

NATURE OF DAMAGE TO PILE FOUNDATION OF BUILDINGS IN RECENT EARTHQUAKES AND SOME PROPOSALS FOR SEISMIC DESIGN

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ABSTRACT: Observations of damage to pile foundation of buildings in recent major earthquakes indicate substantial instances of the damage at deeper part of the piles. Generally such damages tend to be common at interfaces of soil layers with prominent stiffness contrast. It is evident that the damages occurring at deeper part of piles are inherently difficult to detect and practically impossible to repair. Consequently, adequate provision in the design is indispensable to make such damages as unlikely as possible. Previous studies based on nonlinear response analysis of the soil-pile-structure system represented by two-dimensional (2-D) FEM model indicated distinctly large kinematic response forces at soil layer interfaces, demonstrating the nature of stresses developed in piles due to distinct stiffness contrast between soil layers. It is also shown that a simpler 1-D lumped mass model can adequately replicate the nature of actions on piles obtained from the 2-D FEM model. While such detailed analysis may be impractical for general design application, the current design practice does not account for the kinematic actions due to displacement response of ground. Showing the extent of actions disregarded by the current method, a simple design approach to account for the effects of the displacement response of ground is proposed, and its potential for realistically representing the displacement response actions is illustrated.

KEYWORDS: Pile foundations, Earthquake damage, Seismic design, Nonlinear response, Ground movement, Displacement response method

INTRODUCTION

From the investigations following the Miyagiken-oki earthquake of 1978, a number of cases of the shear failure close to the top of PHC (Prestressed High Strength Concrete) piles supporting residential buildings was observed in Sendai City. Following these observations, the necessity for the earthquake resistant design of piles was adequately recognized from the administrative point of view. Accordingly, the design standard (Ministry of Construction Guidelines) for the earthquake resistant design of piles was published in 1984. Being limited to the recommendation based on the allowable capacity design (the so-called primary design), the standard may have been inadequate in fully grasping the failure state of piles in the design. Besides, the standard did not have any legal force. Consequently, the questions of how much the standard was referred to and applied in actual design practice in the years that followed is difficult to understand. Reports on the investigation of buildings with pile foundations affected by the Hyogoken-Nambu earthquake of 1995 indicate reoccurrence of the nature of damage to PHC piles observed in the Miyagiken-oki earthquake of 1978. In addition, another distinctive nature of the damage to relatively long piles was observed in the Hyogoken-Nambu earthquake. Several reports are

available on the damage to piles (e.g. Karkee & Kishida 1997, Karkee et al. 1997, Matsui & Oda 1996 etc.) during the Hyogoken-Nambu earthquake, and many of these reports include instances of failure at deeper parts of relatively long piles, frequently at locations close to distinct soil layer interfaces. Such failure to piles seem to result due to the existence of lateral stiffness contrast between adjacent soil layers, including the liquefaction and loss of strength at an intermediate layer (Fujii et al. 1998). The damage, and sometimes breakage, at lower part of piles, often at several meters below the ground surface, are reported to have been predominant at sites where significant lateral movement of ground occurred, usually due to liquefaction.

The stresses developed in piles under earthquake loading can be regarded as consisting of the inertia effects of the superstructure as well as the kinematic effects of displacement response of ground. The inertia effect is directly related to the extent of shaking to which the structure is subjected. The extent of the kinematic effects depends on the ground condition to which the building is founded as well as the level of incident excitation including nonlinear behavior of soil. The kinematic effect is simply termed as the ‘displacement response effect’ in this paper. Although it is convenient in practice to make a distinction between ‘inertia’ and ‘displacement response of ground’, it should be recognized that the two effects interact with each other during earthquake shaking and any attempt to separate the two effects can be only approximate.

The relative magnitudes of the effects due to the superstructure inertia and the displacement response of ground depend on the ground condition as well as on the level of excitation. The extent of shaking to which the structure is subjected depends on the level of nonlinearity introduced in the ground by the level of incident excitation (Karkee et al, 1992, 1993). In addition, the displacement response of ground for a given incident excitation 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 displacement response effects.

Analytical study (Sugimura et al. 1997 & Karkee et al. 1998) was carried out on a typical reinforced concrete apartment building common in Japan based on the two-dimensional (2-D) nonlinear response analysis using the finite element method (FEM). Three simple variations in the soil condition were considered and the analysis was based on the total stress method. The results show strong influence of soil layering on the response stresses induced in piles. Attempt was made to quantify the contribution of the building part and that of the foundation part to the overall response of the dynamic 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. The contribution of the foundation part, however, was found to depend to a much larger extent on the nature of soil layers interacting with the piles. The results indicate the severity of displacement response effects depending on the ground condition.

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 displacement 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 found to depict adequately the extent and the nature of stresses developed in piles due to displacement response of ground. One-dimensional (1-D) lumped mass model (Sugimura, 1973), generally referred to 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 problem of displacement response effect is investigated in greater detail in this paper based on the 1-D lumped mass model. 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 displacement response effects if the soil parameters are adequately selected to account for the nonlinear

behavior. The results of the SSI analysis are utilized to identify some of the important considerations involved in the earthquake resistant design of piles supporting buildings. A simple practical method to indirectly account for the nonlinear soil behavior is proposed based on the observed behavior and the results of the dynamic SSI analysis. The proposed design approach includes the effects of nonlinearity of the soil as well as the essence of the seismic response of ground.

