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Performance of Large-diameter Piles Subjected to Lateral Spreading of Liquefied Deposits

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ABSTRACT: Design of pile foundations subjected to lateral spreading of liquefied soil deposits would require the lateral force acting on the pile body to be specified. For the back and forth movement of soil deposits as affected by liquefaction during earthquakes, it has been customary to degrade the modulus of subgrade reaction appearing in the Winkler type pile-soil interaction model. However, little has been known on the order of magnitude by which the stiffness of soils as represented by the coefficient of subgrade reaction should be reduced to allow for the effects of liquefaction-induced lateral spreading on the lateral force acting on the pile body. To explore these aspects, back-analyses were conducted for small-diameter piles such as precast, reinforced concrete piles which were damaged at the time of the 1995 Kobe earthquake. The outcome of these analyses was reported in a previous paper by the authors. Additional studies were performed afterwards to examine the degree of stiffness degradation of liquefied soils by focusing attention to the behaviour of large-diameter piles such as the cast-in-place reinforced concrete bored piles. The outcome of these studies revealed that the stiffness of liquefied soils should be reduced by a factor of $2x10^{-4}$ to $2x10^{-2}$. Conduct of back-analyses in this context is described in this paper, along with the main conclusions derived therefrom.

INTRODUCTION

Design of piles for the effects of seismic motions is generally performed by using a soil-pile interaction model in which a vertically-placed beam is supported by a series of spring elements. The beam represents the performance of the pile, and soil properties are represented by the spring constants. The effects of horizontal seismic motions on piles are allowed for by incorporating a horizontal force at the top of the pile, which is equivalent to the inertia force from the superstructure. For this type of analysis the spring constants are determined for conditions of no softening of the soils due to liquefaction. When liquefaction is of concern, the stiffness of the liquefied soils is dramatically reduced, and these effects need to be considered. The Japanese Code of Highway Bridge Design stipulate, for example, that for the majority of cases the spring constants be reduced by a factor of 1/6 to 2/3 depending upon the degree of safety against liquefaction. However, when it comes to the effects of lateral spreading of once liquefied soils, there has been no requirement stipulated in this code for the design of pile foundations. Thus, concerns have been kindled on these effects since the Kobe earthquake in 1995 because of the extensive occurrence of damage to foundation piles apparently due to the lateral spreading. When piles are subjected to the lateral flow of once liquefied soils, lateral forces would be applied directly to the pile body throughout the depth of liquefaction. In assessing this force in the design, there would be two approaches. The first method consists of assessing directly the lateral force on the pile body either based on empiricism or by means of the concept of viscous flow (Chaudhuri et al., 1995; Hamada and Wakamatsu, 1998). This may be called the "Force-based approach". In either way, it would be difficult to introduce a parameter which is indicative of the degree of destructiveness of the ground failure. Thus, the specification of the lateral force would have to be made irrespective of whether the ground displacement is destructively large or small. In the second method, the lateral displacement of the ground is specified through the depth of the deposit where the lateral spreading is induced. This prescribed displacement is applied to the spring system inducing lateral forces acting on the pile body. This procedure may be called the "Displacement-based approach". One of the advantages of this method is that it allows to specify the magnitude of ground displacement which is indicative of the degree of destructiveness or severity of the lateral spreading. In this method however, the choice of the spring constants has a profound influence on the magnitude of the lateral force induced, and as such difficulty is encountered in evaluating correctly this value for design purpose. It is expected that the spring constant in laterally spreading soils is much smaller than that in the case of the back and forth movement of soils as stipulated in the Japanese Code of Highway Bridge Design as mentioned above. While the code basically stipulates 1/6 to 2/3 reduction in the coefficient of subgrade reaction, the reduction is anticipated to be much more drastic if the effects of lateral spreading are allowed for. Thus, it becomes necessary to know the order of magnitude by which the conventionally used coefficient of subgrade reaction should be degraded to account for the interaction phenomenon taking place in the course of lateral spreading of liquefied soils.

Calculations in the above context were reported and discussed in previous papers (Ishihara, 1997; Ishihara and Cubrinovski, 1998). As a result, it was found that the stiffness of soils in laterally deforming deposits is reduced by a factor of 2×10^{-4} to 1×10^{-2}, and the degree of this reduction depends upon the relative displacement between the pile and the surrounding ground. However, in these studies, back-analyses were made only for the case of failure of relatively low-stiffness precast reinforced concrete piles having a diameter of 30 - 40 cm. These piles are used generally for foundation of medium-weight structures such as buildings and warehouses. In contrast to the above, the cast-in-place reinforced concrete bored piles are commonly used for supporting a large body of footings of piers for highway bridges. These piles have a diameter of 1.0 - 2.0 m, and are constructed by what is known as the benoto method. Thus, it is felt necessary to perform similar kind of back-analysis for such large-diameter piles to examine the stiffness degradation characteristics of the surrounding liquefied soils. The outcome of the studies in this context is described in the following pages of this paper.

CHARACTERISTICS OF PILED FOUNDATIONS

Majority of the piles damaged by the lateral flow of liquefied deposits at the time of the Kobe earthquake in 1995 may be divided into two groups, that is, the precast reinforced concrete piles and the cast-in-place reinforced concrete bored piles. The precast concrete pile is hollow-cylindrical and has a diameter of 30 - 40 cm. The length is generally in the range of 10-20 m. The piles are arranged generally in a group of 4-6 piles which are embedded at their tops into a common footing slab about 0.5 - 1.0 m thick. The top of the piles are connected to the footing slab in different ways, and therefore it is difficult to identify whether the top was rigidly connected or not, especially because these piled foundations were constructed more than 20 years ago, and construction details are not known. The footing slab having a thickness of 0.5 - 1.0 m is generally embedded into the surface soil layer above the ground water table which is therefore free from liquefaction. Inasmuch as the thickness or depth of embedment is relatively small, the lateral pressure acting on the sidewall of the slab in the non-liquefied surface layer is considered to be relatively small and therefore the presence of the footing slab may not be pronounced.

In contrast to the above, cast-in-place reinforced concrete bored piles are commonly used for supporting a large body of footings of piers for highway bridges. These piles have a diameter of 1.0 - 2.0 m, and are constructed by the benoto method. In this type of structures, the footing is constructed of massive reinforced concrete and has a thickness of 3 to 4 m. The whole body is embedded in the ground to a depth of 3 - 4 m where the ground water table is encountered. Thus, the effects of the lateral force in the non-liquefied surface layer acting on the sidewall of this embedded footing may not be ignored when making the back analyses for the behaviour of the underlying piles subjected to lateral flow.

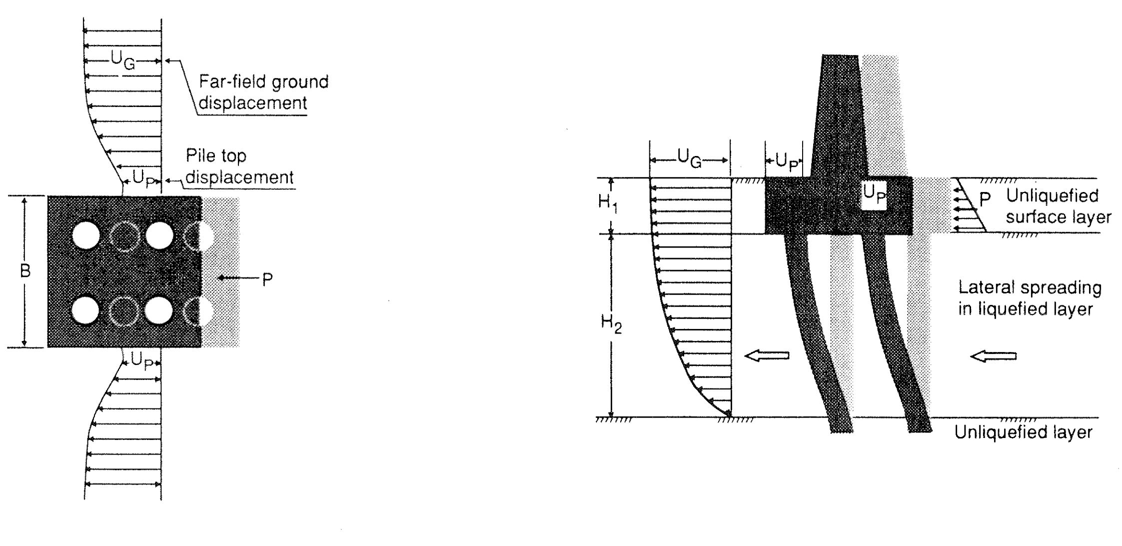
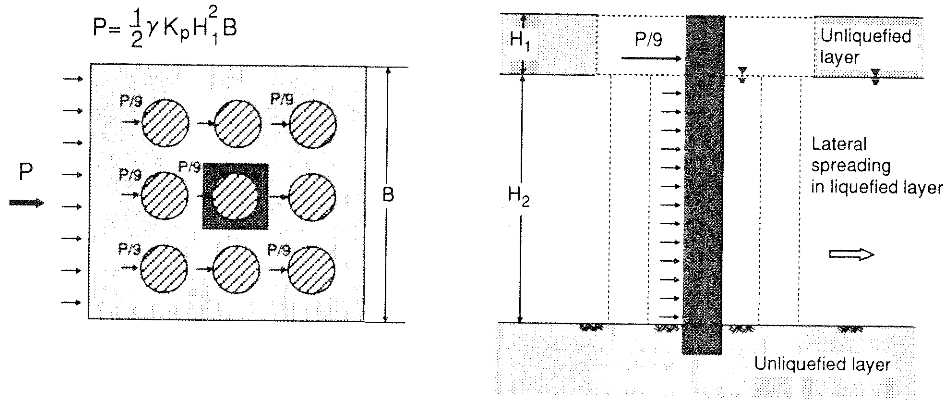
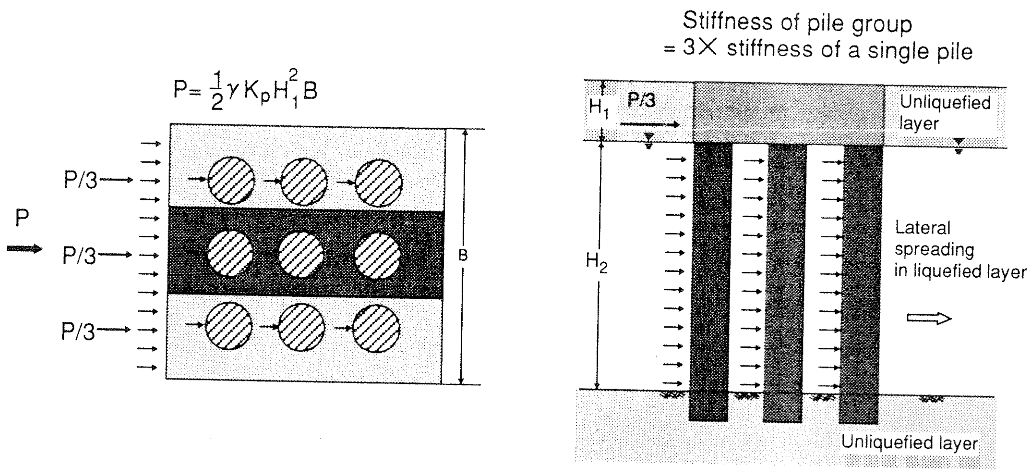


