Significance of ground response effect in piles under earthquake loading

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ABSTRACT: Several instances of damage at intermediate part of piles during the Hyogoken-Nambu earthquake have been clearly identified. Such damages are inherently difficult to detect, and even more difficult to repair, and it is imperative that such damages should be as unlikely as possible. Nonlinear soil structure interaction (SSI) analysis with 2-D FEM model, as well as a simpler analysis with 1-D lumped mass model, provide consistent results concerning the potential and significance of ground response effects responsible for higher moment and shear in the intermediate part of the piles. The ground response effect is not accounted for in the current design practice. The proposed simple approach to evaluate the ground response effect is found to give results compatible with the 2-D and 1-D SSI analysis, indicating that the approach may be development and refinement for application to design practice. Further research needs are identified and emphasized.

1 INTRODUCTION

There are several reports on the damage to piles during the January 17, 1995 Hyogoken-Nambu earthquake (e.g. Karkee & Kishida 1997, Karkee et al. 1997, Matsui & Oda 1996 etc.). The reports include instances of failure in relatively long piles at intermediate location close to soil layer interfaces. Such failure to piles seem to have resulted due to the existence of lateral stiffness contrast between adjacent soil layers, including due to loss of strength of a layer due to liquefaction (Fujii et al. 1998).

The stresses in piles under earthquake loading can be regarded as consisting of the inertia effects of the superstructure as well as the kinematic effects of ground response. The kinematic effect is simply termed as the '*ground response effect*' in this paper. Although it is convenient in practice to make a distinction between 'inertia' and 'ground response' effects, is should be recognized that the two effects interact with each other and an attempt to separate the two may be only approximate.

The inertia effect is directly related to the extent of shaking to which the structure is subjected. The relative magnitude of the inertia and the ground response effects on piles depends on the ground condition as well as on the level of excitation. Level of excitation becomes important because the extent of shaking to which a structure at a given ground condition is subjected depends on the level of nonlinearity introduced in the ground by the incident motion (Karkee et al. 1992, 1993). In addition, the ground response effect for a given incident motion can vary substantially depending on the ground condition. Generally, long piles penetrating deep layered deposits, particularly if there are sudden changes in lateral soil stiffness, are expected to exhibit large ground response effects.

Analytical study (Sugimura et al. 1997 & Karkee et al. 1998) was carried out on a typical reinforced concrete tall building in Japan based on a two dimensional (2-D) nonlinear response analysis using finite element method (FEM). Three simple variation in the soil condition was considered and the analysis was based on the total stress. The results showed strong influence of soil layering on the response stresses induced in piles. Attempt was also made to quantify the contribution of the building part and that of the foundation part to the overall response of the soil-pile-building system under earthquake loading. It was found that the contribution of the building part itself depended on the nature of soil condition. In addition, the contribution of the foundation part to the response stresses in piles was seen to depend to a even larger extent on the nature of soil layers interacting with the piles.

Only the inertia effects tend to be explicitly accounted for in the seismic design of piles. In the Japanese practice, piles are designed to resist the forces due to inertia of the building, and the basement if applicable (BCJ 1984). The ground response effect that may even result in the failure of piles at deeper parts, such as those observed in the Hyogoken-Nambu earthquake, is not explicitly accounted for. The nonlinear analysis of the soil-pile-structure system based on the FEM representation was utilized to depict the extent and nature of stresses in piles attributable to the ground response effect (Karkee et al. 1998). One-dimensional (1-D) lumped mass model (Sugimura, 1973), also known as the modified

Penzien (1964) model, provides an alternative to the FEM analysis such that the computational rigor is greatly reduced while retaining the basic dynamic nature of the problem. The extent of ground response effects is investigated in detail based on the 1-D lumped mass model in this paper. The results are compared with those of the nonlinear response analysis based on the 2-D FEM model reported earlier (Karkee et al. 1998). It is shown that the soil structure interaction (SSI) analysis based on the 1-D lumped mass model may be advantageously utilized to evaluate the ground response effects if the soil parameters are adequately selected to account for the nonlinear behavior. A method to indirectly account for the nonlinear soil behavior is proposed.

Even the 1-D lumped mass model is not simple enough for common everyday design application. A simple method to consider the ground response effect was proposed by the authors (Karkee et al. 1998) based on the concept of distributed load method (Sugimura, 1992) and the principle of the beam on elastic foundation (Hetényi, 1946). The results of the detailed analysis based on the 1-D lumped mass model are compared with the results of the proposed simple method. It is shown that the proposed simple method provides adequate trend of the forces exerted on the pile due to ground response effect. With further investigations and refinement the simple method may find useful application in evaluating the extent of ground response effects in practice.

