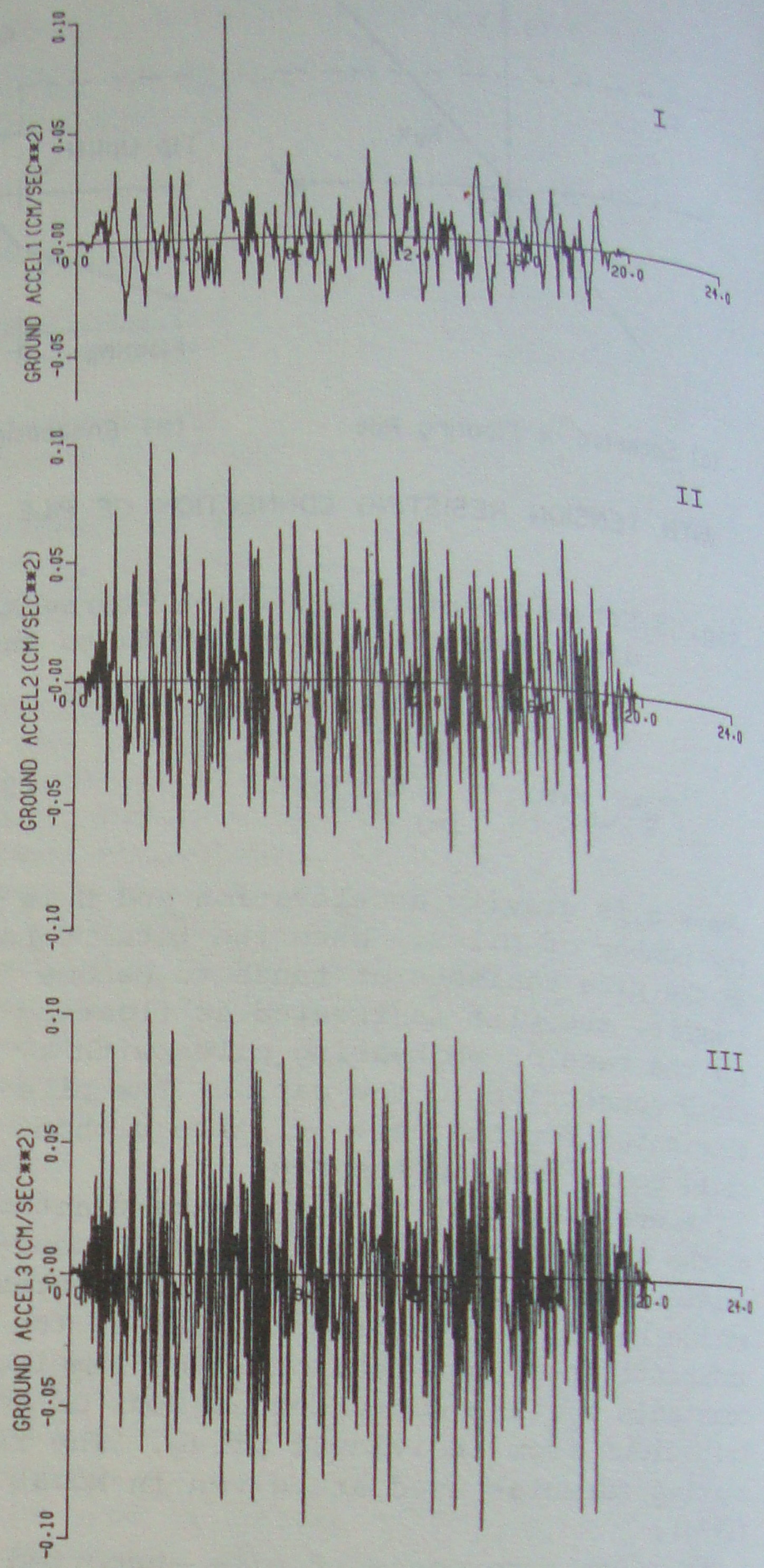


Fig. 5 Time histories of ground acceleration employed

In Eq. (5), the stiffness and damping matrices are

$$[K] = \left[\begin{array}{c|cc} [k] & \{0\} & \{0\} \\ \hline \{0\}^T & k_{uu} & k_{u\psi} \\ \{0\}^T & k_{\psi u} & k_{\psi\psi} \end{array}\right] \tag{7a}$$

$$[c]=\left[\begin{array}{c|cc} [c] & \{0\} & \{0\} \\ \hline \{0\}^T & c_{uu} & c_{u\psi} \\ \{0\}^T & c_{\psi u} & c_{\psi\psi} \end{array}\right] \tag{7b}$$

and the loading vector

$$\{P\} = -[\{m\} \mid m_b + \sum_1^n m_i \; \sum_1^n m_i h_i]^T \ddot{u}_g(t) \tag{8}$$

The mass matrix is given in El Hifnawy and Novak (1986).

The matrices [c] and [k] list the stiffness and damping constants of the building on rigid foundation. The constants k_{ij}, c_{ij} describe the impedance functions that vary with time due to pile tip or cap uplift.

Because of the nonlinear character of Eq. (5) stemming from variation of the impedances of the pile foundation with time, the Wilson-θ method of step-by-step integration with respect to time was used to calculate the building response to simulated ground acceleration. In this method, the floor equivalent earthquake forces $P_i(t)$, the base shear, $Q_b(t)$, and the overturning moment $M_b(t)$ are calculated at each time step as

$$P_i(t)=m_i[\ddot{u}_i(t)+\ddot{u}_b(t)]+m_i h_i \ddot{\psi}(t) \tag{9a}$$

$$Q_b(t)=\Sigma P_i(t)+m_b\ddot{u}_b(t) \tag{9b}$$

$$M_b(t)=\sum_1^n P_i h_i+(I_b+\sum_1^n I_i)\ddot{\psi}(t) \tag{9c}$$

Using the mathematical model and the technique outlined, an extensive parametric study was conducted. The main results are described below.

4 NATURAL FREQUENCIES AND DAMPING RATIOS

The equation of free, undamped vibration follows from Eq. 5 for {P} = {0}. The fundamental natural frequencies and first mode damping ratios for the building shown in Fig. 2 are given in Table 2 for the case in which no uplift was allowed (i.e. as in Fig. 1a). The damping ratios were calculated using the energy approach (Novak, 1973; Novak and El Hifnawy, 1983). The natural frequencies increase and the damping ratios decrease with the number of piles as expected, approaching the fixed base case. Comparing the three spectra of ground acceleration (Fig. 4), it can be seen that the peak of the medium band spectrum (II) is centred around the fundamental frequency of the example building.

