Lateral Reaction of Piles Using Free-Field Response

SRéaction latérale de pieux utilisant la réponse de site

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ABSTRACT

This paper proposes a new method that uses free-field measurements of vertical/horizontal displacements and excess pore pressures to calculate the lateral loads on the piles due to embankment induced soil movements. The variation of lateral earth pressure K and the horizontal effective stresses are calculated and a constitutive relationship is proposed between lateral earth pressure coefficient and modulus of horizontal subgrade reaction. Using this relation the stiffness degradation of the soil with deformation is defined for the cases where piles are installed before or after the embankment construction.

RÉSUMÉ

Cet article propose une nouvelle méthode qui emploie des mesures de déplacements verticaux/horizontaux de terrain et d'excès de pression de pore à fin de calculer les charges latérales sur les pieux dus aux mouvements de la terre induit par des remblais. La variation de la pression latérale du sol K et les efforts efficaces horizontaux sont calculés et un rapport constitutif est proposé entre le coefficient de pression latéral du sol et le module de réaction horizontal du sol de fondation. Utilisant cette relation, la dégradation de rigidité du sol avec la déformation est définie pour les cas où des pieux sont installés avant ou après la construction de remblais.

Keywords : laterally loaded piles, embankments, soil-pile interaction, soft clays

1 INTRODUCTION

Piled bridge abutments for approach embankments have been used extensively in practice as an integral part of the infrastructure projects. The required function of the piles in these cases is to resist embankment induced soil movements. Piles constructed to resist soil displacements from the moving slopes perform their function with the same basic load transfer mechanism. Challenging issues in the design of these structures mostly arise due to the determination of the magnitude and time dependency of the load transfer between the soil and the structure when piles are constructed in soft soils. Researchers devote their concern mainly on the estimation of the pressures acting on the pile from the moving soil and lateral resistance that is developed in the responding soil (Springman (1989), Chow (1996), Chen and Poulos (1997), Goh et al. (1997), Stewart et al. (1994), Ellis and Springman (2001) and many others). The analytical approaches have been classified in two main categories by Stewart et al. (1994); pressure based methods and displacement based methods. Displacement based methods are described as the methods where the distribution of free-field lateral soil displacement is input and the resulting pile deflection and bending moment distribution are calculated.

The free-field soil behaviour carries invaluable implications of the mechanism that is developed in the resisting soil zones. This argument holds true both in the absence and presence of the piles. If there are piles in the system the applied load is taken by the soil-pile composite system and shared among these two components with respect to their relative rigidities. Fig.1 demonstrates these two cases of soil response against the moving soft soil under the embankment loads. As seen the regions outside the toe in both cases are passively loaded and these are defined as the zones of interest for the solution of the problem in this paper. In both cases the works done in passively loaded zones should be the same since they are supposed to dissipate the same amount of energy applied by the acting sides due to the same construction work.

This paper proposes a comprehensive method that uses free-field measurements of vertical/horizontal displacements and excess pore pressures provided from in-situ to calculate soil response. The displacements that are in the region beneath and away from the toe of the embankment are of concern. Using the free-field deformations for this zone, the variation of lateral earth pressure K and thus the horizontal effective stresses can be calculated provided that excess pore water pressures were also known. A constitutive relation is proposed between lateral earth pressure coefficient and modulus of horizontal subgrade reaction. Using this relation the stiffness degradation of the soil with deformation can be defined for the cases where piles are installed before or after the embankment construction.