2 INVESTIGATIONS WITH 2-D FEM MODEL

In the previous study by the authors (Karkee et al. 1998, Sugimura et al. 1997), a typical multi-storied reinforced concrete apartment building in Japan was analyzed in detail considering three simple variations in soil condition. The schematic FEM model used for the nonlinear dynamic analysis is shown in Figure 1. The three soil profiles, designated as *a-soil*, *b-soil* and *c-soil*, are given in Figure 2. Details of the building structure and of the FEM analysis, together with the nonlinear material behavior for soil and concrete materials are as given in Sugimura et al. (1997). It was noted that the maximum shear force and bending moment in the intermediate part of the pile length were largest in case of the c-soil and occurred near the two layer interface. It was also noted that the maximum moment distribution in the pile was distinctly small in case of a-soil compared to that of b-soil when at lower level of excitation, whereas they were practically same at higher level of excitation. The result indicate the importance of considering the level of excitation (Karkee et al. 1992, 1993) as well as the soil condition while evaluating the effects of ground response. The level of excitation may be considered to be closely related to the extent nonlinearity in soil.

Two sets of analysis were undertaken in each case, one for the total soil-pile-building system and the other for the foundation sub-system. The difference between the two sets of analysis was regarded as the effect of building sub-system. As mentioned above, the separation of the contribution of the building sub-system this way is only approximate. Figure 3 shows the maximum moment diagram thus obtained for the building sub-system when the input earthquake motion was El Centro NS scaled to a peak velocity of 50cm/s.

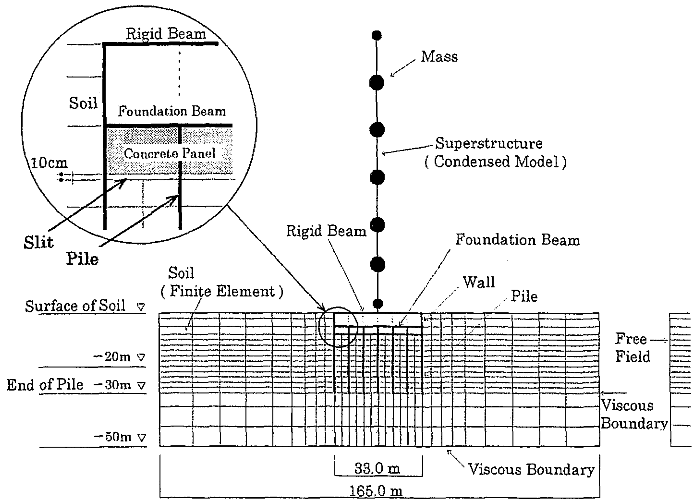

Figure 1. Schematic 2-D FEM representation

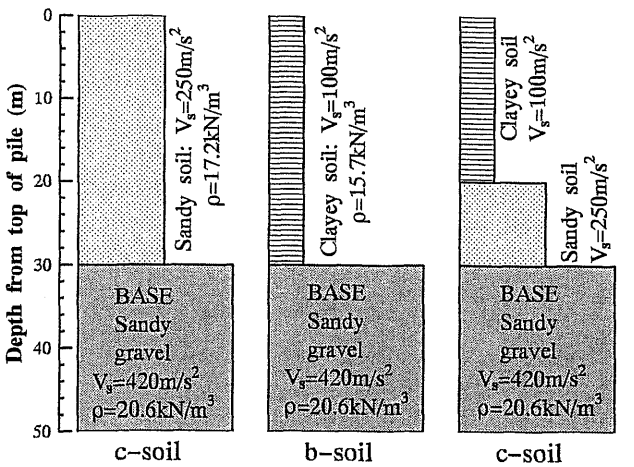

Figure 2. Details of the 3 soil conditions considered

The moment distribution approximated based on the design guide (Sugimura, 1988) by assuming the total lateral force resisted by piles to be $1.5Q_y+kW_B$, is also shown for comparison. Here Q_y (64.0MN) is the base shear capacity of the building, k is the seismic design coefficient for the basement and W_B (98.4MN) is the weight of the basement. It may be noted in Figure 3 that the moment at the pile head obtained for the static horizontal load are quite comparable to the contribution of building sub-system obtained from the nonlinear response analysis using FEM. However, the maximum moment for the total system as shown in Figure 4 is quite different from what might be obtained based on the design guide. The comparison in Figure 3 demonstrates that the current design guide tends to account for only the inertia effects of the building part.