Table 2 Fundamental frequencies and damping ratios for building on piles (with no uplift)

N	26 piles	32 piles	38 piles	44 piles	fixed base
ω (rad/s)	14.08	14.46	14.76	15.05	16.85
D (%)	1.92	1.80	1.70	1.64	1.00

5 EFFECT OF UPLIFT ON SEISMIC RESPONSE

The complete time histories and the peak values of displacements, earthquake forces as well as the total forces on individual piles were calculated for three different excitation conditions: (a) - the earthquake signals had the same peak ground acceleration but different spectra and variances; (b) - the earthquake signals had one spectrum but different peak ground accelerations; and (c) - the earthquake signals were generated from three different spectra having the same variance but different peak accelerations.

5.1 Response to ground motions having different spectra and the same peak acceleration

In this paragraph the effects of pile head attachment to the cap and/or tip socketing are examined for the three earthquake signals shown in Fig. 5 corresponding to the three different power spectra shown in Fig. 4 but having the same peak acceleration $\hat{a}$=0.1g. The variances of the three different spectra are given in Table 3. The variances differ substantially. The broad band spectrum (III) has the highest variance followed by the medium (II) case and then the narrow band (I) spectrum.

Table 3 Variances of ground motions having different power spectra but the same peak acceleration = 0.1g

Type of spectrum	Narrow (I)	Medium (II)	Broad (III)
σ^2 (cm^2/sec^4)	175.61	762.76	1139.72

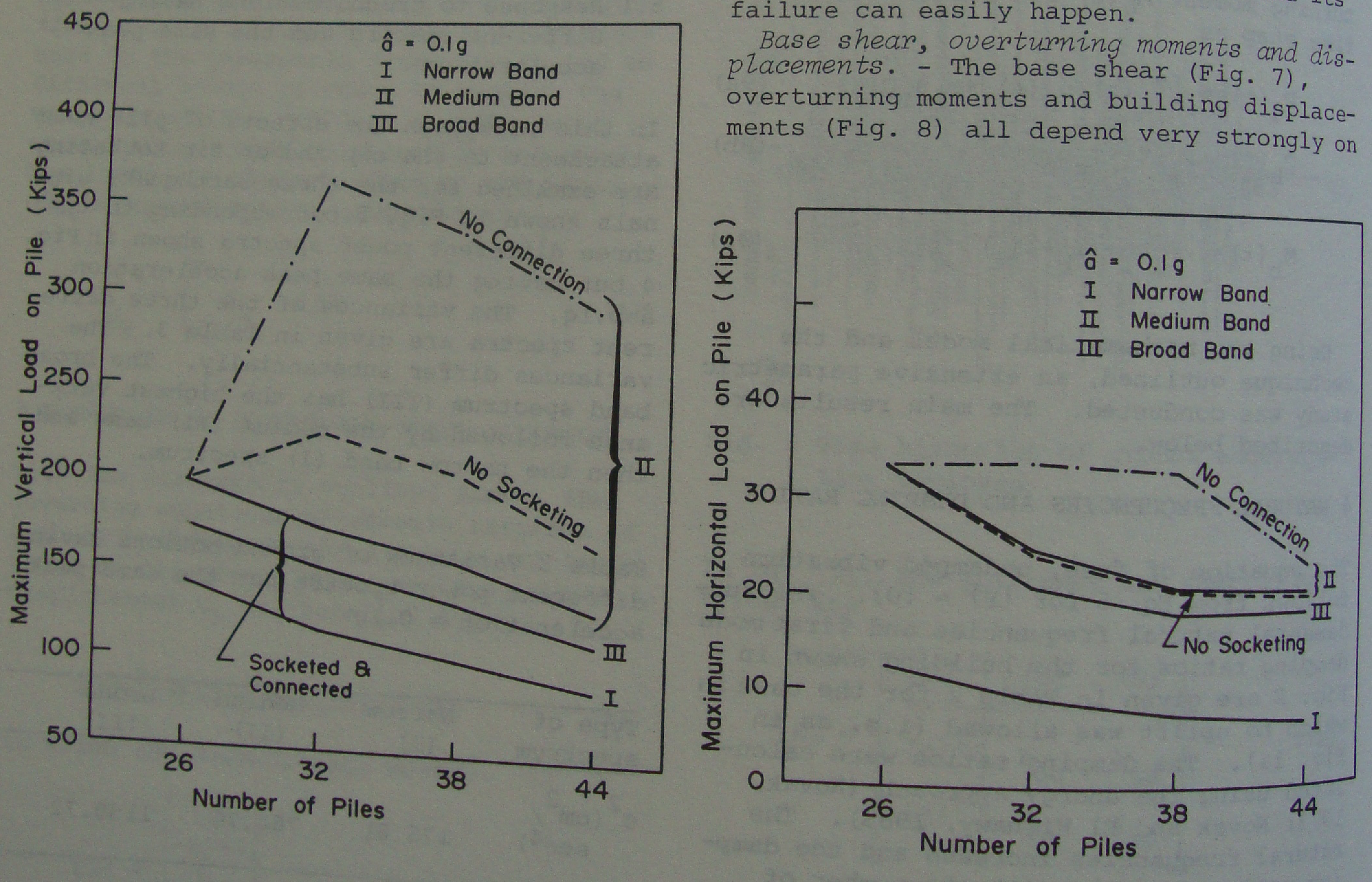
The large differences in the variance, being a measure of the total power of the ground shaking, indicate that marked differences in the building response can be anticipated even for one value of peak ground acceleration.

Pile forces. - The peak values of vertical and horizontal forces per pile are examined for the 8 storey building assumed. For the three different ground motions (spectra), the variations in the peak pile vertical force with the number of piles are shown in Fig. 6a. For the signals having narrow or broad band spectra, the vertical force on the pile is not affected by the absence of the pile connection to the cap or tip socketing because tension does not occur. For the intermediate spectrum, which has its high power region close to the fundamental frequency of the building, tension does occur. Consequently, pile vertical forces increase by about 30% if the piles are not socketed and by 100% if the tension resisting connection is not provided. This dramatic increase in pile forces for the no connection case is caused by the elimination of the piles located at the opposite edge of the building. For floating (friction) piles, large increases in axial loading were observed also as reported previously (El Hifnawy and Novak, 1986).

The horizontal loads on piles (Fig. 6b) are typically about ten times smaller than the vertical loads (but so is the bearing capacity). For the medium band spectrum whose peak coincides with the building fundamental frequency, the horizontal pile forces increase by up to 60% when the pile head connection to the cap is absent. The broad band spectrum gives higher forces in general than the narrow band spectrum owing to the differences in the variances indicated in Table 3.

The substantial increases in both vertical and horizontal pile loading due to elimination of pile connection to the cap can have serious consequences if the piles do not have a sufficient reserve in bearing capacity. Endbearing piles can fail and floating piles may be pushed deeper into the soil resulting in the collapse or tilting of the building. This type of damage was reported in a few Japanese earthquakes (Mizuno, 1987) and it also occurred in the 1985 Mexico City earthquake. Particularly with wood piles, their attachment to concrete caps is difficult to secure and its failure can easily happen.

