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ANALYSIS AND DESIGN OF FOUNDATION UNDER EARTHQUAKES

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ABSTRACT

Design of shallow foundations as well as piles in seismic areas needs special consideration. Shallow foundations may experience a reduction in bearing capacity and increase in settlement due to seismic shaking. The reduction in bearing capacity depends on the type of soil and the ground acceleration. This needs to be accounted for in design.

In pile group behavior, suitable group efficiency factors need to be considered to account for pile-soil-pile interactions. The soil-pile behavior under earthquake loading is generally nonlinear. The non-linearity may be accounted for by defining the soil-pile stiffness in terms of strain dependent soil modulus. A simple procedure to account for non-linearity is presented. Comments are also offered on their design aspects.

KEYWORD

Seismic design, shallow foundations, dynamic bearing capacity, dynamic settlement, piles, pile groups, non-linearity.

INTRODUCTION

Several behavioral and design aspects of shallow foundations and pile groups subjected to earthquake loads have been presented. For the case of shallow foundations the usual design methods are briefly discussed. The method to account for reduction in bearing capacity due to earthquake loading (Richard et al, 1993) is reviewed. For analysis and design of pile groups under seismic loading a simple approach to account for nonlinear behavior of soil-pile system under dynamic loads and group efficiency factors are presented. In this approach strain dependent soil modulus is used to define soil-pile stiffness and radiation damping.

SHALLOW FOUNDATIONS

Bearing capacity of shallow foundations for the static case has been extensively studied and reported in literature. However foundations are often subjected to transient and earthquake loads which may impose dynamic horizontal and vertical loads on them. These loads may induce large permanent deformations. A fundamental definition of the *dynamic bearing capacity* has not yet been found. The response of a foundation to dynamic loads is affected by the (1) nature and magnitude of dynamic loads, (2) number of pulses and (3) the strain rate response of soil. Triandafilidis (1965) presented a solution for the dynamic response of a continuous footing supported by a saturated cohesive soil and subjected to a vertical transient pulse. Prakash and Chummar (1967) investigated the dynamic response of a continuous footing resting on c - ϕ soil and subjected to a horizontal transient pulse. Wallace (1961) presented a solution for the settlement of a continuous footing subjected to vertical transient load assuming punching shear type failure.

Basavana, Prakash and Arya (1981) used finite element approach to determine the load settlement behavior of footings on clay under vertical transient loads. It was observed that even if the shear strength increases due to strain effect, the bearing capacity of clay under transient load may be either higher or lower than that under static load depending upon period of applied load in relation to the natural period of the foundation-soil system.

Bearing Capacity Reduction Under Seismic Loads

Shallow foundations for seismic loads are usually designed by the equivalent static approach. The foundations are considered as eccentrically loaded or as carrying inclined eccentric load and Meyerhof's (1965) method is used for calculating the ultimate bearing capacity. To account for the effect of dynamic nature of the load, the bearing capacity factors are determined by using dynamic angle of internal friction which is taken as 2-degrees less than its static value, Whitman and Healy (1962). The settlement and tilt of the foundation may then be obtained using the method proposed by Prakash (1981).

Richards et al (1993) have observed seismic settlements of foundations on partially saturated dense or compacted soils. These settlements were not associated with liquefaction or densification and could be easily explained in terms of seismic bearing capacity reduction. Richards et al (1993) have proposed a simplified approach to estimate dynamic bearing capacity and seismic settlements of a strip footing. Figure 1 shows the assumed failure surface for the seismic bearing capacity (q_{uE}) analysis of a strip foundation. q_{uE} is given by Eq. 1:

$$q_{uE} = cN_{cE} + qN_{qE} + \frac{1}{2}\gamma BN_{\gamma E} \qquad (1)$$

where q = Surcharge above the base of the footing ,i.e., γD_f

γ = Unit weight of soil

D_f = Depth of the foundation

N_{cE}, N_{qE}, and $N_{\gamma E}$ = Seismic bearing capacity factors which are functions of ϕ and tan ψ, where tan $\psi = k_h \div (1 - k_v)$

k_h = Horizontal coefficient of acceleration due to earthquake

k_v = Vertical coefficient of acceleration due to earthquake

For static case, $k_h = k_v = 0$ and Eq. (1) becomes

$$q_u = cN_c + qN_q + \frac{1}{2}\gamma BN_\gamma \qquad (2)$$

in which N_c, N_q and N_γ are the static bearing capacity factors. Figure 2 shows plots of $N_{\gamma E}/N_\gamma$, N_{qE}/N_q and N_{cE}/N_c with tan ψ and ϕ.