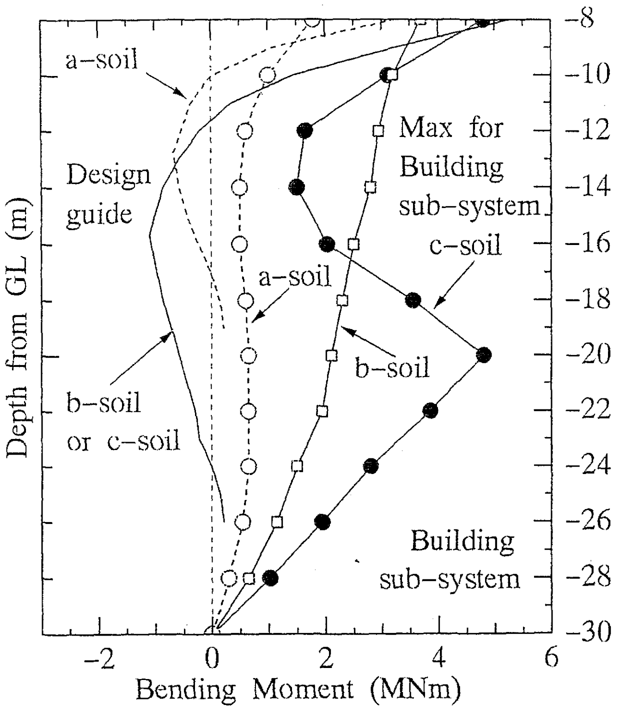

Figure 3. Comparison of maximum moment in pile for the building sub-system from FEM analysis with the moment obtained from the design guide.

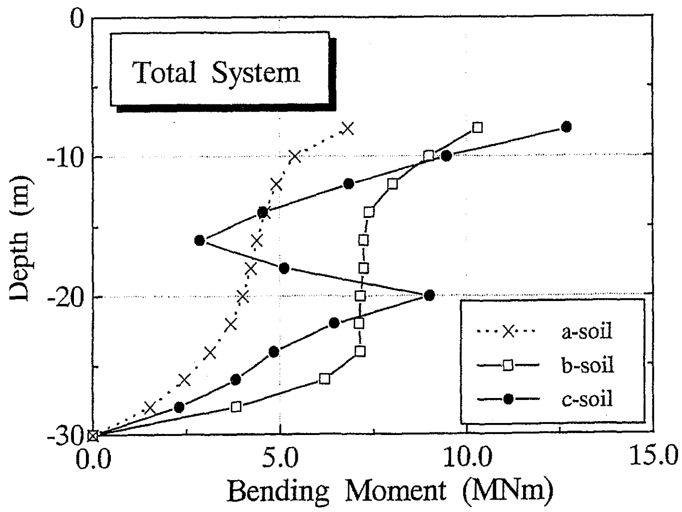

Figure 4. Maximum moment distribution in pile

3 ANALYSIS FOR 1-D LUMPED MASS MODEL

As an attempt to investigate the possibility of evaluating the extent of ground response effects by a simpler method, the method developed by Sugimura (1973) for the SSI analysis considering one dimensional (1-D) lumped mass model was utilized. In this method the soil-pile interaction spring is derived from the Mindlin's second solution and the method basically consists of elastic SSI analysis. As the results of FEM analysis indicated largest ground response effects in case of the c-soil, the 1-D lumped mass analysis was limited to this case. The input motion utilized was the El Centro NS record scaled to a peak velocity of 50cm/s, which is same as one of the input motions utilized in the FEM analysis. The schematic of the soil-pile-building in the 1-D model is shown in Figure 5, where the interconnecting springs and dashpots are omitted for clarity. Rocking of the basement was also considered.

3.1 *Soil parameters for the 1-D SSI analysis*

The 1-D lumped mass model for the SSI analysis considered here basically assumes linear behavior of the soil as well as the structure. Considering the fairly soft soil condition (c-soil in Figure 2) to which building is founded and the strong level of shaking for which the investigation is intended, attempt was made to indirectly account for the nonlinear soil behavior. For this the 1-D nonlinear response analysis of the free field was first undertaken. The nonlinear analysis was utilized to evaluate the maximum shear strain reached in different soil layers interacting with the pile and the basement masses.