Base shear, overturning moments and displacements. - The base shear (Fig. 7), overturning moments and building displacements (Fig. 8) all depend very strongly on



(a) Vertical loads

(b) Horizontal loads

Fig. 6 Maximum loads on piles for different ground motions with the same peak acceleration = 0.1g (1 Kip = 4.448 kN)

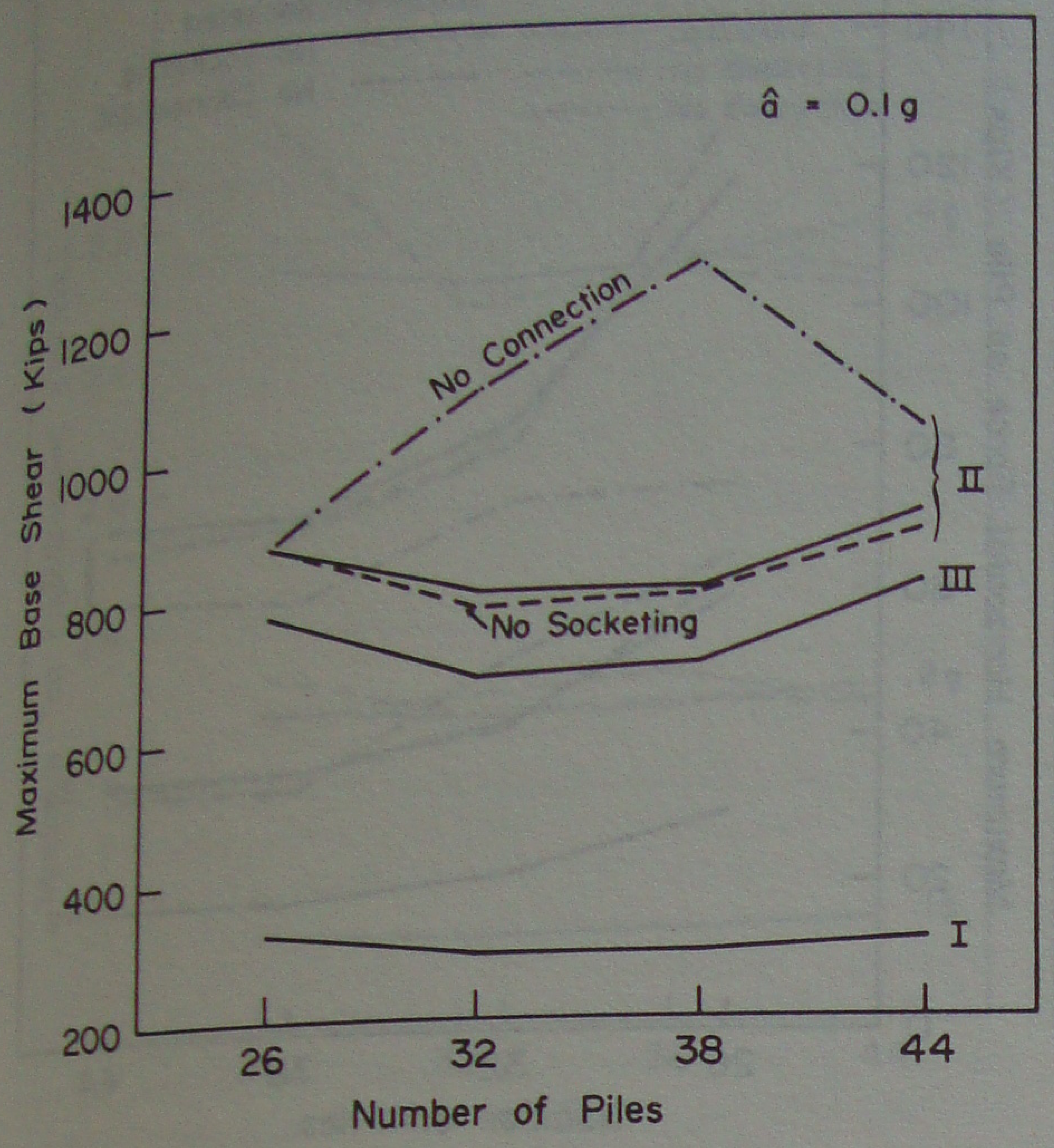


Fig. 7 Maximum base shear for different ground motions with the same peak acceleration = 0.1 g (1 Kip = 4.448 kN)

