PSEUDO-STATIC ANALYSIS OF PILES SUBJECTED TO LATERAL SPREADING

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SUMMARY

Soil liquefaction during strong ground shaking results in almost a complete loss of strength and stiffness in the liquefied soil, and consequent large ground deformation. Characteristics of the liquefied soils and loads on piles are significantly different during the cyclic phase and in the subsequent lateral spreading phase. Thus, it is necessary to separately consider these two phases in the simplified analysis of piles. This paper describes a practical procedure for preliminary assessment of piles subjected to lateral spreading. Effects of a crust of non-liquefied soil at the ground surface, properties of liquefied soils and pile groups are discussed in relation to their modelling in the simplified pseudo-static analysis approach. Particular attention is given to the treatment of unknowns and uncertainties involved in the simplified analysis and the need for parametric studies.

INTRODUCTION

There are several methods available for analysis of piles in liquefied soils including sophisticated dynamic analysis based on the effective stress principle and simplified methods based on the pseudo-static approach. A dynamic effective stress analysis permits rigorous evaluation of the complex seismic soil-pile-structure interaction while considering the effects of excess pore pressures and eventual soil liquefaction. Since this analysis allows detailed modelling of the liquefaction process and dynamic behaviour of soil-pile-structure systems, it is considered appropriate for assessment of the seismic performance of important structures. Its wider application to engineering practice is hindered by the high demands that this method imposes on the user regarding the knowledge and understanding of the phenomena considered and particular features of the adopted numerical procedure.

For preliminary assessment and design of piles, a simplified analysis may be more appropriate since this analysis is based on conventional engineering concepts and therefore is easy to implement in practice. In order to properly simulate the behaviour of piles in liquefying soils and address the design needs, the simplified analysis must satisfy the following requirements: i) the adopted model must capture the relevant deformation mechanism for piles in liquefying soils; ii) the analysis should permit estimating the inelastic response and damage to piles, and iii) the method should allow for variations in key parameters and assessment of the uncertainties associated with lateral spreading.

This paper first introduces the salient features of the behaviour of piles in liquefying soils and resulting typical damage to piles, and then discusses the implementation of the simplified pseudo-static analysis in the context of the three principal requirements mentioned above. Key parameters and uncertainties involved are discussed, in particular their importance and how to address these uncertainties in the simplified analysis of piles. A simplified pseudo-static analysis procedure for assessment of piles subjected to lateral spreading is eventually summarized in several key steps. Even though a particular analysis method has been adopted in this study, the proposed procedure is applicable to the pseudo-static analysis approach in general.

GROUND DISPLACEMENTS IN LIQUEFIED SOILS

When analysing the behaviour of piles in liquefying soils, it is useful to distinguish between two phases in the soil-pile interaction: a cyclic phase in the course of the intense ground shaking and consequent development of liquefaction, and a lateral spreading phase following the liquefaction. During the cyclic phase, the piles are subjected to cyclic horizontal loads due to ground displacements (kinematic loads) and inertial loads due to vibration of the superstructure; the combination of these oscillatory kinematic and inertial loads determines the critical load for the integrity of the pile during the shaking. Lateral spreading, on the other hand, is primarily a post-liquefaction phenomenon that is characterized by very large unilateral ground displacements and relatively small inertial loads. Thus, the characteristics of liquefied soils and lateral loads on piles can be quite different between the cyclic phase and the subsequent lateral spreading phase. For this reason, when utilizing simplified methods for assessment of the pile response, two separate analyses have to be conducted: one for the cyclic phase, and another for the lateral spreading phase of the response. Even though this paper focuses on the analysis of piles subjected to lateral spreading, both phases of the response are briefly discussed in the following paragraphs in order to understand the process of development of liquefaction and subsequent lateral spreading.

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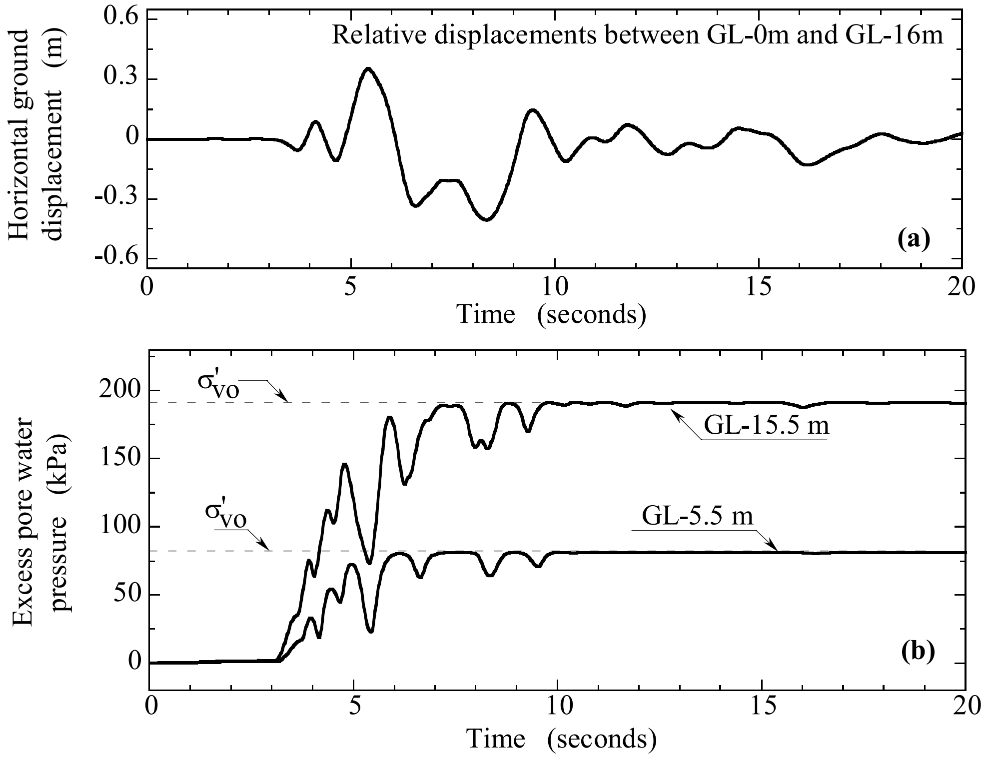

Figure 1: ***Ground response of liquefied deposit in the 1995 Kobe earthquake: (a) Cyclic ground displacement (relative displacement between ground surface and 16m depth); (b) Excess pore water pressure at 5.5m and 15.5m depth.***

Cyclic Ground Displacements

In order to illustrate some important features of ground displacements in liquefied soils, observations from well documented case histories in the 1995 Kobe earthquake are discussed in the following. Figures 1a and 1b show computed horizontal ground displacement and excess pore water pressures that developed in an 18 m thick fill deposit during the intense ground shaking caused by this quake. This ground response is representative for the cyclic phase of the response of free field deposits in areas that were not affected by lateral spreading.

