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Considering ground response effects in the seismic design of piles

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ABSTRACT: There have been several instances of damage to piles at deeper part, generally near the soil layer interfaces, during the past earthquakes. Such damages are inherently difficult to detect and repair, mandating adequate provision in the design to make them as unlikely as possible. Non-linear response analysis of soil-pile-structure system considering a two dimensional FEM model shows distinctly large kinematic response forces near soil layer interfaces, demonstrating the nature of stresses that may develop in piles due to distinct stiffness contrast between soil layers. While such detailed analysis is impractical for general design application, the current practice of designing the pile for a single concentrated load representing the inertia effect of superstructure involves implicit disregard of the kinematic actions due to ground response. A simple approach to account for the ground response effects is proposed and its potential for practical design application is illustrated.

1 INTRODUCTION

Investigations on the damage to piles during the past earthquakes provide some basic information concerning the nature of failures in piles at locations with deep soil deposit under strong ground shaking. Examples include the January 17, 1995 Hyogoken-Nambu earthquake (Karkee & Kishida 1997, Karkee et al. 1997, Matsui & Oda 1996 etc.) and the June 12, 1978 Off-Miyagiken earthquake (Sugimura 1981, 1987). Remarkably significant instance of the damage is reported to have occurred at deeper parts along the pile, particularly in relatively longer piles. Evidently, the location of pile damage at intermediate part in longer piles also tends to coincide with changes in soil layering.

The stresses developed in piles due to the soil-pile-structure interaction under earthquake shaking consist of the superstructure inertia effects as well as the kinematic effects of ground response.

The latter effects are termed simply as the 'ground response effects' in this paper. The relative magnitude of the inertial and the kinematic actions depends on the ground condition as well as the level of excitation. Generally, long piles penetrating a deep layered deposit, particularly where there is a sudden change in soil stiffness, are likely to be exerted by large ground response forces.

However, only the inertia effects tend to be explicitly accounted for in the seismic design practice for piles. The horizontal force to be resisted by the pile consists of the inertia of the building, and the basement if applicable (BCJ 1984), with no explicit recognition of the ground response effects.

Results of the nonlinear response analysis on the soil-pile-building interaction system of a 35 storied reinforced concrete building based on a two-dimensional finite element model (Sugimura et al. 1997) is discussed. Considering three simple variations in soil condition, other structural details remaining the same, it is shown that the influence of soil layering on the stresses developed in piles during earthquake shaking can be very dominant. Of particular interest is the clear dependence of the superstructure inertia itself on the nature of the soil layering system interacting with the pile.

That is, for a given incident motion specified for a region the superstructure inertia depends on the different levels of excitation resulting from the extent of nonlinear response (Karkee et al. 1992) depending on the local site condition. In addition, the ground response effect for the same input earthquake motion is very different site condition. The results of the finite element response analysis clearly illustrate the need for adequate consideration of inertia as well as the ground response effect in pile design.

While the detailed finite element analysis is known to adequately depict the response of the soil-pile-structure system under earthquake excitation, the necessary computational effort can be formidable for its application to everyday design practice. Considering the inadequacy of the current design practice of specifying a single concentrated load at the pile top in lieu of earthquake actions, there is a need to develop a simple design method that can account for the ground response effects realistically. Sugimura (1992) proposed the use of distributed load to represent the effect of ground response on piles. Presumably, the nature and the magnitude of the distributed load should reflect the local soil condition and its dynamic characteristics. A simple approach to evaluate the distributed load and to evaluate the ground response effects is proposed in this paper. Preliminary simulations clearly illustrate the potential of the approach to realistically represent the ground response action on piles that may be expected during earthquakes.

2 DAMAGE TO PILES IN PAST EARTHQUAKES

Reports on investigation of damage to foundations during past earthquakes provide ample instances of damage at the intermediate part of a pile. The location of the damage in piles may provide some indication of the dominance of either the inertia effects of the superstructure or of the kinetic effects of ground response. Generally the inertial forces may be considered to result in the failure of piles near the top, while the ground response effects may be expected to result in damages at the deeper part, particularly where there is a abrupt change in the soil stiffness.

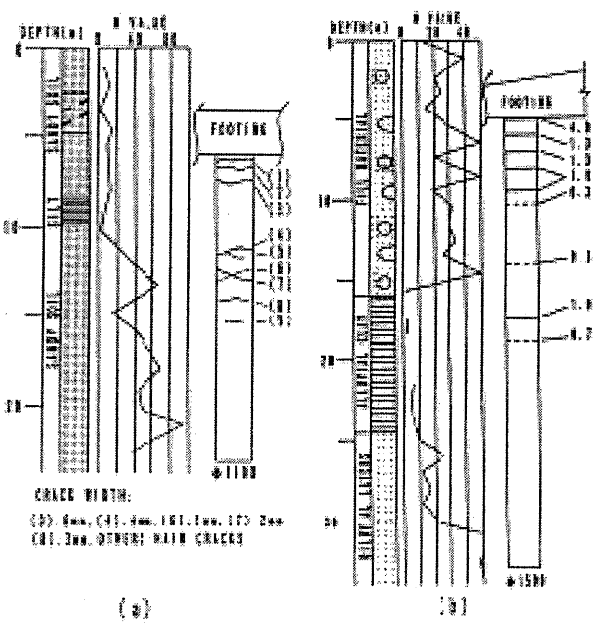

Figure 1. Piles damaged in Hyogoken-Nambu earthquake (a) Building (AIJ 1996) & (b) Highway (HBC 1995)