Even the 1-D lumped mass model is not simple enough for common everyday design application. A simple method to consider the displacement response effect in practice 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 method provides adequate estimation of the forces exerted on the pile due to displacement response effect of ground. With further investigations and refinement the simple method may find useful application in practice.

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 displacement response in the design of piles. The level of excitation may be considered to be closely related to the extent nonlinearity in soil.

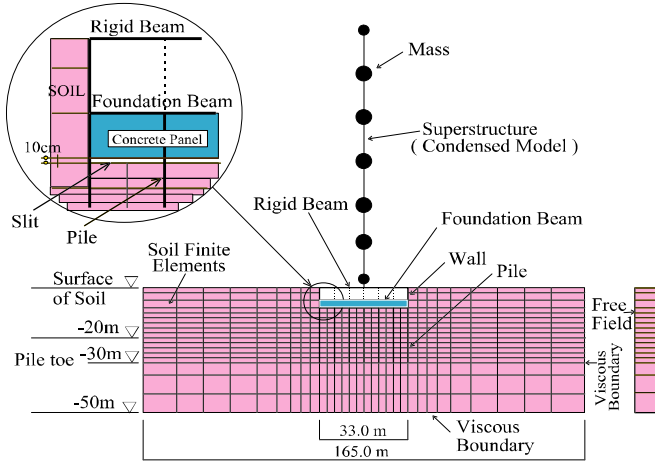


Figure 1. Schematic model of 2-D FEM representation

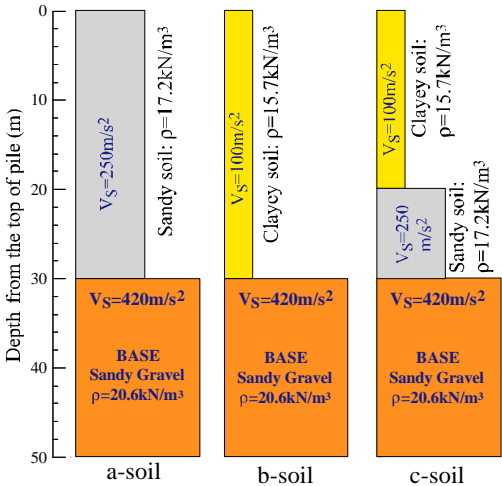


Figure 2. Details of the three types of soil condition considered

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.

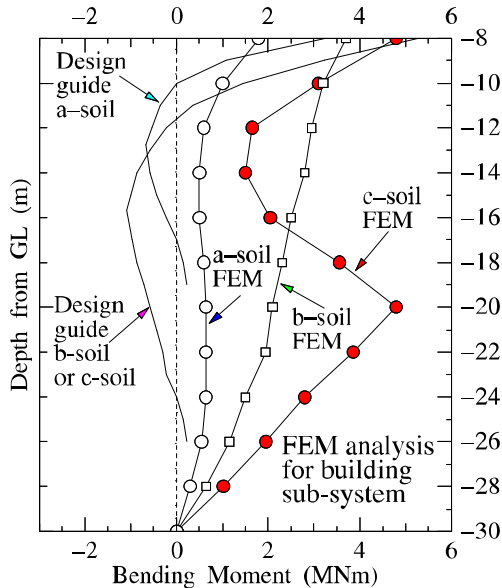


Figure 3. Maximum moment in pile for building sub-system from FEM analysis and the moment from design guide.

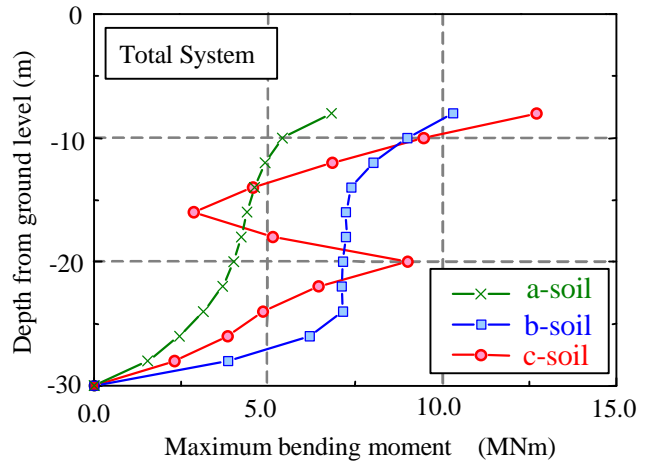


Figure 4. Maximum moment in distribution in pile

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.

ANALYSIS FOR 1-D LUMPED MASS MODEL

To evaluate the extent of displacement response effects by a comparatively 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 thus consists of elastic SSI analysis. As the results of FEM analysis indicated largest displacement 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 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 basement was also considered.

