Design charts for single piles under lateral spreading of liquefied soil

Concevez les diagrammes pour les piles simples sous la propagation latérale du sol liquéfié

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ABSTRACT
When soil liquefaction occurs in slightly sloping ground or near free-face topographic irregularities large horizontal displacements may occur due to the lateral spreading of the liquefied ground. This kind of ground failure may cause extensive damage to pile-supported structures as witnessed in several recent earthquakes (Chi-Chi 1999, Kobe 1995, etc). A systematic analysis of the detrimental effect of lateral spreading requires a sophisticated 3-D numerical analysis. Still, there is need for preliminary estimation of the response of deep foundations based on readily available data, such as the geometric and the mechanical characteristics of the foundation and the maximum anticipated displacement at the ground surface. For this purpose, a set of over 200 parametric numerical analyses were performed, with the pseudo static P-y method, in order to analyze the basic parameters affecting the pile behavior. The results of the parametric analysis have been consequently combined into design charts for the computation of the maximum developed pile head displacement and moment.

RÉSUMÉ
Quand la liquéfaction de sol se produit en inclinant légèrement des irrégularités topographiques de libre-visage moulu ou proche les grands déplacements horizontaux peuvent se produire en raison de la propagation latérale de la terre liquéfiée. Ce genre d'échec au sol peut causer des dommages importants aux structures pile-soutenues comme été témoin dans plusieurs tremblements de terre récents (1999 Chi-Chi, Kobe 1995, etc.). Une analyse systématique de l'effet néfaste de la propagation latérale exige une analyse numérique à trois dimensions sophistiquée. Toujours, il y a besoin d'évaluation préliminaire de la réponse des bases profondes basées sur des données facilement disponibles, telles que les caractéristiques géométriques et mécaniques de la base et du déplacement prévu maximum sur la surface au sol. À cette fin, un ensemble de plus de 200 analyses numériques paramétriques ont été exécutés, avec la pseudo méthode du PY de charge statique, afin d'analyser les paramètres de base affectant le comportement de pile. Les résultats de l'analyse paramétrique ont été par conséquent combinés dans des diagrammes de conception pour le calcul du déplacement et du moment développés maximum de tête de pile.

Keywords : liquefaction, lateral spreading, single piles, numerical investigation

1 INTRODUCTION

One of the most damaging effects of earthquake-induced soil liquefaction is the lateral spreading of soils, where large areas of ground move laterally to lengths ranging from some centimeters to a few meters. This phenomenon may occur in the case of even small surface inclination (e.g. 2÷4%) or small topographic irregularities (e.g. 2÷3m) such as those near river and lake banks.

In such cases, the kinematic interaction of single piles and pile groups with the lateral spreading ground may induce significant additional residual horizontal loads and bending moments to the pile, which cannot be predicted by common design methods for superstructure loading.

2 PSEUDO-STATIC PREDICTION METHODS

The detailed analysis of piles against lateral spreading is a rather complicated soil–structure interaction problem which, strictly speaking, requires a sophisticated numerical simulation, well beyond the limits of common applications. Thus, for simplified computations, a number of pseudo-static methodologies have been developed, where the loads or displacements applied by the laterally spreading ground are being estimated independently and subsequently applied as external loads to the pile. Existing pseudo-static methodologies may be divided in two categories:

(a) The ***P-y method***, which relies upon the substitution of the ground with "Winkler type" springs that are governed by a non-linear load-displacement (P-y) law. According to this methodology an independent estimation of the ground displacement is made and the resulting displacements are applied to the base of the springs in order to evaluate the pile deflection and the corresponding shear forces and moments (e.g. Tokimatsu 1999, Boulanger et al 2003).

(b) The ***limit equilibrium method***, which is based on a pseudo-static estimation of the ultimate pressure that the laterally spreading ground applies to the pile. Pile displacements and bending moments can be consequently evaluated (e.g. JRA 1996, Dobry et al 2003) from beam theory.