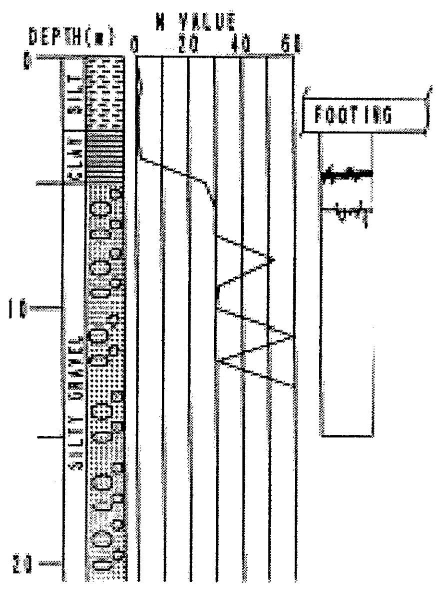

Figure 2. Damage to pile in 1978 off-Miyagi-ken earthquake (Sugimura & Oh-oka 1980)

A typical case of damage to a pile supporting a building during the Hyogoken-Nambu earthquake (AIJ, 1996) is shown in relation to the ground profile in Figure 1(a). The pile is seen to develop cracks near the top as well as at the middle part around where the soil profile changes into a stiffer layer. It seems plausible that the change from a softer to a stiffer soil layer may have contributed to the cracking at the middle part of the pile. Similarly, Figure 1(b) shows the damage to a pile in a highway bridge (HBC, 1995). Again the pile has cracked at the middle part where the soil stiffness decreases suddenly as indicated by the N-value distribution. The cracking of the piles at deeper part noted in Figure 1 were detected by core boring and borehole camera. Figure 2 shows the damage to pile during the off-Miyagiken earthquake (Sugimura & Oh-oka, 1981) in a L-shaped building where the outer corner joining the two wings had undergone a large settlement. This is a case of a pile on the inner side of one the wings. The pile foundation damage was considered to be due to inertial forces of the superstructure. However, the cracks seem to align with changes in soil layering.

Clearly, the damage to piles at deeper part is more problematic in the event of rehabilitation and recovery after the earthquake. While there are successful cases of repair of the damage to pile near the top (e.g. karkee & Kishida 1997), damages at deeper part of the pile are by nature much more difficult to detect and repair. It is imperative that the design approach for piles at earthquake regions should particularly strive to make the damage to piles at the deeper part less likely. Extremely large actions on piles may manifest due to failure of the ground (e.g. liquefaction) accompanied by lateral spreading (e.g. Tokimatsu et al. 1996) and detailed investigation of the local site for such possibility should be included while considering ground response effects in the seismic design of piles.

3 FINITE ELEMENT ANALYSIS FOR NONLINEAR SEISMIC RESPONSE

In Japan, 20 to 45 storied reinforced concrete buildings have been common for apartment building structures. The natural period of these buildings range from 1.2s to 2.5s, falling into a range in the design spectra where the spectral velocity ordinates tend to be uniform. Generally these buildings are supported on cast-in-place concrete piles with enlarged base. A 35 storied building is considered a representative of these structures and is analyzed in detail for its seismic response characteristics.

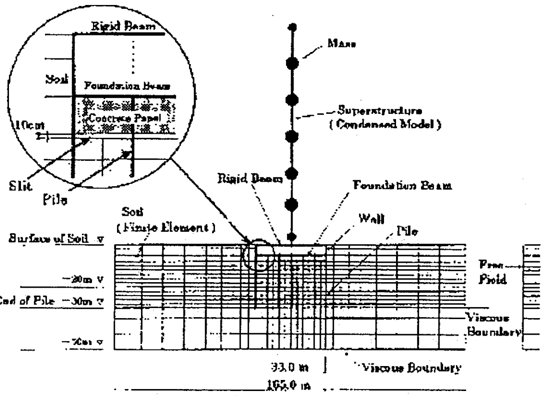

Figure 3. The two-dimensional FEM representation of structure for nonlinear seismic response analysis.

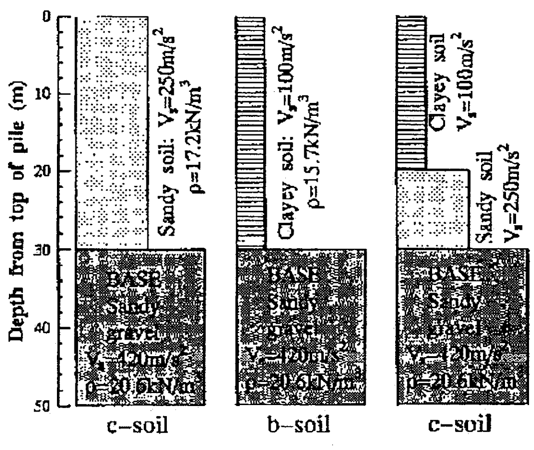

Figure 4. Details of the three soil conditions used in the analysis, other conditions remaining same.

The schematic finite element model for nonlinear dynamic response analysis is given in Figure 3. The basement slab in Figure 3 is assumed to have no direct contact with the soil underneath.