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 basement masses.

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 was set as the values corresponding to strain levels of 65% of the maximum strain during the nonlinear analysis of the free field mentioned above.

RESULTS OF THE 1-D SSI ANALYSIS

Two sets of analysis were carried out, one for the *total system* shown in Figure 5 and the other for the *foundation sub-system*, with the building masses were 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 and at time 2.12s.

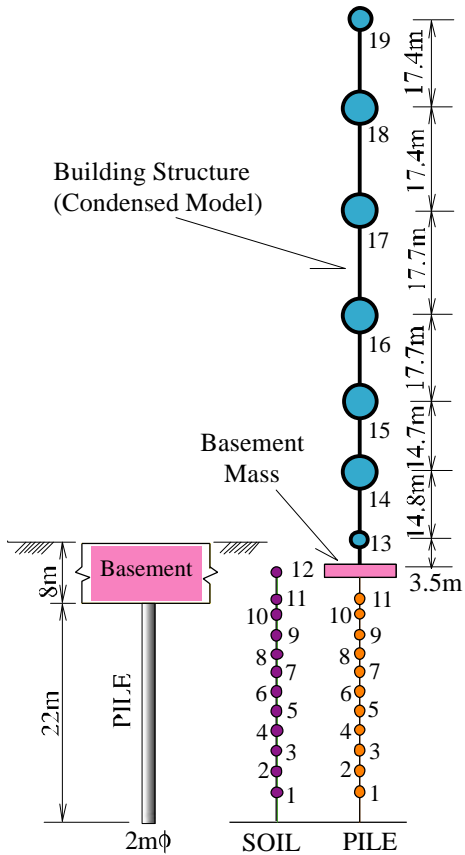


Figure 5. Lumped mass model for the 1-D SSI analysis (spring & dashpot details not shown)

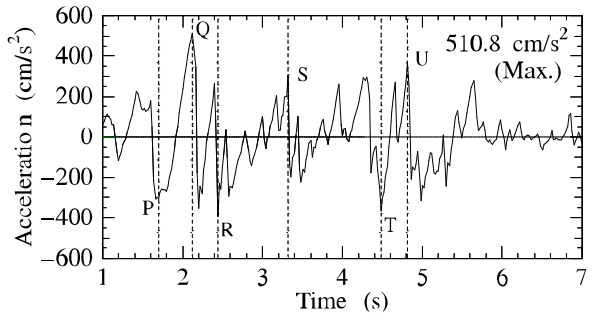


Figure 6. Strong part of the input motion time history

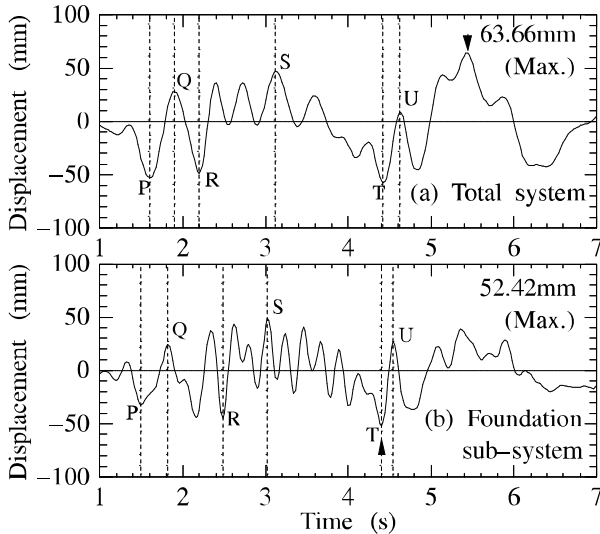


Figure 7. Pile head displacement response history

The displacement 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 the (a) total system and the (b) foundation sub-system. It is clear that displacement response at pile head for the total system is larger and dominated by longer periods when compared to that for the foundation sub-system. That is, the building part contributes not only to the magnitude of the displacement response but also to the period content of the response.

Figure 8 compares the bending moment response at around the mid-point of the pile (actually pile mass # 5 in Figure 5) for the total system and the foundation sub-system. The period content in the bending moment response for the total system closely correspond to that of the pile head displacement response history shown in Figure 7. Similarly, the period content in the bending moment response at mid-point of pile tends to correspond to that in the pile head displacement response in case of the foundation sub-system. These observations indicate that the bending moment response at the middle part of the pile length tends to correlate well with the displacement response at the pile top. Considering that the stresses exerted in piles at the middle part of the pile tend to be dominated by the displacement response effects rather than the inertial effect of the superstructure, it can be further surmised that the displacement response effects on piles during earthquake loading tend to relate closely to the displacement response.

COMPARISON BETWEEN 2-D FEM AND 1-D RESULTS

Figure 9 compares the distribution of the maximum bending moment in piles obtained from the FEM analysis and the 1-D analysis. It is seen that the maximum bending moment diagram for the foundation sub-system obtained by the two methods compare quite well. The maximum moment distribution at the middle part of the pile is quite similar in Figure 9 for the total system as well. However, the maximum bending moment at the pile head is much larger in case of the total system and also in case of foundation sub-system to some extent. This is because the bending stiffness is assumed to be constant in case of the 1-D analysis.