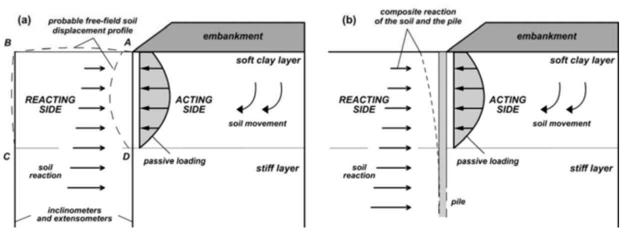

Figure 1. Soil movements beneath an embankment (a) free-field (b) with piles

2 THE CUBZAC-LES-PONTS TEST SITE

In-situ measurements of Cubzac-les-Ponts test embankment were used to validate the proposed method. This heavily instrumented test embankment was constructed without piles and a complete set of displacements and pore pressures for free-field case were provided (Magnan et al. 1983). The upper layer of the soil profile was soft to very soft clay layers with a thickness of 9.0 meters, followed by sand-gravel and marn layers. An instrumentation system as seen in Fig. 2 with inclinometers, extensometers and piezometers made it possible to monitor the deformations and excess pore water pressures in time. The instrumentation did not reach to the stiff layers. Trial embankment construction took place in seven days with a final height of 2.3 meters.

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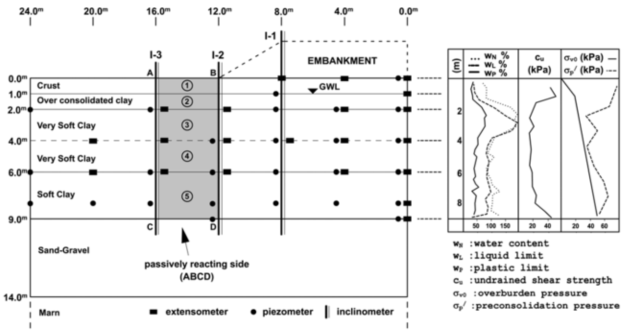

Figure 2. Cubzac-les-Ponts Test Embankment (Magnan et al. 1983; Oztoprak and Cinicioglu 2005)

3 LATERAL EARTH PRESSURE COEFFICIENT, K

Zhang et al. (1998) showed the dependency of lateral earth pressure coefficient, K on strain increment ratio, R_ε. The continuous relationship between K-R_ε (Fig. 3), that is constituted by Zhang et al. (1998) makes it possible to calculate the values of K for any intermediate or limit stress state, provided that the minor and major principal strains for these states are available. The strain increment ratio R_ε is defined as the ratio between the minor and major principal strains. The continuous variation of the earth pressure coefficient K with strain increment ratio R_ε in Fig. 3 is also defined in mathematical form with equations (1) and (2). ϕ' is the angle of internal friction in terms of effective stress.

$$K = (1 - \sin\phi')/(1 - \sin\phi'.R_\varepsilon) \rightarrow \Delta\varepsilon_1 = \Delta\varepsilon_{vertical} \quad (1)$$

$$K = (1 - \sin\phi'.R_\varepsilon)/(1 - \sin\phi') \rightarrow \Delta\varepsilon_1 = \Delta\varepsilon_{lateral} \quad (2)$$

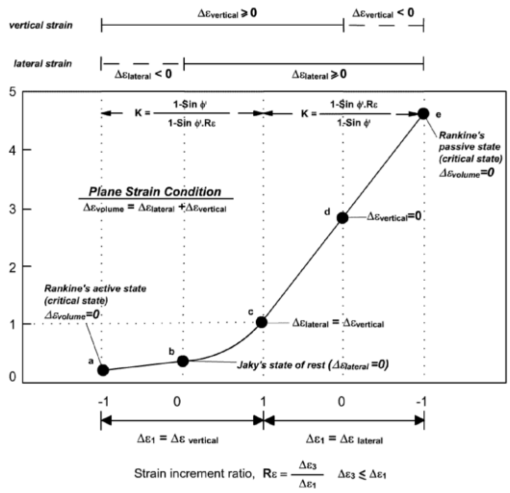

Figure 3. Variation of K-R_ε (Zhang et al. 1998)

Eqns. (1) and (2) were used with the Cubzac-les-Ponts test data and the lateral earth pressure coefficients mobilized during the construction and post-construction stages of the fill are calculated for each shaded zone in Fig. 2, numbered from 1 to 5. As seen in Fig.4 after the embankment construction (consolidation or post-construction period) the lateral movements or flows in the soft clay layers are dominant, specifically for zones 2-3-4, thus these zones are near to passive limit stress states. However, zone 1 which is near to the soil surface and likely to heave and the lower zone 5 which is probably less affected from lateral movements are close to at-rest stress states during post-construction period.