Three simple soil profiles shown in Figure 4 and designated as a-soil, b-soil and c-soil overlying a sandy gravel layer, typically found in urban areas in Japan at depths of 30m or more, is considered. Three sets of response analysis are carried out for the three soil conditions, other structural details remaining the same. Details of the building structure and the method of analysis, together with the assumed nonlinear behavior of soil and concrete are as given by Sugimura et al. (1997).

3.1 *Base input motion for the analysis*

Two input motions are considered to investigate the effect of relative difference in the level of excitation. One is the well known El Centro NS record and the other is the Hyogoken-Nambu earthquake record of the Kobe marine observatory (Kobe JMA NS). The El Centro motion is scaled to a maximum velocity of 50cm/s while the Kobe record is used as it is. The corresponding peak accelerations are 510.8cm/s^2 and 818.0cm/s^2 respectively. The response spectra of the two input motions are given in Figure 5, where it is seen that the spectral velocity ordinates for the Kobe record are significantly larger than those for El Centro in the period range 0.3-3.0s.

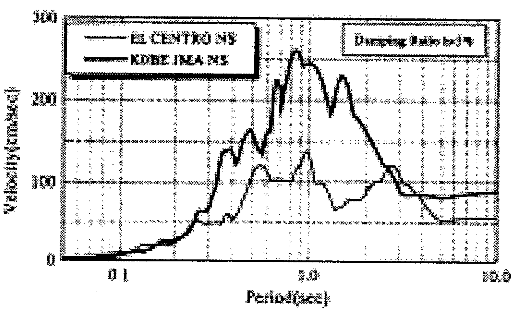

Figure 5. Response Spectra of input motions.

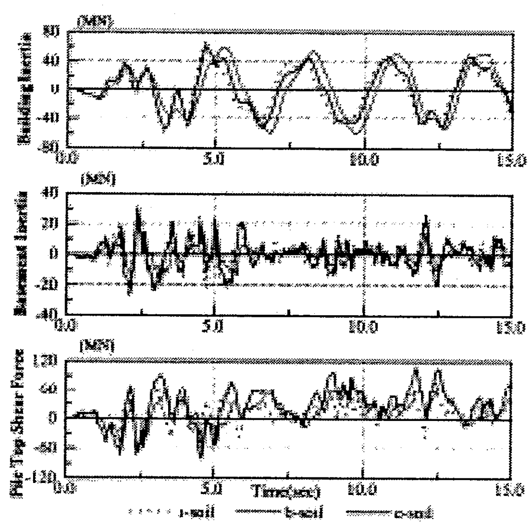

Figure 6. Time histories of some typical response forces.

3.2 *The Results of the Response Analysis*

Some of the time histories of the response forces for El Centro input are shown in Figure 6. From the time history of the building inertia, the predominant period of the building can be noted to be about 2.9s. The corresponding period of the building was about 3.3s for Kobe JMA NS. Compared to the elastic period of 2.1s, the period of the building has elongated by a factor of 1.4 and 1.6 respectively, larger factor indicating stronger shaking in case of the Kobe input. Figure 6 also shows that the basement inertia is larger in stiffer soil (a-soil) compared to soft soils (b and c-soils). The time histories for pile top shear force contain long as well short period components, unlike the building inertia, where the component attributable to the predominant period of the building dominates.

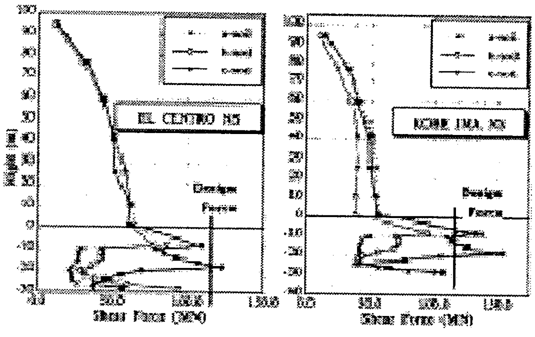

Figure 7. Maximum shear force response of the Building and the piles for the two input motions.

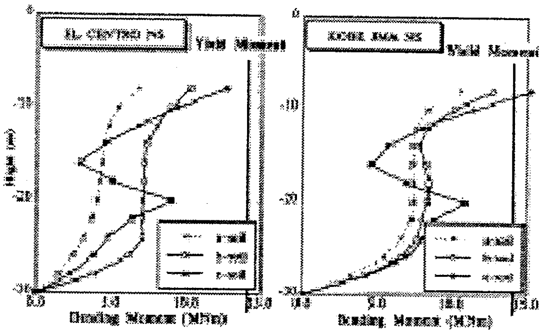

Figure 8. Maximum bending moment in piles under the action of the two input motions.

Figure 7 shows the maximum shear force response of the structure corresponding to the two incident motions. In the current design practice for tall buildings in Japan, the piles are generally designed to have the ultimate shear capacity defined as the sum of 1.5 times the base shear capacity of the building (64.0MN) and the seismic coefficient for basement times the weight of basement (98.4MN). Assuming the seismic coefficient for basement to be 0.2, the ultimate shear capacity works out to be 115.8MN, which is designated as the design force in Figure 7. The maximum shear force in piles is largest in case of c-soil for both the input motions and occurs at the interface of the two soil layers. The cracking of piles at the layer interfaces observed during past earthquakes, as noted in Figure 1, may have been caused by this tendency of increased shear force.