Recently, Ashford & Juirnarongrit (2004) concluded that the P-y method is the most reliable, after comparing the two most commonly used limit equilibrium methods (JRA, 1996 and Dobry et al., 2003) with a simple P-y method that used the curves proposed from Reese et al. (1974) for sands, degraded with a factor $\beta = 0.1$ in order to take into account the soil liquefaction. Bhattacharya et al. (2003) also concluded that the limit equilibrium method of JRA (1996) is systematically non-conservative. Thus, on the ground of these independent findings, the P-y method has been chosen to derive the design charts in this paper.

More specifically, the method proposed by Branderberg (2002) has been selected, according to which the P-y curves of API (1995) for the non-liquefied sands should be used, after

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being degraded with a loading factor β. This factor represents the effect of liquefaction on the mechanical characteristics (soil strength and deformation) of the natural soil and can be computed according to Table 1, in terms of the corrected blow count of the Standard Penetration Test $(N_1)_{60\text{-}CS}$.

The aforementioned methodology has been chosen among seven (7) compatible methodologies (Ishihara & Cubrinovski, 1998, Cubrinovski et al., 2006, Rollins et al., 2005 & 2007, Tokimatsu, 1999, High Pressure Gas Safety Institute of Japan, 2000, Railway Technical Research Institute of Japan, 1999, and Matlock, 1970) following an extensive evaluation through comparison to three centrifuge experiments (Abdoun 1998) and one large shaking table experiment (Cubrinovski et al. 2004).

Table 1. Proposed degradation factors β after Branderberg (2000)

$(N_1)_{60\text{-}CS}$	β
<8	0 to 0.1
8-16	0.1 to 0.2
16-24	0.2 to 0.3
>24	0.3 to 0.5

3 PARAMETRIC ANALYSES

The numerical analyses have been performed with the help of the finite elements program NASTRAN (MacNeal-Schwendler Corp. 1994). Simulation of the liquefied soil layers was based on the P-y methodology outlined in the previous paragraph. The non-liquefied soil layers have been simulated with the P-y curves proposed by API (1995, 2002) without the use of a degradation factor. It should be mentioned that, as long as the non-liquefiable base layer does not fail, the exact P-y curve used for its simulation does not affect significantly the results, as its stiffness is almost 100 times larger than that of the liquefied soil above it.

Based on a previous study of the lateral spreading phenomenon (Valsamis et al., 2007), the variation with depth of the lateral displacements of the liquefied soil was assumed as a quarter sine, with the maximum displacement developing near the top of the layer and zero displacement at the bottom of the layer. On the other hand, the displacement of the non-liquefied soil layers was assumed to remain constant with the depth.

In total, 162 such parametric analyses have been performed, which refer in three basic combinations of pile and soil profiles (Figure 1):

- "2-layered geometry", where a single pile with free head conditions rests inside a uniform liquefied soil layer that overlies a non-liquefiable soil stratum.
- "3-layered geometry", that differs from the previous case due to the existence of a non-liquefiable soil crust
- "Fixed pile head" case, which differs from the 2-layered case due to the restraining of the pile head movement, in accordance with real cases where the existence of a superstructure prevents the pile head movement.

This categorization is justified on the grounds that any possible constraints on the free pile head displacement and rotation, either due to the non-liquefiable soil crust or a superstructure, proved to be among the the most important factors controlling the pile response. Note that, Ishihara & Cubrinovski (1998), Brandenberg (2002), Rollins et al. (2005) have also reached similar conclusions for the effect of the pile head constraint enforced by a non-liquefiable soil crust.

Sixty six (66) parametric analyses have been performed for the 2-layerd case and the following pile & soil input parameters:

- degradation factor β = 0.05 to 0.4
- soil friction angle φ = 32 to 42° (D_r = 35~90%)
- thickness of liquefied soil layer H_{liq} = 6 to 10m
- Pile Elastic modulus E = 30 to 210 GPa
- Pile diameter D = 0.15m to 0.6m,
- Pile stiffness EI = 16 to 1336 MN·m², and
- Maximum ground surface displacement D_h = 0.125m to 1.20m.

