Soil-Pile Interaction In Liquefied Deposits Undergoing Lateral Spreading

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ABSTRACT: For the design of foundation piles subjected to lateral spreading of the ground, it is necessary to specify the lateral force acting on the pile body from the surrounding soil deposit which is moving as a result of liquefaction. In view of many design codes incorporating spring-supported beam system to model the soil-pile interaction, it may be reckoned reasonable to investigate the effects of the lateral flow in terms of how much the spring constant should be degraded as a result of softening of the soil due to liquefaction. In an attempt to clarify this aspect, back-analyses were made for cases of damage to foundations for which field data such as soil conditions and damage features are made available. As a result, it was found that the degree of degradation in the spring constants is dependent on the relative movement of the piles and surrounding soils. Analysis results showed that, for example, if the pile displacement is about 50 % of the surrounding soil, the stiffness in the spring constant should be reduced to 1/1000 ~ 1/100 of the value which is generally assumed in the design practice considering non-liquefied conditions.

1 INTRODUCTION

Design of piles considering the effects of seismic motions is generally performed through the use of 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 superstructures, but 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 soils is reduced and these effects need to be considered. The Japanese Highway Code stipulate, for example, that for the majority of cases the spring constants be reduced by a factor of 1/6 - 2/3 for making the analyses. However, if it comes to the effects of lateral spreading of once liquefied soils, there has been no requirement stipulated in the code for the design of pile foundations. 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 types of approach. 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). This may be called "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 "Displacement-based approach". One of the advantages of this method is that it is possible to specify the amount of ground displacement which is indicative of the degree of destructiveness or severity of the lateral spreading. 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. 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 Design as mentioned above. While the

Figure 1. Soil-pile model and numerical computation scheme

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 the lateral spreading. Some preliminary calculation was performed in a previous paper (Ishihara, 1997), but in needs of more data additional case studies were undertaken recently in the above context and the results of such investigation are presented below.

2 SUBGRADE REACTION MODEL

In this interaction model, the soil is assumed to be a Winkler-type material in which the lateral force, F , is proportional to the relative displacement between the pile and the soil,

$$F = \beta k (U_G - U_P) \quad (1)$$

where U_G and U_P are lateral displacements of the ground and pile, respectively, and k is the coefficient of subgrade reaction. If the soil is brought to a state of liquefaction thereby developing lateral spreading, the stiffness of the soil would be reduced drastically leading to a reduction of the k -value. The degree of this stiffness reduction is expressed by β in Eq. (1)

which will be referred to as the "Stiffness degradation parameter". This may be alternatively interpreted as representing highly nonlinear characteristics of the soil in a state of lateral spreading. The soil-pile interaction model adopted in this study is displayed schematically in Fig. 1. Main features of the model are outlined as follows.

(1) The stiffness of the springs 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.

(2) Generally, there is an unliquefied 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 soil mass in the unliquefied layer may be modeled generally in two 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 an earth pressure is applied to the upper portion of the pile in the same direction as the flow of the underlying liquefied soil layer, as illustrated in Fig. 2a. 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. The other conceivable mode of movement would be to assume the surface soil to displace by an amount smaller than that occurring in the underlying deposit, as illustrated in Fig. 2b. In this case, the pile head is considered to move by an

Figure 2. Modes of surface soil displacement and direction of force application

amount equal to or greater than the surrounding soil in the upper layer and an earth pressure is postulated to act in the direction opposite to the soil movement. When the pile head moves by an amount equal to the surrounding soil, the earth pressure becomes equal to zero. It is to be noticed that, whichever the mode of displacement is, the movement of the soil in the underlying liquefied deposit should be assumed to be greater than the displacement of the pile. Otherwise, the direction of the force to the pile body is reversibly oriented and the basic assumption of the liquefied soil pushing the pile would become invalid.