Figure 8 shows the maximum bending moment in piles. Again a very strong influence of the soil layering is apparent from the distinctly different shape of the maximum bending moment distribution in case of c-soil. Larger moment at the soil layer interface is seen in c-soil for both the El Centro and the Kobe input. Larger bending moment at deeper part of the pile is particularly problematic because it tends to act in combination with decreased axial load in pile with depth. In addition to larger moment at the layer interface, it is seen in Figure 8 that the bending moment at the pile head is largest in c-soil for both the input motions.

Another interesting result is that the bending moment distribution in a-soil is nearly half of that in b-soil in case of El Centro input, while the bending moment distribution for a-soil and b-soil is practically same in case of Kobe input. This may be attributed to the much higher level of shaking in the later case with the increased nonlinear effect resulting in similar stiffness in a-soil and b-soil at higher level of excitation (Karkee et al. 1993, 1992). This may be possible because the soil stiffness degradation tends to saturate at higher level of excitation.

3.3 *Building and foundation components of total response*

Attempt is made to separate (Sugimura et al. 1997) the response of the total system into those of the building and the foundation systems. The contribution of the building superstructure and that of the soil and foundation to the response of the total system depicted in Figure 3 is thus obtained.

Figures 9 and 10 show time histories of actions contributed by building and foundation systems respectively.

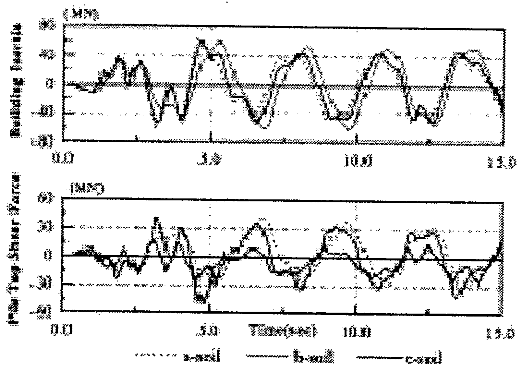

Figure 9. Response forces from building system.

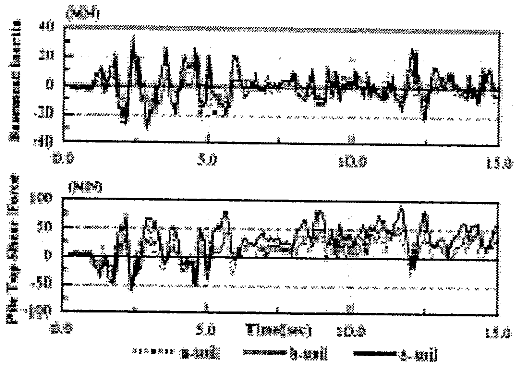

Figure 10. Response forces from foundation system.

It may be noted in Figure 9 that the building inertia and the pile top shear force tend to be in opposite phase, while both are dominated by the predominant period of the building as noted above.

Figures 9 and 10 show that the contribution of the foundation system consists of relatively short period components when compared to that of building system. However, the contribution of both the building system and the foundation system to the pile top shear force clearly depends on the soil condition.

Figure 11 shows distribution of the maximum shear force in a single pile. While the magnitude and the nature of distribution of maximum shear force is strongly dependent on the ground condition, the contribution of the superstructure inertia to the shear force in piles is distinctly smaller than that of the foundation system. Thus the shear force obtained based solely on the inertia with out regard to the local site condition tend to grossly underestimate the maximum shear forces that may be expected during strong ground shaking. In fact, the maximum shear force responses of the total system and that of the foundation system are seen to be practically coincident in all the three soil types in Figure 11, particularly at deeper part, indicating domination of the foundation part. The result shows that the shear forces in piles may even be represented by the response of the foundation portion alone.

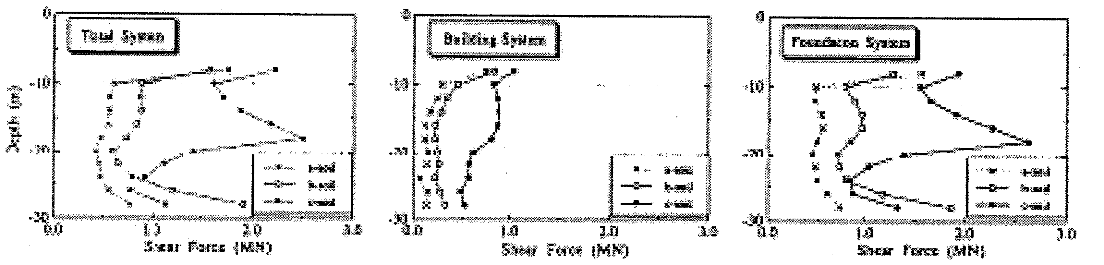

Figure 11. Distribution of maximum shear force in pile for total, building and foundation systems.