Several features of the ground response shown in Figure 1 are relevant for the behaviour and analysis of piles in liquefying soils. First, the cyclic horizontal ground displacements in the course of the strong shaking are very large with peak values of about 0.35-0.45 m. These displacements roughly correspond to an average shear strain of about 3% to 4% in the liquefied layer. Next, it is important to note that at the time when the ground displacement reached a large value of about 0.3m for the first time since the start of the shaking, i.e. at approximately 5.3 sec, the excess pore water pressure was well below the initial effective overburden stress thus indicating that the soil has not fully liquefied, at this stage. These large cyclic displacements were accompanied with high ground accelerations of about 0.4g at the ground surface. This type of behaviour, where large ground displacements and high accelerations concurrently occur just before or at the onset of liquefaction, has been also observed in shake table experiments, thus highlighting the need to carefully consider the combination of inertial loads from the vibration of the superstructure and kinematic loads from the ground movement when analysing the behaviour of piles during the cyclic phase. The magnitude of these loads depends on a number of factors including the excess pore water pressure build-up, relative stiffness between the soil and the pile, and relative predominant periods of the ground and superstructure, among others. Clear and simple rules for combining the ground displacements (kinematic loads) and inertial loads from the superstructure in the simplified pseudo-static analysis of piles have not been established yet, though some suggestions may be found in [1], [2] and [3].

Lateral Spreading Displacements

In the 1995 Kobe earthquake, the ground distortion was particularly excessive in the waterfront area where many quay walls moved several metres towards the sea and lateral spreading occurred in the backfills. The spreading progressed inland as far as 200 m from the revetment line. Ishihara et al. [4] investigated the features of movements of the quay walls and ground distortion in the backfills by the method of ground surveying and summarized the measured displacements in plots depicting the permanent ground displacement as a function of the distance inland from the waterfront, as shown in Figure 2. Here, the shaded area shows the range of measured displacements along several sections of Port Island in the N-S direction while the solid line is an approximation for the average trend; the horizontal dashed line indicates the level of cyclic ground displacements in the free field. Note that permanent ground displacements were about 1-4m at the quay walls and exceeded 0.5m up to about 50m from the revetment line. Since lateral spreading develops after the cyclic phase and is typically a post-liquefaction phenomenon, it is associated with higher excess pore pressures and hence lower stiffness of liquefied soils as compared to those in the preceding cyclic phase. Clearly the magnitude and spatial distribution of ground displacements, as well as the stiffness of liquefying soils are quite different between the cyclic phase and lateral spreading phase. Thus, in order to properly account for these differences, separate simplified analyses of the pile have to be performed for the cyclic phase and lateral spreading phase of the response.

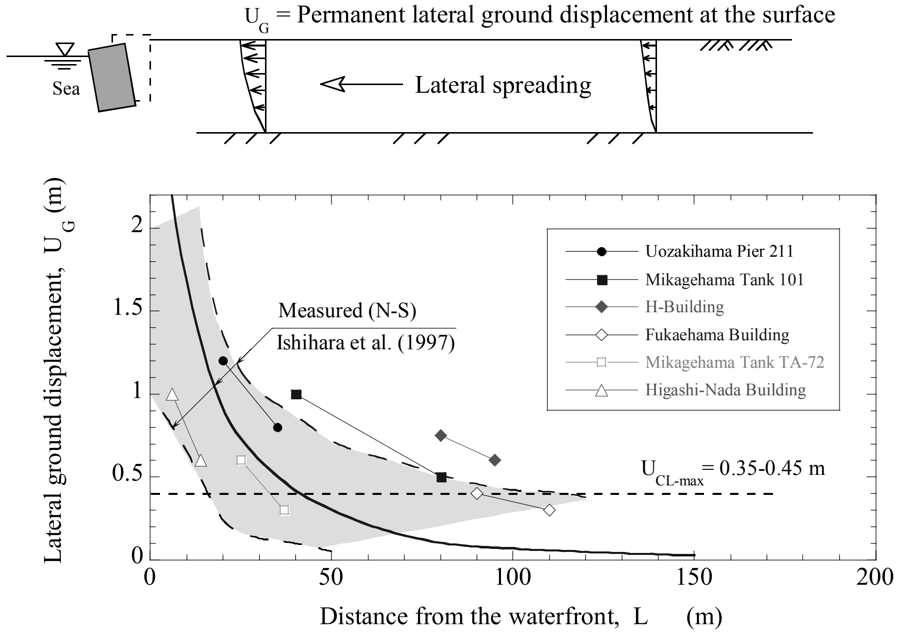

Figure 2: ***Permanent lateral ground displacements due to spreading of liquefied soils in the 1995 Kobe earthquake.***

TYPICAL DAMAGE TO PILES

A large number of pile foundations of buildings, storage tanks and bridge piers located in the waterfront area of Kobe were damaged in the 1995 Kobe earthquake (e.g., [5], [6], [7], [8]). Detailed field investigations were conducted on many piles by lowering borehole video cameras and inclinometers in order to inspect the distribution of cracks and deformation of the pile throughout its length. Visual inspection of the damage at the pile head was also carried out by excavating the surface soils surrounding the pile top. By and large, the observed damage to the piles can be summarized as follows:

1. For most of the piles, largest damage was observed at two locations: at the pile head and in the zone of the interface between the liquefied layer and the underlying non-liquefied layer (Figure 3).
2. Piles in the zone of large spreading displacements were consistently damaged at depths corresponding to the interface between the liquefied layer and the underlying non-liquefied layer. Since this interface was at large depths where inertial effects from the superstructure are known to be relatively insignificant, this damage can be attributed to the kinematic loads arising from the excessive lateral ground movement due to spreading.
3. Damage at the pile head was encountered both for piles in the free field and piles located within the lateral spreading zone, near quay walls. Both inertial loads from the superstructure and kinematic loads due to lateral ground displacements contributed to the damage at the pile head.
4. The variation in lateral spreading displacements with the distance from the waterfront shown in Figure 2 may result in different lateral loads being applied on individual piles, depending on their position within the pile-group [9]. This in turn may lead to significant cross-interaction effects and consequent bending deformation and damage to piles in accordance with these interaction loads from the pile-cap-pile system. In some cases, these pile-group effects were dominant and effectively governed the mode of failure in the piles [10].

In addition to the typical damage patterns described above, other less significant and less common damage features were found consistently throughout the length of piles thus reflecting the complex dynamic nature of loads and behaviour of piles in liquefying soils.