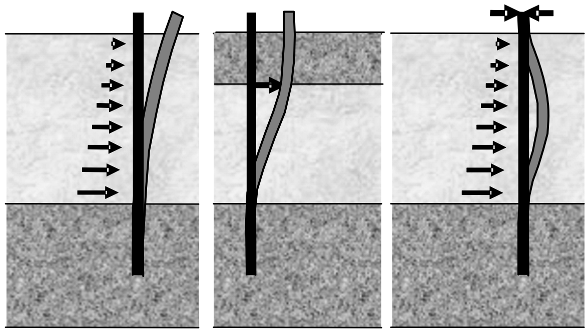

Figure 1. Static models for the (a) 2-layered, (b) 3-layered and (c) fixed pile head cases

Fifty (50) more parametric analyses have been performed for the 3-layered case, with the thickness of the non-liquefied soil crust ranging from 1 to 4m and the previously mentioned range of the remaining input parameters. Finally, another forty six (46) parametric analyses were performed for the fixed pile head case and the above mentioned range of parameters.

4 DESIGN CHARTS

Pile design against lateral spreading must assure that, following the seismic excitation:

(a) no structural failure of the pile has occurred (no development of plastic hinges at any depth), and

(b) no performance failure of the superstructure should be encountered due to excessive superstructure displacements.

To check against these criteria, both the maximum developing moment and the maximum displacement of the pile head are needed.

The depth of the maximum bending moment is in general variable. For the cases considered in this article (Figure 1), it is known before-hand that maximum moments develop at the interface between the liquefied soil layer and the non-liquefied base layer. Similarly, it is known before hand that the maximum pile displacement develop at the pile head, for the 2- and the 3-layered cases, and near the mid-depth of the liquefiable soil layer for the fixed pile head case. For these reasons, the statistical analysis of the parametric analyses results has been focused upon the magnitude of those two design parameters and not upon the respective location along the pile.

It should also be mentioned that the statistical processing was not "blind", e.g. based only on some algorithm that minimizes the error of the empirical predictions. On the contrary, a general form of the prediction relations was initially obtained based on analytical solutions of the static models presented in Figures 1a, 1b and 1c and subsequently the statistical processing was used to calibrate the general relations against the results of the parametric analyses.

Figures 2a and 2b present the proposed design charts for the maximum pile displacement and the associated bending moment for the <u>2-layered</u> soil profile. Alternatively, the pile head displacement D_{pile}(m) may be computed from Figure 2a and the respective maximum bending moment M_{max}(kN/m) may be subsequently estimated as:

$$M_{\max} = 2.2 \frac{EID_{pile}}{H_{liq}^2} \tag{1}$$

where H_{liq}(m) is the liquefied soil layer thickness and EI(kN/m²) is the pile stiffness.

Observe that the relation in Figure 2a is strongly non-linear. This is due to the fact that the Winkler springs representing the soil are elasto-plastic and thus, after a certain soil displacement,