Fig. 1. Pattern of lateral displacements of the ground and pile during lateral spreading of liquefied soil



(a) Single pile hypothesis



(b) Pile row hypothesis

Fig. 2. Two hypotheses for partitioning a load from the footing to piles

BASIC ASSUMPTIONS AND RULES OF LOAD PARTITIONING

When making the back-analyses for the large-diameter group piles with a massive footing, it is necessary to assume, in one way or another, the magnitude and direction of the lateral force applied to the footing slab from the non-liquefied surface layer, and to set up hypotheses as to how the total load on the footing is apportioned among the individual piles to which the analysis is to be conducted.

If the ground surrounding the footing moves by an amount larger than the footing slab itself, it is apparent that the lateral force applied to the footing is oriented in the direction of the ground flow as illustrated in Fig. 1. It may be assumed that the lateral force would be equal to or smaller than the passive earth pressure. The total lateral force, P, due to this earth pressure which is deemed as the maximum possible value is given by

$$P = 1/2\gamma K_p H_1^2 B \tag{1}$$

where $K_p = \tan^2(45°+\phi/2)$, ϕ is the angle of internal friction, H_1 is the thickness of the unliquefied surface layer, and B is the width of the footing. In apportioning this total force to the individual piles, there would be two concepts, as follows, which are regarded as two extremes within which the actual conditions lie.

(1) Single pile hypothesis

Suppose there are nine piles arranged at an equal spacing as shown in Fig. 2. The simplest concept would be to assume that the total lateral force is carried equally by each pile. Thus, the back-analysis may be made for a pile with a given flexural stiffness as illustrated in the side view of Fig. 2 (a).

(2) Pile row hypothesis

The three piles immediately adjacent to the upstream wall are considered to carry larger portion of the lateral force as compared to the other two piles located downstream, as illustrated in Fig. 2 (b). However, it would be acceptable to postulate that one-third of the total load is transmitted equally to each row of the pile alignment in the direction of the ground deformation, as illustrated in Fig. 2 (b). In this modeling, the flexural stiffness of the pile row as a whole is assumed to be three times the flexural stiffness of a single pile, and the modeled three-pile unit is assumed to be subjected to the lateral force of P/3 near the head of the pile.

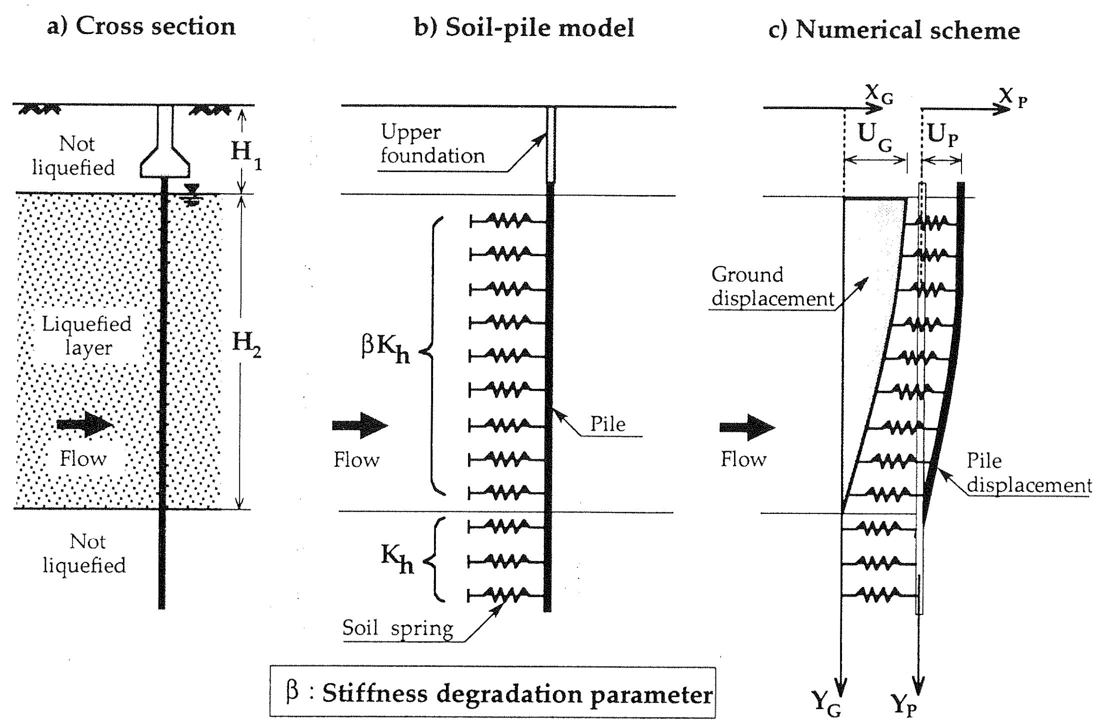


Fig. 3 Soil-pile model and numerical scheme

The above is an illustration for the case of 9 piles equally spaced. For the footing having more piles with complicated arrangements in plan, other hypotheses with similar context will be made in the following back-analyses. It is to be noted that, no matter which is the rule of load partitioning, the piles in the liquefied deposits are assumed to behave independently as a single pile. Thus, group effects of interaction amongst the piles are not taken into consideration in the present analysis.

SCHEME OF BACK-ANALYSES

The behaviour of piles is assumed to be represented by the model in which the lateral force, F, acting on the piles is proportional to the relative displacement between the pile and the soil in far-field condition. This may be written as

$$F = \beta k d \ (U_G - U_P) \qquad (2)$$

where U_G and U_P are lateral displacements of the ground and pile respectively, d is the effective area, and k is the coefficient of subgrade reaction. The model is schematically illustrated in Fig. 3. If the soil is brought to a state of liquefaction and consequent lateral flow, the stiffness of the soil would be reduced drastically leading to a reduction in the k-value. The degree of this stiffness reduction is expressed by β in Eq. (2), which will be referred to as the "Stiffness degradation parameter". The main aim of the present back-analyses is to pursue the range of this degradation parameter. The steps of the analysis to be followed are described below.

(1) The stiffness of the spring is assumed to decrease by a factor β upon liquefaction and lateral spreading of the soil through the depth of liquefaction, H_2. The spring constant in the underlying non-liquefied zone is assumed not to be degraded. It is further assumed that the stiffness degradation occurs uniformly throughout the depth H_2 where lateral spreading is taking place.

(2) Generally, there is a non-liquefied layer to a certain depth H_1 near the surface. This depth may be roughly defined to be equal to the depth to the ground water table. The movement of this non-liquefied soil mass may be modeled in different ways. One is to assume the surface layer to move in unison with the underlying liquefied stratum. In this case, it may be assumed that the passive earth pressure is applied to the wall of the footing on the upstream side in the same direction as the flow of the underlying liquefied soil layer, as illustrated in Fig. 1. In this type of surface soil movement, the displacement of the pile head is considered always smaller than the overall movement of the surrounding soil.

(3) The ground displacement due to lateral spreading is specified and given to the springs in the liquefied portion of the soil deposit; displacements, bending moments and lateral forces acting on the pile body are calculated, whereby the pile is assumed to deform in an elasto-plastic manner where the moment-curvature relation is represented by a trilinear relationship.