Seismic Settlement of Foundations

Bearing capacity-related seismic settlement takes place when the ratio $k_h/(1 - k_v)$ reaches a critical value $(k_h/1 - k_v)^*$. If $k_v = 0$, then $(k_h/1 - k_v)^*$ becomes equal to k_h^*. Figure 3 shows the variation of k_h^* (for k_v = c = 0; granular soil) with the static factor of safety (FS) applied to the ultimate bearing capacity

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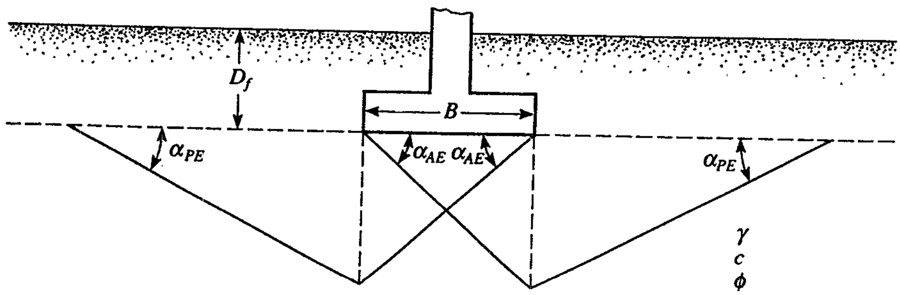

FIGURE 1 Failure surface in soil for seismic bearing capacity (After Rechards et al, 1993)

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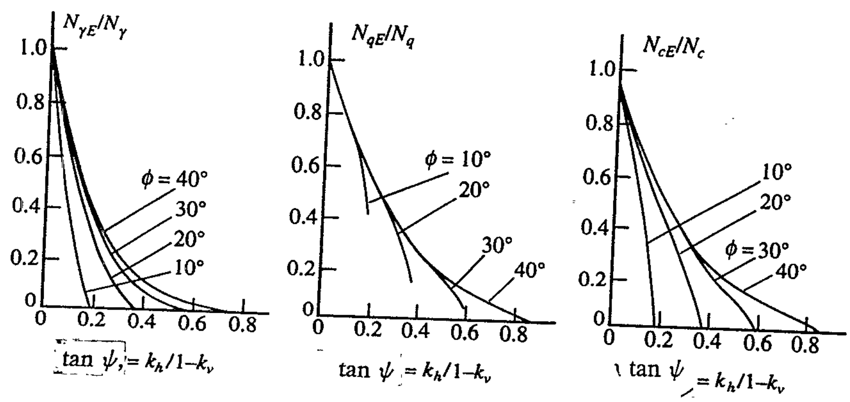

FIGURE 2 Variation of $N_{\gamma E}/N_\gamma$, N_{qE}/N_q and N_{cE}/N_c with ϕ and tan ψ (After Richards et al 1993)