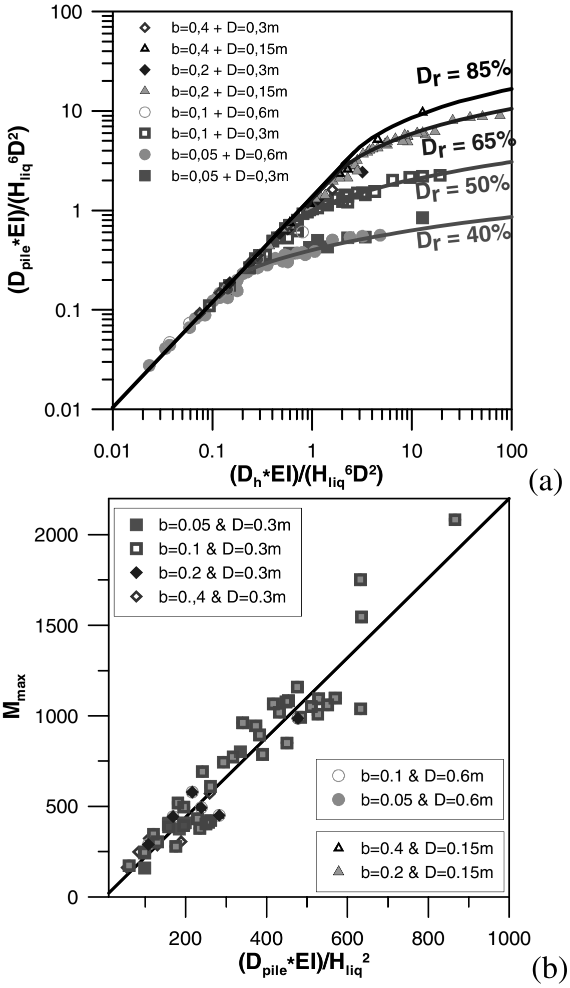

Figure 2. Design charts (a) for the maximum pile displacement and (b) for the maximum developing bending moment of the pile, for the 2-layered soil profile

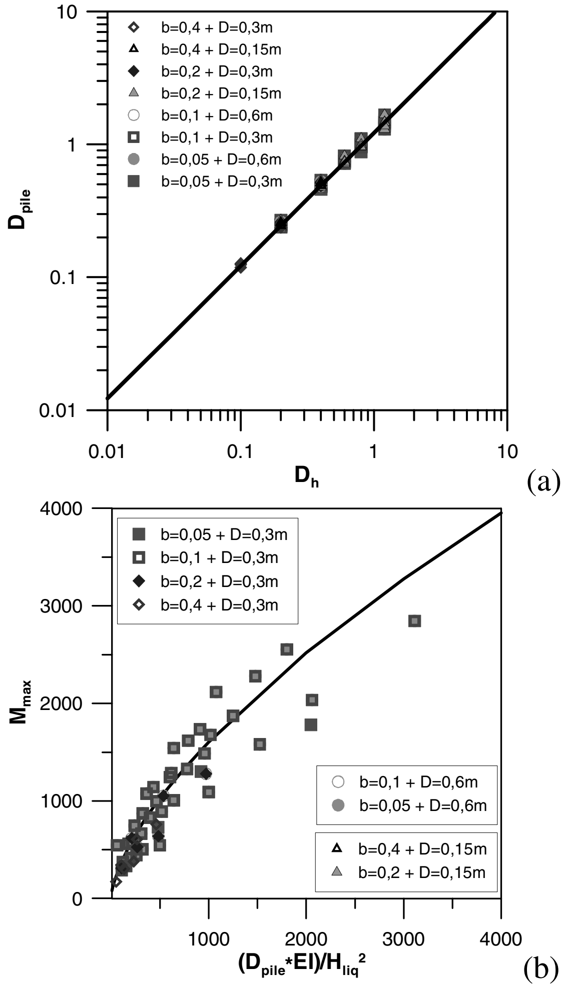

Figure 3. Design charts (a) for the maximum pile displacement and (b) for the maximum developing bending moment on the pile, for the 3-layered soil profile

the loads due to the lateral movement of the soil remain constant. This elastoplastic response of the soil springs is the main reason why the derivation of a simple analytical expression for the pile displacement was not possible. Moreover, note that the correlations of Figure 2a are not dimensionless and thus they should always be used in conjunction with the international system unit SI (kN, m).

The design charts for the 3-layered soil profiles are shown in Figures 3a and 3b. Observe that the pile head displacement follows systematically the non-liquefied soil crust displacement. This observation has been also confirmed from centrifuge experiments (Abdoun, 1999) which show that pile head displacements are only slightly larger than soil surface displacements. In this case, it was possible to develop simplified analytical relations, both for the pile head displacement and the developing bending moments, namely:

$$D_{pile} = 1.22 \cdot D_h \tag{2}$$

$$M_{\max} = 18\left(\frac{EID_{pile}}{H_{liq}^2}\right)^{0.65} \tag{3}$$

Finally, Figures 4a and 4b present the design charts for the maximum pile displacement and bending moment respectively, in the case of the fixed pile head. In this case also, it was possible to establish simple to use analytical relations for the estimation of the above mentioned design parameters, namely:

$$D_{pile} = H_{liq}^{12} D_h^{0.3} \frac{\beta^2 D^2}{(EI)^2} \tag{4}$$

$$M_{\max} = 18 \frac{EID_{pile}}{H_{liq}^2} \tag{5}$$

where β is the degradation factor for the soil strength due to the liquefaction which can be taken from Table 1.