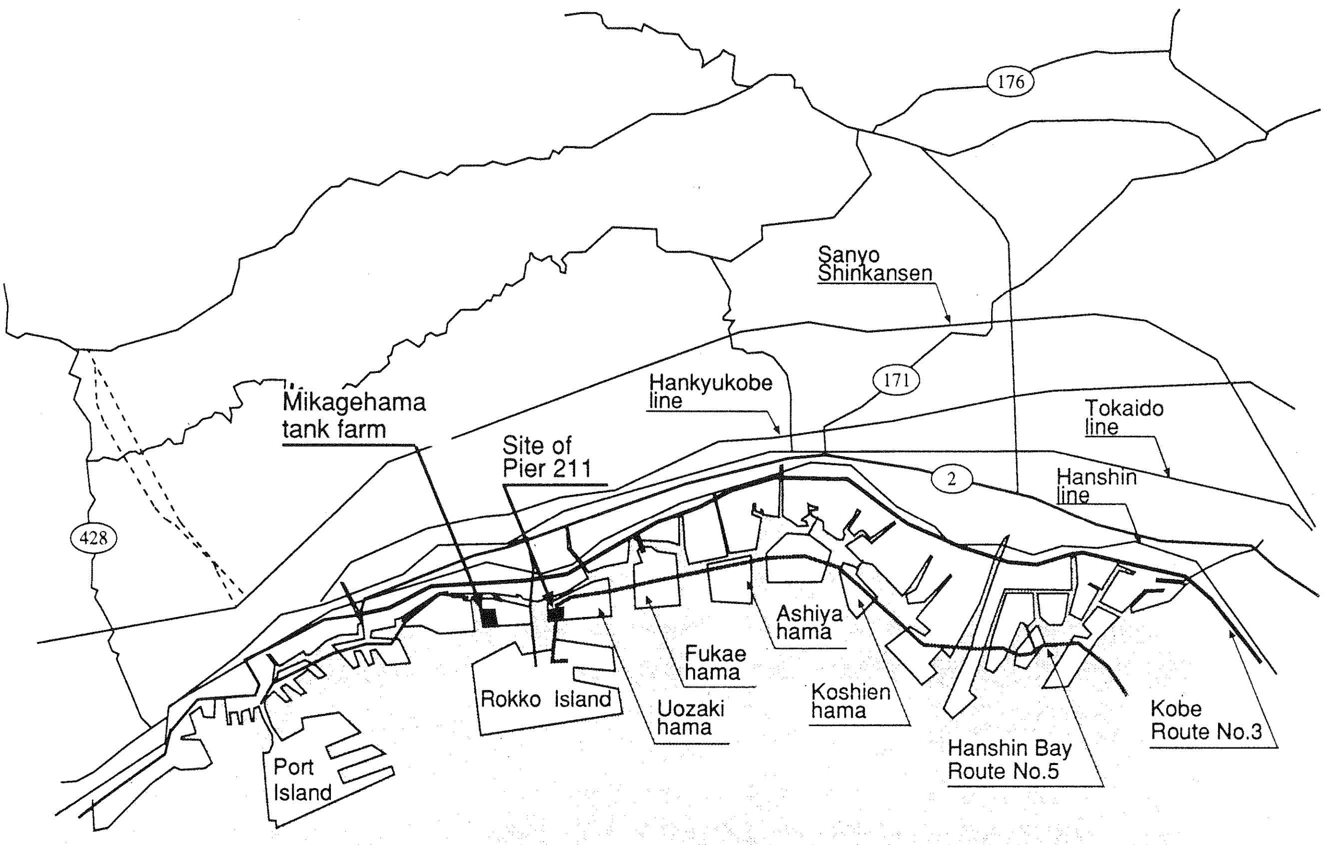


Fig. 4. Locations of the bridge pier and tank farm in the coastal zone of Kobe

ANALYSES FOR HIGHWAY BRIDGE PIER

The piled foundation considered in this back-analysis consists of bored piles 1.5 m in diameter and 41.5 m in length which belong to Pier 211 supporting the elevated structure of the Hanshin Expressway Bay Route No. 5 in Kobe. The location of the pier is shown in Fig. 4 and features of the ground displacements as inferred by the method of air-photo interpolation are presented in Fig. 5, where it may be seen that a seaward lateral displacement of about 1.0 m is estimated at the site of Pier 211. The arrangements of the piles supporting Pier 211 are presented in the plan view in Fig. 6 where it is seen that the footing 22.5 m by 14.5 m is located about 30 m behind the revetment in the south. Side views of Pier 211 are shown in Fig. 7. It can be seen that the footing embedded to a depth of 4.18 m is supported by 22 cast-in-place reinforced concrete piles constructed by the bored-hole method. The cross sectional view of the piled footing and soil profile in its vicinity is shown in Fig. 8, where it may be seen that the reclaimed deposit composed of gravel, sand and silt exists to a depth of about 20 m where liquefaction and consequent lateral spreading are assumed to have occurred.

As one of the investigations for examining the damage of the piles caused by the Kobe earthquake, two holes 7 cm in diameter were drilled from the top surface through the footing slab down into the piles P4 and P10 shown in Fig. 7. A video camera was lowered into the holes to inspect features of the crack development and the outcome is shown in Fig 9. It may be seen that the cracks are concentrated near the pile head and also in the vicinity of the interface between the liquefied deposit and underlying non-liquefied layer. In the analysis, the flexural stiffness characteristics of a single pile were assumed to be those shown in Fig. 10 which were obtained from the

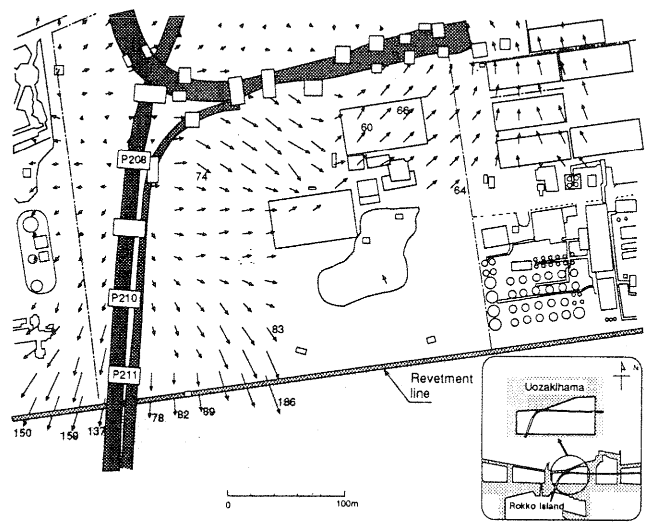


Fig. 5. Ground displacements in the vicinity of Pier 211 (Hanshin Highway Authority, 1996)

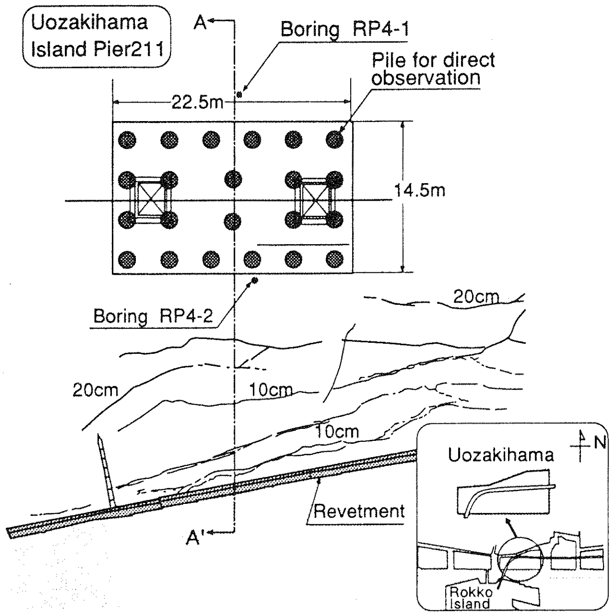


Fig. 6. Ground distortion and location of Pier 211 (Hanshin Highway Authority, 1996)

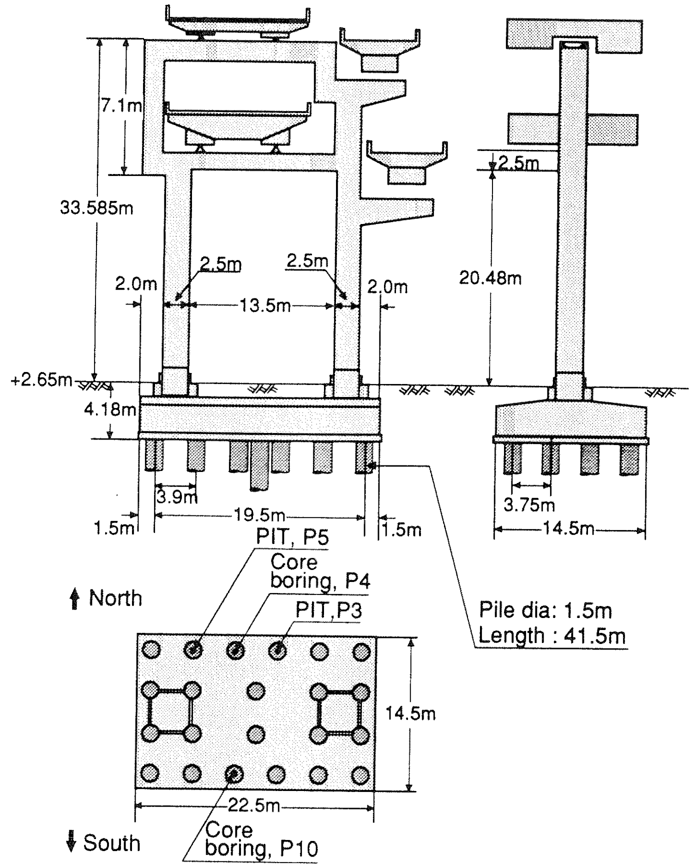


Fig. 7. Plan and side views of Pier 211 (Hanshin Highway Authority, 1996)

cross sectional characteristics of the bored pile. Characteristics of the soil model used for the piled foundation of Pier 211 are illustrated in Fig. 11, where the depthwise distribution of the K_h-value ($K_h = \mathrm{k}D$) as estimated by an empirical formulae in the Japanese Code of Highway Bridge Design is given together with the depth of liquefaction as evaluated from an independent analysis. Liquefaction is assumed to have developed to a depth of 20 m.