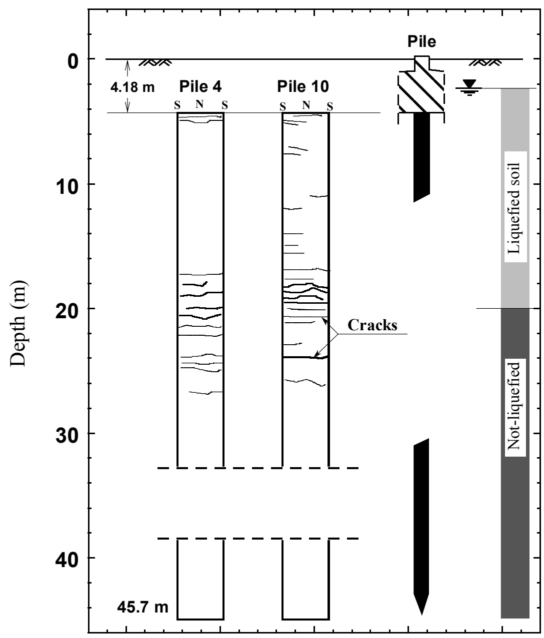

Figure 3: ***Typical damage to piles observed in the 1995 Kobe earthquake: Uozakihama bridge pier P211.***

PSEUDO-STATIC APPROACH FOR SIMPLIFIED ANALYSIS

The most frequently encountered soil profile for piles in liquefied deposits consists of three distinct layers [11]. As, illustrated in Figure 4, in this typical configuration the liquefied layer is sandwiched between a non-liquefied crust layer at the ground surface and a non-liquefied base layer.

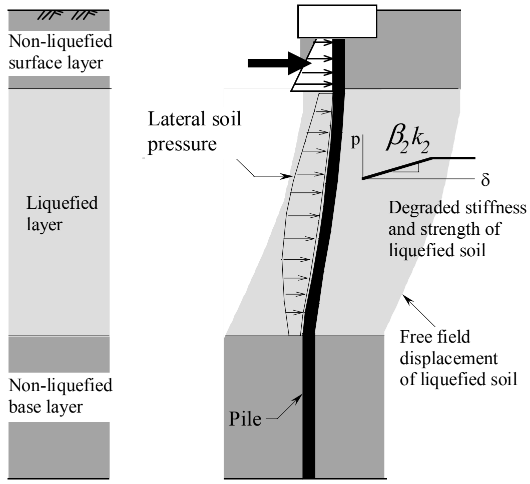

Figure 4: Three-layer deposit for piles in liquefied soils.

Liquefaction during strong ground shaking results in almost a complete loss of strength and stiffness of the liquefied soil, and consequent large ground deformation. As demonstrated in the previous section, particularly large and damaging for piles are ground displacements due to spreading of liquefied soils. During spreading, the non-liquefied surface layer is carried along with the underlying spreading soil, and when driven against embedded piles, the crust layer is envisioned to exert large lateral loads on the piles. Thus, when evaluating the pile response to lateral spreading with the simplified pseudo-static approach, one should take into account the significant degradation of stiffness and strength of the liquefied soil, excessive lateral ground displacements and relatively large lateral loads from the non-liquefied layer at the ground surface. Needless to say, the proposed pseudo-static approach is not limited to the three-layer model, and for example, a FEM beam-spring model that permits consideration of a multi-layer deposit with variable soil properties can be used. A typical beam-spring model is shown in Figure 5 in which the pile is represented by beam elements while the soil is modelled with a series of horizontal springs which are distributed throughout the length of pile. As indicated in Figure 5, springs with degraded properties are used for the liquefied layer and ground displacement representing the lateral spreading is applied at the free end of the springs. This input ground displacement corresponds to the displacement of the deposit in the free field which is not affected by the response or presence of piles.

Since one of the key requirements in the simplified analysis is to estimate the inelastic response and damage to the pile, in the proposed model simple but nonlinear load-deformation relationships are adopted for the soil-pile system. These are summarized in Figure 6 together with the input parameters of the computational model. Bilinear springs defined by a subgrade reaction coefficient *(k_i)* and an ultimate pressure from the soil *($p_{i\text{-}max}$)* are adopted for the soil. For the non-liquefied layers, the ultimate lateral pressure from the soil is defined using conventional concepts, e.g. the Rankine passive pressure, whereas an estimate for the residual strength of liquefied soils is used as a basis for evaluation of the ultimate pressure from the liquefied soil on the pile. As illustrated in Figure 6, the stiffness of the springs in the liquefied soil is degraded by a factor β_2 (where $\beta_2 < 1.0$). A tri-linear moment-curvature relationship (M-ϕ) is adopted for the pile indicating threshold levels at cracking, yielding and ultimate states typically used for reinforced concrete piles. In general, however, any nonlinear moment-curvature relationship can be used for the pile.

KEY PARAMETERS AND UNCERTAINTIES INVOLVED

As described in the previous sections, the behaviour of piles subjected to lateral spreading is very complex involving rapidly changing dynamic loads and deformational characteristics of liquefying soils. A key issue in the implementation of the pseudo-static analysis is therefore how to determine appropriate values for the parameters of the simplified model in view of the highly dynamic nature of the problem and the adopted static analysis approach. Clearly, it is very difficult to estimate the stiffness and strength of liquefying soils or to predict the magnitude and spatial distribution of lateral spreading displacements. In addition, there are significant uncertainties associated with the ground