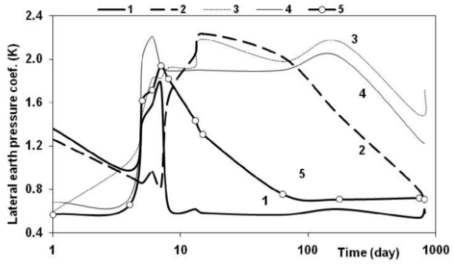

Figure 4. Variation of K during pre/post-construction states of the fill

4 DEGRADATION OF THE SOIL STIFFNESS WITH DEFLECTION

In Fig. 5, two different cases of embankment construction on soft soils is given. Fig. 5a matches to the case without piles and Fig. 5b gives the basic features of the behaviour with piles. In both of the cases, the shaded regions outside the toe of the embankment are defined as the zones of interest for the solution of the problem. The works done in these shaded regions should be the same provided that the embankment geometry and loading conditions are identical, thus the same amount of energy imparted by the same construction work. The first case can be analyzed by Zhang et al. (1998) to calculate the horizontal passive resistance of the soil $p=\sigma'_h=K.\sigma'_v$ mobilized during/after embankment construction. The soil resistance in the piled case can be defined with Winkler soil model as $p= k_h.y$, that is given in Fig. 5b. These two approximations of the resisting soil behavior should be the same to satisfy the condition for the same amount of energy dissipation. This condition is fulfilled with equation (4) which links real field response that is interpreted in terms of $K.\sigma'_v$ to the modeling equation of $k_h.y$. In the above expressions K is the lateral earth pressure coefficient, σ'_v and σ'_h is the vertical and horizontal effective pressures, y is the deflection of the structure and k_h is the modulus of horizontal subgrade reaction. Eqns. (1) and (2) can be rewritten as (4) and (5) to express k_h in terms of fundamental soil parameters (Kelesoglu 2006, Cinicioglu and Kelesoglu, 2006).

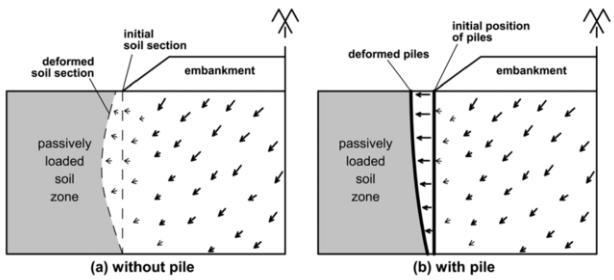

Figure 5. Passively Loaded Zones for the Conditions (a) without piles; (b) with piles.

$$k_h.y = K.\sigma'_v \rightarrow k_h = [K/y]\sigma'_v \quad (3)$$

$$k_h = [(1-\sin\phi')/(1-\sin\phi'.R_\varepsilon).y].\sigma'_v \quad \text{for} \quad \Delta\varepsilon_1 = \Delta\varepsilon_{vertical} \quad (4)$$

$$k_h = [(1-\sin\phi'.R_\varepsilon)/(1-\sin\phi').y].\sigma'_v \quad \text{for} \quad \Delta\varepsilon_1 = \Delta\varepsilon_{lateral} \quad (5)$$

In eqns. (4) and (5), k_h is defined as a function of ϕ', R_ε, y and σ'_v. Maximum internal friction angle, ϕ' stands as the strength parameter, R_ε covers the indications of structural changes in the soil, σ'_v reflects the influence of depth. Using these equations for each shaded zone given in Fig. 2, field degradation curves of Cubzac-les-Ponts test embankment were calculated. Degradation curves presented in Fig. 6 and for the simplicity of the subsequent calculations these curves are simplified to the best-fit curves, the equations of which can then be found.