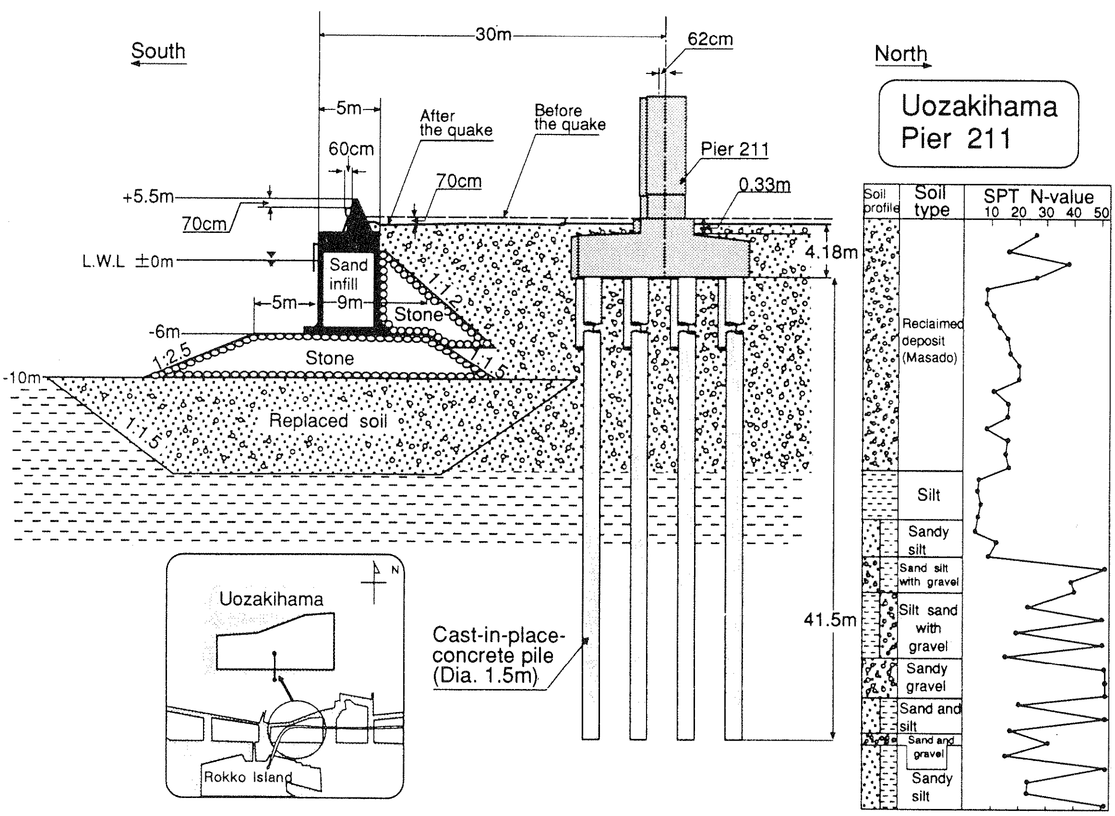


Fig. 8. Quay wall, foundation and soil conditions at Pier 211 (Hanshin Highway Authority, 1996)

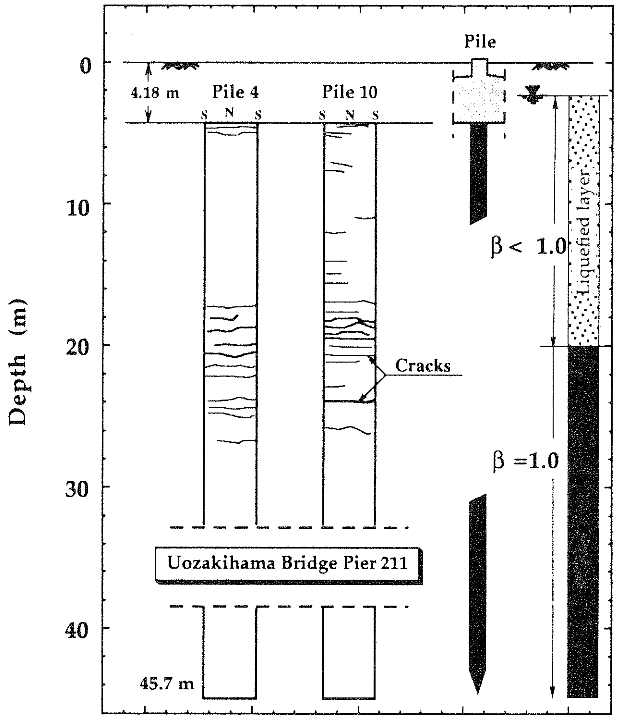


Fig. 9. Crack distribution in the bored piles at Bridge Pier 211 in Uozakihama Island

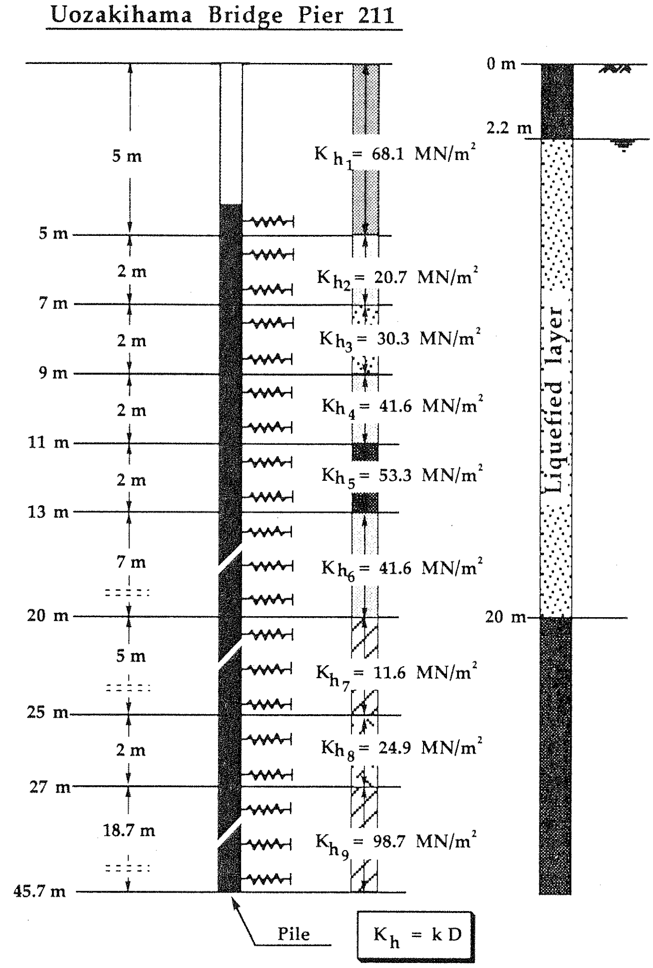


Fig. 11. Soil stiffness properties for the cast-in-place RC pile at the Bridge Pier 211 in Uozakihama Island

In the first analysis, the hypothesis of a single pile as illustrated in Fig. 2 (a) is adopted, where the lateral force from the unliquefied surface layer is applied to the footing in the direction of flow deformation. The coefficient of passive earth pressure, K_p, was obtained by assuming ϕ=35°. Based on the data shown in Fig. 5, the lateral displacement at the Pier 211 was assumed to be 1.0 m on the ground surface in the free-field condition and distributed with a cosine function through the liquefied layer down to a depth of 20 m as displayed in Fig. 12. In the actual computation, the presence of the footing was represented by an equivalent beam and a single pile

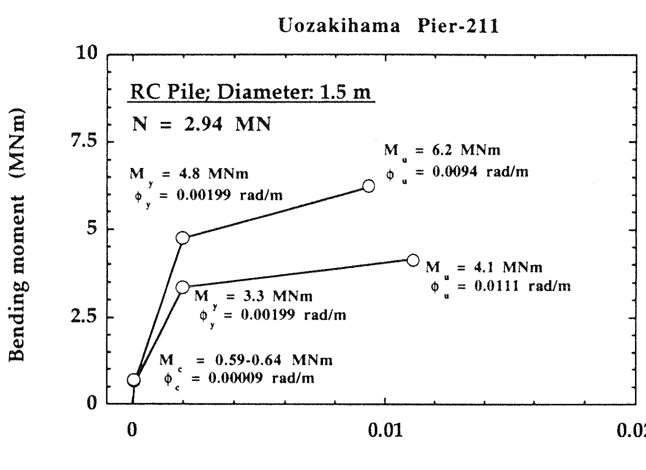


Fig. 10. Bending moment versus curvature relation for the large diameter piles at Bridge Pier 211

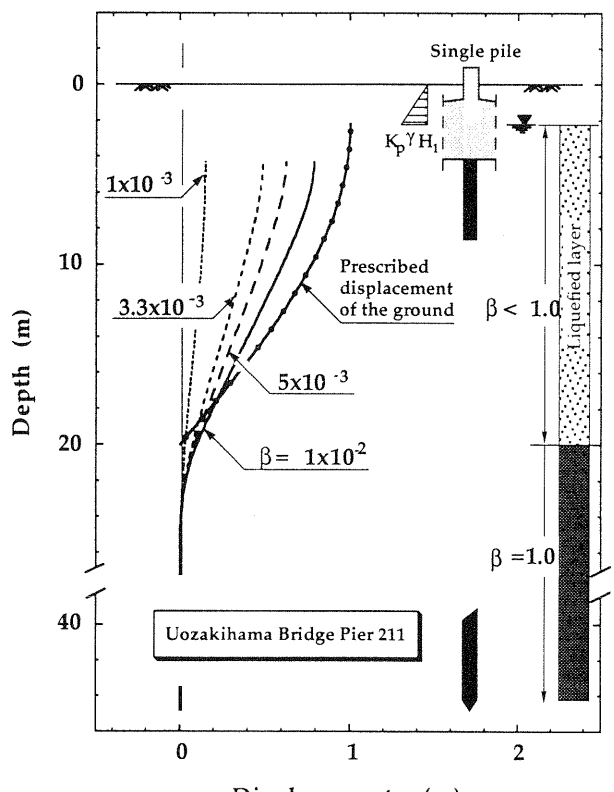


Fig. 12. Lateral displacement of the pile at Pier 211: Analysis with the single-pile hypothesis