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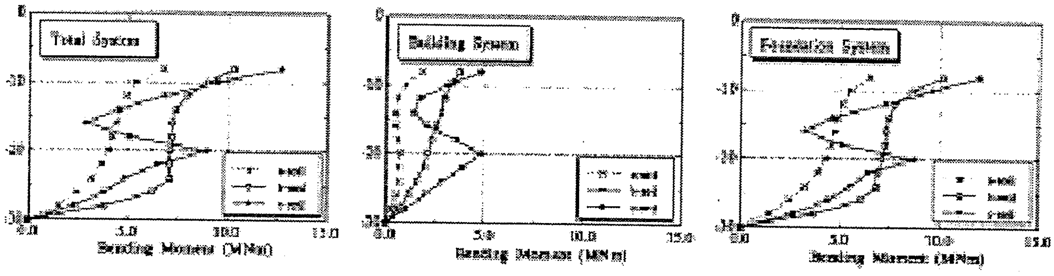

Figure 12. Distribution of the maximum bending moment in pile for total, building and foundation systems.

Similar to the case of maximum shear force, Figure 12 shows distribution of maximum bending moment in a pile. Again, the contribution of the foundation system to the maximum bending moment distribution is distinctly large and practically same as that of the total system. This follows logically from Figure 11, because integration of shear force gives the bending moment. However, maximum bending moment does not necessarily correspond to the maximum shear force, and it is important to note strong domination of foundation system in the bending moment acting on the piles.

The results of nonlinear response analysis of the soil-pile-building system depicted in Figure 3 clearly illustrate the inadequacy of the current practice of considering the inertia of superstructure and basement (BCJ, 1984) with implicit disregard of the soil condition in the seismic design of piles. Considering that finite element analysis discussed above is impractical for everyday design, the need to develop a suitable method to adequately account for the ground response effects is evident.

4 SIMPLE DESIGN APPROACH TO CONSIDER GROUND RESPONSE EFFECTS

As mentioned above, seismic design practice in Japan allows for inertial forces to be resisted by the pile. However, the reports on damage to piles during past earthquakes, as well as the results of finite element response analysis, show strong domination of ground response effects on the internal forces developed in piles. A simple method based on the principle of beam on elastic foundation, together with an approach to evaluate the distributed load, is proposed for adequate consideration of the ground response effects in the seismic design of piles.

4.1 *Principle of beams on elastic foundations*

Considering a Winkler soil model and assuming the pile to be a massless beam on elastic foundation, the basic equation relating the horizontal deflection y of the pile may be given by Equation 1.

Here, D is the diameter, E is the Young's modulus and I is the sectional area moment of inertia of the pile. The constant k_h is the coefficient of horizontal subgrade reaction of the soil and x is the depth in soil. The soil movement $f(x)$ in Equation 1 represents the free field ground response displacement during the earthquake and equating it to zero gives the equation for the static loading case.

$$EI\frac{d^4y}{dx^4} + k_h D\{y - f(x)\} = 0 \tag{1}$$

If $k_h Df(x)$ in Equation 1 is considered to be a force $p(x)$ at a depth x required to displace the soil there by $f(x)$, then we have:

$$EI\frac{d^4y}{dx^4}+k_hDy=p(x) \tag{2}$$

Equation 2 can be solved for a given distribution of load *p(x)* along the pile length. From a practical point of view, solutions for three simple load distribution, consisting of concentrated load, uniformly distributed load and triangularly distributed load, can be appropriately combined to depict a more general distributed load approximately. These solutions are in fact already available from Hetényi (1946). This framework for the solution of a beam on elastic foundation is utilized in the proposed simple analysis method to indirectly account for the kinematic effects of ground response.

4.2 *The current design method in Japan (MKS units)*

Based on the Japanese design guide (Sugimura, 1988), the design external force Q_p for earthquake loading is given in terms of the base shear Q and the number of piles n as:

$$Q_p=(1-\alpha)\frac{Q}{n}; \qquad \alpha=1-0.2\frac{\sqrt{H_B}}{\sqrt[4]{H_f}} \tag{3}$$

Here, α is the participation factor of embedment given in terms of the height above ground H_B and the depth of embedment H_f, both in meters. Based on the engineering judgment derived from parametric analysis, it is recommended that the value of α should not be greater than 0.7. The design guide suggests that the internal forces and displacement can be computed based on the theory of beam on elastic foundation. For a given degree of fixity α_r at the pile head and a horizontal concentrated load Q_p acting at the top, horizontal displacement at pile head y_o is obtained from Equation 4. When not determined from horizontal loading test, it is suggested that k_h in kg/cm^3 be estimated from the Equation 6, where E_o is the elastic modulus of soil in kg/cm^2 and D is in cm.

$$y_o=\frac{Q_p}{4EI\beta^3}(2-\alpha_r) \tag{4}$$

$$\beta=\sqrt[4]{\frac{k_hD}{4EI}} \tag{5}$$

$$k_h=0.8E_oD^{-\frac{3}{4}} \tag{6}$$

As can be noted, the design method considers only the inertia effect of the superstructure and needs to be recognized as such. While the method may be adequate to account for the inertia effects, it does not attempt to addresses the kinematic effects due to interaction between the pile and the surrounding soil that may undergo significant nonlinear response during strong ground shaking.

From detailed analysis, it is noted above that the ground response effects can be very significant.