It appears that the first type of the surface soil movement might be encountered in the case such as a single pile where there is no object in its vicinity as illustrated in Fig. 3a. In the case of group piles, free movement of the surface soil within the pile group might be restricted due to constraints from the surrounding piles, or by the presence of horizontal connecting beams as illustrated schematically in Fig. 3b. Thus, the second type of movement of the surface soil illustrated in Fig. 2b will become valid. It is to be noticed that the mode of displacement of the surface layer would be significantly influenced by various factors including the stiffness of the piles and characteristics of the foundation raft.

It will be assumed in the following back-analysis that the constraints as above will not exert any influence on the overall movement and flow in the underlying liquefied soil deposit. Thus, no matter whichever the mode of displacement of the surface layer is, the interaction of the pile in the underlying layer is not influenced.

(3) The ground displacement is specified and given to the springs in the liquefied portion of the soil deposit, and displacement and bending moment of the pile are calculated, whereby the pile is assumed to deform in an elasto-plastic manner.

Figure 3. Modes of the surface soil movement

Figure 4. Location of the building in Niigata investigated by excavation

3 CASE STUDIES

3.1 Family Court House Building

In the area of extensive liquefaction at the time of the Niigata Earthquake in 1964, lateral flow of liquefied sands took place exerting deleterious effects on foundations of structures. Among many of those injured by the lateral spreading, the reinforced concrete piles (RC-piles) supporting a three-story reinforced concrete building were those investigated in details by excavating the surrounding soil deposits to a depth of about 10 m (Yoshida and Hamada, 1991). The location of the building is shown in Fig. 4. The movement of the ground as identified by virtue of the air-photo interpretation is also reported in the paper as shown in Fig. 5, where it may be seen roughly that the movement of the ground consequent upon liquefaction was about 1.0 m in the north-east

Figure 5. Permanent ground displacements (cm) near the FCH Building (Yoshida and Hamada, 1991)

Figure 6. Foundation plan of the FCH Building (Yoshida and Hamada, 1991)

direction at the place where the Family Court House was located. The layout of the piled foundations are displayed in Fig. 6 where the exact locations of two investigated footings (No. 1 and No. 2) are indicated. These footings 1m x 1m in size were supported by single reinforced concrete piles 35 cm in diameter as indicated in the inset of Fig. 6. The investigations were conducted in 1989, 25 years after the earthquake. Such an opportunity presented itself

Figure 7. Soil boring data in the vicinity of FCH Building (Yoshida and Hamada, 1991)

when the building was completely demolished. Sheet piles were driven to enclose the foundation pile and while being dewatered, the ground was excavated to expose the complete body of the pile as it had been buried.

Soil profiles were investigated in 1987 by means of borings at two sites near the building as indicated in Fig. 5. Results of the soil investigation are shown in Fig. 7 where it can be seen that deposits of clean sand prevail to a depth of 30 m with SPT N-values less than 10 to a depth of about 12 m.

The outcome of field measurements on the lateral deformation of the two piles made in the excavated pits is demonstrated in Fig. 8. According to the design chart, both piles were designed to have a length of 14 m, but perhaps because of a stiff sand layer encountered, the pile driving for the Pile No. 1 must have been stopped at a depth of 10-12 m. This situation is conjectured from the absence of any injury to the Pile No. 1 around the bottom of the excavated pit (Yoshida and Hamada, 1991). The Pile No. 2 appeared to have been driven to a deeper stratum with the design length of 14 m as evidenced by many cracks that had developed near the bottom of the pit. This is indicative of a large bending moment having exerted during the lateral flow of the liquefied sand. The Pile No. 2 was completely chopped off about 2 m below the pile head, as indicated by the discontinuity in the measured pile deformation shown in Fig. 8.