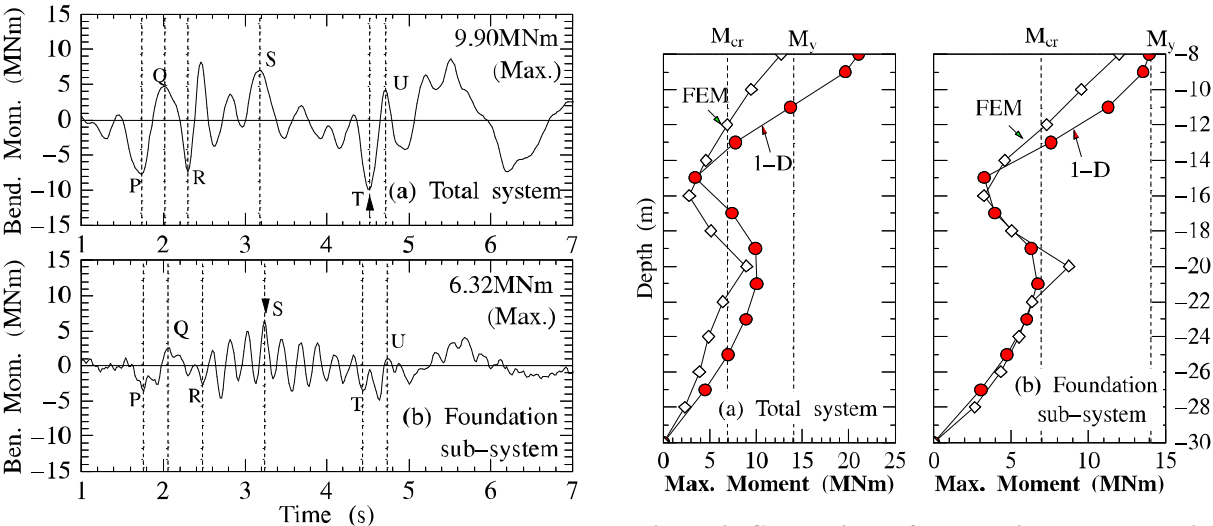


Figure 8. Pile midpoint bending moment response

Figure 9. Comparison of the maximum moment in piles obtained from 2-D FEM & 1-D SSI analysis.

In case of the FEM analysis the bending moment resistance is made to depend on the curvature (Sugimura et al, 1997), such that the bending moment increases with decreased rate after cracking moment M_{cr} is reached and it does not increase beyond the yield moment M_y . This has not been accounted for in the 1-D method. The ranges

of M_{cr} and M_y are shown in Figure 9 by dotted lines for direct comparison. It is seen that the maximum bending moment distribution from FEM and 1-D analysis are very close when bending moment is less than or slightly higher than cracking moment M_{cr} .

Similarly, Figure 10 compares the maximum shear force distribution obtained from the two methods of analysis. There is quite a difference in the shear force distribution for both the total system and the foundation sub-system. However, the extent of the values of shear force applicable for design of piles obtained by the two methods are of the same order. It seems that the 1-D method tends to give higher maximum shear force values for design compared to the FEM analysis, thus providing some conservatism. Also the general trend of the variation of maximum shear forces along the pile length, obtained by FEM and 1-D methods, are seen to be quite similar in Figure 10.

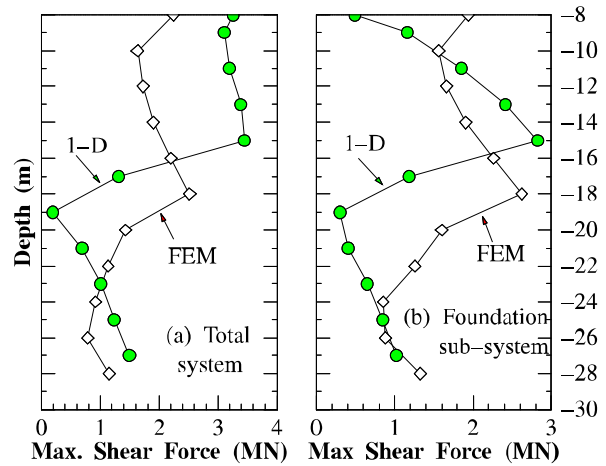


Figure 10. Comparison of maximum shear force in piles obtained from the 2-D and 1-D SSI analysis

BENDING MOMENT DISTRIBUTION DURING SHAKING

Figures 9 and 10 show the distribution of the maximum bending moment along the pile length under earthquake loading. Nature of the time dependence of the bending moment distribution can be understood from Figure 11, where the bending moment distribution at selected times during strong part of the shaking is given. Bending moment due to the total system (black circles) and the foundation subsystem (open circles) are plotted. The corresponding shear force distribution are plotted in Figure 12.