5 CONCLUDING REMARKS

In the previous paragraphs, diagrams and relations were presented for the approximate evaluation of the maximum displacement and bending moment of the pile due to liquefaction-induced lateral spreading. The charts concern three different combinations of pile and ground conditions, often encountered in practice. The proposed design charts and relations should be used with the following limitations:

(a) They were derived pseudo-statically, taking only into account the final displacement of the ground, at the end of shaking. Any effects of the superstructure inertia are ignored.

(b) The expected free-field maximum ground surface displacement should be computed independently, based on the (many) available empirical relations which are published in the literature (e.g. Hamada, 1999, Youd et al, 2002).

(c) All the above mentioned charts and relations, and more specifically those concerning the 2-layered soil profile case,

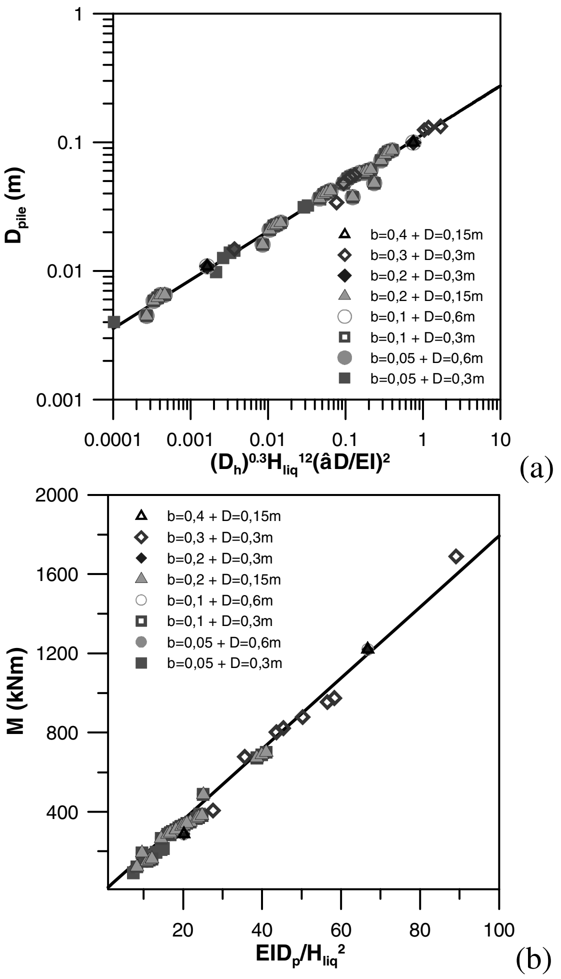

Figure 4. Design charts for the (a) maximum developed pile displacement and (b) the maximum developing bending moment on the pile, for the fixed pile head case

should be applied only when the soil has the capability to "flow" freely around the pile under investigation. In all other cases (e.g. small distance between the piles, sheet-pile wall, etc) they may lead to non-conservative predictions of the pile displacement and bending moment.

(d) It has been assumed that the pile has been adequately embedded to the non-liquefiable base soil layer so as to guarantee fixed bottom conditions during lateral ground spreading. When the pile has not been driven adequately to the bottom (non-liquefiable) soil layer, there is the possibility of pile extortion or significant pile base rotation which results to larger displacements for the pile head and smaller developing moments.

Finally, note that the results presented herein have been also evaluated against numerical 2-D and 3-D numerical analyses, where the coupled pile-laterally spreading ground response has been simulated with the aid an effective stress elastoplastic analyses (Valsamis 2009). These comparisons, not shown here due to length limitations, show that for the 3-layered geometry and the 2-layered geometry with fixed pile head the simplified P-y methodology gives accurate results. However, for the 2-layered geometry where the pile head can move freely, the P-y methodology slightly overpredicts the expected pile head displacements and thus the maximum developed moments which are calculated from them.

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