When the excavation was conducted in 1989, the pit was dug first to a depth of 2.0 m while breaking and removing the footing slab. According to the design chart, the pile top was located at an elevation 1.65 m below the ground surface. The elevation of

Figure 8. Lateral deformation of Piles No. 1 and No. 2 in FCH Building (Yoshida and Hamada, 1991)

the ground water table could be somewhat variable, but based on the design chart and soil investigations it may be assumed to have been about 1.0 - 1.5 m deep as accordingly indicated in Fig. 7. The feature in which the pile head was connected with the footing slab is not known unfortunately. When the piles suffered injury due to the lateral flow at the time of the 1964 Niigata Earthquake, the soil mass above the ground water table may be assumed to have moved together with the liquefied soil deposit underlying the ground water table. This appears to have been the case, however, in the free field conditions where there was no constraint near the piles. In the foundation system of the FCH Building, a number of footing slabs were connected to each other by underground horizontal beams about 1.0 m in height and 0.8 m in width. Existence of such beams is considered to have exercised some constraint on the movement of the surface soil deposit in the vicinity of the single piles being studied. Thus, it may well be assumed that the deformation pattern as illustrated in Fig. 2b has occurred at the time of the lateral spreading in 1964 Niigata Earthquake. If this assumption is valid the lateral force must be acting in the direction opposite to that of the flow and its magnitude may be assumed to be equal to the passive earth pressure $K_p \gamma H_1^2 / 2$. It is also envisaged that the near surface deposit might have been ravaged by violent motions of the ground due to liquefaction involving cracking and spurting of mud water. Thus, the earth pressure acting on the pile would have been much smaller than that mobilized in the passive earth pressure conditions. As an extreme case, there would have been no lateral force acting on the pile-top. This could be the case where the pile and

Niigata FCH Building (Pile 2)

Figure 9. Spring constants for the RC-Pile No. 2 in FCH Building, Niigata

surrounding surface-soil move together without any relative displacement. With the considerations as above, it will be of interest to see the effects of the lateral force by assuming two cases, that is, zero earth pressure and passive earth pressure $K_p \gamma H_1^2 / 2$.

Though the connection between the footing and pile top is not known, it may be acceptable to assume for simplicity that the pile-footing system is modeled by a single continuous beam throughout the depth as illustrated in Fig. 1. The single pile is postulated to be supported by a series of springs and subjected to the zero or passive earth pressure to a depth of $H_1 = 1.25$ m as illustrated in Fig. 9. Regarding the lateral force below the depth H_1 , it is assumed to have been applied through the springs which are deformed in consistence with an amount of displacement specified at each depth of the liquefied layer.

In the analysis of the Piles No. 1 and No. 2 in the building of the Family Court House, the spring constants were evaluated by using an empirical formulae for the coefficient of subgrade reaction k stipulated in the Japanese Code of Bridge Design.

The spring constant K_h was determined by multiplying the coefficient of subgrade reaction k by an effective width of the pile

$$K_h = k d \quad (2)$$

The value of K_h evaluated in this way with reference to the SPT N-value of Fig. 7 is shown in Fig. 9 for the representative soil profile at the site of Family Court House.

Figure 10. Bending moment versus curvature relation for the RC-piles in the FCH Building

In making the analysis, it is necessary to specify the relation between the bending moment and curvature of the beam representing the flexural characteristics of the reinforced concrete pile. This relation was established based on the cross section and deformation characteristics of the piles as shown in Fig. 10. The analysis of the piles undergoing the lateral displacement were performed by a series of computations as described below.

(1) First, the lateral displacement is prescribed throughout the depth. In the case of the Family Court House site, the sand deposits are postulated to have developed liquefaction to a depth of 9.0 m and the entire soil mass above it to have moved with a cosine distribution of lateral displacement with its maximum value of 1.0 m at a depth of 1.25 m.

(2) The displacement of the ground as postulated above was applied to the equivalent linear spring model and analysis was made for the spring-supported beam to obtain the lateral displacement of the pile. In conducting the analysis, the pile head was assumed to be free to move horizontally whereas the lower end of the pile was assumed to be free only to rotate for the Pile No. 1 at the depth of 9.5 m and fixed for the Pile No. 2 at the depth of 14.0 m. In the analysis, the spring constant for the supposedly liquefied layer was reduced in a wide range taking the stiffness degradation parameter of $\beta = 5 \times 10^{-4} - 1 \times 10^{-2}$ as defined by Eq (1).