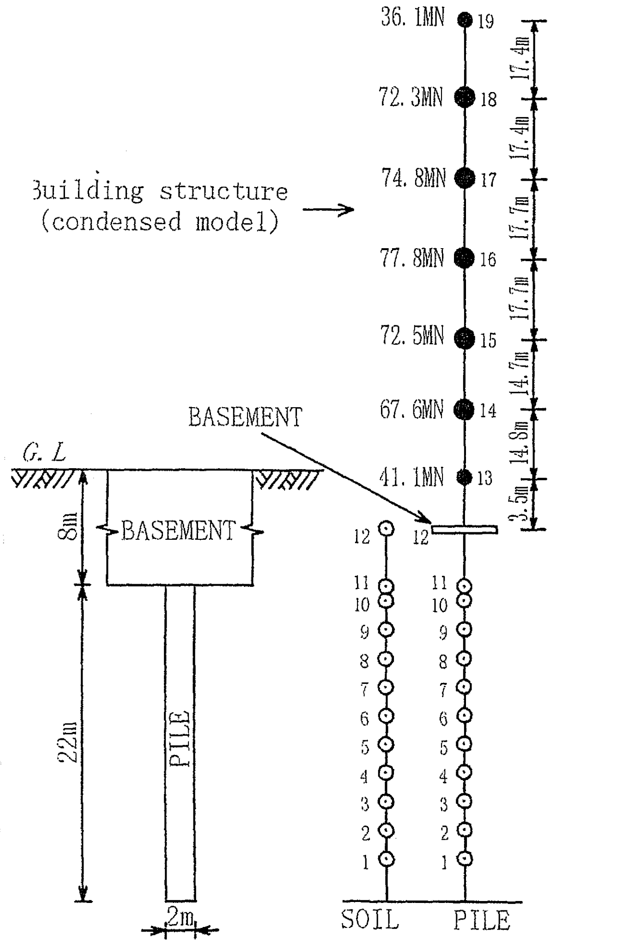

Figure 5. Lumped mass model for 1-D SSI analysis

The nonlinear relationship representing the strain dependence of the shear modulus and the damping factor was same as that utilized in the earlier FEM analysis (Sugimura et al. 1997). In the 1-D SSI analysis the pile was divided into 11 lumped masses and the basement was considered as a single lumped mass. Accordingly, the number of interacting soil masses was altogether 12 as shown in Figure 5. The shear modulus and the damping factor for these interacting soil layers were set as the values corresponding to strain levels of 65% of the maximum strain during the nonlinear analysis of the free field mentioned above.

3.2 *Results of the1-D SSI analysis*

Two sets of analysis were carried out, similar to those in the FEM analysis, one for the *total system* shown in Figure 5 and the other for the *foundation sub-system*, with the building masses removed. Figure 6 shows the strong part of the input motion where the 6 peaks of larger than 300cm/s^2 have been marked with letters P to U. The peak acceleration of *510.8cm/s^2* is seen to occur at point Q in Figure 6.

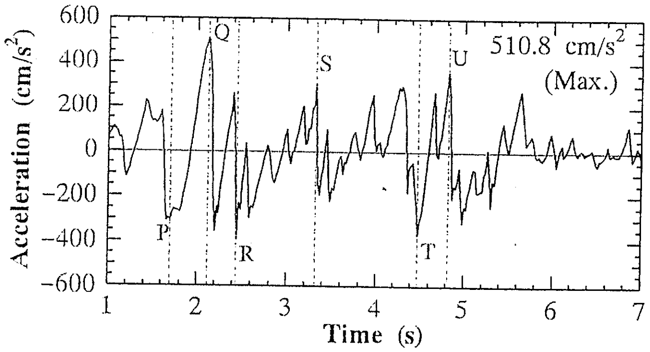

Figure 6. Strong part of the input motion history

The ground response effects may be regarded as more closely related to the displacement response rather than to the acceleration response. Figure 7 compares the time history of the pile head displacement response relative to the base layer for (a) total system and (b) foundation sub-system. While the overall magnitudes are quite comparable, the displacement response at pile head for the total system is seen to be dominated by longer period components when compared to that for the foundation sub-system. The observation indicates that the contribution of building part is primarily in the frequency content of the displacement response rather than its magnitude. That is, the magnitude of the displacement response depends mostly on the foundation sub-system.

Figure 8 compares the bending moment response near the mid-point of the pile (actually pile mass # 5 in Figure 5) for (a) total system and (b) foundation sub-system. The period contents in the bending moment response for the total system in Figure 8(a) correspond closely to the pile head displacement response history shown in Figure 7(a). Although not as close, similar tendency can be noted for foundation sub-system when Figures 7(b) and 8(b) are compared. The observations indicate that the bending moment response at the middle part of the pile length tend to correlate with the pile top displacement response. Since the magnitude of displacement response primarily depends on the foundation sub-system as noted above, it follows that the ground response effects on piles during earthquake loading can be estimated from the displacement response of the foundation sub-system.

It may also be noted that there are no maximum peaks in Figures 7 and 8 corresponding to the peak acceleration denoted by Q in Figure 6, indicating the difficulty in estimating the ground response effects based on a certain value of peak input acceleration.