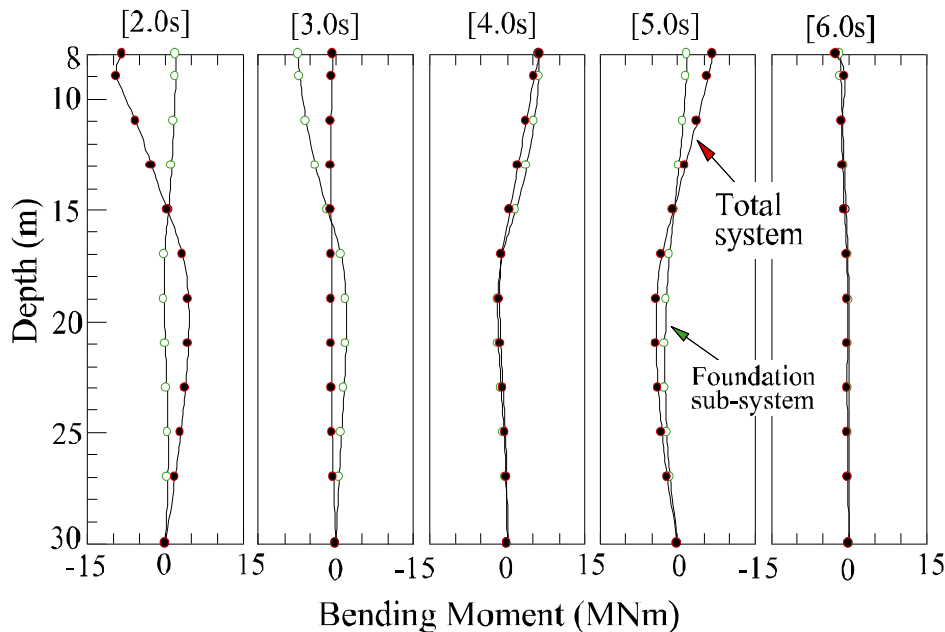


Figure 11. Bending moment distribution in piles at selected times obtained from 1-D SSI analysis

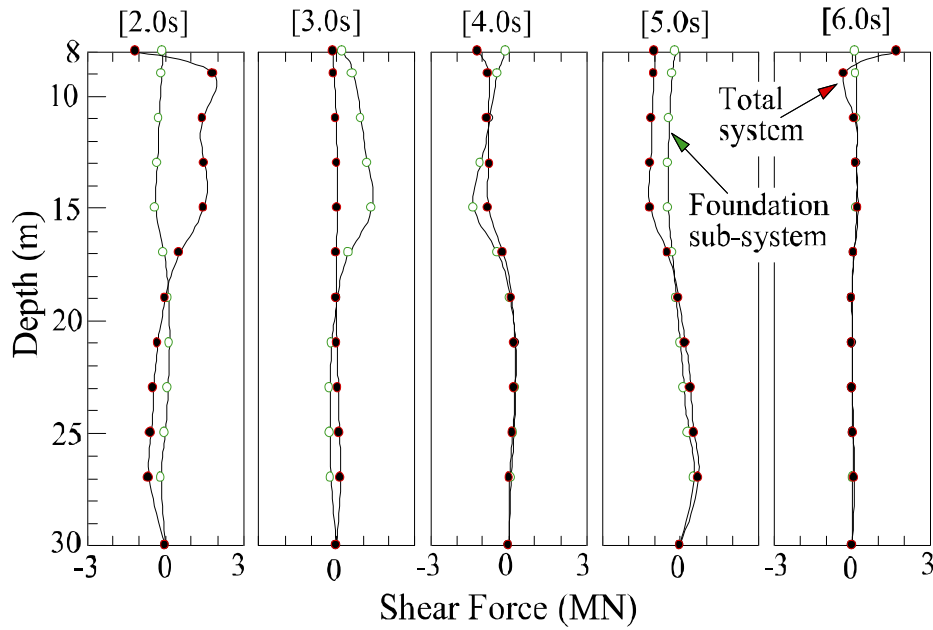


Figure 12. Shear force distribution along pile length at selected times obtained from 1-D SSI analysis

It is clear from Figures 11 and 12 that the bending moment and the shear force due to the foundation sub-system 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 work out to be larger, as indicated by Figures 9 and 10.

DISPLACEMENT RESPONSE IN PRACTICE

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

OUTLINE OF THE PROPOSED METHOD

The method is based on the principle of beam on elastic foundation and utilizes the distributed load acting on the beam rather than a concentrated load at the top. The magnitude and the nature of the distributed load $p(x)$ is determined taking into consideration the local site condition and the nonlinear soil behavior. 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 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. The input motion for the design of high-rise buildings is generally defined in terms of V_{max} in Japan.

$$U_g = V_{max} \left(\frac{2H}{pV_s} \mathbf{f} \right) \quad (1)$$

The parameter f 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 $f \geq 1$. The introduction of this parameter constitutes an attempt to account for the nonlinear effects in soil response. Japanese design guideline (BCJ, 1992) recommends a value of f as 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.

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

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

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 its value affects 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 f times corresponds to a soil stiffness degradation by $1:f^2$. 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 infinite beam on elastic foundation.