(3) As briefly mentioned above, the passive earth pressure was applied in the upper portion of the pile to a depth of $H_1 = 1.25$ m, where the angle of internal friction of $\phi = 30^\circ$ and $\gamma_t = 17.6$ kN/m³ were assumed to be the case. In the remaining portion of the pile to the depth of 9.0 m, the displacement with a cosine distribution was applied to the springs. Analysis was made as well for the case of no earth pressure in the upper portion of the piles with the same conditions otherwise.

Figure 11. Lateral displacements of Pile No. 1 computed for different β -values: Analysis with passive earth pressure (Niigata FCH Building)

Figure 12. Bending moments of Pile No. 1 computed for different β -values: Analysis with passive earth pressure (Niigata FCH Building)

Figure 13. Lateral pressure on Pile No. 1 computed for different β -values: Analysis with passive earth pressure (Niigata FCH Building)

(4) From the layout of the piled foundations shown in Fig. 6, the Pile No. 1 and No. 2 may be considered as single piles and as such the analysis was conducted for a pile with a width of $d = 35$ cm.

The outcome of the analysis for the Pile No. 1 is presented in Fig. 11 through Fig. 16 and the results for the Pile No. 2 are shown in Figs. 17 through 22.

As shown in Figs. 11 and 14, the lateral displacement for the Pile No. 1 is assumed to be zero at the depth of 9.5 m. Assuming this to be reasonable, Fig. 11 shows that the case of analysis with passive earth pressure using the degradation parameter $\beta = 8 \times 10^{-3}$ can provide the best fit to the displacements observed in the in-situ investigation. It is to be noticed that the displacement of the pile at the top of the liquefied layer or at the bottom of the unliquefied surface layer was shown to be 0.6 m which is smaller than the value of 1.0 m of the ground displacement prescribed in the analysis at this elevation. The degradation parameter $\beta = 8 \times 10^{-3}$ is plotted in Fig. 31 versus the relative displacement $U_G - U_P$ normalized to the thickness of the liquefied layer, H_2 .

In the case of the analysis with no earth pressure in the surface layer, the results of calculations are presented in Fig. 14. It may be seen that the best fit between the observed and calculated behaviour is achieved with $\beta = 1 \times 10^{-3}$ where the lateral pile-top

Figure 14. Lateral displacements of Pile No. 1 computed for different β -values: Analysis without earth pressure (Niigata FCH Building)

Figure 15. Bending moments of Pile No. 1 computed for different β -values: Analysis without earth pressure (Niigata FCH Building)

Figure 16. Lateral pressure on Pile No. 1 computed for different β -values: Analysis without earth pressure (Niigata FCH Building)

displacement is obtained also as 0.6 m in the analysis. The value of the normalized relative displacement $(U_G - U_P)/H_2$ is plotted in Fig. 31 versus the value of β .

Thus, if the liquefied layer is assumed to have induced lateral spreading overall in the area of FCH Building without any constraint from surrounding objects, a relative displacement of $1.0 - 0.6 = 0.4$ m is considered to have occurred at the top of the liquefied layer. Then, the stiffness degradation parameter is reckoned to have taken a value between $\beta = 1 \times 10^{-3}$ – 8×10^{-3} which depends generally upon the magnitude of the lateral pressure acting on the upper portion of the pile in the unliquefied surface layer.

The bending moment induced in the pile body is shown in Fig. 12 for the case of passive earth pressure application. It may be seen that at depths of 1.5 m to 7.5 m the induced bending moment far exceeds the yield value M_y at which larger cracks are considered to develop in the concrete. Observation of the in-situ investigation revealed that predominant cracks existed at the depths of 2.5 m to 4.5 m as accordingly indicated in Fig. 12. In the case of no earth pressure application, the range of depth at which the calculated bending moment exceeds the yield value coincides reasonably well with the depth of observed cracks as indicated in Fig. 15, but the value of the calculated moment for this case is smaller as com-