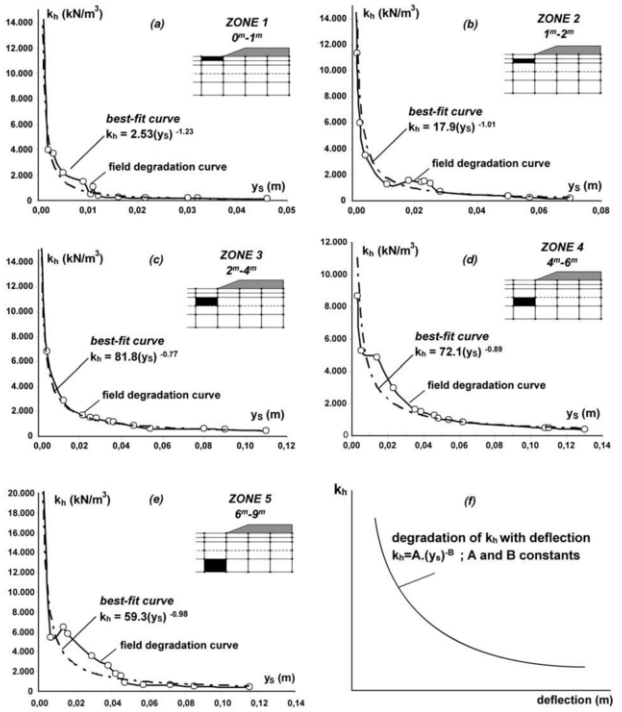

Figure 6. Field and best-fit degradation curves of k_h with deflection.

5 PILE DESIGN WITH DEGRADATION CURVES

Lateral loads on the piles, induced by mobilized lateral soil movements due to embankment construction, is to be shared between the resisting component (i.e. piles and soil) when there are piles in the system. The amount of the load exerted to the soil or the pile is directly related to the stiffness values of these resisting components. The relationship defining the proportioning of the applied load with respect to relative stiffness values of each component was used by several researchers (Chow 1996, Goh et al. 1997) and is given as

$$[K_p + K_s]\{\Delta y_p\} = [K_s]\{\Delta y_s\} \tag{6}$$

where $\{\Delta y_p\}$ is the incremental deflection of the pile and $\{\Delta y_S\}$is the incremental free-field soil movement. On the other hand, $[K_P]$ and $[K_S]$ are the pile and soil stiffness matrices. Pile stiffness matrix, $[K_P]$ is formed by pile geometry and material properties, whereas soil stiffness matrix, $[K_S]$ is constituted by the moduli of horizontal subgrade reaction values calculated from the best-fit curve equations of the stiffness degradation curves given in Fig. 6.

The use of the response curve to solve interaction properties is demonstrated in Fig. 7. The response curve is an envelope to the whole behavior either with or without piles. Any incremental stage can be considered by the area under the curve extending from the deformation state corresponding to the start of that stage and the deformation state corresponding to the end of it. If the piles are constructed after the embankment, then in this case the displacements in the soil prior to the piles must be considered as mobilized and dissipated in the soil, hence they provide no additional load on the piles. This case can be calculated, easily, with the proposed method in the manner of a regular stage, considering the preceding soil deformations as the start of the current stage. As a result of this property, it is possible to postulate that the proposed method is capable of analyzing the behavior for all possible cases: either when piles are constructed before, during or after embankment construction.