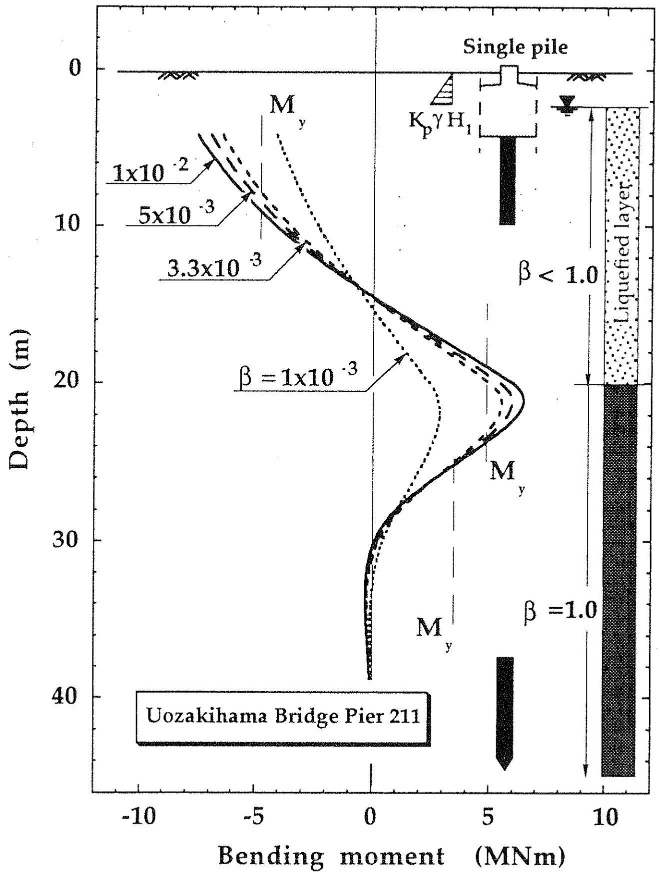


Fig. 13. Bending moment of the pile at Pier 211: Analysis with the single-pile hypothesis

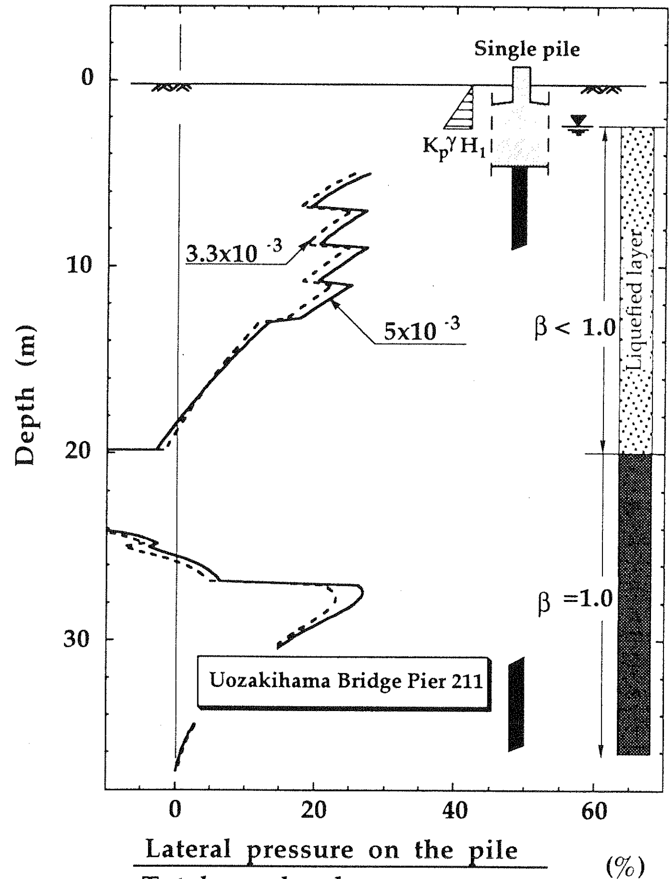


Fig. 14. Lateral pressure on the pile at Pier 211: Analysis with the single-pile hypothesis

was postulated to extend to the ground surface with the lateral force applied near its head as indicated in Fig. 12. Computation was made for values of the stiffness degradation parameter in the range between $\beta=1\times10^{-3}$ - 1×10^{-2}. The deformation of the pile obtained from the analyses with different β–values is also shown in Fig. 12. Pier 211 is reported to have displaced about 60 cm as a result of the ground movement towards the sea. With this in mind, the degradation parameter of $\beta=5\times10^{-3}$ seems to yield the best fit to the observed performance of Pier 211. The bending moment of the pile resulting from the computation is shown to be distributed as displayed in Fig. 13. For the case of $\beta=5\times10^{-3}$, the computed bending moment is in excess of the yield moment, M_y, near the pile head and also around the depth of 20 m where discontinuity is encountered from the liquefied to the unliquefied layers. This observation is consistent with the location of the cracks as demonstrated in Fig. 9. For reference sake, the lateral pressure acting on the pile body is shown in Fig. 14 versus the depth. It may be seen that the lateral pressure on the pile is less than 30 % of the total overburden pressure throughout the depth of the liquefied layer.

In the second analysis, the pile row hypothesis as explained in Fig. 2 (b) was incorporated with respect to the lateral force applied near the top of the pile. Results of the analysis on the lateral displacement of

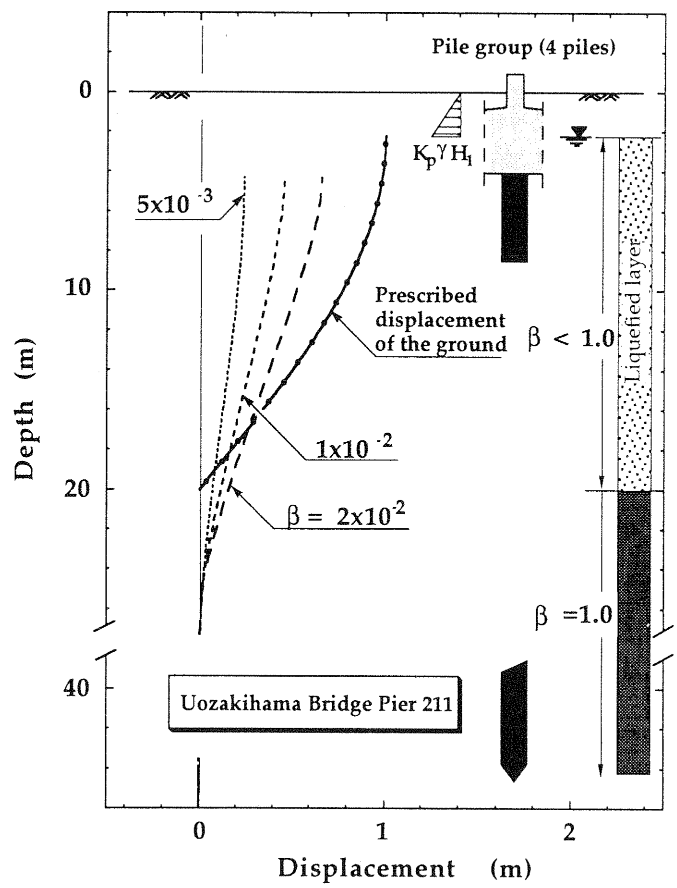


Fig. 15. Lateral displacement of the pile at Pier 211: Analysis with the pile-row hypothesis

the pile are displayed in Fig. 15 in the same fashion as in the case of the previously described analysis. In this case, the stiffness degradation parameter of β=2x10^{-2} seems to give the best agreement with the observed displacement of the pile being 60 cm on the ground surface. The calculated bending moment distribution as displayed in Fig. 16 is also in consistence with the observed locations of the crack development which were near the head and also near the interface between the liquefied and unliquefied layers. The calculated lateral pressure on the pile shown in Fig. 17 indicates that at any depth in the liquefied zone, the lateral pressure is below 30 % of the total overburden pressure.

With the considerations as above, it may be mentioned conclusively that, for the bored piles of the Bridge Pier 211 in the Uozakihama Island, the stiffness degradation of the liquefied sandy soil due to the lateral spreading might have been probably on the order of β=5x10^{-3} - 2x10^{-2} as compared to the stiffness value in normal conditions without liquefaction.

ANALYSES FOR FOUNDATION OF STORAGE TANK

A cylindrical tank with a capacity of 20,000 kl for storage of LPG (Liquid Propane Gas) located in Mikagehama was severely shaken at the time of the Kobe earthquake. Although gas leakage occurred from the breakage at the inlet-outlet valve at Tank 101, no structural damage was incurred. Details of the accident and soil conditions in this island are described in a paper by Ishihara (1997). The location of the island is shown in Fig. 4, and more detailed arrangements of facilities near the Tank 101 under consideration are presented in Fig. 18. The plan and side views of the Tank 101 are shown in Fig. 19, together with a soil profile at this site. This tank 39 m in diameter is supported by a total of 97 cast-in-place bored piles 1.1 m in diameter and 27 m in length. Each of the piles spaced at 3.7 m is connected by underground horizontal beams arranged in lattice as displayed in Fig. 19. The horizontal beam 2.0 m in height was embedded into the ground. Then, it was assumed in the back-analysis that this 2 m thick horizontal beam lattice system constitutes a large footing which is considered rigid enough to be regarded as a massive footing. On this basis, the two assumptions described in Fig. 2 were incorporated for estimating the lateral force acting on the piles from the unliquefied surface layer.

At the center of the Tank 101, located at a distance of 50 m from the quay wall, the lateral displacement which would have occurred had the tank not been there is estimated to be 1.8 m, based on the field measurements nearby (Ishihara and Cubrinovski, 1998). Exact amount of the lateral movement of the

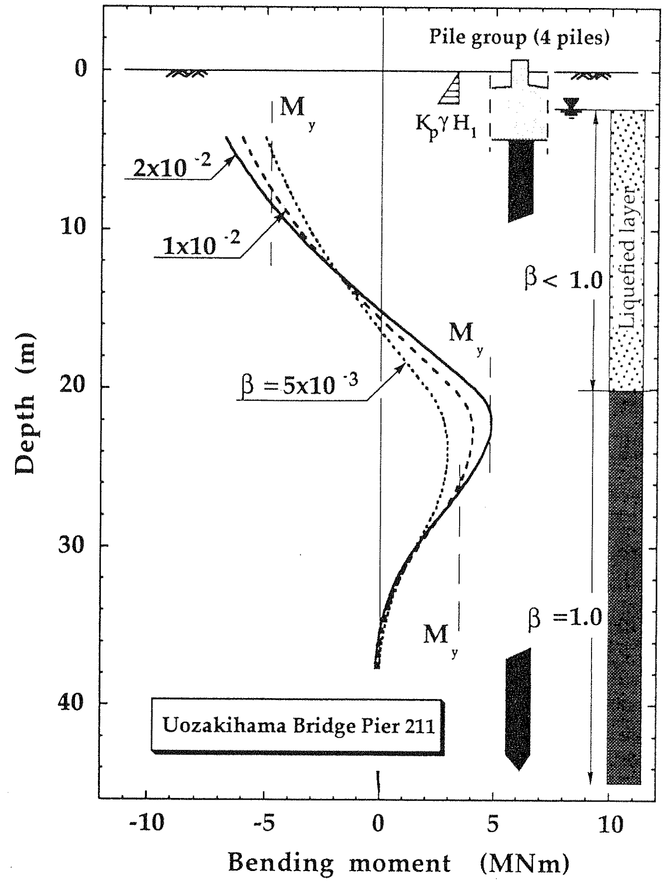


Fig. 16. Bending moment of the pile at Pier 211: Analysis with the pile-row hypothesis