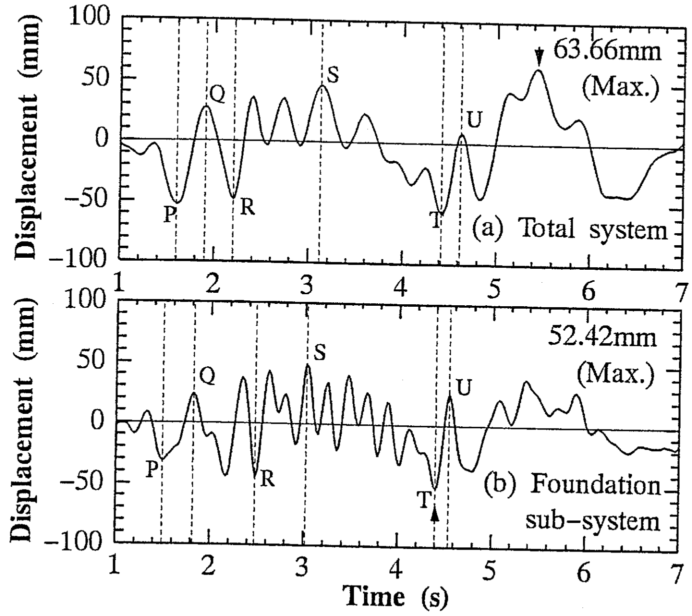

Figure 7. Pile head displacement response history

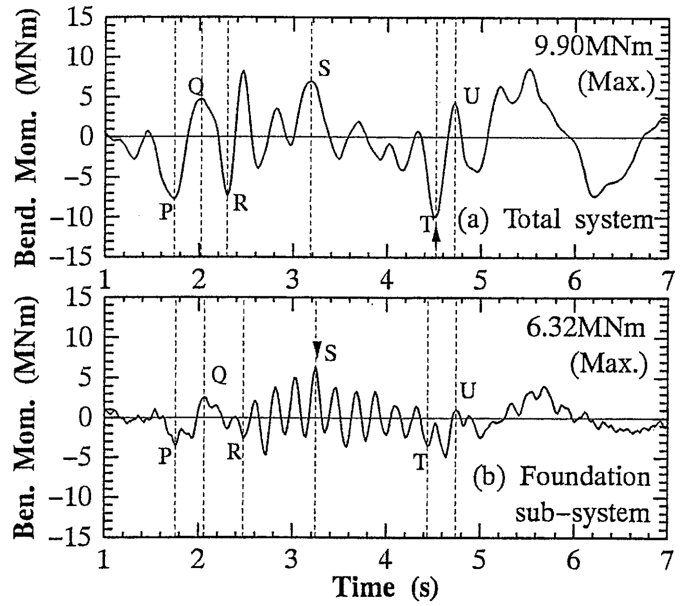

Figure 8. Pile midpoint bending moment response

3.3 *Comparison between 2-D FEM and 1-D results*

Figure 9 compares the distribution of the maximum bending moment in the pile obtained from the 1-D lumped mass analysis with that obtained earlier from the 2-D FEM analysis (Karkee et al. 1998). It is seen in Figure 9(b) that the maximum bending moment diagram for the foundation sub-system obtained by the two methods compare very well, particularly at the middle and lower part of the pile. The results are comparable at the middle and lower part of the pile for the total system as well, as seen in Figure 9(a). However, the maximum bending moment near the pile top tends to be larger in case of the 1-D analysis. The difference is particularly large for the total system. This is because the bending stiffness of the pile is assumed to be constant in the 1-D analysis.

Actually, a plastic hinge is expected to be formed when the yield level is reached, such that the moment does not increase significantly beyond yield level.

In case of the FEM analysis the bending moment was made to depend on the curvature (Sugimura et al. 1997), such that the bending moment increases with decreased rate after cracking moment M_{cr} and it does not increase any further beyond yield moment M_y. This is not accounted for in the 1-D method. The ranges of M_{cr} and M_y are shown by dotted lines in Figure 9, where larger differences between 1-D and 2-D FEM methods are prominent beyond M_{cr} and become very large beyond M_y. The maximum bending moment distribution from FEM and 1-D analysis are very close in the range of less than or slightly higher than the cracking moment M_{cr}.

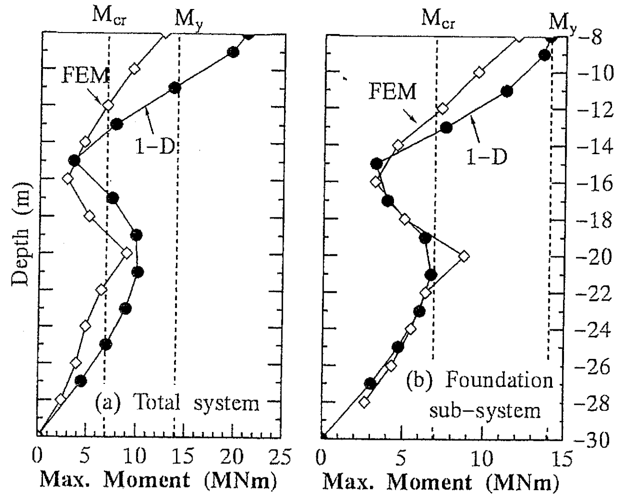

Figure 9. Maximum moment in piles from the SSI analysis based on FEM and 1-D lumped mass model.

Figure 10 compares the maximum shear force distribution obtained from the 1-D and the 2-D FEM methods, where agreement is not so good for both the total system and the foundation sub-system. However, the maximum shear force, considered the design values, are quite comparable, with the 1-D method giving slightly higher values for the total system. The general trend of the distribution is also quite comparable.

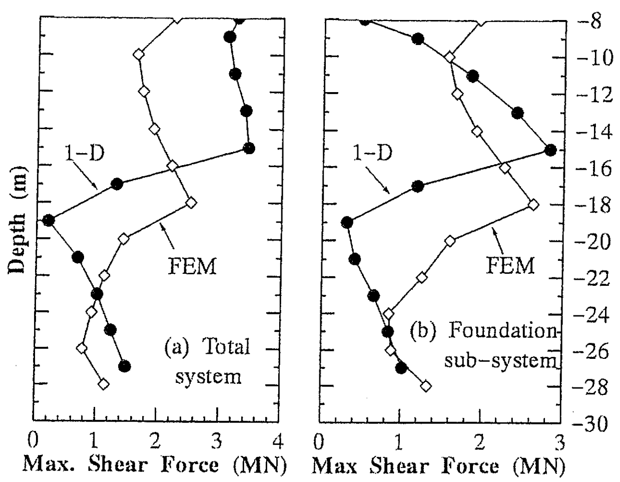

Figure 10. Maximum shear force distribution in piles from the FEM and the 1-D lumped mass model.