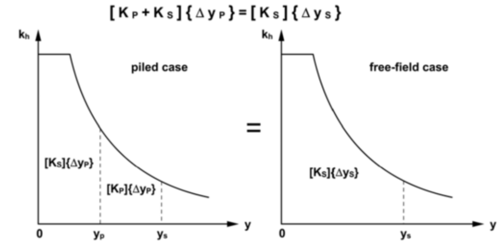

Figure 7. Use of same soil response envelope to find (left figure) load sharing mechanism between the pile and the soil (right figure) the applied pressure at any deflection state, y_s

6 RESULTS

Results of the proposed method are compared with finite element analysis carried out with Soft Soil Creep-SSC soil model in Plaxis and Modified Cam Clay-MCC soil model in Crisp. Soil parameters are taken from the detailed site and laboratory research of Magnan et al. (1983) and also from Wood (1990) as given in Table 1. The undeformed meshes of both models are given in Fig. 8.

Table 1. Soil Parameters Used in the FEM Analyses (Magnan et al. 1983)

Material	κ	λ	e_{CS}	M	k_X	k_Y
Crust	0.017	0.12	1.0	1.29	$4.6\text{x}10^{-9}$	$9.0\text{x}10^{-9}$
OC clay	0.022	0.53	2.6	1.16	$1.4\text{x}10^{-9}$	$1.2\text{x}10^{-9}$
VS clay	0.085	0.75	3.2	1.03	$2.6\text{x}10^{-9}$	$7.0\text{x}10^{-10}$
VS clay	0.048	0.53	2.25	1.03	$1.5\text{x}10^{-9}$	$1.0\text{x}10^{-9}$
Soft clay	0.043	0.52	2.3	1.03	$1.5\text{x}10^{-9}$	$1.0\text{x}10^{-9}$

κ : swelling index, λ : compression index, e_{cs} : critical void ratio, M : slope of the critical state line, k_X, k_Y : coefficient of vertical/horizontal permeability [m/s]

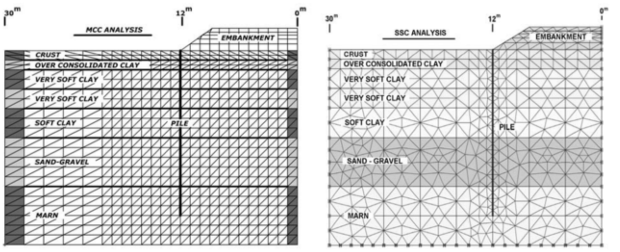

Figure 8. Mesh and stratification of the FE models in MCC and SSC

From the degradation curves given in Fig.6 that are obtained using the proposed method, it is clear that once the steep initial part of the degradation curve has been exceeded the influence of degradation diminishes.

As consolidation proceeds (Fig. 9) the deviation between the maximum bending moment values estimated with the proposed method and the other applied FE methods (Crisp and Plaxis) increases. The estimations of Crisp and Plaxis are close to each other although Crisp gave greater values. For the case where piles are constructed after the embankment (end of 7^{th} day), the bending moment values reduce significantly. The ratio of bending moments on piles for the case where piles are constructed after the embankment (Case II) to the case where piles are constructed before embankment (Case I) is 14%, 28% and 50% for the days 15, 174 and 817, respectively.