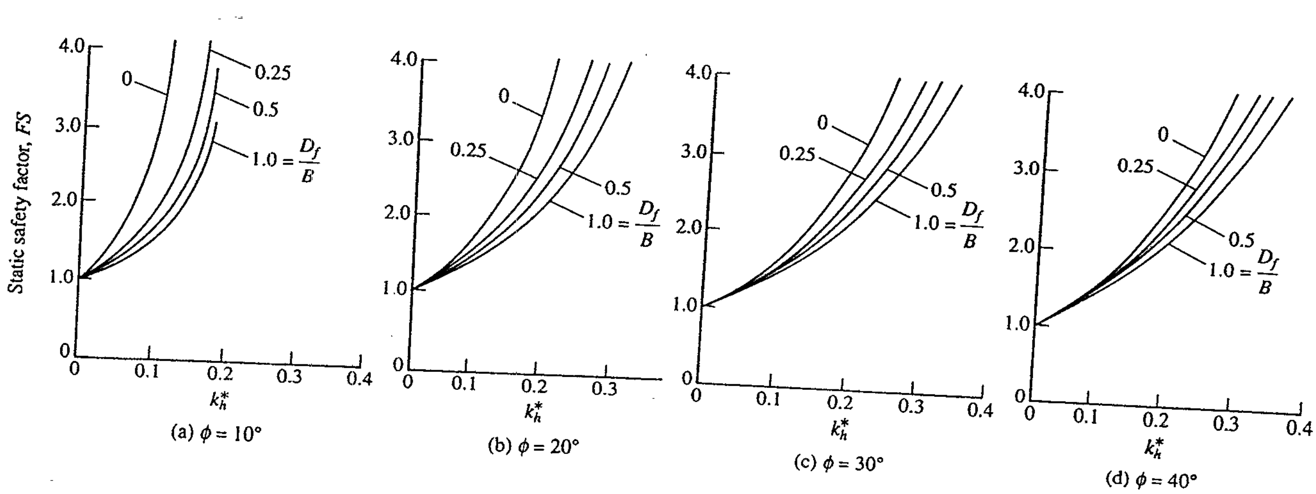

FIGURE 3 Critical acceleration k_h^* for $k_v = c = 0$ (After Richards et al 1993)

[Eq, (2)], for values of ϕ of 10°, 20°, 30°, and 40° and D_f/B of 0, 0.25, 0.5 and 1.0.

The settlement (S_{Eq}) of a strip foundation due to an earthquake can be estimated (Richards et al, 1993) as

$$S_{Eq}(m) = 0.174 \frac{V^2}{Ag} \left|\frac{k_h^*}{A}\right|^{-4} \tan \alpha_{AE} \tag{3}$$

where V = Peak velocity for the design earthquake (m/sec)
A = Acceleration coefficient for the design earthquake
g = Acceleration due to gravity (9.81 m/sec^2)
$\tan \alpha_{AE}$ = Depends on ϕ and k_h^*

In Figure 4, variation of $\tan \alpha_{AE}$ with k_h^* for ϕ of 15° - 40° is plotted.

Suppose a typical strip foundation is to be constructed on a sandy soil with B = 2 m, and D_f = 0.5 m, γ = 18 kN/m^3, ϕ = 34°, and c = 0. The value of k_h = 0.3 and k_v = 0 and the velocity V induced by the design earthquake is 0.4 m/sec. The static ultimate bearing capacity for this footing for vertical load will be 1,000 kN/m^2 (Eqn 2). The reduced ultimate bearing capacity for vertical load (Richards et al., 1993) is calculated as 290 kN/m^2 Eqn 1. If the footing is designed using a FS = 3 on the static ultimate bearing capacity (i.e., for an allowable soil pressure of 333 kN/m^2), the additional settlement due to earthquake will be 20.5 mm. This settlement reduces to 7.0 mm if FS of 4 is used.

Besides ensuring that the footing soil system does not experience a bearing capacity failure or undergo excessive settlement, the foundations should be tied together using interconnecting beams (Applied Technology Council 1978, see also Prakash 1981).

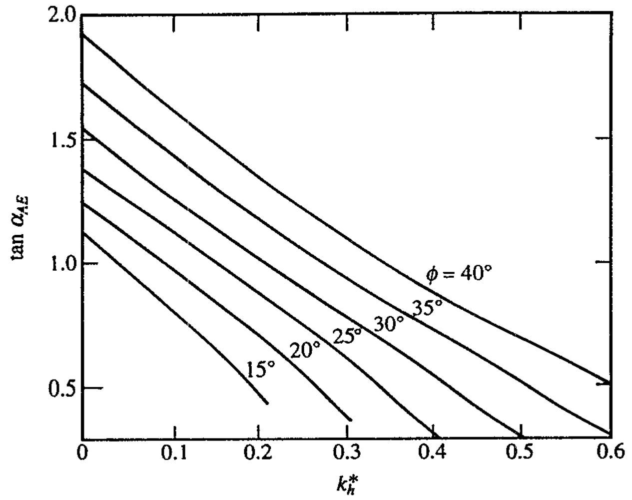

FIGURE 4 Variation of $\tan \alpha_{AE}$ with k_h^* and ϕ (After Richards et al 1993)