3.4 *Bending moment distribution during shaking*

Figures 9 and 10 show the distribution of the maximum bending moment along the pile length under earthquake loading. Next the nature of the time dependence of the bending moment and shear force distribution is depicted in Figures 11 and 12 respectively. Figure 11 shows the bending moment distribution at selected times during strong part of the shaking. Bending moment due to the total system (black circles) and the foundation sub-system (open circles) are plotted for comparison. Corresponding shear force distributions are given in Figure 12.

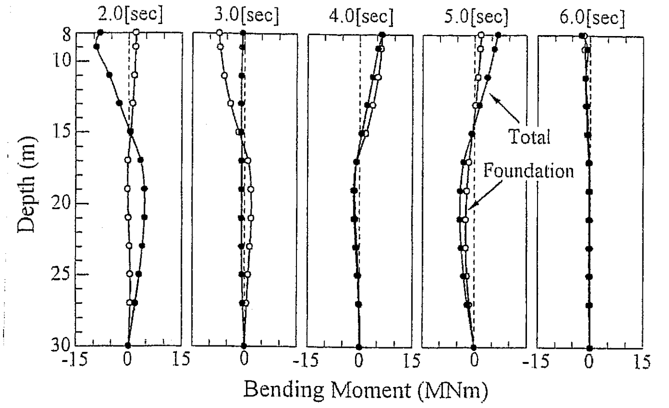

Figure 11. Distribution of bending moment along the pile length at selected times during strong shaking

Figures 11 and 12 indicate that the moment and the shear in the pile for the foundation sub-system at certain times can be even larger than those due to the total system. However, when the envelop of the maximum values during shaking is considered, the bending moment and shear force due to the total system seem to work out to be larger, as indicated by Figures 9 and 10.

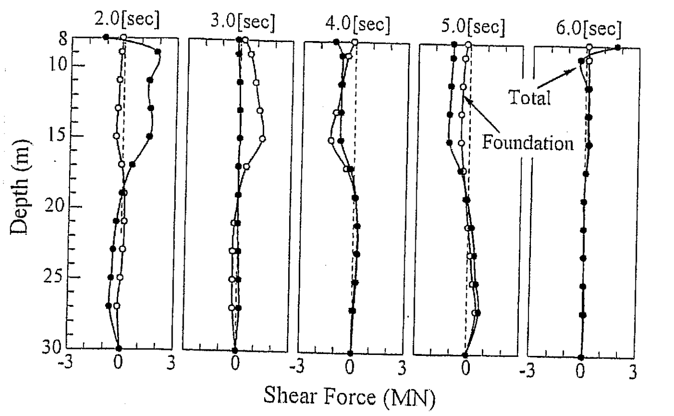

Figure 12. Distribution of the of shear force along the pile length at selected times during strong shaking

4 SIMPLE APPROACH FOR EVALUATION OF GROUND RESPONSE EFFECTS

As mentioned above the current design method tend to consider only the inertial effect of the superstructure and may be quite adequate when recognized as such. However, the ground response effects can be substantial and dominant depending on the ground condition and the level of excitation encountered. A simple approach was proposed (Karkee et al. 1998) for the evaluation of the ground response effect applicable to everyday design practice. Attempt is made here to see how the proposed simple method compares with the results of 1-D analysis. As noted above, the ground response effects tend to relate well with the displacement at pile top. This provides a basis for the proposed simple method directly based on the estimation of the extent of displacement.

4.1 *Outline of the proposed method*

The analysis method is based on the principle of beam on elastic foundation that utilizes a distributed load rather than a concentrated load at the top. The magnitude and the nature of the distributed load *p(x)* is estimated based on the local site condition with the nonlinear soil behavior considered indirectly. For this the displacement response *f(x)* of the free field under earthquake loading is estimated. Considering the first mode of the free field motion, the maximum displacement at the top of a soil layer of thickness H and shear wave velocity V_S may be given in terms of the peak velocity of the input motion V_{max} by Equation 1. It may be noted that the input motion for the design of high-rise buildings is generally defined in terms of V_{max} in Japan.

$$U_g = V_{\max}\left(\frac{2H}{\pi V_S}\phi\right) \quad (1)$$

The parameter ϕ in Equation 1 is the factor by which the ground period may elongate due to nonlinear effect (Karkee et al. 1992, 1993) during strong ground shaking, such that $\phi \geqq 1$. The introduction of this parameter constitutes an attempt to account for the nonlinear effects in soil response indirectly. Japanese design guideline (BCJ, 1992) recommends of ϕ =2.2 for deep deposits in bay areas. With the displacement U_g relative to the bottom of a layer known, the ground displacement *f(x)* at any depth x may be obtained by assuming a cosine function (Hasegawa & Nakai, 1992) for the distribution across the depth H of the layer. Thus the displacement *f(z)* relative to the bottom of a soil layer, where z varies from 0 at the top to H at the bottom, may be given by Equation 2. Once the displacements relative to the bottom of each layer is computed, the overall displacement *f(x)* relative to the base layer can be easily obtained, such that *p(x)* is given by Equation 3.