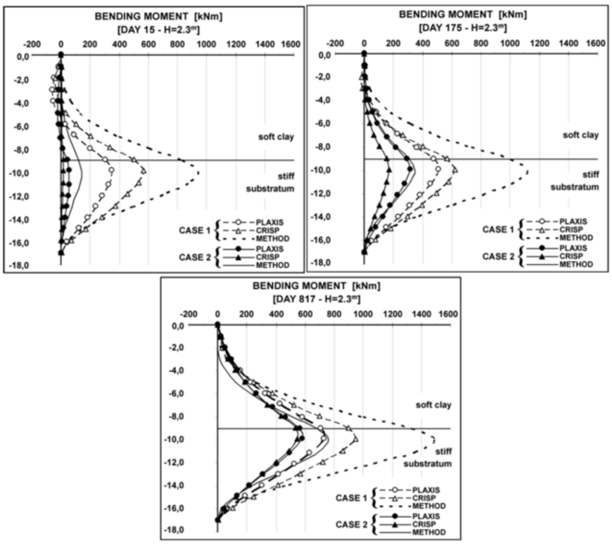

Figure 9. Comparison of the pile bending moments for Case I and II

Lateral pile head deflections found with the applied methods are shown in Figs. 10 and 11 for both Cases I and II. For Case I, it can be stated that Plaxis and Crisp calculate smaller deflection states than the proposed method during consolidation periods following the completion of the construction of the embankment. During construction, calculated pile head deflections with all the applied methods are similar especially between the 5th and 6th days. There is a steady increase in the pile head deflection values calculated with the proposed method all the way through the construction and consolidation stages. For Case II all the methods yield similar results. It is possible to state that the displacements are reduced about 50% in Case II compared to the results of Case I.

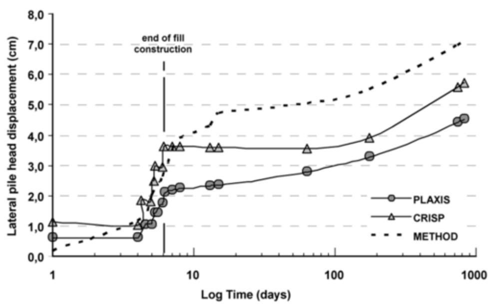

Figure 10. Pile head displacements with time – Case I

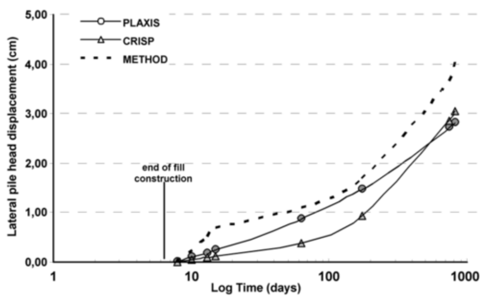

Figure 11. Pile head displacements with time – Case II

The findings of the proposed method can continuously be controlled against the failing states to provide a design safety factor. The bearing capacity relationship by Fleming et al. (1992) was used to calculate the ultimate bearing capacity of $\phi \neq 0$ soils under the thrust of laterally loaded piles where K_P is the passive lateral earth pressure coefficient, γ is the effective unit weight, z is the considered depth and D is the pile diameter. Results of this control for the bearing capacity of the soil under the thrust of the pile are given in Fig. 12.

$$p_{ult} = K_p^2 \gamma z D \tag{7}$$

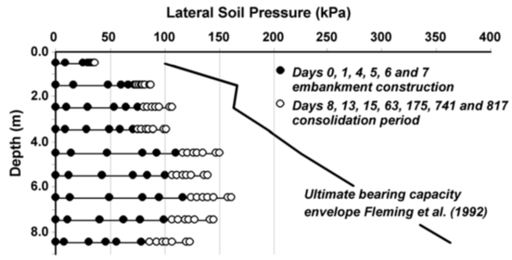

Figure 12. Ultimate Lateral Bearing Capacity of the Soft Soil Layers

7 CONCLUSIONS

A new theoretical approach based on the soil response curves calculated from field measurements has been introduced to solve the problem of laterally loaded piles subjected to soil movements induced by the construction of a nearby embankment. Free-field measurements such as vertical and horizontal deformations and excess pore pressures beneath and away from the toe of the embankments are used as input. To validate the proposed method field measurements of Cubzac-les-Ponts test embankment is used. Results of the proposed method are compared with two different finite element softwares and soil models. The comparisons of the results obtained with the application of these different approaches indicate the strength of the proposed method. This underlying mechanism is used to find loads, pile deflections and bending moments on the piles for the both cases where the piles are constructed before and after the embankment.

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