4.3 *The proposed method for evaluation of ground response effects*

As is noted above in Equation 2, if the magnitude and the nature of the distributed load *p(x)* could be determined taking into consideration the local site condition and soil nonlinearity, it may be possible to solve the problem of the beam on elastic foundation to estimate the forces acting on piles due to the kinematic effects of ground response. For this the displacement response *f(x)* of the free field under the action of the earthquake needs to be estimated. Considering the first mode of the free field motion, the elastic fundamental period of a soil layer of thickness H and shear wave velocity V_S may be approximated by $4H/V_S$. Then the maximum displacement U_g occurring at the top of the i^{th} soil layer may be given in terms of the peak velocity of the input motion V_{max} by Equation 7.

$$U_g = V_{\max} \times \left(\frac{2H}{\pi V_S} \phi \right)_i \tag{7}$$

The parameter ϕ in Equation 7 is the 'ground period elongation factor' representing the extent of elongation in the ground period due to the nonlinear effect (Karkee et al. 1992, 1993) during strong ground shaking, such that $\phi \geq 1$. If all the soil layers at a site are assumed to contribute equally to the ground period elongation, a constant ϕ may be assumed for all the soil layers. The introduction of the parameter ϕ constitutes an attempt to account for the effect of nonlinear soil response in a simple manner. From the dynamic analysis using the El Centro input (Sugimura et al. 1997), predominant ground period T_P was seen to be about twice the elastic fundamental ground period T_G. The result indicates the value of ϕ to be about 2.0. The Japanese guideline (BCJ, 1992) recommends a value of 2.2 for ϕ in case of strong shaking at bay areas with deep soil deposit. It seems reasonable to assume the overall ground period elongation factor ϕ at a deep soil site to be in the range of 2.0 to 3.0.

For a given value of the ground period elongation factor ϕ at a site, consideration of a variable value ϕ_i for different soil layers may be considered appropriate depending on the site condition.

One of the way this could be done is to assume the value of ϕ_i for the i^{th} layer of thickness H and shear wave velocity V_S to be in proportion to the value of H/V_S such that overall ground period elongation factor ϕ remains the same. Thus, if the value of H/V_S for the i^{th} layer is defined as τ_i, then the period elongation factor ϕ_i for the layer may be given by $\alpha\tau_i$, where α is given by Equation 8.

$$\alpha = \phi \times \frac{\sum_i \tau_i}{\sum_i \tau_i^2} \qquad \& \quad \phi_i = \alpha\tau_i \quad \textit{Where} \quad \phi_i \geq 1 \tag{8}$$

With the displacement U_g relative to bottom of the i^{th} layer obtained from Equations 7, the ground displacement $f(z)$ relative to the bottom of the soil layer, where z varies from 0 at the top to H at the bottom of each layer, may be given by the cosine function of Equation 9. Once the displacements relative to the bottom of each layer is computed, the overall displacement $f(x)$ relative to the pile toe can be easily obtained, such that $p(x)$ is given by Equation 10.

$$f(z) = U_g \cos\left(\frac{\pi}{2H}\right) z; \quad 0 \leq z \leq H \tag{9}$$

$$p(x) = k_h D f(x); \quad 0 \leq x \leq L \tag{10}$$

One of the crucial aspect in the proposed method is the proper evaluation of the coefficient of subgrade reaction k_h in Equation 10, because its value affects the magnitude of the distributed load $p(x)$ to a large extent once $f(x)$ is determined. In this respect, it may be noted that the elongation of the ground period by ϕ_i noted in Equation 7 corresponds to the soil stiffness degradation by $1:\phi_i^2$. This can be adequately accounted for in estimating the value of k_h, which may be computed from Equation 11 derived by Vesic (Poulos and Davis 1980) considering an infinite beam on elastic foundation.

$$k_h = \frac{0.65}{D}\left\{\frac{E_S}{1-\nu^2}\right\} \times \left\{\frac{E_S D^4}{EI}\right\}^{\frac{1}{12}}; \quad \frac{E_0}{E_S} = \lambda \tag{11}$$

In Equation 11, EI is the bending stiffness of the pile as defined earlier, ν is the Poisson's ratio of soil and λ is the 'stiffness degradation factor' indicating the extent to which the soil stiffness reduces during strong shaking, resulting in the ground period elongation by ϕ_t as mentioned above. If there is a single soil layer over the pile embedment depth, the value of λ for it may be taken as equal to ϕ^2, otherwise the stiffness degradation factor λ_i for the i^{th} layer would be ϕ_i^2, where ϕ_t is obtained from Equation 8. If necessary the value of k_h for a layer may be taken as zero to depict liquefied situation.

It may be noted that although the effect of soil stiffness degradation due to nonlinear response would be clearly crucial in the evaluation of the kinematic stresses in the piles due dynamic soil-pile interaction, it is also likely to be important in evaluating the inertia effects. However, the simple design method proposed here is primarily concerned with the aspects of soil response effects, and no attempt is made here to investigate the effect of soil nonlinearity on the evaluation of the inertial forces in piles.

4.4 *Computation example based on the proposed method*

Attempt is made to compute bending moment and shear force distribution in piles for the case of c-soil by assuming the value of ϕ to be 2.0 corresponding to the El Centro input case mentioned above. As noted in Figures 11 and 12, the ground response forces are most significant in c-soil, which consists of a clayey soil layer underlain by a stiffer sandy soil layer as shown in Figure 4.

