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CERTIFICATION COURSE

EARTHQUAKE RESISTANT DESIGN OF FOUNDATIONS

Lecture 26: Design of Piles in Liquefiable Soils -2

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Pile Foundations

Module -3: Chapters

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5. Combined Pile Raft Foundation (CPRF)
6. Design of Piles in Liquefiable Soils



Acknowledgement - Reference

Most of the contents of this lecture are taken from following literature.
This is gratefully acknowledged:

Madabhushi G., Knappett J. and Haigh S., (2010). Design of Pile Foundations in Liquefiable Soils. Imperial College press.

Design of Piles in Liquefiable Soils - 2

- Inertial Loading
- Kinematic Loading
- Kinematic Loading in Laterally Spreading Soil
- Limit Equilibrium Analysis in laterally spreading soils
- Limit Equilibrium Analysis with nonliquefied crust
 - Stiff clay as a nonliquefiable layer
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Inertial Loading

The loading requirement imposed by seismic event on piles require different geotechnical and structural design of these elements compared with the design of piles under static loading.

Considering the case of pile foundations in level ground comprising a soft, horizontal soil layer in which no soil liquefaction occurs, overlying a stiff, horizontal soil layer. The pile foundation passes through the soft layer and rests on the stiff soil layer as shown in Fig. 2.1.

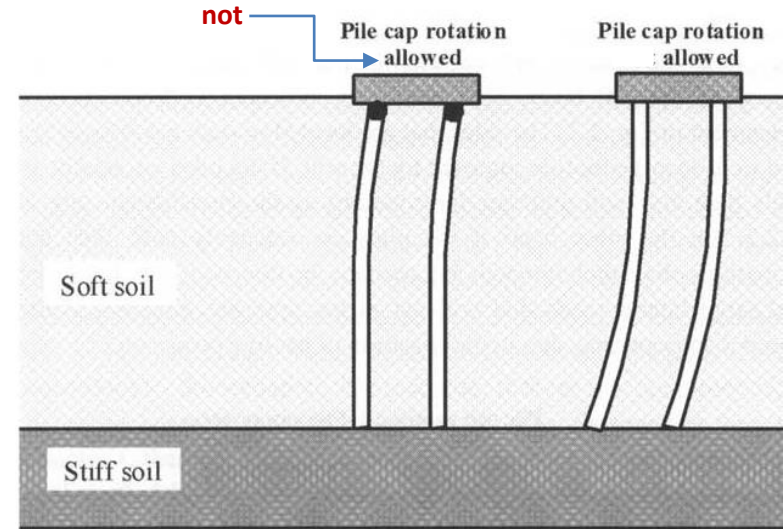


Fig.2.1 inertial interaction between piles and soils

Inertial Loading (Continued...)

The earthquake motion can be transmitted from the stiff soil stratum into the softer layer and this motion can be amplified as it propagates through the softer layer and finally transferred to the piles and the superstructure. This sets up structural vibrations in the superstructure imposing inertial loading on the pile cap.

This inertial load has to be carried by the piles. If the inertial load is large, then piles can suffer significant lateral displacements. Further, depending on the stiffness of the superstructure and the pile cap's bearing capacity, the pile cap can either be prevented from suffering any rotation or some rotation can occur. If the pile cap rotations are prevented, then the piles will undergo elastic bending and in extreme cases plastic hinges can form at the pile/pile cap interface as indicated in Fig. 2.1.

Inertial Loading (Continued...)

On the other hand, if the pile cap can undergo some rotation, then the piles will suffer elastic bending but the pattern of bending is opposite to the previous case (Fig. 2.1). These types of behaviour will be dominated by the stiffness of the shallow soil layers as the lateral inertial load on the piles will only displace the soil in this region. If the shallow layers in this region are soft, this can be accomplished easily. On the other hand, if the shallow layers below the pile cap comprise stiffer/competent soil then the lateral displacement of piles and the rotation of the pile cap will be smaller, but the piles may attract large lateral loads that oppose the inertial loads due to mobilisation of the passive soil pressures in this region.

Clearly 'inertial loading' on the pile is a dynamic load and its peak magnitude would depend on the dynamic characteristics of both piles and the soil layers. In particular the inertial load will depend on the stiffness of the pile relative to that of the surrounding soil.