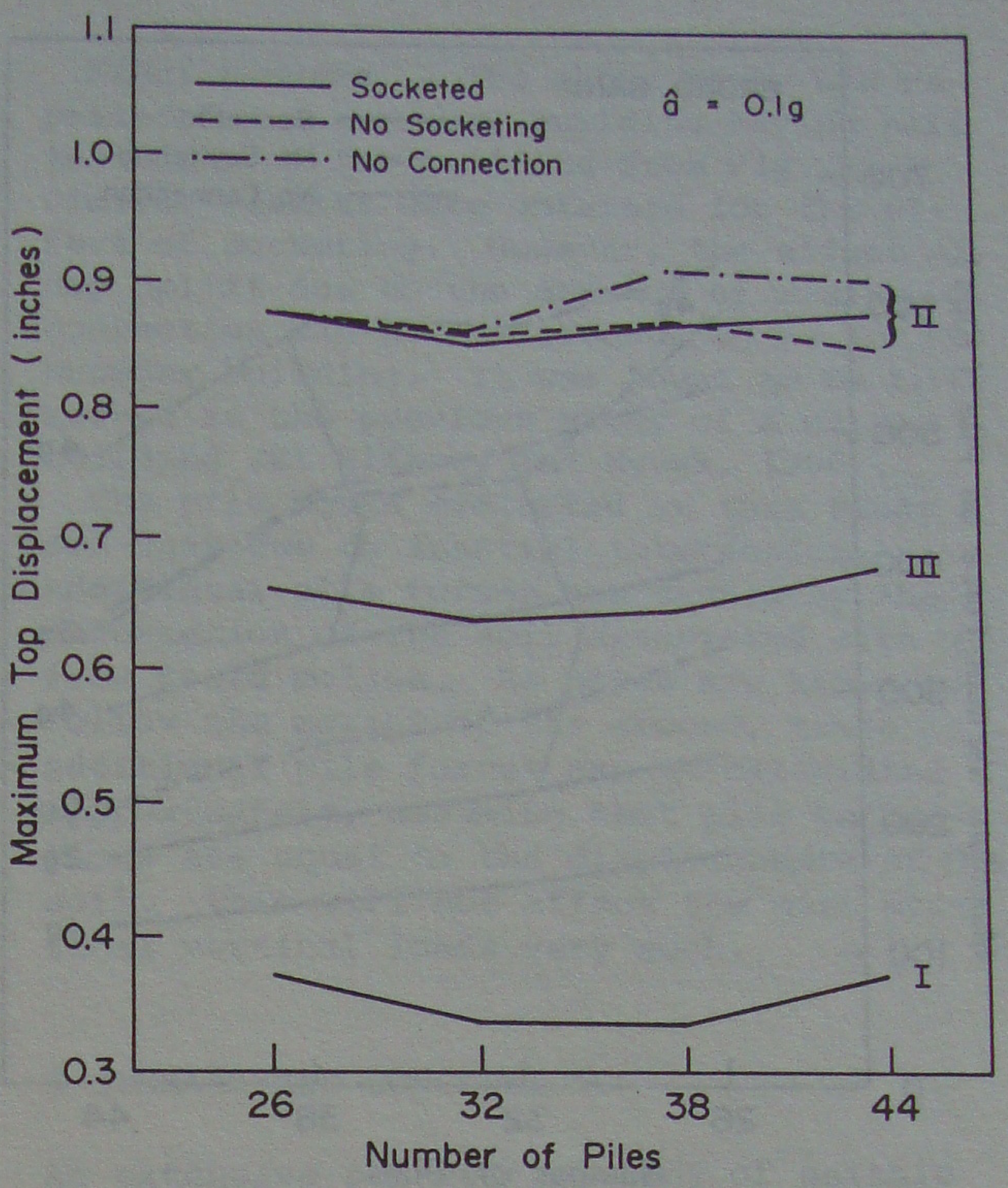


Fig. 8 Maximum top displacement of building for different ground motions with the same peak acceleration = 0.1 g (1 in. = 25.4 mm)

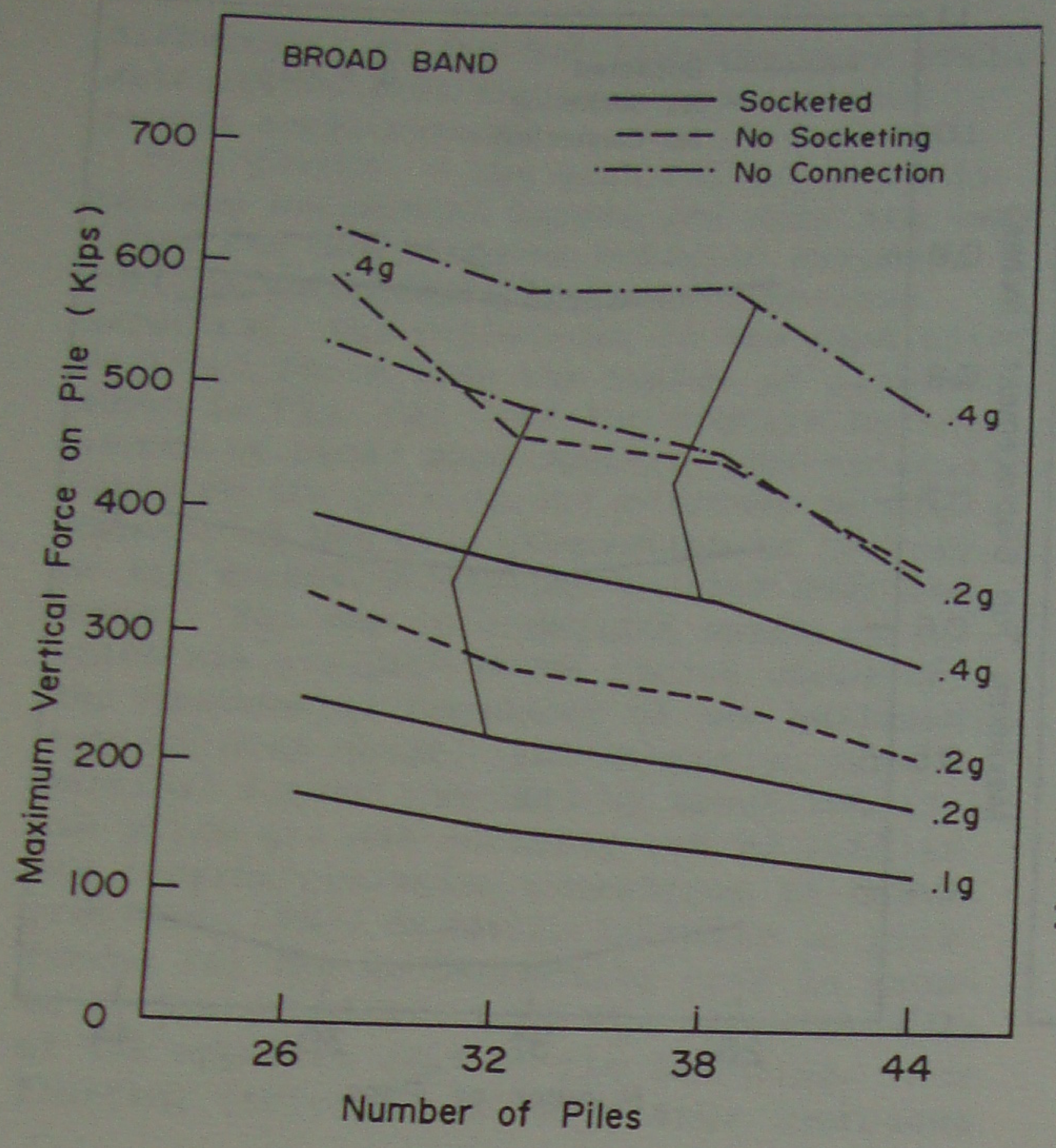
the type of the ground motion with the narrow band excitation giving the lowest response. The effect of socketing is very slight and occurs only for the spectrum II. However, the absence of pile connection to the cap, intentional or unintentional, may result in a dramatic increase in seismic loading on the building reaching up to 60% for spectrum II (Fig. 7). Storey displacements are practically independent of the pile connection and socketing but vary markedly with the type of ground motion (Fig. 8).

The above parametric study was conducted once more on the same building subjected to signals whose peak accelerations were doubled to 0.2g but whose time history outside the spike with the maximum acceleration remained the same. Results very close to those shown in Figs. 7 and 8 were obtained with the conclusion that doubling the peak acceleration alone has no effect on the seismic response of the building. Obviously, peak ground acceleration is not a very suitable measure of ground motion intensity for engineering purposes, as already inferred from the data shown in Table 3.