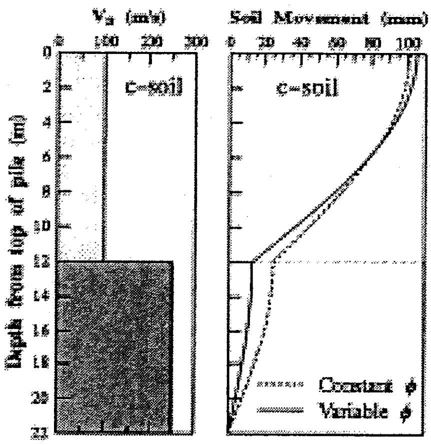

Figure 13. Shear wave velocity profile and the ground displacement for constant & variable ϕ.

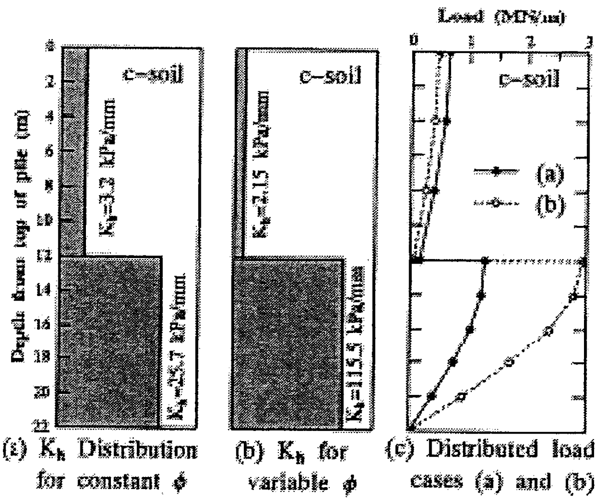

Figure 14. Distribution of subgrade reaction for constant and variable ϕ and the corresponding distributed loads.

Figure 13 shows the shear wave velocity profile together with the soil displacement estimated by Equation 9. Figure 13 shows the soil displacement distribution for the constant ϕ case as well as for the variable ϕ based on Equation 8. The ground displacement depicts the shear wave velocity profile logically, giving much larger displacement of softer layer compared to the stiffer layer, with even better contrast in case of variable ϕ. The ground displacement near the pile top is about 100mm.

The coefficient of subgrade reaction k_h for the ground response effect is obtained from Equation 11 by assuming the Poisson's ratio ν of the soil to be 0.45. Resulting distribution of k_h for constant ϕ is shown in Figure 14(a). Assumption of a variable ϕ based on Equation 8 results in a k_h distribution of Figure 14(b). The distributed load $p(x)$ obtained from Equation 10 corresponding to the two cases of constant and variable ϕ are shown in Figure 14(c). The cosine distribution of the distributed load is approximated by uniformly varying loads at discrete intervals in Figure 14(c).

The distributed lateral load on the pile shown in Figure 14(c) may not act all at once throughout the duration of ground shaking, specially in case of multiple soil layers. This may be understood from different possible modes of ground movement. This was previously confirmed (Sugimura et al. 1997) by the elastic eigenvalue analysis of the three systems corresponding to the three soil conditions of Figure 4. It was found that the second mode of c-soil showed the worst of the three soil type cases, indicated by large bending moment and shear force around the two soil layer. To indirectly account for such effects in the simple design method proposed here, it would be logical to consider different combinations of distributed load over different sections of the pile length. Figure 15 shows three possible distributed load combinations assuming a constant value of ϕ, and in consideration to the two distinct soil layers. The shear force and bending moment diagrams for the three distributed load cases of Figure 15 and the k_h distribution of Figure 14(a) are given in Figure 16.

The shear force and bending moment diagrams for the three distributed load cases representing the ground response effects is obtained by assuming the pile top to be restrained against rotation with the pile tip free. It is seen in Figure 16 that there is a large shear force near the interface of the two soil layers similar to that seen in Figure 11, and that the bending moment too is large around there.

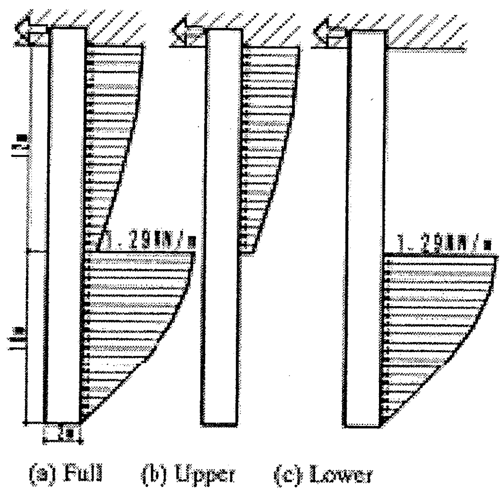

Figure 15. Three possible distributed load cases in c-soil with two distinct soil layers.

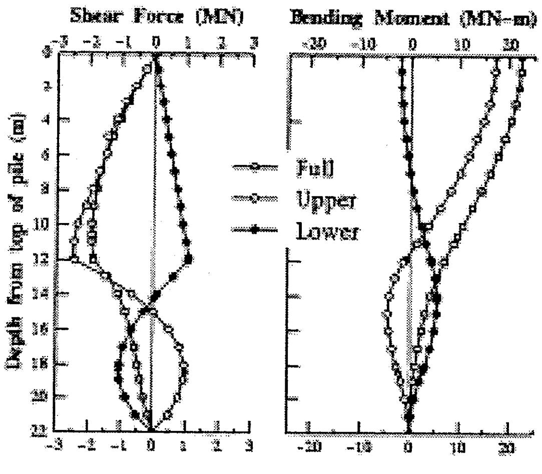

Figure 16. Shear force and bending moment diagrams for the three distributed load cases in Figure 15.