PILES AND PILE GROUPS UNDER EARTHQUAKE LOADINGS

Pile foundations are used extensively to support heavy loads. The bulk of loads are static which forms the basis for fixing the section (size), embedded length and configuration (spacing and arrangement) of the piles in the group. Dynamic loads are caused by nature e.g. earthquakes, winds, waves and by man-made vibrations, e.g., machines, traffic and blasts. Prakash and Sharma (1990) have recommended the following procedures for design of single pile against earthquakes:

1. Estimate the dead load on the pile. The mass at the pile top which may be considered vibrating with

the piles is only a fraction of this load.

2. Determine the natural frequency ω_{n1} and time period in first mode of vibrations.
3. For the above period, determine the spectral displacement S_d for the appropriate damping (radiation + material).
4. Using the pile displacements in (3) above, determine the bending moments and shear forces for structural design of the pile.

ANALYSIS OF PILE GROUPS

Pile groups have been analyzed by geotechnical engineers neglecting the foundation-structure interaction. The structure has been analyzed by the civil/structural engineer by either neglecting the flexibility of the foundations or using a fixed value of the soil spring. The soil stiffness (spring) is strain dependent and soil behavior during earthquakes is strongly non-linear. Therefore, analysis and design of structures need to consider the total pile-soil-structure interaction.

The following steps are involved in analysis of a pile group

1. Determine the stiffness and radiation damping of the single pile
2. Determine the stiffness of the pile group by applying appropriate pile-soil-pile interaction factors.
3. Determine natural frequencies
4. Estimate the response of the pile group for the given input motion.

The pile cap displacements (translation and rotation) may be used as input parameters for superstructure analysis.

Stiffness and Radiation Damping of Single Piles

Stiffness of single piles in horizontal-translation and rotation may be determined as follows:
Novak and El-Sharnouby (1983) and Gazetas (1991) have recommended expressions for horizontal sliding stiffness (k_x), rocking stiffness (k_ϕ) and cross-coupling stiffness $k_{x\phi}$. It has been shown by Kumar (1996) that the above two solutions generally give similar results for homogeneous soil profile.

Corresponding expressions for radiation damping coefficients C_x, C_ϕ and $C_{x\phi}$ (sliding, rotational and cross-damping) have been developed by both the investigators.

Stiffness and Radiation Damping of Pile-Group

Gazetas (1991) has recommended pile-soil-pile interaction factors for both the stiffness and radiation damping. Stiffness and radiation damping of pile groups can also be computed by using appropriate efficiency or interaction factors. A detailed discussion on the Pile Group Efficiency Factors (ϵ) for horizontal sliding has been presented in a companion paper to this conference (Prakash et al 1996). Based on experimental studies on piles, following simple expression has been developed for Group Efficiency Factors (ϵ) in horizontal vibrations.

$$\epsilon = \left(\frac{1}{2}\right)^{3/2}\left(\frac{S}{d}\right)^{\frac{1}{2}}\left(\frac{1}{N}\right)^{\frac{1}{5}} < 1 \qquad \textbf{(4)}$$

Group Efficiency Factor (ϵ) is defined as:

$$\epsilon = \frac{Group\ stiffness\ (k_{xg})}{N\ X\ Single\ pile\ stiffness\ (k_x)} \tag{5}$$

where s = Center to center spacing of piles
d = Diameter or width of pile
N = Number of piles in group

Similarly group radiation damping in sliding (C_{xg}) is given by Eqn 6.,

$$\frac{Group\ damping\ (C_{xg})}{N\ X\ Single\ pile\ damping\ (C_x)} = \epsilon \tag{6}$$

The above equation follows from the recommendations for pile group effects by Novak (1974).

Gazetas (1991) has recommended that group efficiency factors for k_ϕ, C_ϕ, $k_{x\phi}$ and $C_{x\phi}$ may be taken as one. In fact the results of Gazetas et al (1991) also show that interaction factors for rotational and cross rotational vibrations are negligible; this means that group efficiency factor is unity.