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

$$p(x) = k_h D f(x); \quad 0 \le x \le L \quad (3)$$

One of the crucial aspect in the proposed method is the proper evaluation of the coefficient of subgrade reaction k_h in Equation 3, because the magnitude of the distributed load *p(x)* for a given *f(x)* depends on the value of k_h. It may be noted that the elongation of the ground period by ϕ times corresponds to a soil stiffness degradation by $1{:}\phi^2$. This can be adequately accounted for in estimating the value of k_h, which may be computed from Equation 4 derived by Vesic (Poulos and Davis 1980) considering an infinitely long beam on elastic foundation.

$$k_h = \frac{0.65}{D}\left\{\frac{E_S}{1-\nu^2}\right\} \times \left\{\frac{E_S D^4}{EI}\right\}^{\frac{1}{12}} \quad (4)$$

$$\lambda = \frac{E_0}{E_S} \quad (5)$$

In Equation 4, *EI* is the bending stiffness of the pile, ν is the Poisson's ratio of the soil and λ is the stiffness degradation factor. λ indicates the extent to which the stiffness is reduced during strong shaking, which in turn results in the elongation of the ground period. If the different soil layers at a local site are assumed to contribute equally to the ground period elongation ϕ, the value of λ may be assumed to be ϕ^2.

4.2 *Computations based on the proposed method*

Attempt is made to compute the bending moment and shear force distribution in pile for the case of c-soil by assuming the value of ϕ to be 2.0, corresponding to the ground period elongation for c-soil under the El Centro input (Sugimura et al. 1997). Figure 13 shows the distribution of the coefficient of subgrade reaction k_h obtained from Equation 4 together with the soil displacement estimated by Equation 1. The Poisson's ratio ν of the soil is assumed to be 0.45. It is seen that the displacement of the softer soil layer is much larger compared to the underlying stiffer layer, such that the ground displacement near the pile top is about 100mm. The distributed load *p(x)* obtained from Equation 3 is also shown in Figure 13. The cosine distribution of the load was approximated by uniformly varying loads at discrete intervals denoted by the black dots in Figure 13.

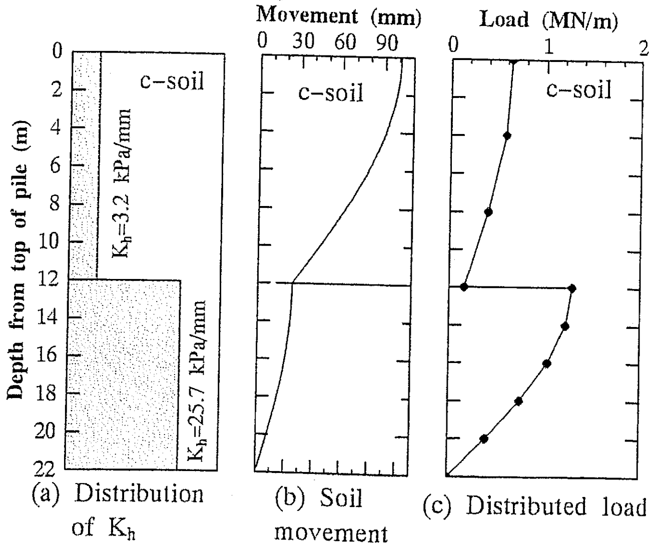

Figure 13. Subgrade reaction and soil displacement profiles and the distributed load for the analysis.

The analysis was carried out by considering the pile as a beam on elastic foundation under the action of the distributed load of Figure 13(c). Two cases of distributed load were considered as shown in Figure 14, including the case of distributed load on upper and lower layers in opposite directions. The boundary condition at the pile top was assumed to be fixed with horizontal displacement allowed. The pile tip was assumed to be free.