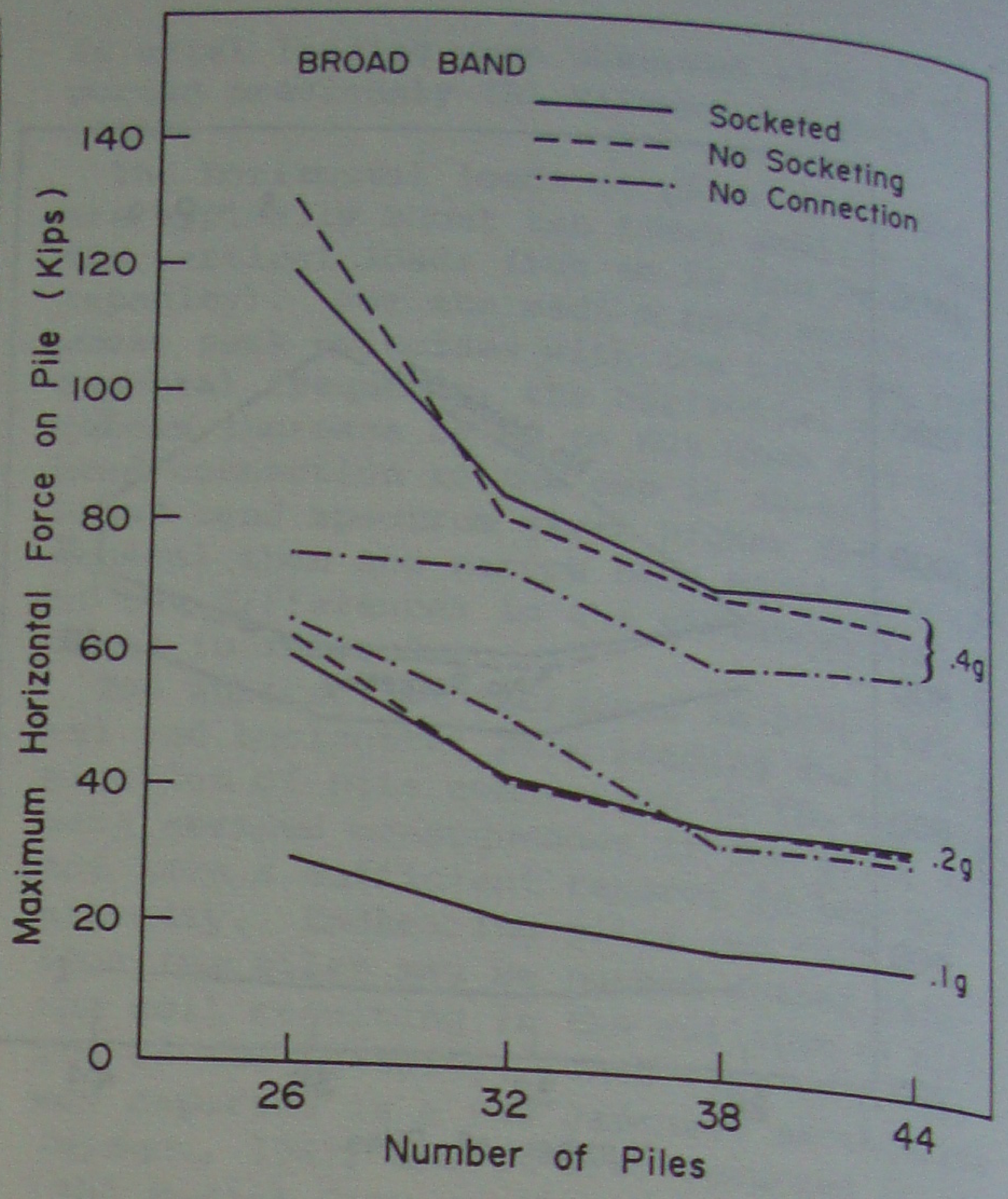
5.2 Response to ground motions having different peak accelerations and the same frequency content

In this paragraph, the effect of earthquake intensity on building response and pile forces is examined for the broad band signal shown in Fig. 5c scaled to different levels. This signal has peak acceleration = 0.1 g and variance = 1139.72 cm^2/sec^4. The analysis was repeated for the whole signal multiplied by 2 and 4, respectively, giving it peak accelerations of 0.1, 0.2 and 0.4 g without any change in frequency content.

The *pile forces* generally increase with the increase in ground shaking intensity as would be expected (Figs. 9a, 9b). For small ground acceleration such as .1 g, tension does not occur. For high intensity ground shaking (0.2 or 0.4 g), the vertical force on the extreme piles increases modestly when socketing is not provided but quite dramatically for piles with no tension resisting connection to the cap (Fig. 9a). However, with such a connection provided and high earthquake intensity, the tensile force per pile is quite high and



(a) Vertical loads



(b) Horizontal loads

Fig. 9 Maximum loads on piles for ground motions with different peak accelerations and the same frequency content (1 Kip = 4.448 kN)

socketing may be necessary. The horizontal force per pile (Fig. 9b) increases or decreases depending on the intensity of ground shaking and the presence or absence of socketing and rigid connection.

The peak *building displacements* increase with ground shaking intensity (Fig. 10). Socketing the piles or connecting them to the cap may increase or decrease the top displacement slightly for moderate earthquake intensity. For high ground acceleration such as .4 g, the top displacement decreases if the cap uplift occurs. This is so because of the decrease in pile group stiffness due to the elimination of piles in tension.

Figures 11 and 12 show how the *seismic forces* are affected by earthquake intensity, pile socketing or pile connection to the cap. For moderate intensity shaking (.1 and .2 g), socketing the pile and connecting it to the cap make no practical difference in the seismic loading of the building; but for higher ground acceleration (0.4 g) the building loading is reduced when cap uplift occurs. This reduction in seismic forces is again caused by the reduction in stiffness.

5.3 Response to ground motions having different spectra but the same variance

Finally, the response of the eight storey building supported by different pile groups was analyzed for three ground motions characterized by the three different power spectra shown in Fig. 4 but normalized to have the same variance equal to 1139. cm^2/sec^4. For this variance, the peak acceleration of the broad band signal (III) is 0.1 g, is more than that for spectrum II and is maximum for the narrow band spectrum (I). This follows from Table 3. The results indicate that in this case the narrow band spectrum gives the highest response and forces followed by the intermediate one and then the broad band spectrum. The results also indicate that socketing the piles or connecting them to the cap has a modest effect on seismic forces and top displacements. However, the pile vertical force increases significantly if the piles are not socketed and dramatically if not rigidly connected to the cap. These trends can be seen in more detail from Figs. 13 and 14.