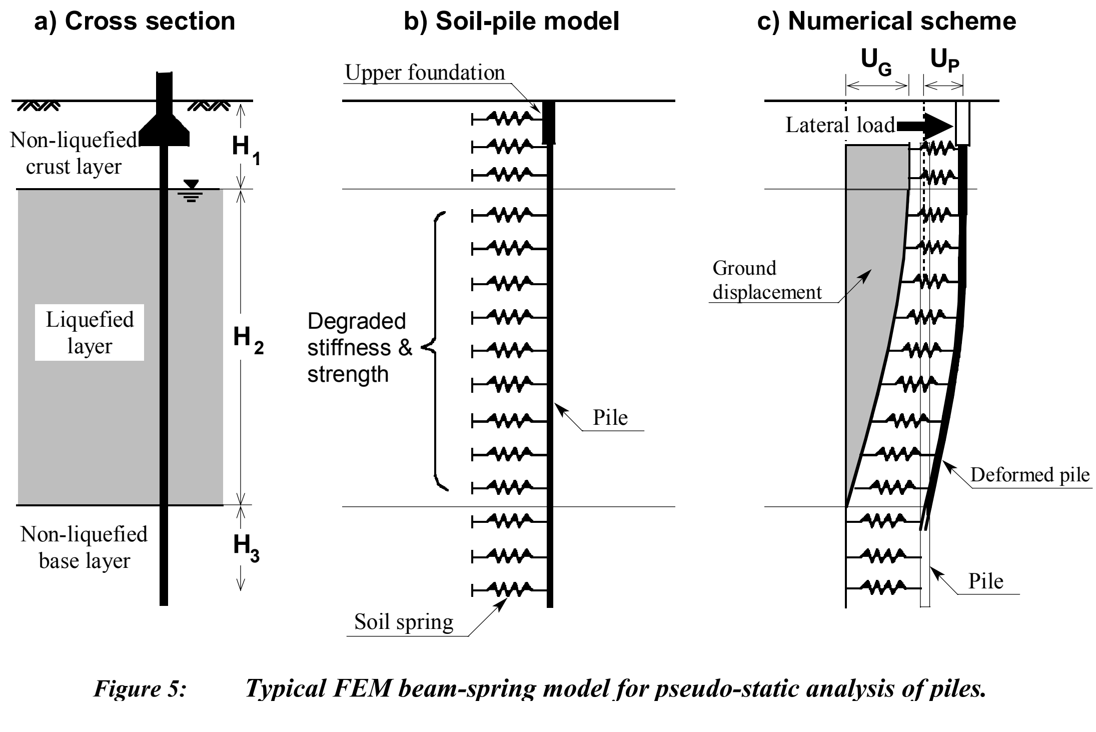

Figure 5: Typical FEM beam-spring model for pseudo-static analysis of piles.

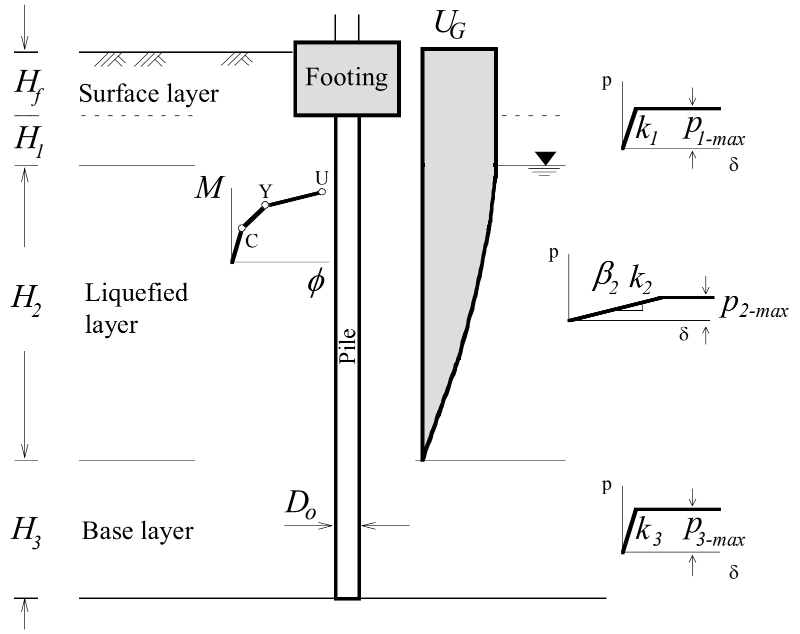

Figure 6: ***Characterization of load-deformation relationships and input parameters of the model.***

motion itself which in turn defines the characteristics of the dynamic excitation and response of the soil-pile-structure system. In other words, there are considerable unknowns and uncertainties associated with the earthquake excitation, consequent response and deformation of the ground, and in how to appropriately analyse the dynamic problem by an equivalent static analysis approach. In what follows, the determination of key parameters of the simplified model is discussed in light of these uncertainties and the objectives of the pseudo-static analysis.

Lateral Ground Displacements

The permanent ground displacement due to lateral spreading (U_G) can be evaluated using empirical correlations based on case histories from strong earthquakes (e.g., [4], [8], [12], [13]). It is important to recognize that in most cases it would be very difficult to make a reliable prediction for the spreading displacements. This difficulty is well illustrated in Figure 2 where a large scatter in the permanent ground displacements is seen even though these data are from a single earthquake event and generally similar ground conditions. In this context, Youd et al. [13] suggested the use of a factor of 2 for the displacements predicted with their empirical model, in order to cover the expected range of variation in spreading displacements. In other words, if a lateral ground displacement of 1.0 m has been predicted with the empirical equation, then the actual displacement is expected to be in the range between 0.5 m and 2.0 m. The obvious question is then what value to use for the ground displacement in the pseudo-static analysis of piles.

Cyclic ground displacements can be estimated more accurately by means of an effective stress analysis, for example, but the use of an advanced analysis for determination of the input in a simplified analysis is difficult to justify and certainly is not practical. For this reason, it seems more appropriate to estimate the peak cyclic displacements for the simplified analysis by using empirical charts correlating the maximum cyclic shear strains in the liquefied layer with the cyclic stress ration and SPT blow count, as suggested by Tokimatsu and Asaka [8], for example. Note that in both cases of cyclic and spreading displacements, the lateral ground displacement that is used as an input in the simplified analysis of piles is a free field ground displacement.

Crust Layer

The lateral load from the non-liquefied crust layer may often be the critical load for the integrity of the pile because of its large magnitude and unfavourable position as a "top-heavy" load acting above a laterally unsupported portion of the pile embedded in the liquefied soil. For the adopted bilinear p-δ relationship for the crust layer shown in Figure 6, the key parameter affecting the pile response is the ultimate lateral pressure, p_{1-max}.

The ultimate soil pressure from the surface layer per unit width of the pile can be estimated using a simplified expression such as, $p_{1-max} = \alpha_u\, p_p$, where $p_p(z_1)$ is the Rankine passive pressure while α_u is a scaling factor to account for the difference in the lateral pressure between a single pile and an equivalent wall. Figure 7 shows the variation of α_u with the

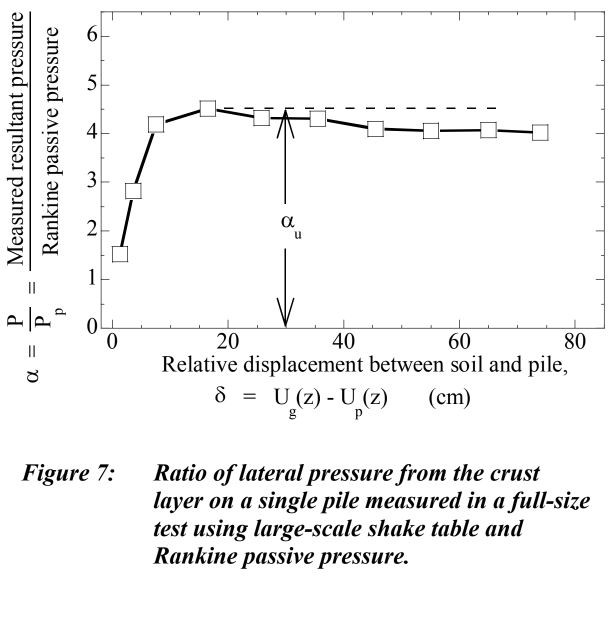

Figure 7: ***Ratio of lateral pressure from the crust layer on a single pile measured in a full-size test using large-scale shake table and Rankine passive pressure.***