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

$$f = \frac{E_0}{E_s} \quad (5)$$

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. Value of λ indicates the extent to which 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 f , the value of λ may be assumed to be f^2 and the same for all the layers.

COMPUTATIONS BASED ON THE PROPOSED METHOD

Attempt is made to compute the bending moment and shear forces in piles for the case of c-soil by assuming the value of f 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 distribution estimated by Equation. The Poisson's ratio ν of the soil 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 at 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 distributed load was approximated by uniformly varying loads at discrete intervals denoted by the black dots in Figure 13.

The analysis was carried out by considering the pile as a beam on elastic foundation based on the distributed load shown in Figure 13(c). Actually two cases of distributed load were considered as shown in Figure 14, including the case of distributed load on upper and lower layers of the two layers in c-soil. The boundary

condition at the pile top was assumed to be fixed with horizontal displacement allowed. The pile tip was assumed to be free.

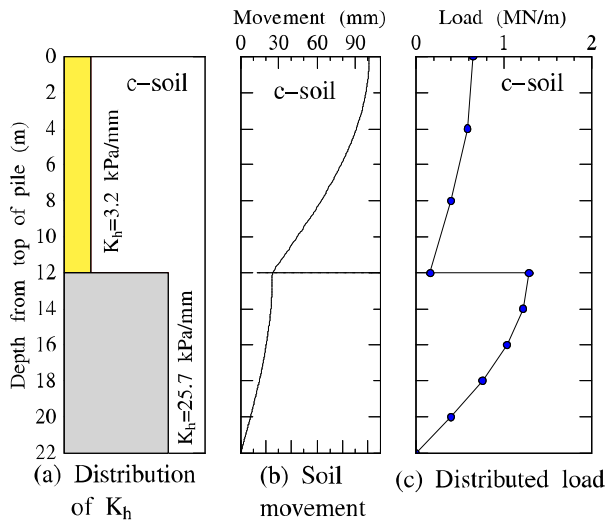


Figure 13. Subgrade reaction and soil displacement profiles and the distributed load for analysis.

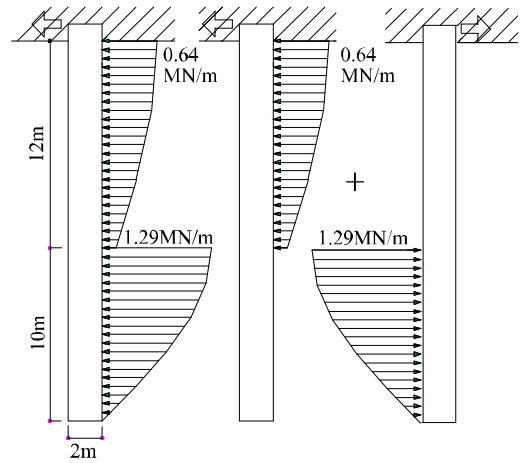


Figure 14. Two loading cases of the distributed load

The bending moment and the shear force diagrams under this condition 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 assumed to be in opposite directions (two sides).

COMPARISON WITH 1-D ANALYSIS RESULTS

The forces exerted by the distributed load on the pile shown in Figure 14 may be considered to indicate a reasonable trend in their distribution. It would be interesting how the trend compares with the results of more detailed response analysis. As the pile itself is considered to behave elastically in the analysis for beam on elastic foundation, it may be reasonable to compare with the results of 1-D analysis, which also assumes the pile to behave elastically. Such comparison is shown 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 analysis above, it may be logical to consider the maximum values at the middle part of the pile for the foundation sub-system. For example, the maximum bending moment at the middle part of the pile for the foundation sub-system occurs at time 3.24s as shown in Figure 8(b). The comparison in this manner is given in Figure 15.

Of course the distribution of the shear force and the bending moment obtained by the simple method proposed for consideration of the displacement response effect in Figure 15 does not compare point to point with the results of 1-D SSI analysis. However, the general trend and the extent can be seen to be reasonable, indicating strong potential for the development of the approach into a method of accounting for the displacement response effects in pile design practice.

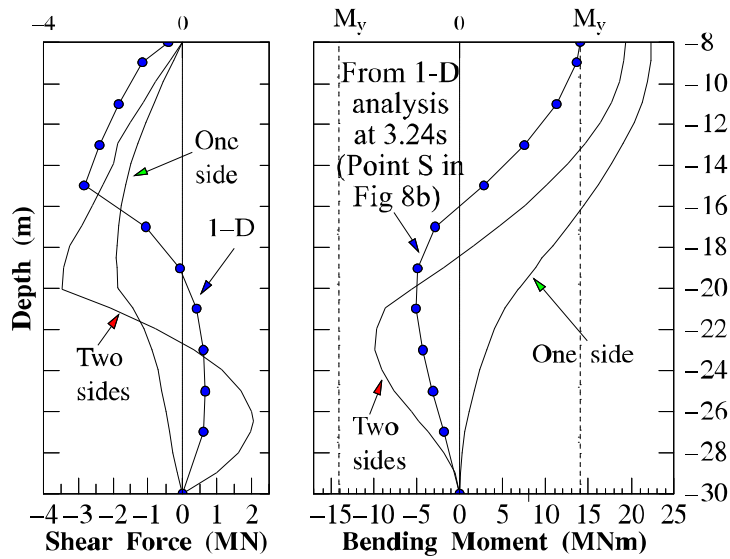


Figure 15. Comparison of shear force and bending from the proposed simple method with the maximum values obtained from 1-D SSI analysis results.