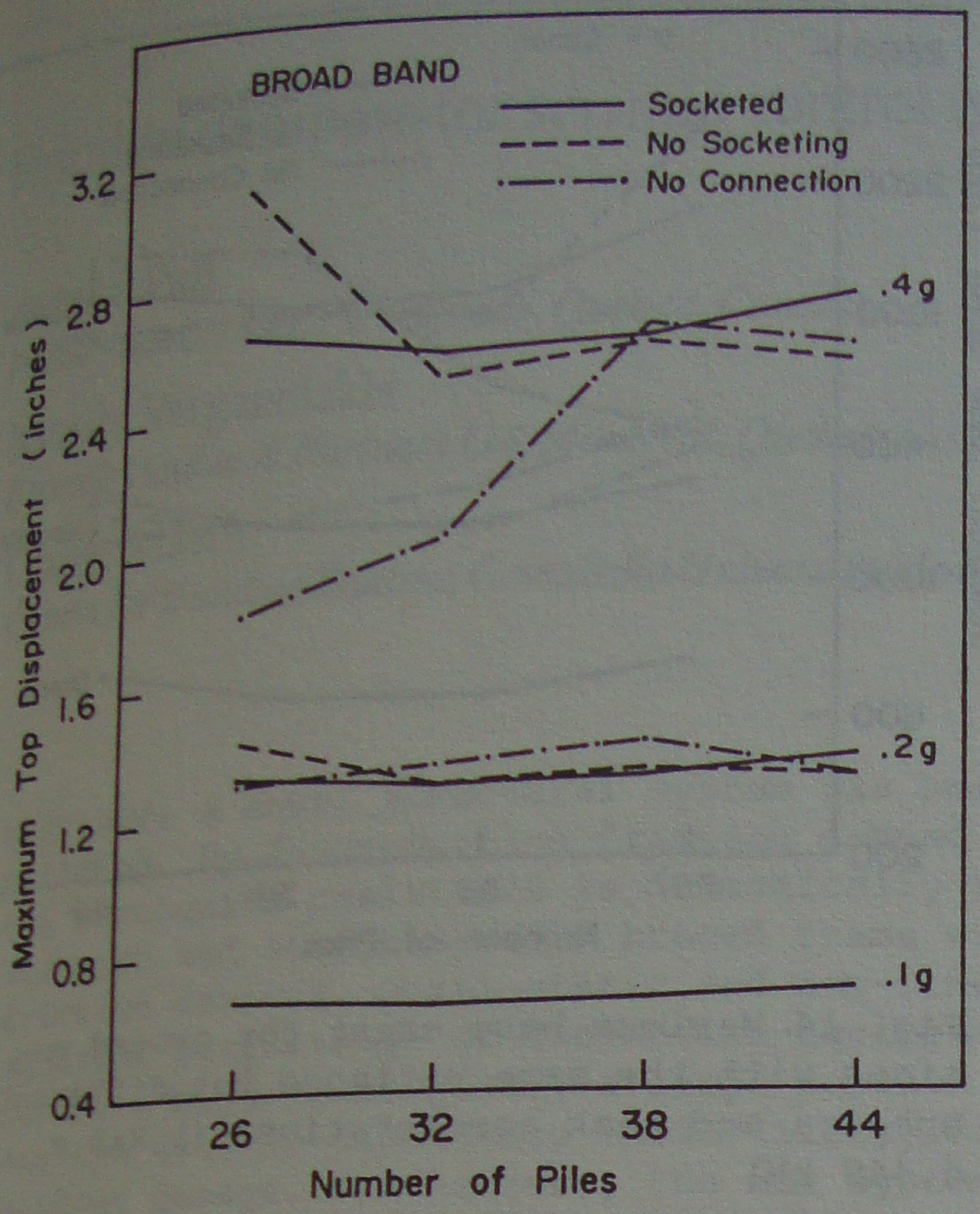


Fig. 10 Maximum building displacements for ground motions with different peak accelerations and the same frequency content (1 in = 25.4 mm)

Final remarks. - The above study was repeated for a narrower building having half the width of the building from Fig. 2. Similar results were obtained for the effect of socketing. However, the effect of cap uplift due to the absence of rigid pile connection was not as pronounced as for the broader building. It was found to be quite marked in the previous study of a narrow building (El Hifnawy and Novak, 1986).

The pile loads evaluated in this study are only due to inertial interaction. Additional pile forces are caused by the deformation of the soil associated with the free field motion. As piles are known to follow the motion of the ground, these additional pile forces can be calculated approximately, assuming that pile deflections are equal to the displacements of the soil. This will not affect the most critical vertical loads very much.

6 SUMMARY AND CONCLUSIONS

An extensive parametric study of seismic response of buildings supported by piles was conducted using artificial earthquake signals. The following conclusions can be made:

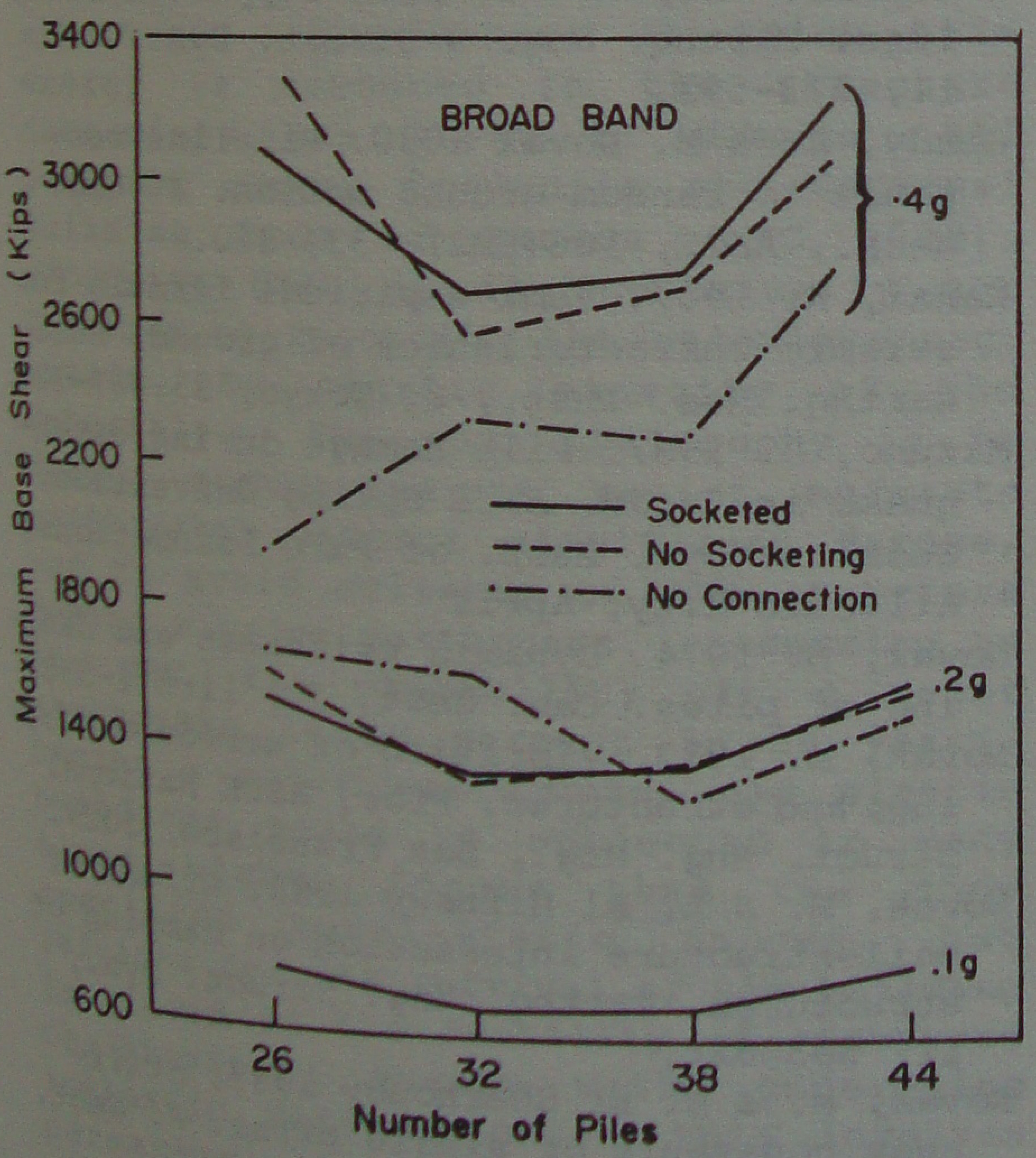


Fig. 11 Maximum base shear for ground motions with different peak accelerations and the same frequency content (1 Kip = 4.448 kN)