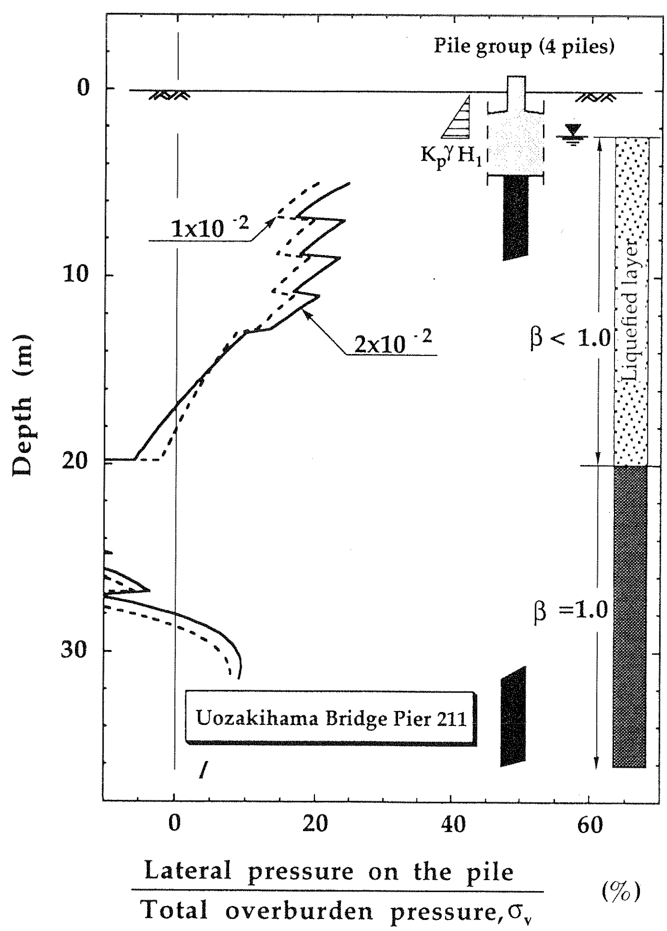


Fig. 17. Lateral pressure of the pile at Pier 211: Analysis with the pile-row hypothesis

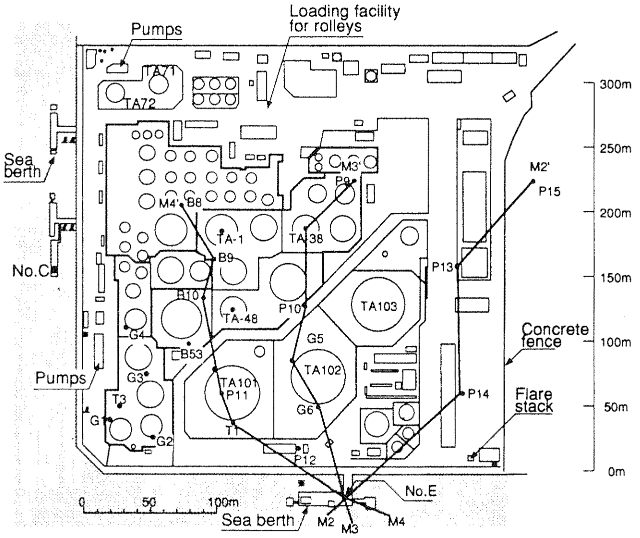


Fig. 18. Location of the Tank 101 in Mikagehama Island

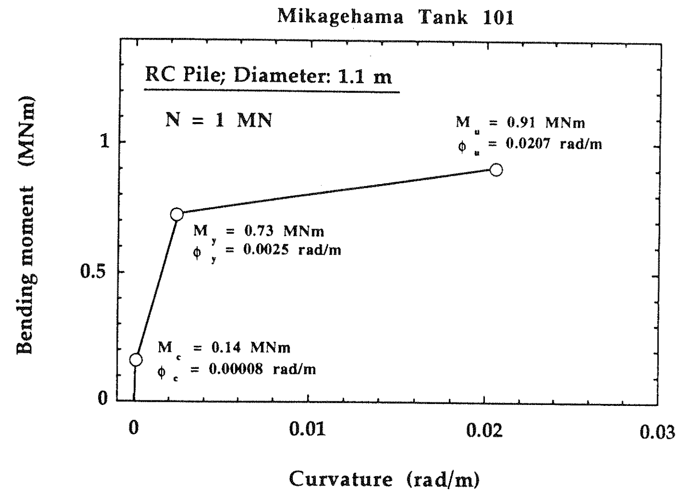


Fig. 20. Bending moment versus curvature relation for the bored piles at Tank 101

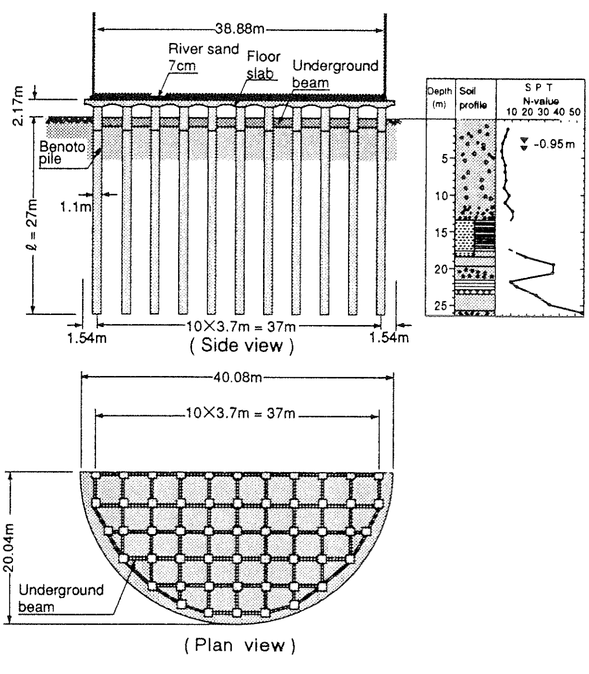


Fig. 19. Side and plan views of the pile foundations of the cylindrical Tank 101

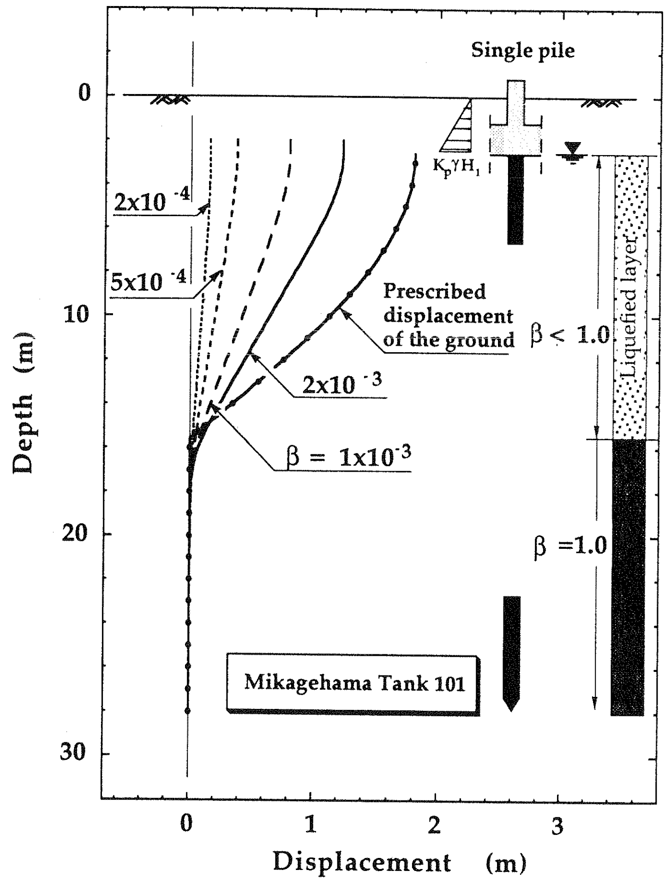


Fig. 21. Lateral displacement of the bored pile at Tank 101: Analysis with the single-pile hypothesis

Tank 101 itself is not known, but in view of that there having been practically no damage to the tank body, the lateral displacement is inferred most likely to have been on the order of about 20 cm. The flexural stiffness characteristics of the bored pile are shown in Fig. 20 in terms of trilinear relation between the bending moment and curvature of the pile.