EXAMPLE OF PILE FAILURE DUE TO GROUND MOVEMENT

Extensive liquefaction and lateral flow of ground is known to have occurred during the Hyogoken-Nambu earthquake of January 17, 1995, resulting in the tilting of building structures. Most prominent were the inclination towards sea of the buildings at reclaimed ground in coastal area, where large lateral ground movement accompanied by the failure of quay walls occurred. Nakazawa et al. (1996) report on the nature of damage to prestressed concrete (PC) piles supporting an apartment building. Their observations and measurements are summarized in Figure 16.

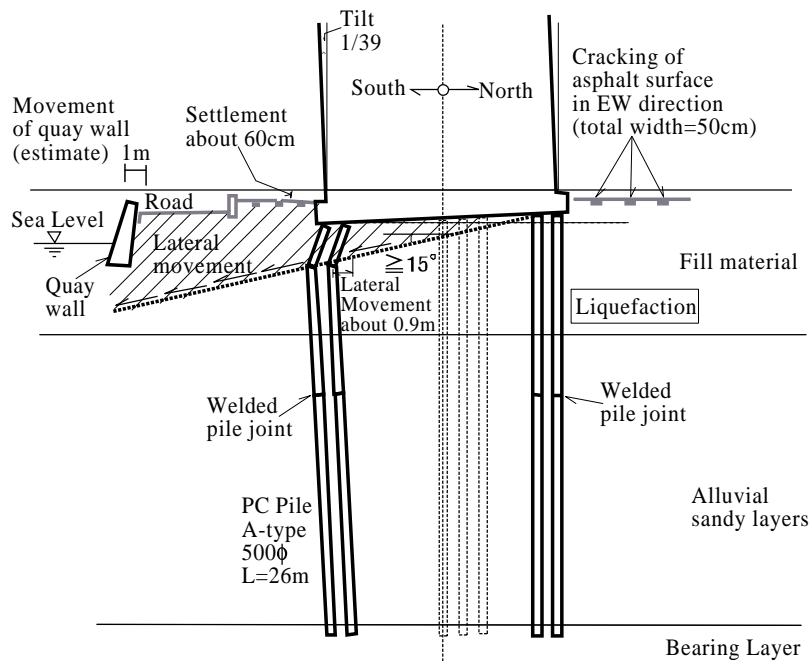


Figure 16. Failure of PC pile due to lateral ground movement (after Nakazawa et al. 1996)

As illustrated in Figure 16, investigation undertaken by Nakazawa et al. indicate that the PC piles under southern footing of the apartment building had moved laterally by about 90cm and the ground had settled by about 60cm. They have also reported the failure of ground along a slip surface inclined downwards towards sea at an angle of about 15 degrees, resulting in the lateral movement and settlement of the filled layer, which may also have undergone liquefaction. Consequently, the piles on the southern footing were broken at a depth from pile top of about 3.4, coinciding with the slip surface. This failure is regarded as the action of ground movement along slip surface. It may be noted in Figure 16 that the piles in the footings on the northern side are not included in the slip area. Accordingly, those piles are reported to have failed only at the top, which is considered by Nakazawa et al. as the effect of inertial forces.

As explained above, Figure 16 is the case of pile failure primarily due to the soil movement along the soil slip surface rather than due to displacement response effect discussed in the preceding sections. Attempt is made here to represent the loading on the pile due to the slip movement as a concentrated load at the slip surface. For this, the angle of internal friction of the fill consisting of the silt mixed coarse material is assumed to about 24 degrees and the soil unit weight is assumed to be 16kN/m^3 . The friction along the slip surface shared by the pile is then assumed to the concentrated load, which is estimated to be about 703kN. The concentrated load acts at a distance of 3.4m from the top of pile, which is regarded as the beam on elastic foundation. The subgrade reaction of the fill material and underlying soil layers is estimated based on the SPT N-values (blow counts) from the soil exploration before construction. The resulting bending moment and shear force diagrams are given in Figure 17.