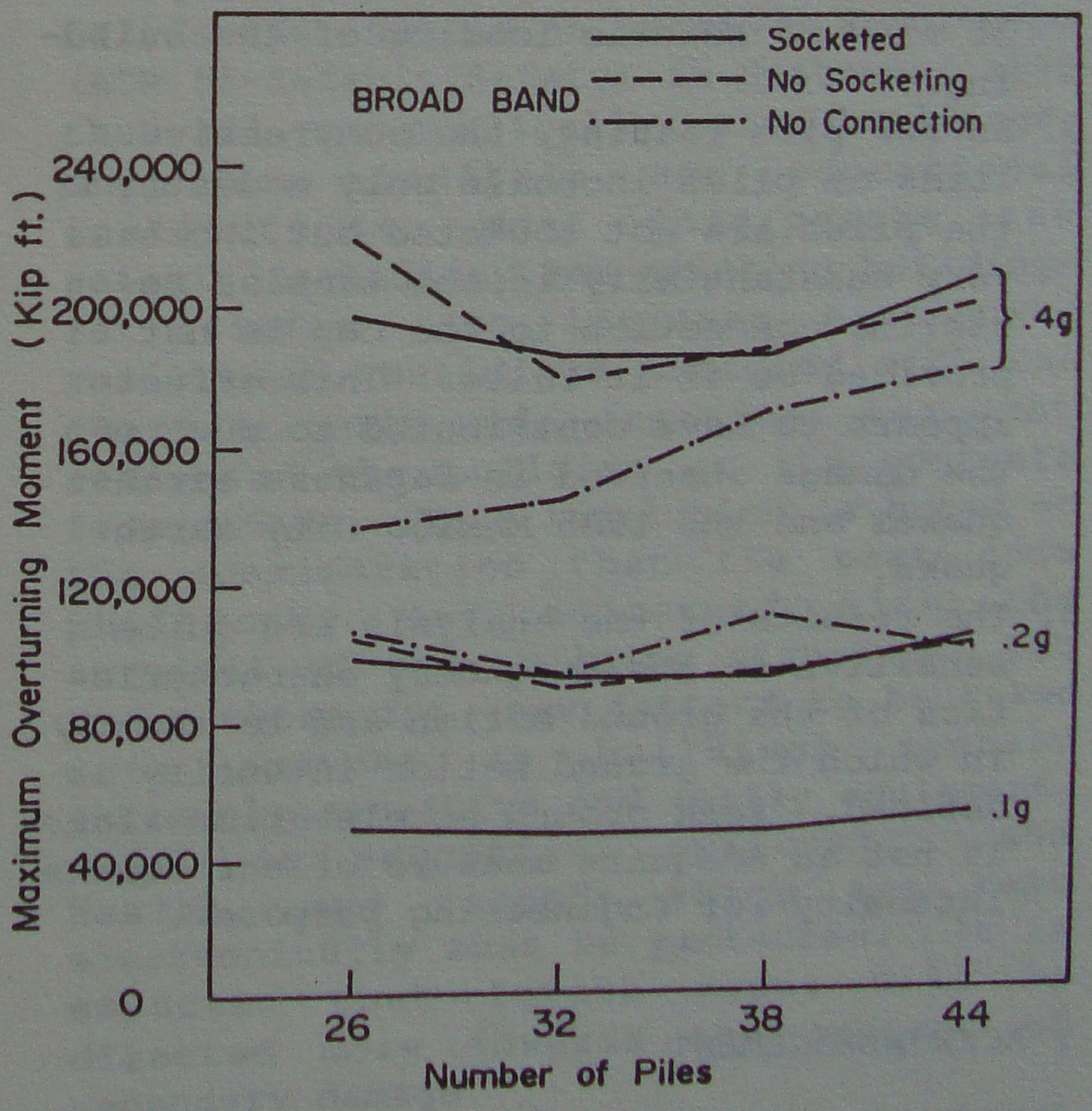


Fig. 12 Maximum overturning moment for ground motions with different peak accelerations and the same frequency content (1 Kip ft = 1.356 kNm)