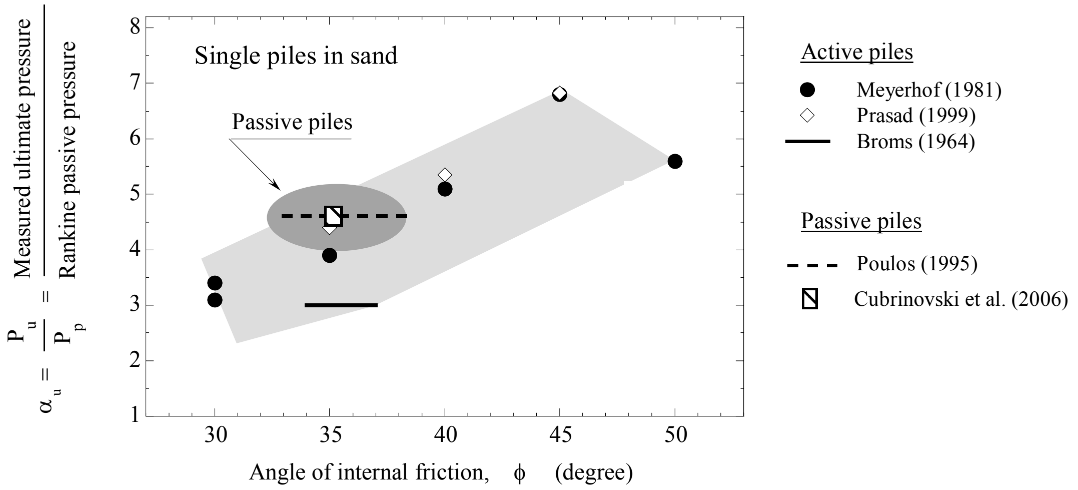

Figure 8: ***Ratio of ultimate pressure from the crust layer on a single pile and Rankine passive pressure obtained in experimental studies.***

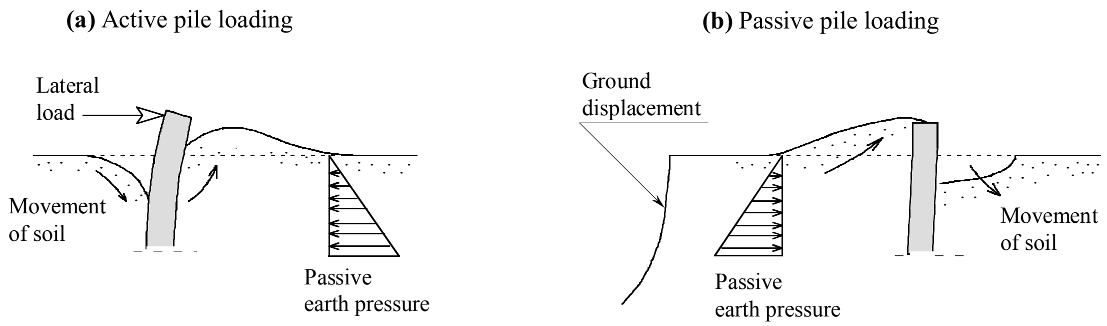

Figure 9: ***Schematic illustration of lateral loading of piles: (a) Active pile loading; (b) Passive pile loading.***

relative displacement observed in a benchmark lateral spreading experiment on full-size piles [14]; the maximum lateral pressure on the single pile was about 4.5 times the Rankine passive pressure. Data from other experimental studies summarized in Figure 8 also indicate very large values for the parameter α_u, clearly indicating that excessive lateral loads can be applied from the crust layer to the pile. When estimating the value of α_u it is important to distinguish between two loading conditions for the pile, so-called *active pile loading* and *passive pile loading* respectively. In the case of active pile loading, a horizontal force at the pile head is the causative load for the pile deformation, as shown in Figure 9a; in this case, the mobilized earth pressure resists the pile movement and provides a resisting force. In the case of passive pile loading, a distributed load along the pile body is applied due to lateral ground movement; hence, in this case, the mobilized pressure from the crust layer provides a driving force for the pile deformation, as illustrated in Figure 9b. In this context, it is worth mentioning the method of Broms [15] which is often referred to in codes dealing with piles under lateral loads. In this method, Broms suggested a value of α_u = 3 to be used when determining the ultimate pressure from the soil on the pile. The test data used by Broms was on active pile loading with values for α_u predominantly in the range between 3 and 6. On this basis, Broms adopted the lower-bound value of α_u = 3 as a conservative estimate for active piles (since lower soil resistance leads to larger pile deformation in this case). This value has been adopted in many design codes for assessment of piles under lateral loads, however, it is important to note that this value may be unconservative for passive piles (since a smaller value of α_u will reduce the load and hence the deformation of the pile, in the case of passive pile loading). As shown in Figure 8, the two sets of experimental data on passive piles suggest a higher value of α_u = 4.5 for passive pile loading which clearly is the relevant mode for piles subjected to lateral spreading.

Figure 7 shows that a large relative displacement of nearly 20 cm was needed to mobilize the ultimate lateral pressure from the crust layer on the pile in the full-size test. This relative displacement δ_u at which p_u is mobilized depends on the relative density of the sand, as illustrated by the experimental data summarized in Figure 10. Here, H denotes the height of the model wall or pile cap used in the particular test. It is evident that for dense sands with D_r = 70% to 80%, the ultimate pressure was mobilized at a relative displacement of about δ_u = 0.02H to 0.08H and that much larger displacement was needed to mobilize the passive pressure in loose sand. Rollins and Sparks [16] suggested that the presence of a low strength layer below the surface layer may increase the required displacement to mobilize the passive pressure, and this appears to be a relevant observation for a crust layer overlying liquefied soils. In any case, it should be acknowledged that in the field a relative displacement between the soil and pile on the order of several centimetres or even tens of centimetres would be needed to mobilize the maximum pressure from the crust layer on the pile.