The scheme of the back analysis, based on the single pile hypothesis, is schematically shown in Fig. 21. The ground displacement was given in a form of the cosine function with its maximum of 1.8 m on the ground surface. The resulting displacements of the pile are also shown in Fig. 21 for varying values of the stiffness degradation parameter postulated in the analysis. It may be seen that the best fit for the pile deformation of 20 cm is achieved with the parameter $\beta=2\times10^{-4}$. The bending moment induced in the pile is computed as displayed in Fig. 22 where it may be seen that the parameter $\beta=2\times10^{-4}$ gives a bending moment at the pile head which is about the yield value M_y, whereas the bending moment is less than the yield value at the interface. In the case of Tank 101, no

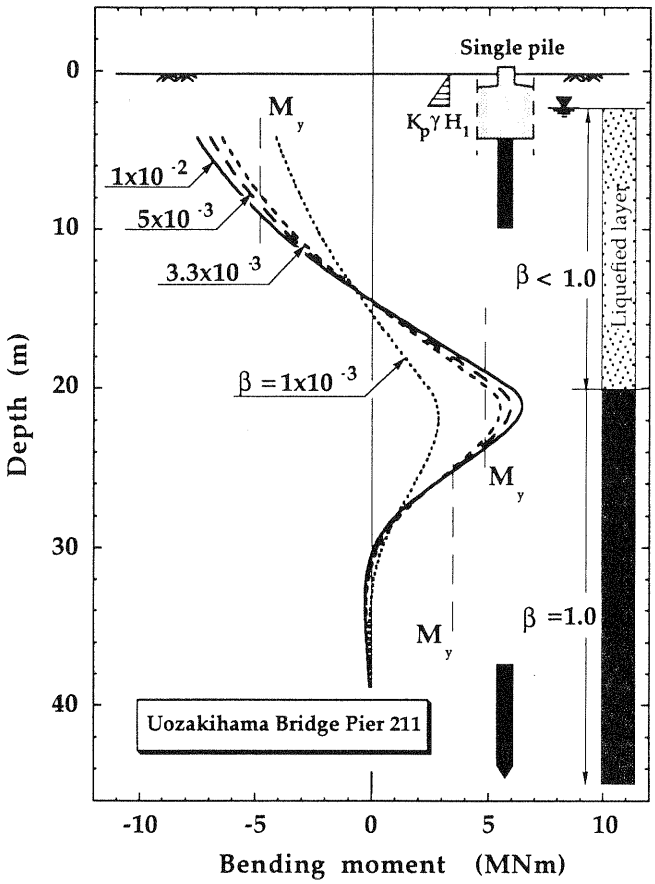


Fig. 13. Bending moment of the pile at Pier 211: Analysis with the single-pile hypothesis

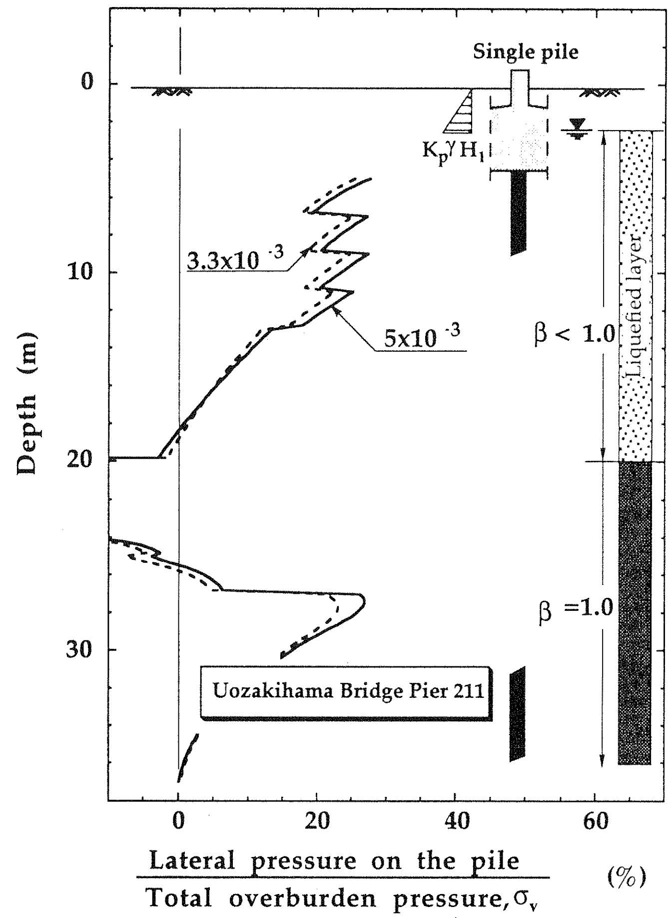


Fig. 14. Lateral pressure on the pile at Pier 211: Analysis with the single-pile hypothesis

was postulated to extend to the ground surface with the lateral force applied near its head as indicated in Fig. 12. Computation was made for values of the stiffness degradation parameter in the range between $\beta=1\times10^{-3}$ - 1×10^{-2}. The deformation of the pile obtained from the analyses with different β–values is also shown in Fig. 12. Pier 211 is reported to have displaced about 60 cm as a result of the ground movement towards the sea. With this in mind, the degradation parameter of $\beta=5\times10^{-3}$ seems to yield the best fit to the observed performance of Pier 211. The bending moment of the pile resulting from the computation is shown to be distributed as displayed in Fig. 13. For the case of $\beta=5\times10^{-3}$, the computed bending moment is in excess of the yield moment, M_y, near the pile head and also around the depth of 20 m where discontinuity is encountered from the liquefied to the unliquefied layers. This observation is consistent with the location of the cracks as demonstrated in Fig. 9. For reference sake, the lateral pressure acting on the pile body is shown in Fig. 14 versus the depth. It may be seen that the lateral pressure on the pile is less than 30 % of the total overburden pressure throughout the depth of the liquefied layer.

In the second analysis, the pile row hypothesis as explained in Fig. 2 (b) was incorporated with respect to the lateral force applied near the top of the pile. Results of the analysis on the lateral displacement of

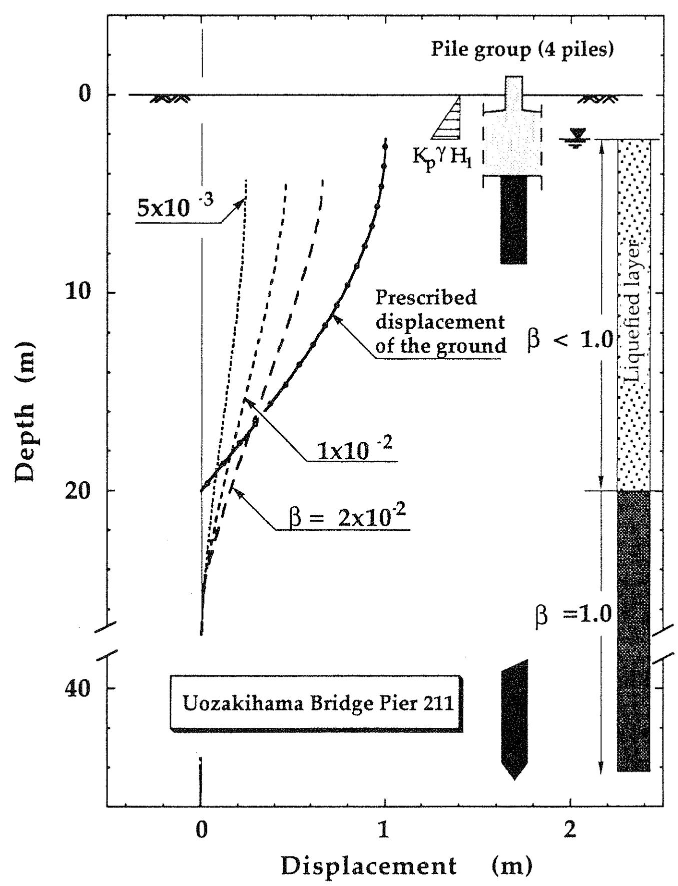


Fig. 15. Lateral displacement of the pile at Pier 211: Analysis with the pile-row hypothesis

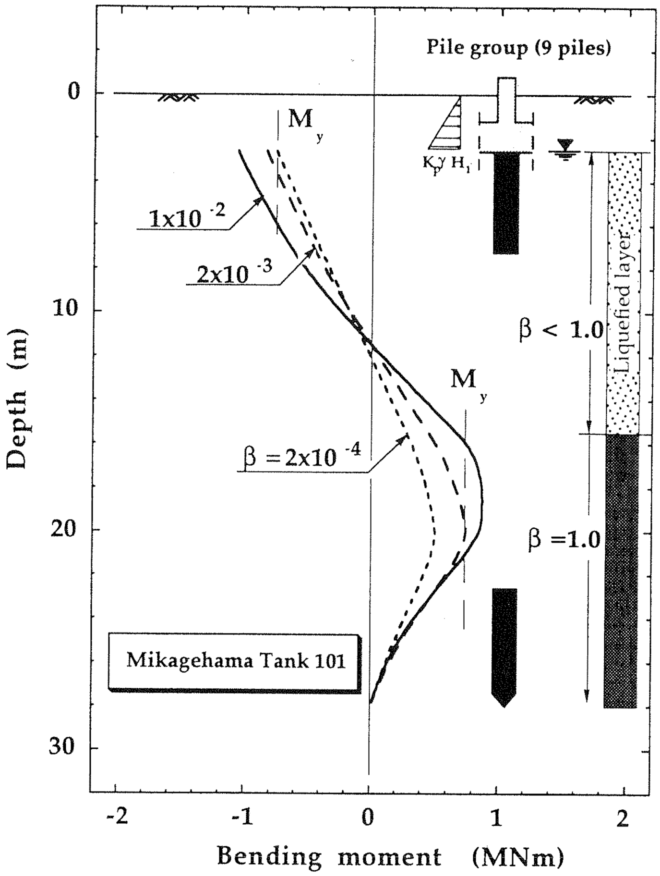


Fig. 25. Bending moment of the bored pile at Tank 101: Analysis with the pile-row hypothesis

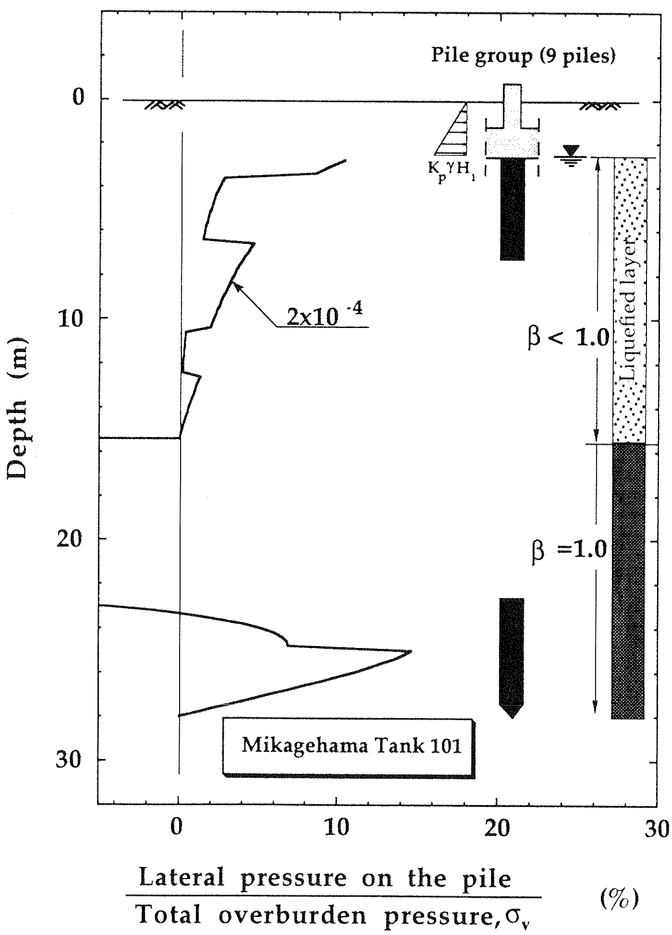


Fig. 26. Lateral pressure of the bored pile at Tank 101: Analysis with the pile-row hypothesis

on the pile row hypothesis explained in Fig. 2 (b).