Figure 17. Lateral displacements of Pile No. 2 computed for different β -values: Analysis with passive earth pressure (Niigata FCH Building)

pared to the moment computed by considering the passive earth pressure. It is to be noticed that the bending moments are the least affected by the degradation parameter β , but they are significantly influenced by the condition whether or not there is a lateral pressure from the surface layer. It would be of interest to see how much lateral force is applied to the pile body in the lower layer from the surrounding soil deposit when it is undergoing lateral spread. For this reason, the lateral pressure per unit area of the pile body was computed and normalized to the total overburden pressure σ_v at each depth. The results of such analysis are demonstrated in Figs. 13 and 16 where it may be seen that, while the influence of the stiffness degradation parameter β is very small, effects of the lateral pressure in the unliquefied surface layer is significant on the lateral force. In the case of no earth pressure application, the lateral pressure on the pile is found to be somewhere between 5 and 20 % of the total overburden pressure whereas that computed by considering the passive earth pressure is predominantly in the range between 10 % and 70 % of σ_v .

The outcome of the analysis for the Pile No. 2 at FCH Building foundation is demonstrated in Figs. 17 through 22. With respect to the pile deformation, the best degree of coincidence is obtained, as shown in Figs. 17 and 20, when a value of $\beta = 8 \times 10^{-3}$ and

Figure 18. Bending moments of Pile No. 2 computed for different β -values: Analysis with passive earth pressure (Niigata FCH Building)

Figure 19. Lateral pressure on Pile No. 2 computed for different β -values: Analysis with passive earth pressure (Niigata FCH Building)

3.3×10^{-3} is assumed for the degradation parameter. Regarding the bending moment and lateral stress, the value of β does not generate much difference, but the magnitude of the earth pressure in the unliquefied surface layer is a dominant factor. In both cases being considered, the computed bending moment is in excess of the yielding moment and far above the crack-inducing moment within the depth range in which cracks were observed at the time of the in-situ investigation. Therefore, the bending moment would not be regarded as a quantity by which validity of the assumption is checked regarding the influence of the earth pressure in the surface layer.

For the Pile No. 2, Figs. 19 and 22 indicate that the lateral pressure on the pile in the liquefied deposit is in the range of 10 to 80 % and 5 to 50 % of the total overburden pressure for the cases with and without earth pressure application, respectively.

3.2 Building in Fukaehama

The damage to foundation piles of a 3-story building at the time of the 1995 Kobe Earthquake is reported by Tokimatsu et al. (1997). The building was situated 6 m inland from the quay wall on a reclaimed fill in Higashi-Nada, Kobe. It was constructed early in 1980's and supported by prestressed concrete piles (PC-piles) 40 cm in diameter and about 20 m long.

Figure 20. Lateral displacements of Pile No. 2 computed for different β -values: Analysis without earth pressure (Niigata FCH Building)

Figure 21. Bending moments of Pile No. 2 computed for different β -values: Analysis without earth pressure (Niigata FCH Building)

Figure 22. Lateral pressure on Pile No. 2 computed for different β -values: Analysis without earth pressure (Niigata FCH Building)

The building sustained a tilt of 3 degrees due to the lateral spreading of the soils. The piles beneath the foundation were exposed and detailed investigations were carried out in-situ to see features of injury on precast reinforced concrete piles. The investigations included visual observation of cracks around inside walls of the hollow cylindrical piles by means of a television camera and measurements of tilt through the depth by using a slope indicator lowered into the inside hole of the piles.

The plan view of the building foundation is shown in Fig. 23 with precise locations of the piles investigated. The soil profiles obtained at locations of boreholes No. 1 and No. 2 are presented in Fig. 24 where it may be seen that the reclaimed fills of the Masado soil exist to a depth of about 8.5 m. Liquefaction and lateral spread seem to have occurred in this reclaimed deposit. The results of the ground survey reported by Tokimatsu et al. (1997) are shown in Fig. 25 in terms of the lateral displacement of the ground surface plotted versus the distance from the waterfront. It may be seen that the ground displacement at the place of Pile S-7, 6 m inland, was about 1.2 m towards the sea. From an independent survey involving measurements of the inclination of the building structure and displacement of the roof, the displacement of the head of the Pile S-7 was estimated to be about 80 cm.