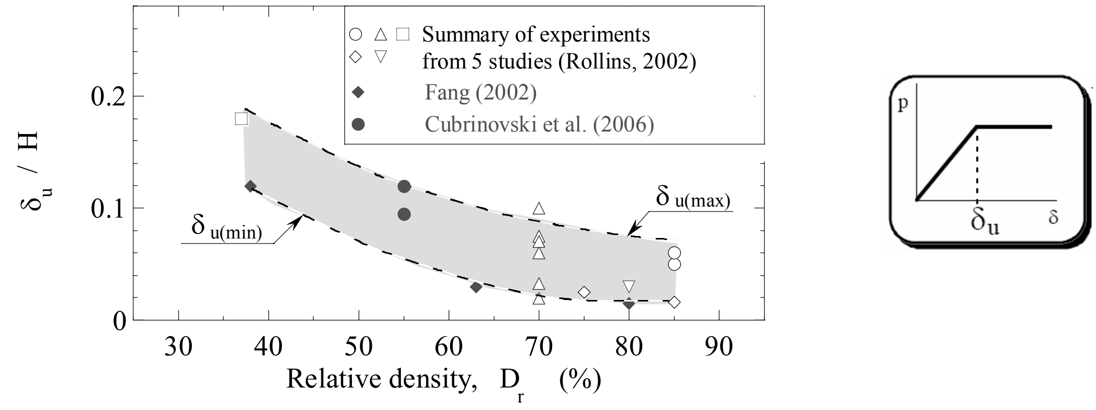

Figure 10: ***Relative displacement required to fully mobilize the passive pressure as a function of the relative density of sand: summary of data from experimental studies.***

Liquefied Layer

Key parameters controlling the lateral load from the liquefied soil on the pile are the stiffness reduction factor β_2 and the ultimate pressure from the liquefied soil $p_{2\text{-}max}$.

The factor β_2, which specifies the reduction of stiffness due to liquefaction and nonlinear stress-strain behaviour ($\beta_2 k_2$), is affected by a number of factors including the density of sand, excess pore pressure level, magnitude and rate of ground displacements, and drainage conditions. Typically, β_2 takes values in the range between 1/50 and 1/10 for cyclic liquefaction and between 1/1000 and 1/50 in the case of lateral spreading. The values of β_2 back-calculated from the full-size tests on piles [14] are shown in Figure 11 as a function of the lateral ground displacement, illustrating that β_2 is not a constant, but rather it varies in the course of lateral spreading. In general, the value of β_2 should be related to the soil properties and anticipated ground deformation. For example, lower values of β_2 are expected for very loose soils because such soils are commonly associated with high and sustained excess pore water pressures and large ground deformation. While this sort of qualitative evaluation of β_2 should be carefully considered, the quantification of this parameter is very difficult and subjective because of the inherent uncertainties associated with properties of liquefying soils.

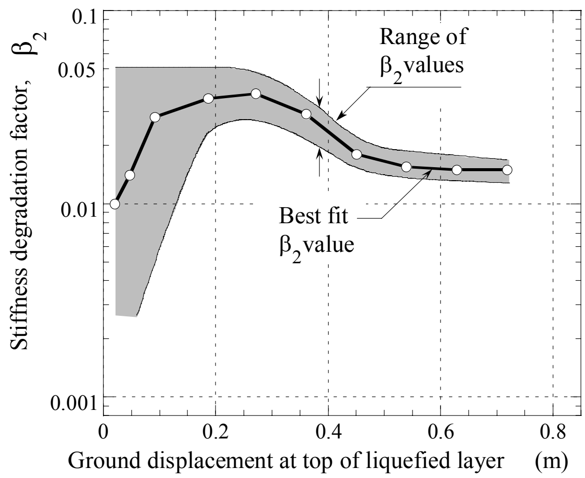

Figure 11: ***Degradation of stiffness in the liquefied layer observed in full-size test on piles.***

The residual strength of liquefied soils S_r could be used in the evaluation of the ultimate pressure from the liquefied soil on the pile, $p_{2\text{-}max} = \alpha_{ur} s_r$. Here, the residual strength s_r can be estimated using empirical correlation between the residual strength of liquefied soils and SPT blow count, such as that proposed by Seed and Harder [17] or Olson and Stark [18]. Note that the former correlation assumes a constant residual strength for a given blow count while the latter one uses a normalized residual strength of the form (s_r/σ'_{vo}) and hence implicitly assumes that s_r increases with depth for a given SPT blow count. Even though s_r is sometimes referred to as the undrained or steady state strength, in effect the values of s_r obtained from the above-mentioned empirical charts are significantly lower than the corresponding undrained strength values, and strictly speaking, they do not correspond to the so-called steady state of deformation.

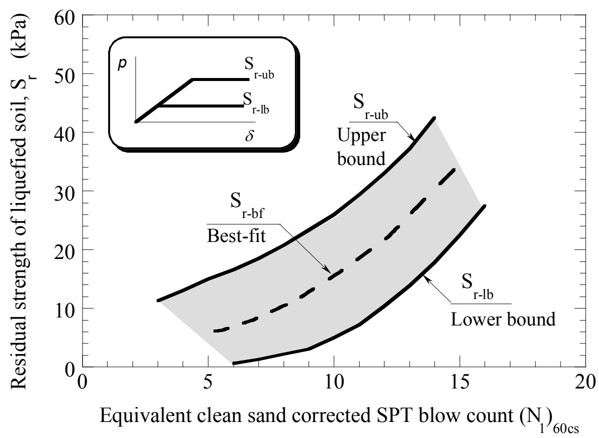

Figure 12: ***Residual strength (S_r) of liquefied sandy soils back-calculated from case histories (after Seed and Harder, 1991).***

The shaded area in Figure 12 shows the correlation between S_r and the normalized SPT blow count for clean sand $(N_1)_{60cs}$ proposed by Seed and Harder [17]. A large scatter exists in this correlation indicating significant uncertainty in the value of S_r for a given $(N_1)_{60cs}$ value. For example, for $(N_1)_{60cs} = 10$, the value of S_r can be anywhere between 5 kPa (lower bound value) and 25 kPa (upper bound value). In addition, the multiplier α_{ur} ($p_{2\text{-}max} = \alpha_{ur}\, s_r$) is also unknown and subject to uncertainties. Note that α_{ur} is different from the corresponding parameter α_u for the crust layer previously discussed, because the interaction and mobilization of pressure from surrounding soils on the pile is different between liquefied and non-liquefied soils.