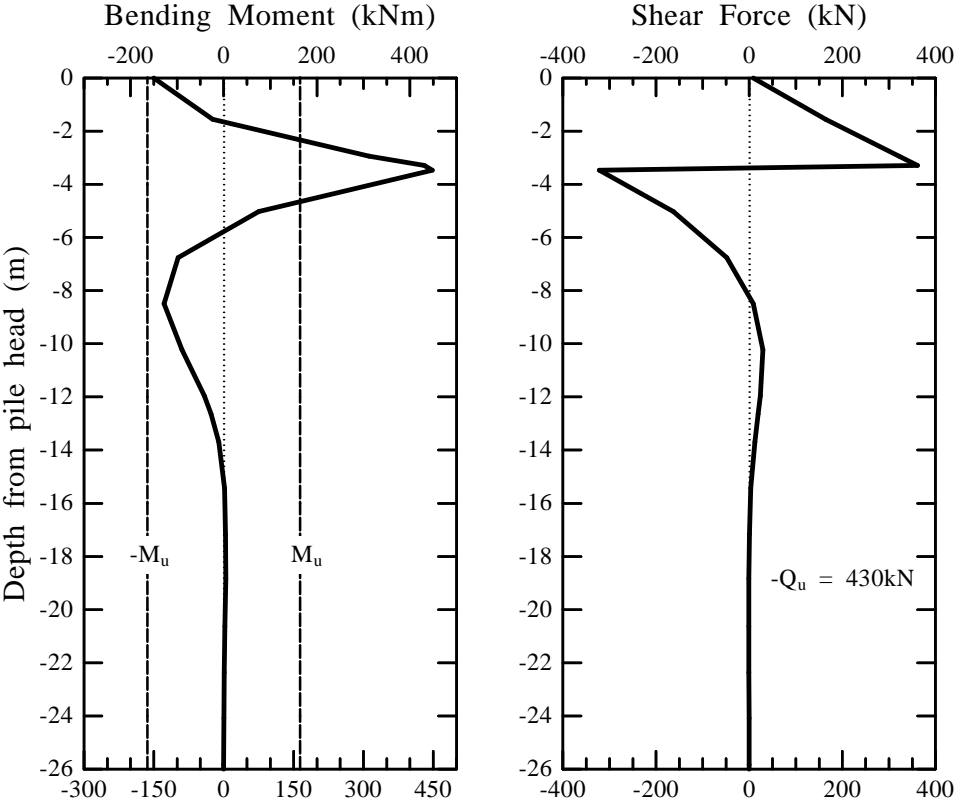


Figure 17. Estimated bending moment and shear force diagrams for PC pile on south side in Figure 16.

Since the analysis based on the beam on elastic foundation assumes elastic behavior, the bending moment in the PC pile at the slip surface is seen to be much larger than the ultimate bending resistance of the pile. Actually, the bending moment cannot exceed the ultimate value. In addition, the shear force is very large and comparable

to the estimated ultimate shearing resistance of the pile. The simple computation indicates that the failure of the PC piles on the south side in Figure 16 at the top as well as at a depth of about 3.4m have been caused by the action of lateral ground movement along the slip surface.

FURTHER RESEARCH ON DISPLACEMENT RESPONSE EFFECTS

The simple approach proposed to account for the displacement response effects in practice shows ample promise for practical application. However, further investigations based on a large number of simulations considering different soil conditions to compare the results of detailed response analysis with those from the proposed simple approach would be required for further refinement and development of the method for actual practical application. Experimental investigations and objective comparison with observations on the behavior of piles during earthquakes would provide valuable insight in the process.

It may also be noted that the current design guide in Japan seems to provide adequately for the inertial effects of the superstructure. However, the coefficient of subgrade reaction k_i utilized in the proposed method for displacement response effect is smaller and the displacement near pile top larger than the corresponding values applicable in the 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 displacement response effects or to have a combined method to account for both.

CONCLUSIONS

Reports on investigation of damage to foundations during past earthquakes indicate ample instances of damage to piles that can be directly or indirectly attributed primarily to displacement response effects, rather than the inertial effects of the superstructure usually considered in design. As an attempt to simulate the response forces that may result in such cases, a 35 storied reinforced concrete building was analyzed by considering two-dimensional (2-D) FEM representation of the foundation-soil system. Three simple variations in ground condition were considered for the same structure. Results indicate significantly large displacement response forces on piles, particularly at the soil layer interface of abrupt stiffness contrast.

While greatly reducing the computational rigor of the 2-D FEM analysis, 1-D lumped mass method of soil structure analysis provides results quite similar to 2-D analysis, and provides further credence to the significance of displacement response effects. The nonlinear soil behavior in the 1-D SSI was accounted for indirectly by selecting the shear modulus damping values for soil corresponding to 65% of the maximum shear strain during the nonlinear response 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 analysis is not accounted for. Otherwise, the results of 1-D and 2-D analysis are quite compatible.

The result of the analysis demonstrates the importance of considering the displacement response effects in the design of piles and illustrates the inadequacy of the present design practice of accounting only for the superstructure inertial forces. Of the three soil profiles considered in the FEM analysis, c-soil with two distinct soil layers gives rise to the largest response forces in piles for the same incident motion, indicating the influence of soil layering.

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. However, when the displacement response effects are included, bending moment and shear force can be substantially larger depending on the soil condition and the level of incident excitation. Proposed simple approach based on the concept of distributed load

and the beam on elastic foundation provides adequate representation of the bending moment and shear force distribution in piles. The proposed simple approach shows potential for design application as a concise and practical method of accounting for the effects of displacement response of the ground. Further research needs and directions are pointed out and emphasized.

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