Figure 23. Arrangements of the foundations in plan view of the building in Higashi-Nada, Kobe (Tokimatsu et al., 1997)

The scheme of the back analysis for the performance of Pile S-7 is indicated in Fig. 26. The pile 20 m long is assumed to be rigidly-connected to the overlying footing slab and embedded to stiff strata extending from 9 m to 20 m which did not develop liquefaction. The soil above the ground water table may be assumed to have moved together with the underlying liquefied soil, but because of the constraint

Figure 24. Soil profile beneath the foundations of the building in Higashi-Nada, Kobe (Tokimatsu et al., 1990)

Figure 25. Lateral displacement of the ground surface at the site of the building in Higashi-Nada, Kobe (Tokimatsu et al., 1990)

produced by the horizontal underground beams between the front and back rows of the footing slabs, the soil in the unliquefied surface layer appears not to have been free to move with the underlying liquefied soil stratum. Thus, it appears highly likely that the pile in the unliquefied layer has moved nearly in unison with the surrounding soil stratum. With this assumption, the earth pressure in the surface layer may be neglected. If the pile is assumed to have moved ahead of the surrounding soil, the condition as illustrated in Fig. 2b might have been produced, thereby inducing the passive earth pressure acting against the movement of the pile. The computation for

Figure 26. Spring constants for the PC-pile of the building in Higashi-Nada, Kobe

Figure 27. Bending moment versus curvature relation for the PC-pile of the building in Higashi-Nada, Kobe

this case is not presented in this paper.

The model of the beam-spring system used in the analysis is illustrated in Fig. 26 where K_h -values estimated by the empirical formula in the design code are indicated. The nonlinear stiffness characteristics of the PC-pile are shown in Fig. 27 in terms of the bending moment versus curvature relation.

Figure 28. Lateral displacements of the Pile S-7 computed for different β -values: Analysis without earth pressure (building in Higashi-Nada, Kobe)

Figure 30. Lateral pressure on the Pile S-7 computed for different β -values: Analysis without earth pressure (building in Higashi-Nada, Kobe)

Figure 29. Bending moments of the Pile S-7 computed for different β -values: Analysis without earth pressure (building in Higashi-Nada, Kobe)

Displacement distributions computed in analyses of Pile S-7 for the case of zero earth pressure in the upper layer are shown in Fig. 28. Superposed in this diagram is the displacement of the pile measured by means of the inclinometer. Comparison of the observed displacement with the computed one appears to indicate that the stiffness degradation parameter should take a value of $\beta = 5 \times 10^{-3}$ in order for the displacement of the pile head to become equal to the observed value of 0.8 m. The value of $\beta = 5 \times 10^{-3}$ obtained above is shown in Fig. 31 for comparison sake plotted versus the normalized relative displacement $(U_G - U_P)/H_2$. Distribution of the computed bending moment is shown in Fig. 29 versus depth where it may be seen that the moment in excess of the yield value M_y occurs at two depth ranges which are roughly coincident with the depths of crack development observed by virtue of the video camera. Figure 30 shows the lateral pressure acting on the pile which is normalized to the total overburden pressure σ_v . It may be seen that the lateral pressure on the pile is less than 50 % of the total overburden pressure.

3.3 Foundation of spherical tank

At the time of the Kobe Earthquake in 1995, spherical tanks in a man-made island were damaged by extensive

Figure 31. Normalized relative displacement between the ground and pile versus the stiffness degradation parameter β : Summary plot for the three case histories

liquefaction and consequent lateral flow of the ground. Details of the damage feature and accounts of pile behaviour were introduced elsewhere (Ishihara, 1997) with reference to the back analysis made for the damaged piles. The back analysis was conducted in the same fashion as described above in order to assess the range of the stiffness reduction factor for piles subjected to lateral flow of the liquefied deposit. In that analysis scheme, the earth pressure on piles in the upper unliquefied layer was assumed to be a passive earth pressure. It was possible to perform the back analysis by assuming zero earth pressure on the piles in the surface layer. The results of the analysis for the two extreme cases as above are presented in Fig. 31 in terms of the relation between the normalized relative displacement and the stiffness degradation parameter.