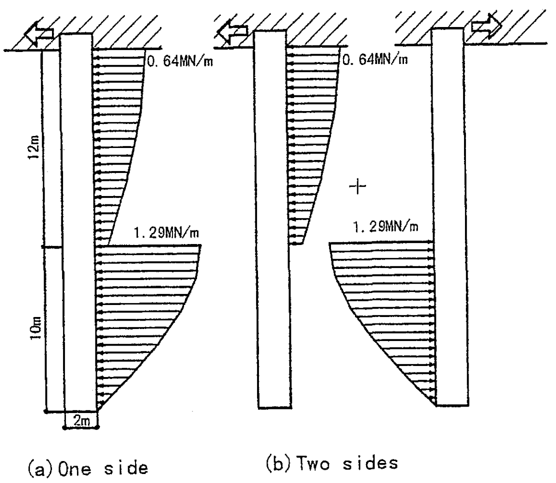

Figure 14. Two loading cases of the distributed load

The bending moment and the shear force diagrams are given in Figure 15. Again it may be noted that the bending moment at the pile top exceeds the yield moment M_y. Actually, a plastic hinge may be assumed to be formed when the moment reaches M_y, thus keeping the maximum bending moment at the top to yield level. Alternatively, a certain degree of fixity may be specified and accounted for in the analysis for beam on elastic foundation. It may be noted in Figure 15 that the shear force and bending moment tend to be large around the middle part when the distributed load is acting in opposite directions.

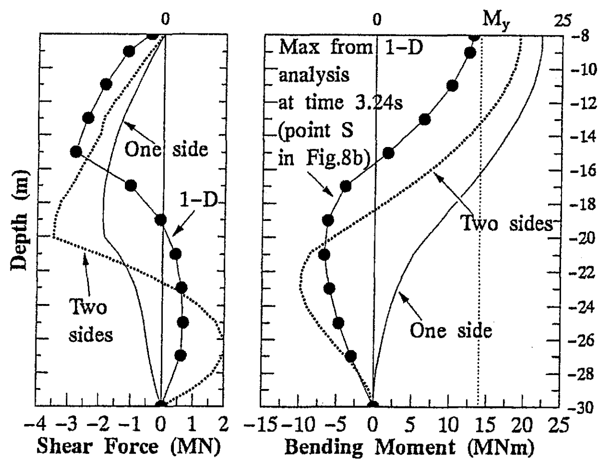

Figure 15. Shear force and bending moment diagrams

4.3 *Comparison with 1-D analysis results*

The forces exerted by the distributed load on the pile shown in Figure 15 may be considered to indicate a reasonable trend. As the pile itself is considered to behave elastically in the analysis considering a beam on elastic foundation, it may be reasonable to compare the results with the results of 1-D analysis, which also assumes the pile to behave elastically. The comparison is also given in Figure 15.

As observed in Figures 11 and 12, the bending moment and shear force distribution in the pile during the 1-D SSI response analysis depends on the time under consideration. For the purpose of the comparison with the results of distributed load method, it may be logical to consider the distribution when the maximum value occurs at the middle part of the pile. For example, the maximum bending moment at the middle part of the pile for the foundation sub-system occurs at time 2.34s as shown in Figure 8(b). The distribution of shear and bending compared in Figure 15 are obtained in this manner.

Of course the distribution of the shear and the moment distribution obtained by the proposed simple method does not compare point to point with the results of 1-D SSI analysis. However, the general trend and the extent of values compare reasonably, indicating strong potential for the development of the approach into a concise method of accounting for the ground response effects in pile design practice.

4.4 *Further research on ground response effects*

The proposed practical approach to evaluate the ground response effect shows ample promise and potential for design application. Further investigation based on a large number of simulations considering different soil conditions to compare the results of detailed response analysis would be required for further refinement and development of the method for design applications. Experimental investigations and objective comparison with observations on the behavior of piles during earthquakes would provide validation and valuable insight in the process.

It may also be noted that that the current design guide in Japan seems to provide adequately for the inertia effects of the superstructure. However, the coefficient of subgrade reaction k_h utilized in the proposed approach to evaluate the ground response effect is generally smaller and the displacement near pile top larger that the corresponding values applicable in the present design guide. Further investigations would be required to clarify these aspects. There is also the question of whether it is more convenient to have separate analysis for inertial and ground response effects or to have a combined method to account for both.

5 CONCLUSIONS

Reports on behavior of pile foundations during the Hyogoken-Nambu earthquake show several instances of damage attributable to ground response effects. Results of nonlinear response analysis based on the 2-D FEM representation illustrate the significance and the need to account for ground response effects in design. While greatly reducing the computational rigor of 2-D FEM analysis, 1-D lumped mass method provides results comparable to those of the 2-D FEM analysis, and provides further credence to the significance of ground response effects. The nonlinear soil behavior in the 1-D SSI analysis was accounted for indirectly by selecting the shear modulus and damping values of soil corresponding to 65% of the maximum strain during the nonlinear analysis of the free field. The 1-D method gives higher moment at pile top because the cracking and yield moment capacity considered in the 2-D FEM analysis is not accounted for in the 1-D analysis.

The moment at the pile top that may be obtained from the current design guide is quite close to the maximum moment contribution of the superstructure obtained from the SSI analysis. When the ground response effects are included, the moment and shear along the pile length can be much larger depending on the soil condition and the level of earthquake excitation. Proposed simple approach based on the concept of distributed load and the beam on elastic foundation provides adequate representation of moment and shear distribution, similar to those obtained from detailed SSI analysis. The proposed simple approach shows ample potential for design application as a concise method of evaluating the ground response effects. Further research needs and directions are pointed out and emphasized.

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