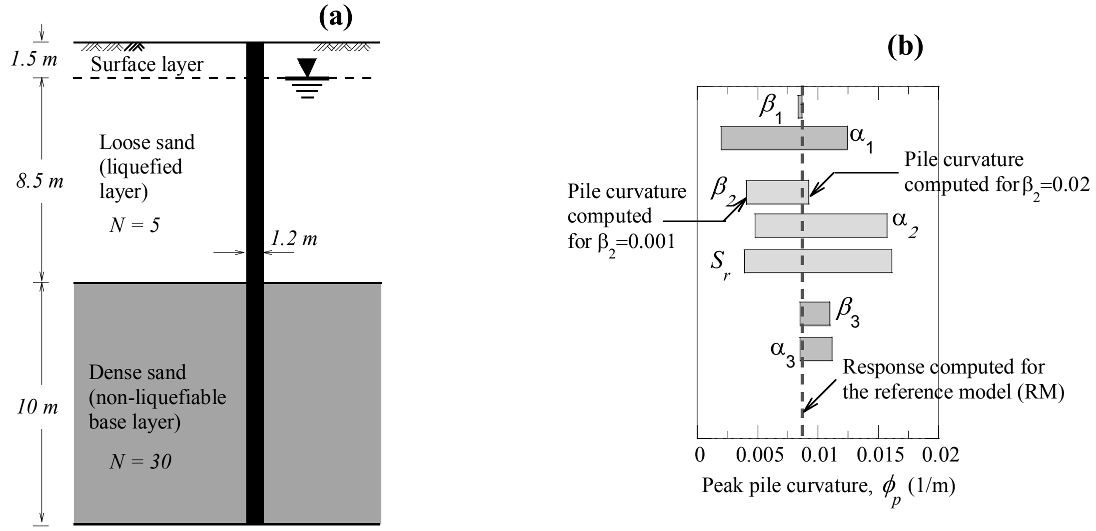

Figure 13: ***Effects of variation in soil parameters (α_i, β_i and S_r) on the pile response: (a) Soil-pile model; (b) Tornado chart indicating the computed peak curvature of the pile for different values of key parameters.***

Effects of Uncertainties on the Computed Pile Response

Clearly, there are significant unknowns in the simplified pseudo-static analysis of piles associated with the lateral spreading and properties of liquefied soils. To illustrate the effects of these uncertainties, the response of a single pile model (Figure 13a) computed using the pseudo-static analysis is considered below [19]. The model represents a 1.2m diameter pile embedded in a deposit consisting of a loose sand (liquefiable) layer overlying a non-liquefiable layer of dense sand. The water table is assumed to be at 1.5m depth, thus defining a crust layer at the ground surface with a thickness of H_1 = 1.5 m overlying a liquefiable layer with a thickness of H_2 = 8.5 m, as illustrated in the figure. Parameters of the computational model were determined based on the adopted pile properties, penetration resistance and previously discussed relevant range of values for each of the model parameters. Since many of the parameters are not uniquely defined but rather may vary over a relatively wide range of values, first a representative set of values was adopted for a so-called reference model (RM). The adopted values of the parameters for the reference model with respect to the stiffness degradation (β_i), multiplier for the ultimate soil pressure (α_i) and residual strength of the liquefied layer (S_r), which are the focus of the presented parametric study, are summarized in Table 1. The computed response of the reference model for an applied free field ground displacement of U_G = 1.0 m is shown with the broken line in Figure 13b in terms of the peak pile curvature, ϕ_p = 0.0087m^{-1} (the largest curvature along the length of the pile).

Table 1. Summary of model parameters.

Parameter	Reference value (RM)	Lower bound value	Upper bound value
β_1	1.0	0.3	1.0
α_1	4.5	3.0	5.0
β_2	0.01	0.001	0.02
α_2	3.0	1.0	6.0
S_r	8.0	1.0	15.0
β_3	1.0	0.3	1.0
α_3	9.0	5.0	9.0

In addition, a series of parametric analyses were conducted in which an extreme value was used for one of the listed parameters in Table 1 (either the upper-bound or the lower-bound value from the relevant range of values for the parameter) while all other parameters were kept at their reference value (RM). The horizontal bars of the tornado chart shown in Figure 13b indicate the computed response in such parametric analyses. For example, a peak pile curvature of ϕ_p = 0.0046m^{-1} was computed when β_2 = 0.001 was used while ϕ_p = 0.0088m^{-1} was computed for β_2 = 0.02 respectively. Hence, the width of the horizontal bars indicates the sensitivity of the pile response on the variation of the examined parameter. It is evident from this plot that the computed pile response changes significantly when the value of a single parameter is varied within its relevant range of values. This clearly shows that the uncertainties associated with the parameters of the model have to be considered in the simplified pseudo-static analysis of piles in a parametric study.

Pile-Group Effects

Pile groups may generally affect the behaviour of piles in liquefying soils in two ways, first through the cross interaction among the piles within the group, and second, by influencing the key parameters controlling the pile response such as the stiffness of the liquefied soil or magnitude and spatial distribution of spreading displacements. Both effects are briefly discussed in this section.

Piles in a group are almost invariably rigidly connected at the pile head, and therefore, when subjected to lateral loads, all piles will share nearly identical horizontal displacements at the pile head. During lateral spreading of liquefied soils in the waterfront area, each of the piles could be subjected to a different lateral load from the surrounding soils, depending upon the particular location of the pile within the group and the spatial distribution of the spreading displacements. This is illustrated schematically in Figure 14, for a pile group. Consequently, both the interaction force at the pile head and the lateral soil pressure along the length of the pile will be different for each pile, thus leading to a development of different stresses and deformation along the length of individual piles in the group (Figure 15). This response feature, in which the piles share identical displacements at the pile head but have different deformations throughout the depth, is considered to be a significant feature of the deformational behaviour of pile groups subjected to lateral spreading. These pile-group effects can be easily captured by a simplified method of analysis using a single pile model [9]. Needless to say, the presented beam-spring model is not restricted to a single pile model and it can be applied to a pile group, which in turn permits to directly account for the aforementioned cross-interaction effects.

The second influence of the pile-group regarding its effects on the magnitude and distribution of ground displacements, stiffness characteristics of spreading soils and ultimate soil pressure, is more difficult to quantify. Experimental data on these effects for piles in liquefiable soils is scarce and not conclusive. Figure 16, for example, illustrates a clear tendency for reduction in the ultimate lateral soil pressure with an increasing number of piles within the group. These data are for a pile spacing of 2.5-3 diameters, and include both active and passive piles, though the trend is basically derived from active piles. Further evidence for the pile-group effects on key parameters controlling the pile response in liquefying soils such as U_G, β_2, α_2, $p_{2\text{-}max}$ and $p_{1\text{-}max}$ discussed herein is urgently needed.