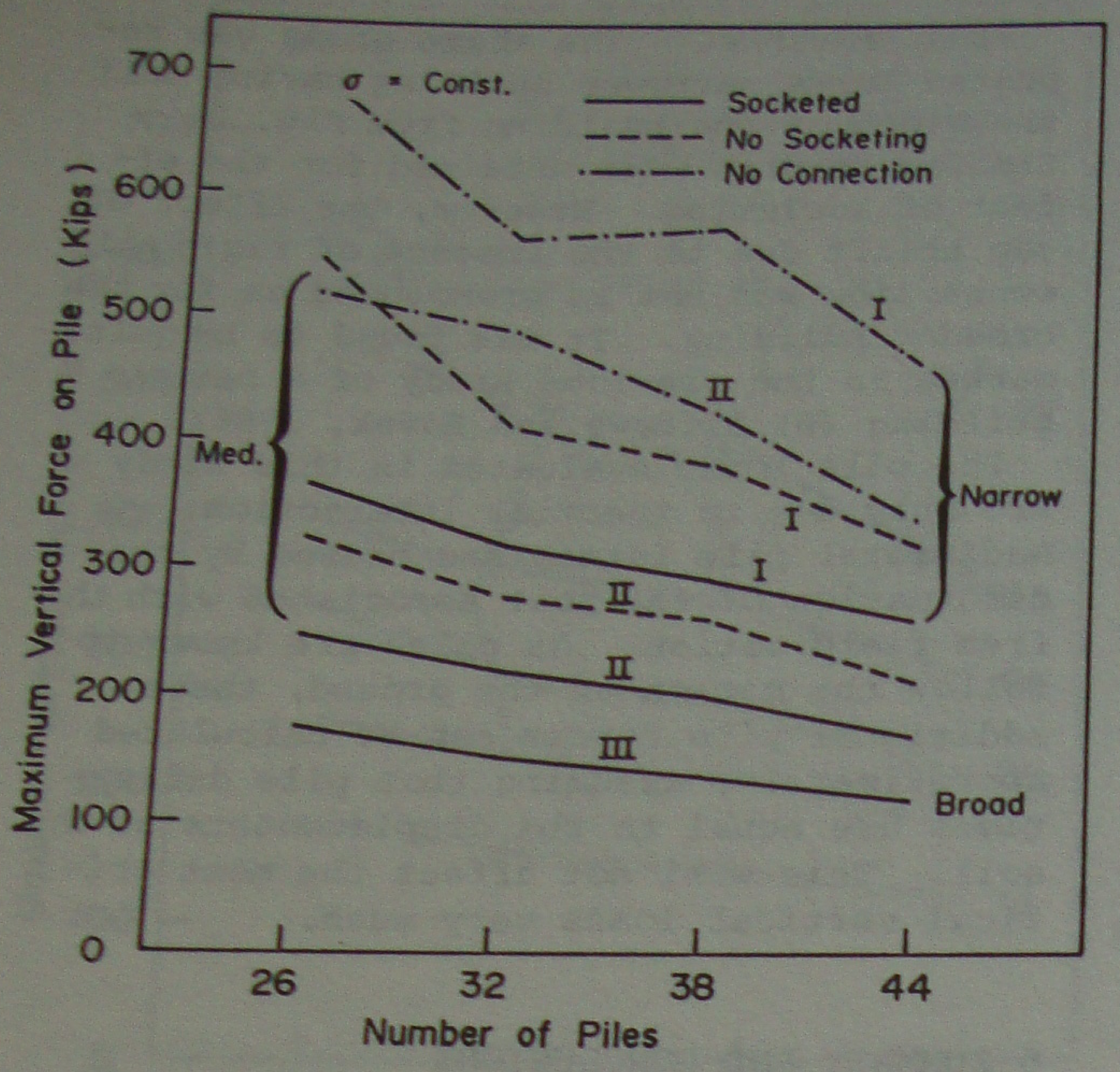


Fig. 13 Maximum vertical loads on piles for ground motions with the same variance but different spectra and peak accelerations (1 Kip = 4.448 kN)

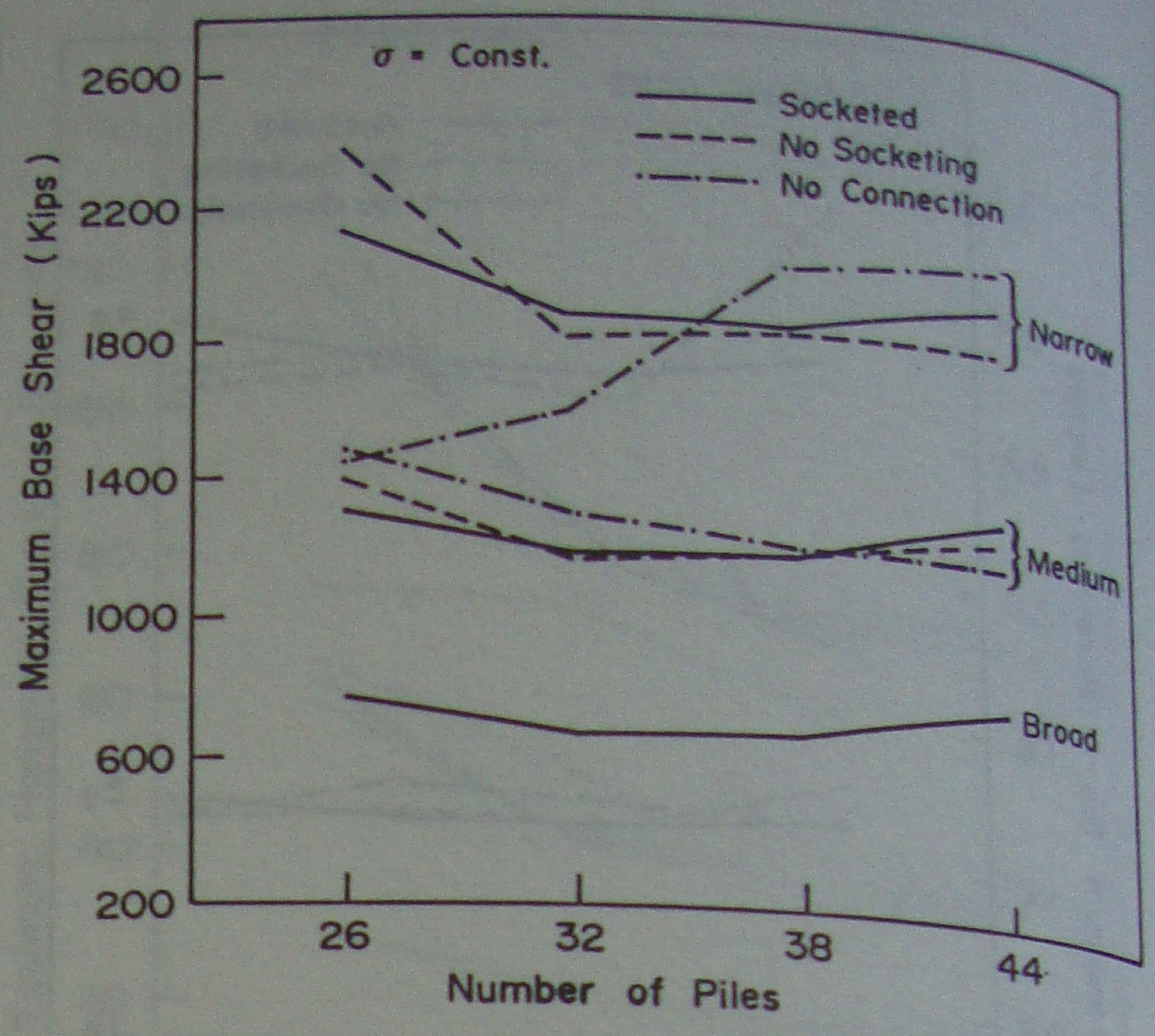


Fig. 14 Maximum base shear for ground motions with the same variance but different spectra and peak accelerations (1 Kip = 4.448 kN)

1. Rigid connection of piles to the cap and pile socketing may not be necessary for low seismic intensity or from the point of view of seismic loading of the building.
2. As for pile loading, the compressive loads on piles increase only modestly if the piles are not socketed but increase very substantially if the tension resisting connection to the cap is not provided or if it fails. This effect appears to have contributed to much of the damage observed in Japanese earthquakes and the 1985 Mexico City earthquake.
3. The results of the analysis are quite sensitive to the frequency characteristics of the ground motion and to the way in which the ground motion intensity is defined. Peak ground acceleration alone is not an adequate measure of earthquake intensity for engineering purposes.

7 ACKNOWLEDGEMENT

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