The pile cap stiffness and damping needs be added to the k_{xg}, $k_{\phi g}$ and C_{xg} and $C_{\phi g}$ computed above (Prakash and Sharma 1990).

Pile Group Natural Frequencies

The next step is to determine natural frequencies of the system, which may be determined from the following equation (Kumar and Prakash 1995)

$$\left[\left\{\omega^4 - \left(\frac{k_x}{m} + \frac{k_\phi}{M_m} + \frac{C_x C_\phi}{mM_m} - \frac{C_{x\phi}^2}{mM_m}\right)\omega^2 + \left(\frac{k_x k_\phi}{mM_m} - \frac{k_{x\phi}^2}{mM_m}\right)\right\}^2 + \left\{\left(\frac{k_x C_\phi}{mM_m} + \frac{k_\phi C_x}{mM_m} - \frac{2k_{x\phi}C_{x\phi}}{mM_m}\right)\omega - \left(\frac{C_\phi}{M_m} + \frac{C_x}{m}\right)\omega^3\right\}^2\right]^{1/2} = 0 \tag{7}$$

In this equation, k's and c's represent the total stiffness and damping (i.e., sum of piles and cap's stiffness and damping respectively). This equation in terms of stiffness and damping value is very useful in analysis of soil non-linearity as compared to the frequency equations routinely presented in published literature (Prakash and Sharma 1990).

Pile Group Response

The response of the pile group may be determined by any standard structural dynamic method of analysis. A detailed time history solution for a given ground motion may be obtained and the maximum response values be selected for design. The above response may be used as input motion for the superstructure to

Alternatively, a spectral response method be used as explained below.

a. For given ground motion, develop response spectra for various values of damping.
b. For the natural frequencies (periods) determined in step 'a' above, determine the spectral response at appropriate damping level. For the 2-degrees of freedom system considered above, two values of spectral response, S_1 and S_2 are determined. The design response may be determined as the RMS value as (Prakash 1981):

$$(S)_{design} = \sqrt{S_1^2 + S_2^2} \tag{8}$$

Piles only can be analyzed for this case. Foundation-structure interaction may not be solved.

NON-LINEAR ANALYSIS

Prakash et al (1996) have shown that soil-pile behavior is strongly non-linear. Therefore, for analysis and design of pile foundations in seismically active regions, non-linearity of soil must be considered. Various relationship between soil shear modulus (G) and normalized shear modulus (G/G_{max}) with strain have been developed (Vucetic and Dobry, 1991). The strains in soils during earthquakes may be of the order of 10^{-4} to 10^{-2}. A procedure for non-linear analysis is recommended .

Procedure to Perform Nonlinear Analysis

1. As a first approximation, assume soil shear modulus to be equal to low strain modulus, i.e., modulus corresponding to 10^{-6} strain.
2. Compute displacements at the foundation level for the assumed soil modulus.
3. Compute shear strain in the soil.
4. Obtain the value of shear modulus using the appropriate shear modulus-strain relationship, corresponding to the strain computed in Step 3.
5. If the difference between the assumed shear modulus and that computed in Step 4 is within the tolerance limit (2 to 5% is recommended for all practical purposes), then move to the next time step.
6. If the difference computed in Step 5 is not within the tolerance limit, assume the value of shear modulus computed in Step 4 as the new value of shear modulus and restart from Step 2.
7. Repeat Steps 2 to 6, till the difference between the assumed and computed shear modulus is within the tolerance limit.

It should be noted that a perfect match between the predicted and assumed strains or moduli may not occur. In that case, we have to accept the match as close as it gets. This may happen because the prediction model may not correctly simulate the actual phenomenon. Also the assumptions in the analysis may not be realized fully in practical cases.

CONCLUSIONS

1. A method to estimate seismic bearing capacity and settlements of strip foundations has been reviewed.
2. A procedure to determine non-linear response of soil-pile systems using stiffness based on strain-dependent soil modulus is discussed.
3. Further research involving carefully conducted model and field tests on shallow foundations and piles and their analysis are needed to develop reasonable design procedure for earthquake resistant design of foundations.

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