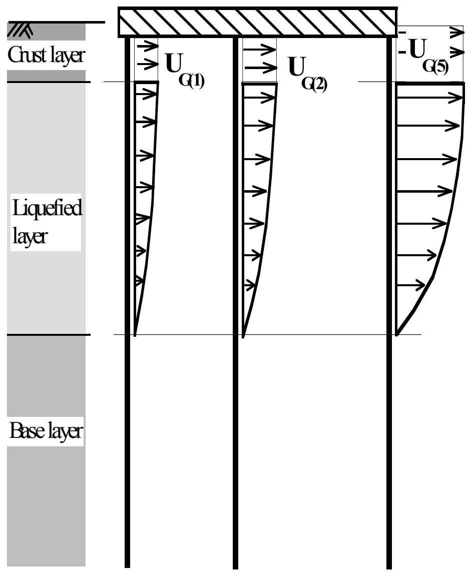

Figure 14: ***Non-uniform distribution of lateral spreading displacements acting on pile a group in the waterfront area.***

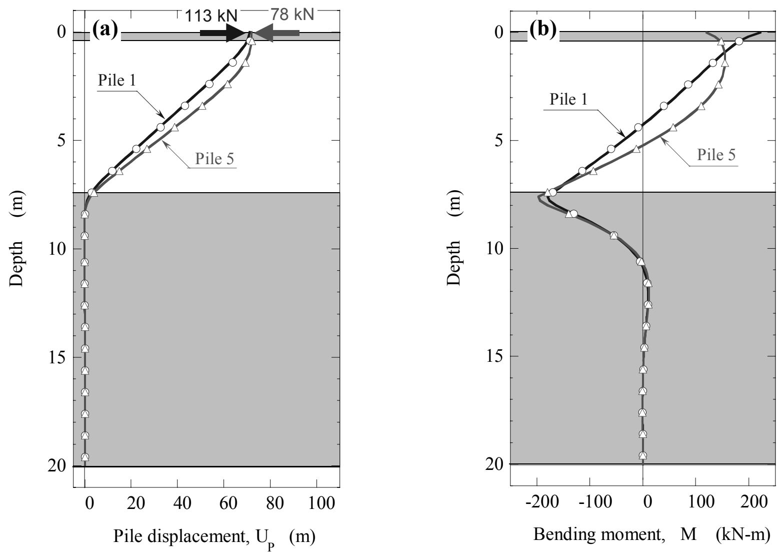

Figure 15: ***Illustration of cross-interaction effects on end piles subjected to different ground displacements (small ground displacement acting on Pile 1; large ground displacement acting on Pile 5): (a) Pile displacements, and (b) Bending moments.***

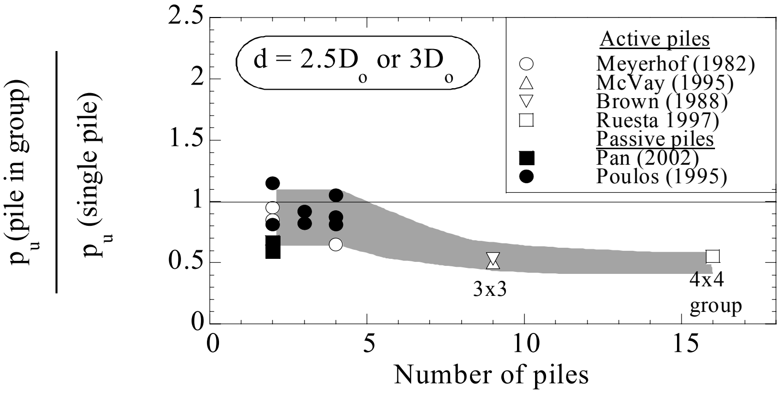

Figure 16: ***Reduction of lateral soil pressure due to pile-group effects.***

SUMMARIZED PROCEDURE FOR PSEUDO-STATIC ANALYSIS

The proposed practical procedure for preliminary assessment and design of piles subjected to lateral spreading can be summarized in the following steps:

1. A simplified three-layer (or multilayer) model is first determined for the soil deposit, where the liquefied layer is sandwiched between a non-liquefied crust layer at the ground surface and a non-liquefied base layer. The water table may be used to define the thickness of the non-liquefiable crust layer at the ground surface. A single pile model with a nonlinear moment-curvature relationship is adopted for the analysis.
2. The magnitude of lateral spreading displacement can be estimated using empirical relationships for displacements of lateral spreads. In view of the uncertainties involved in the assessment of these displacements, a range of values for the ground displacement needs to be considered in the analyses. It seems practical to assume a cosine distribution for the ground displacement throughout the depth of the liquefied layer and to adopt that the surface layer will move together with the top of the liquefied soil.
3. The initial stiffness in all p-δ relationships can be defined based on empirical correlations between the subgrade reaction coefficient and SPT blow count or elastic moduli.
4. The initial stiffness should then be degraded in order to account for the effects of nonlinearity and large relative displacements that are required to fully mobilize the lateral soil pressure on the pile. Stiffness degradation of liquefied soils is generally in the range between 1/50 and 1/10 for cyclic liquefaction and 1/1000 to 1/50 for lateral spreading.
5. Ultimate lateral pressure from the crust layer can be approximated as being 4.5 times the Rankine passive pressure. On the other hand, empirical relations for the residual strength can be used in the evaluation of the ultimate lateral pressure from the liquefied soil on the pile.
6. A static analysis in which the pile is subjected to the ground displacement defined in step 2 is performed and pile displacements and bending moments are computed.
7. The analysis should be repeated while parametrically varying the values of key model parameters such as the magnitude of applied ground displacement, stiffness and strength of liquefied soil and multipliers used for the ultimate soil pressure.
8. Eventually, pile group effects should be considered including cross interaction among the piles within the group through the pile-cap-pile system, and effects on key parameters controlling the pile responses.

For analysis of the cyclic phase of the pile response, key requirement is to concurrently consider the effects of ground displacements and inertial loads from the superstructure, and to properly consider the characteristics of liquefaction and subsequent ground displacements during the cyclic phase of the response. Cyclic ground displacements can be evaluated based on empirical models for the maximum cyclic shear strain of liquefied soils while the inertial load from the superstructure can be estimated using the peak ground acceleration in conjunction with recently proposed empirical criteria (e.g., [1], [3]).

CONCLUSIONS

Lateral ground displacements of liquefied soils can be very large and damaging for piles both during the intense shaking or cyclic phase of loading and especially during the post-liquefaction lateral spreading phase. Since properties of liquefied soils and loads on piles can be remarkably different during the cyclic phase and subsequent spreading phase, it is necessary to separately consider these two phases in the simplified analysis of piles.

When evaluating the response of piles subjected to lateral spreading, the uncertainties associated with the spreading of liquefied soils need to be carefully considered. Empirical correlations can be used in the evaluation of stiffness and strength of liquefied soils though one should acknowledge the large scatter in such correlations and the need for parametric studies. The lateral load from a non-liquefiable crust layer at the ground surface may often be the critical load for the integrity of piles subjected to lateral spreading, and therefore, special attention needs to be given to the modelling of the surface layer and its effects on the pile response. Cross-interaction effects may be significant for pile foundations especially in areas where significant spatial variability of ground displacements occurs. Effects of group interaction on key parameters controlling the pile response need to be considered in perhaps reducing the severity of the ground movement effects. Based on these premises, a practical procedure for preliminary assessment and design of piles subjected to lateral spreading has been proposed.

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