Kinematic Loading

Earthquake loading differs from other form of environmental and machinery induced cyclic loading because the in-ground motion produce pile loadings in addition to the pile loadings derived from the motion of the supported structure. The in-ground motion generates 'kinematic interaction' between the piles and the soil. As the soft soil layer undergoes vibrations due to the arrival of cyclic shear stresses from the stiffer layer below, it will vibrate in the first mode, with a natural frequency f_n

$$f_n = \frac{V_s}{4H} \quad (1)$$

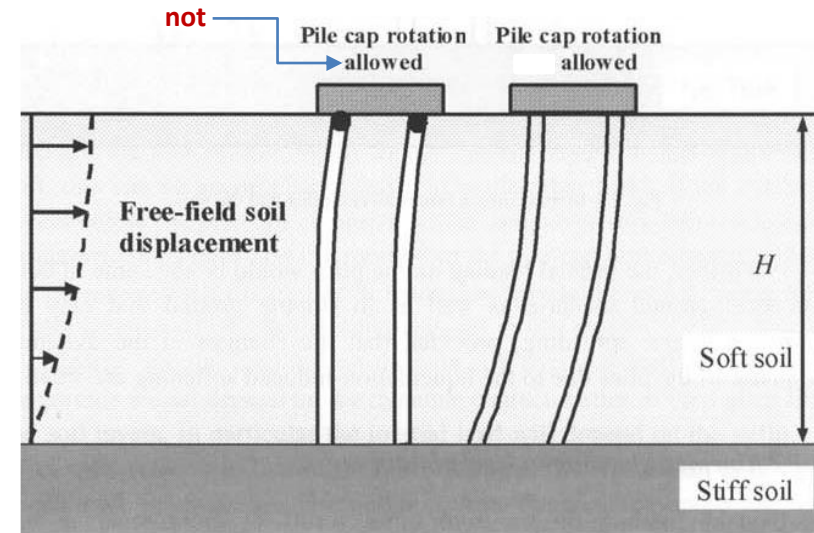


Fig.2.2 kinematic interaction between piles and soils

Kinematic Loading (Continued...)

This motion is indicated as free-field soil displacement in Fig. 2.2. The piles that are located in such a soil layer are forced to follow the motion imposed by the soil. If the piles are relatively flexible then they will undergo deformations as shown schematically in Fig. 2.2. On the other hand if the piles are relatively stiff, then the deformations they undergo will be smaller. In this way, the piles can attract large lateral loads that will act in the opposite direction to the imposed displacements, due to mobilisation of passive pressures.

Thus, due to the kinematic interaction between the soil and pile, the deformation in the piles occurs in the deeper regions as indicated in Fig. 2.2.

Kinematic Loading in Laterally Spreading Soil

A more serious lateral loading condition on piles may arise when there is a nonliquefied crust situated above a liquefiable soil deposit on sloping ground. Such a condition will invariably result in lateral spreading following an earthquake event, thereby inducing large lateral loads on the piles and pile cap.

Often the nonliquefied layer that may suffer lateral spreading may be a cohesive layer. If that is the case, the lateral load applied by a cohesive crust can be determined by using shallow foundation bearing capacity factors. For example, if the clay crust overlying the liquefied layer had an undrained shear strength of S_u , then the lateral pressure q applied on a rough, square pile cap in this region can simply be calculated using Eq. 2, following Randolph and Houlsby (1984).

$$q = (3\pi + 2) \cdot S_u \quad (2)$$

Limit Equilibrium Analysis of Piles under Earthquake Loading

Limit equilibrium analyses are quite popular in the design of retaining walls, shallow foundations and other geotechnical structures. It is possible to carry out limit equilibrium analyses to calculate the bending moments and shear forces in piles in laterally spreading soils. Such analyses are considered as an alternative to the p-y analysis.



Limit Equilibrium Analysis of Piles in laterally spreading soils

Let us first consider the simple case of single piles in laterally spreading soils overlying a dense soil layer, as shown in Fig. 2.3a. There is no nonliquefied crust above the spreading soil.

In this case it is straightforward to establish a limit equilibrium model using the free body diagram shown in Fig. 2.3b. This type of analysis was presented by Dobry et al. (2003) for relatively stiff piles. The results from the limit equilibrium analysis were validated against centrifuge tests conducted in loose deposits of saturated Nevada sand with the dense layer simulated by lightly cemented Nevada sand. (Abdoun et al., 2003).