4 DISCUSSIONS

In the routine practice, the design of piles is often made through the use of a Winkler-type model in which soil-pile interaction is represented by the coefficient of subgrade reaction k . When soil deposits are identified to be softened as a result of liquefaction, this coefficient is reduced drastically. For the level ground conditions the Japanese Code of Highway Bridge foundation recommends that the effect of liquefaction should be taken into account by reducing the value of k by a factor ranging between 1/6 and 2/3 for design of embedded foundations. If this norm

is to be extended to include sloping grounds undergoing lateral spreading, it would be logical for all practical purposes to seek how much reduction in the k -value would farther be necessary to allow for the effect of lateral spreading on the design of piles embedded in such ground. While many laboratory model tests have been performed using 1-g shaking table to evaluate effects of level-ground liquefaction on the k -value, little has been known on the effects of the lateral spreading on the reduction of the k -value.

As it is generally difficult to conduct model tests, it would be worthwhile trying to investigate these effects based on back-analysis of known behaviour of piles in the field during earthquakes. Thus, the above data of back-analyses were arranged so as to find out governing factors influencing the stiffness degradation parameter. In an effort in this vein, it was discovered that the stiffness degradation parameter, β , is closely related with the amount of pile deflection relative to the displacement of the ground. It was further shown that the deflection of the pile does depend on the thickness of the liquefied stratum H_2 . Thus, the normalized relative displacement $(U_G - U_P)/H_2$ was chosen as a parameter influencing the value of β . In this context, plots were made of the back-analyzed data as shown in Fig. 31. It may be seen in this diagram that the value of β tends to decrease with increasing normalized relative displacement. Note that the above relation is dependent on the flexural characteristics of the pile being considered. Thus, the relation shown in Fig. 31 does hold valid only for the

case of reinforced concrete piles with a diameter of 30-40 cm. If the pile of greater stiffness is considered such as cast-in-place concrete pile with a diameter of 1.0-1.5 m, the relation would be different.

As seen in Fig. 31, the correlation between the normalized relative displacement and the stiffness degradation parameter tends to change depending upon the lateral force being applied in the upper portion of the pile in the non-liquefied surface soil deposit. It is apparent that, if the passive earth pressure is applied in the direction opposite to the flow, the value of β giving an equal normalized displacement would become larger as compared to the case of no lateral force.

In all the cases studied above, the piles being considered were located within the space enclosed by neighboring piles or close to the horizontal connecting beams. Thus, strong constraints appears to have been present for the upper portion of the piles under consideration. This indicates that the surface unliquefied soil layer must have moved together with the piles in question thereby producing no earth pressure against the upper portion of the piles. With this fact in mind, the relation for the case of no earth pressure in Fig. 31 is regarded as reflecting actual situations developed in the field during the earthquakes. It is to be noted that the piles in the liquefied layer must have been displaced behind the soil deposit, producing a fairly large relative displacement. Then attention should be drawn solely to the soil-pile interaction in the liquefied deposit. It is apparent that the greater the reduction in the stiffness of the liquefied soil, the larger the relative displacement would be between the soil and piles.

5 CONCLUSIONS

As a result of the back analyses of three case histories of piles injuries during the past earthquakes, it was found that the spring constants representing the soil-pile interaction are by a factor of 1×10^{-3} - 1×10^{-2} smaller in the laterally spreading soils as compared to

the normal soil condition where there is no liquefaction. It was also pointed out that the degree of stiffness degradation in the laterally flowing soil deposits is related to the displacement of the pile relative to the surrounding soil, with the stiffness degradation becoming more pronounced with increasing relative displacement between the pile and the ground. The lateral force acting on the pile during the lateral spreading was found to be less than 50-60 % of the total overburden pressure.

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