With all other conditions taken identical to the case of the single pile hypothesis, the analysis was conducted and the results on displacements are shown in Fig. 24 for varying β-values. It may be seen that the value of $\beta=2x10^{-4}$ gives the best fit to the pile displacement of about 60 cm at the top. The depthwise distribution of the calculated bending moment are shown in Fig. 25. The results indicate the maximum value at the pile head is equal to the yield moment whereas at the interface zone the moment is well below its yield value. This observation also provides an evidence of $\beta=2x10^{-3}$ to be a reasonable estimate for the stiffness reduction parameter. The lateral pressure on the pile body as displayed in Fig. 26 shows that it is less than about 10 % for the liquefied portion of the deposit.

No matter which hypothesis is adopted for the load partitioning, it may be mentioned conclusively that for the bored piles supporting the Tank 101 in Mikagehama tank farm which underwent lateral spreading at the time of the Kobe earthquake, the stiffness of the liquefied soil seems to have been degraded by a factor $\beta=(2–5)x10^{-4}$ as against to the stiffness in normal conditions without liquefaction.

DISCUSSIONS

The results of the analyses for the two cases of the large-diameter bored piles are summarized in Fig. 27 where the stiffness degradation parameter β is plotted versus the relative displacement between the ground surface and pile top which is divided by the displacement of the ground surface. It can be seen that the stiffness degradation parameter tends to increase as the displacement of the pile relative to the ground becomes smaller. If the pile movement is negligible while the ground is moving largely, the β-value is shown to take a value as small as $2x10^{-4}$ indicating little interaction, if any, between the pile and surrounding soil. If the relative displacement is about 30 % of that on the ground surface, the stiffness degradation is shown to take a value of the order of $2x10^{-2}$. Also superimposed in Fig. 27 are the results of the back-analyses performed previously (Ishihara and Cubrinovski, 1998) for the small-diameter precast reinforced concrete piles. It may be seen that, for a given relative displacement, the stiffness of the liquefied soil should be reduced much more drastically for the small-diameter pile as compared to the large-diameter pile. This consequence is interpreted intuitively as reasonable, because the rigid pile would require a greater magnitude of lateral force to be applied in order to achieve a given level of relative displacement, which is tantamount to the requirement of greater stiffness and hence greater β -value of the surrounding soils. It is to be noted that

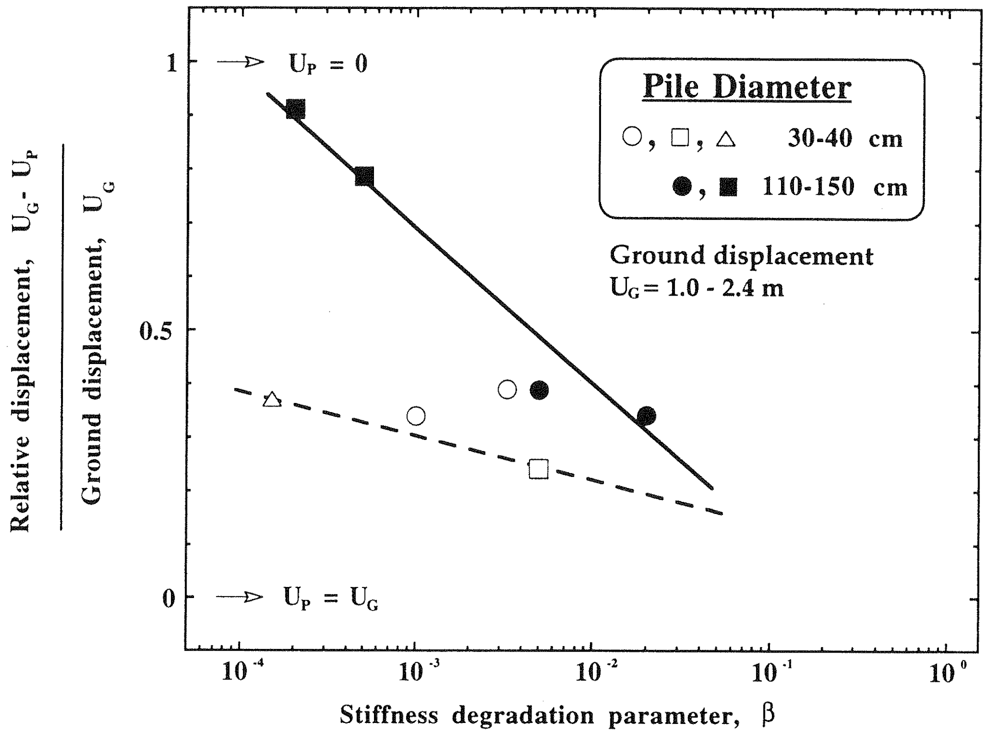


Fig. 27. Relations between the relative displacement normalized by the ground surface displacement and stiffness degradation parameter β

the relative displacement normalized in this way does not indicate explicitly the effects of the magnitude of ground displacement.

Another aspect of the stiffness degradation characteristics would be explored by plotting the results of the back-analyses in terms of the β -value versus the relative displacement divided by the thickness of the liquefied layer H_2. The computed data arranged in this way are demonstrated in Fig. 28 for the two kinds of piles as mentioned above. The relative displacement normalized in this way is indicative of the absolute magnitude of average shear strain of the soil deposit and the flexural deflection of the pile body. It may be seen in Fig. 28 that there is some difference between

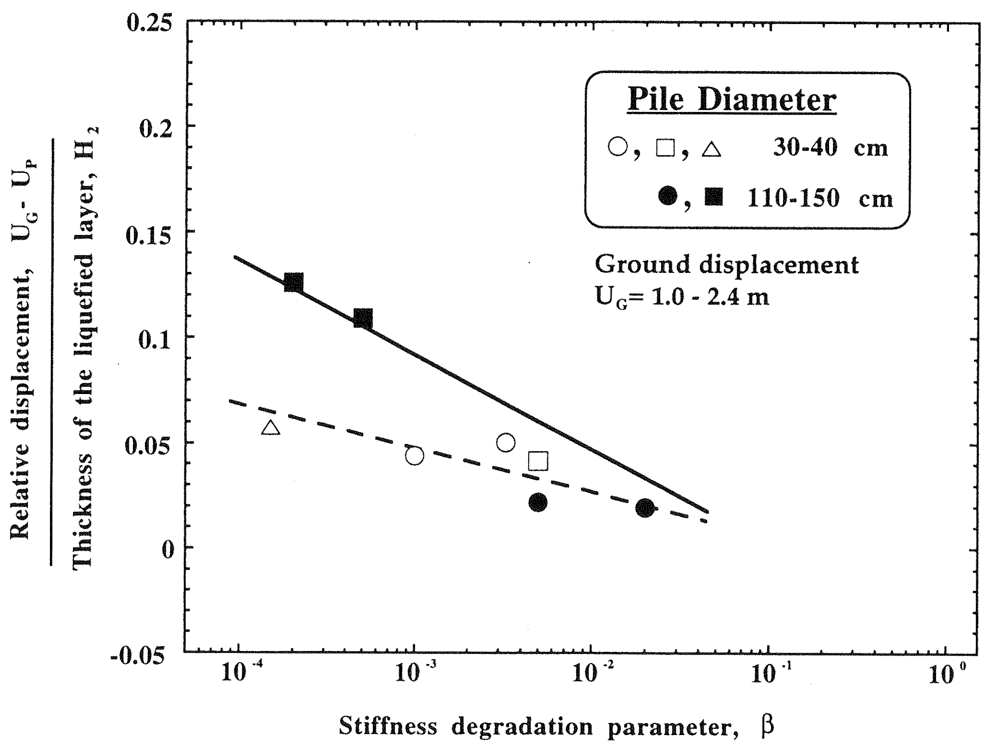


Fig. 28. Relations between the relative displacement normalized by the thickness of the liquefied layer and stiffness degradation parameter β

the small-diameter and large-diameter piles with respect to the relation between the β-value and the global relative strain. Thus, it may be mentioned that the degree of stiffness degradation required to produce a certain level of global relative strain is smaller for the rigid pile with large diameters as compared to that required for the more flexible pile with small diameters.

CONCLUSIONS

Back-analyses were performed for two cases of large-diameter reinforced concrete bored piles which were more or less damaged at the time of the 1995 Kobe earthquake in Japan. The results of the analyses are expressed in terms of the degree of stiffness degradation indicative of pile behaviour interacting with surrounding soil deposits undergoing lateral spreading. As a result of the back-analyses, it was pointed out that under otherwise identical conditions, the stiffness degradation should be less pronounced in the case of the large diameter pile as compared to the small diameter pile in order to achieve the same level of relative displacement between the pile and the surrounding soil. In any event, the order of magnitude for the degradation of stiffness of the surrounding soils was shown to be as small as $2x10^{-4}$ - $2x10^{-2}$.

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