It would not be logical to make a point to point quantitative comparison of the shear force and the bending moment diagrams in Figure 16 with those for c-soil in Figures 11 and 12, which are actually the envelop of maximum forces rather than the actual distribution. In addition, Figures 11 and 12 also include the inertia of the basement. When this fact is recognized, the maximum shear force of about 2.5MN and the maximum bending moment of about 5.5MNm around the middle part of the pile in Figure 16 are qualitatively comparable to those of 2.5MN and 8.0MN-m in Figures 11 and 12 respectively. This indicates that the simple solution proposed here is capable of accounting for the ground response effects in piles that may be expected during ground shaking.

The bending moment near pile top in Figure 16 is of the order of about 20.0MNm compared to about 13.0MNm for c-soil in Figure 12. In contrast, the maximum bending moment of 5.5MNm at the middle part in Figure 16 is smaller than that of about 8.0MNm in Figure 12. This difference is most likely because of the assumption of full restraint of pile top against rotation, which is unlikely to be the case due to nonlinear response of the concrete piles during strong earthquake shaking. In fact, the yield moment of the pile is about 14.0MNm as indicated in Figure 8. Beyond yield level the pile top would tend to rotate, resulting in only a partial restraint against rotation. When this condition is accounted for in the analysis by assuming a certain degree of restraint against rotation rather than the full restraint, the maximum bending moment obtained from the detailed nonlinear analysis can be closely approximated by the proposed simple method. For example, assuming the degree of restraint to be 0.7 in Figure 15 would logically amounts to limiting the moment at pile top to the yield level. Under this condition, the bending moment at the middle part of the pile in Figure 16 would be closer to about 8.0MNm obtained from the detailed nonlinear response analysis.

Investigations based on large number of simulations would be required for further refinement of the proposed method for actual application. However, the values of the coefficient of subgrade reaction k_h and the ground displacement near pile top, applicable for realistic estimation of the ground response effects based on the proposed simple method, tend to be significantly different from those recommended in the current design practice. Evident the values of k_h recommended in Japanese code for the seismic design based on the inertial forces can not be adopted for the evaluation of ground response forces the simple method proposed here. From this standpoint, it seems logical to evaluate the inertial forces and the ground response forces separately, assuming different values of subgrade reaction k_h as applicable, and then design the pile for the envelop distribution of the largest of the two response forces along the pile length.

5 CONCLUSIONS

Reports on the investigation of damage to piles during past earthquakes show ample instances of damage to piles that can directly or indirectly attributed to ground response. Nonlinear response analysis based on finite element analysis indicates large response forces at the soil layer interfaces, demonstrating the importance of ground response effects in pile design, while also illustrating the inadequacy of the present design practice that accounts for the inertia effects alone.

When the response of the total soil-pile-building system is decomposed into those of the building system and the foundation system, the contribution of the foundation part is by far dominant, and comparable to that of the total system. In addition, the contribution of the building system itself is strongly dependent on the soil condition. The results clearly indicate the influence of the local site condition on the inertial actions as well as on the kinematic actions of ground response on piles.

While demonstrating the importance of considering the ground response in the design of piles, the finite element response analysis also indicates the different levels of earthquake excitation to which the superstructure might be subjected depending on the soil condition of the site.

The proposed simple approach based on the beam on elastic foundation framework is promising in capturing the essence of the ground response effects on piles that may be expected from nonlinear response of the soil-pile-building system, if the necessary parameters are selected to reflect the ground dynamic characteristics adequately. The essential parameters in the proposed method include peak input velocity, ground period elongation factor, soil stiffness degradation factor and degree of restraint at the pile head. In addition, the dynamic characteristic of ground such as the elastic shear wave velocity, Poisson's ratio, thickness of soil layers etc are utilized.

To indirectly account for the different modes of ground movement in the proposed simple method, it is logical to consider different combinations of distributed load depending on the number of distinct soil layers over the pile length. Worst combination of the distributed load over different sections of the pile may be considered in evaluating the ground response effects.

The ground period elongation factor for a given site depends on the local site condition as well as the level of earthquake excitation. For extreme level of excitation at sites with deep soil deposit, where the ground response effect is likely to dominate, may be assumed to be in the range 2.0 to 3.0. The soil stiffness degradation may be obtained from the ground period elongation factor.

The coefficient of subgrade reaction obtained based on the proposed method tends to work out to be much smaller than that generally recommended for seismic design of piles based on the inertial forces alone. The inertia and ground response effects may be evaluated logically by assuming different values of subgrade reaction for the two cases as appropriate. If necessary, the coefficient of subgrade reaction for a given soil layer may be taken as zero to depict liquefied situation.

The degree of restraint of the pile head against rotation may be a major consideration in the evaluation of the bending moment distribution in piles due to ground response effects. If the moment at the pile head exceeds the yield level, partial restraint at the pile top may be logically assumed to limit the moment there to close to yield level. Conversely, if the bending moment at a section of the pile exceeds the yield moment capacity, the moment may be redistributed to limit the moment to yield level.

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