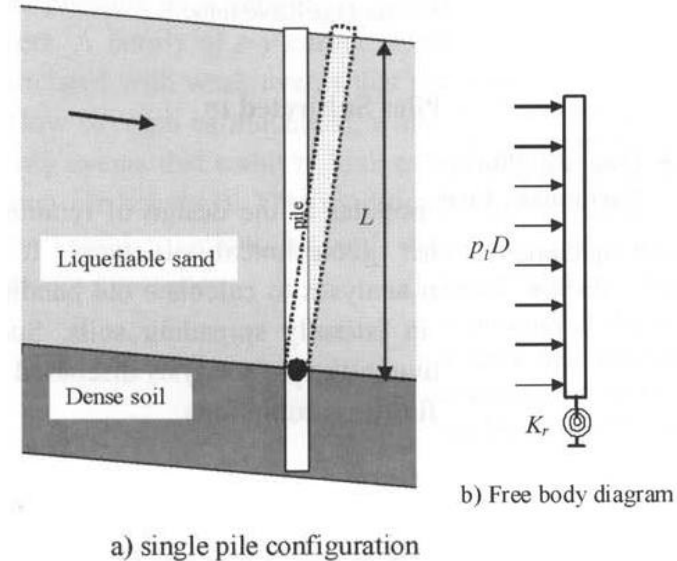


Fig.2.3 Single pile in laterally spreading soil

Limit Equilibrium Analysis of Piles in laterally spreading soils (Cont..)

Let the soil resistance per unit area offered by liquefied soil be p_l . Various researchers have estimated the value of p_l to be between 8kPa and 20kPa based on centrifuge tests on flexible and rigid piles in laterally spreading soils and based on centrifuge tests in which shear wave velocity measurements were made in liquefied soils. (Dobry et al., 2003; Haigh, 2002; Ghosh and Madabhushi, 2002).

Using this, the force applied by the liquefied soil on the pile can be calculated following Eq. 3.

$$F_p = p_l DL \quad (3)$$

The maximum bending moment will occur at the interface with the dense soil and its magnitude will be;

$$M_{max} = \frac{1}{2} p_l DL^2 \quad (4)$$

Limit Equilibrium Analysis of Piles in laterally spreading soils (Cont..)

If a pile cap of resisting area A_c is present at the top of the pile within the laterally spreading soil, the force acting on the pile will increase by $p_l A_c$. The maximum bending will also increase as shown in Eq. 5.

$$M_{max} = p_l L \left[\frac{1}{2} DL + A_c \right] \quad (5)$$

The rotational spring K_r is not required for estimating the maximum bending moment. However, a value of K_r is required if the lateral displacement of the pile top is required.

If the lightly cemented dense sand is assumed to have a rotational stiffness of 5738kNm/rad quoted by Dobry et al. (2003), then the lateral displacement at the pile top can be calculated as

$$\delta_{top} = \frac{M_{max}}{K_r} L \quad (6)$$

Limit Equilibrium Analysis of Piles in the presence of nonliquefied crust

Stiff clay as a nonliquefiable layer

Let us now consider the case of a single pile of diameter D passing through a stiff clay layer of thickness h and a liquefiable sand layer of thickness L before entering dense, nonliquefiable soil as shown in Fig. 2.4. The soil stratification is susceptible to lateral spreading and the stiff clay layer above the liquefiable sand will undergo lateral spreading and is capable of subjecting the pile to large lateral loads as passive soil pressures are generated.

Dobry et al. (2003) describe a limit equilibrium-based procedure for a similar soil configuration for a relatively flexible pile passing through a lightly cemented soil layer at the top and then through a thicker layer of liquefiable sand before entering yet another layer of nonliquefiable material at the bottom, also made up of lightly cemented sand. The same procedure can be modified for the case of a stiff clay layer at the top acting as the nonliquefiable crust. The general procedure of limit equilibrium analysis is described.

Limit Equilibrium Analysis of Piles in the presence of nonliquefied crust

Stiff clay as a nonliquefiable layer

When the sloping ground suffers lateral spreading following an earthquake event, both the liquefiable layer and the stiff clay layer on the top will suffer lateral spreading. This will cause the pile to deform as shown schematically in Fig. 2.4a in dashed lines. Free body diagrams of the soil pressures and hence forces acting on the pile can be sketched as shown in Fig. 2.4b.

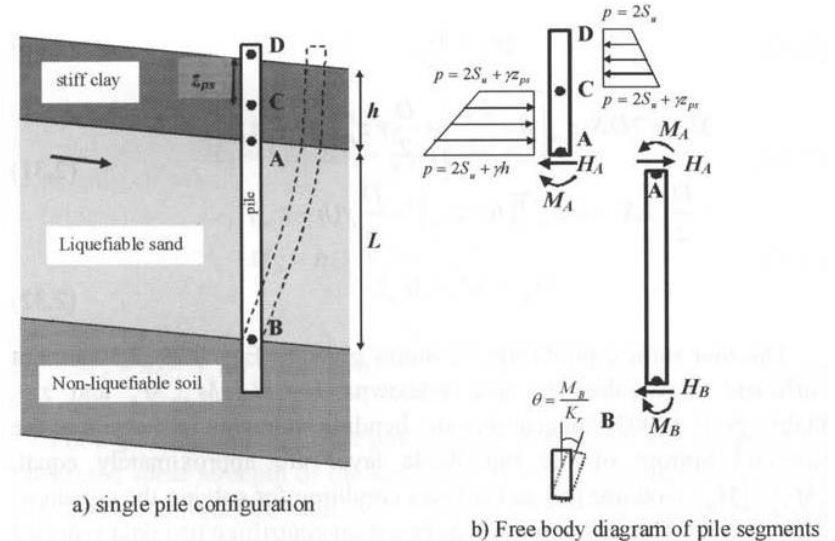


Fig. 2.4 Single pile passing through a laterally spreading soil and nonliquefied crust

Limit Equilibrium Analysis of Piles in the presence of nonliquefied crust

Stiff clay as a nonliquefiable layer

Let us consider points A and B on the pile at the top and bottom of the liquefiable layer. Let point D be the top of the pile. At a certain depth z_{ps} below the top of the pile, a point C is considered. This point C demarcates a shallow region of the stiff clay layer which applies a passive pressure from the down slope side on to the pile and the deeper region of the stiff clay layer which applies a passive pressure from the upslope side on to the pile as indicated in Fig. 2.3b. Further, a rotational spring with stiffness K_r is assumed at point B. This spring models the resistance offered by the lower layer of nonliquefiable soil to the pile.

Let us assume that the stiff clay layer above the liquefiable layer has a unit weight of γ and undrained shear strength of S_u . Consider the passive soil pressures in this layer at points D, C and A. From point D to C the passive pressure varies linearly from $2S_u$ to $2S_u + \gamma z$, and acts from right to left. Similarly from points C to A the passive pressure varies from $2S_u + \gamma z$ to $2S_u + \gamma h$ and acts from left to right on the pile.

Limit Equilibrium Analysis of Piles in the presence of nonliquefied crust

Dense sand as a nonliquefiable layer

Another common condition that is encountered in the field is a dense sand layer above the liquefiable layer. This can occur in the field due to natural deposition or densification efforts prior to construction. The water table could be present at a depth of h , thereby rendering the shallow soil layer above this to be nonliquefiable. A typical configuration is shown in Fig. 2.5. For this case let us include a fixing moment M_{Top} at the pile head. This moment could arise due to resistance to rotation offered by the framing action of a pile group and/or the rotational restraint offered by the superstructure.

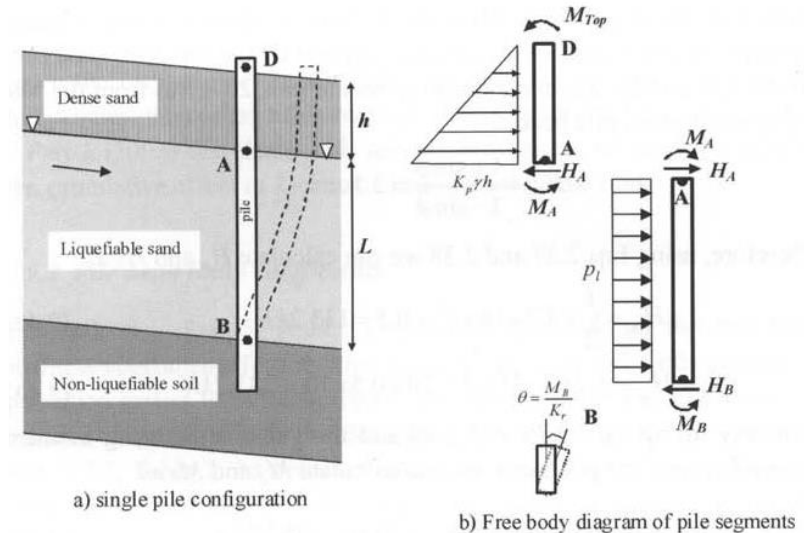


Fig. 2.5 Limit state for a single pile passing through a laterally spreading stiff clay layer

Limit Equilibrium Analysis of Piles in the presence of nonliquefied crust

Dense sand as a nonliquefiable layer

For the limit state shown in Fig. 2.5 the following equilibrium equations can be written, assuming that full passive earth pressure coefficient K_p in the dense sand layer is mobilized at the limit state

$$H_A = \frac{1}{2} K_p \gamma h^2 D \quad (7)$$

$$H_B = \frac{1}{2} K_p \gamma h^2 D + p_l L D \quad (8)$$

$$M_A = \frac{1}{6} K_p \gamma h^3 D - M_{Top} \quad (9)$$

$$M_B = \frac{1}{6} K_p \gamma h^3 D + \frac{1}{2} K_p \gamma h^2 L D + \frac{1}{2} p_l D L^2 - M_{Top} \quad (10)$$

Thank You

