Shallow Foundations Under Dynamic Loads

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ABSTRACT: The shallow foundations may be subjected to dynamic loads by machine operation or by seismic loading. A comparison of observed and predicted response is presented for the case of machine foundations based on observed data obtained by conducting vibration tests on two concrete blocks. For case of shallow foundations for seismic loading the recent developments in determining bearing capacity and settlements are discussed and the need for research for developing rational design procedures in this area is highlighted.

I INTRODUCTION

The shallow foundations in the form rigid concrete blocks are commonly used to support reciprocating type machine foundations. These foundations are also commonly used for supporting structural loads which may be static loads or a combination of static and seismic loads. When any foundation is used to support combination of static as well dynamic loads, it must satisfy the stability criteria for both static as well dynamic loads.

1.1 Machine foundation case

The response of rigid block type machine foundations is generally obtained by making appropriate spring-mass-dashpot models and using principles of mechanics. The spring and damping values are obtained by using elastic half space approach or they may be obtained from impedance compliance functions (Prakash and Puri 1988). This paper presents comparison of the computed and observed response of two block foundations made of concrete. The test blocks were excited into vertical vibrations and their natural frequencies and amplitudes were measured. The pertinent soil properties were determined by conducting in-situ tests and oscillatory shear tests in the laboratory. The response of the test blocks was then computed by using the (i) elastic half space method and (ii) the impedance function approach. A comparison was made of the observed and predicted response of the test blocks.

1.2 Foundations for seismic load case

Shallow foundations in seismic areas may be on non-liquefiable soils or on liquefiable soils. Shallow foundations may experience a reduction in bearing capacity and increase in settlement and tilt due to seismic loading. The reduction in bearing capacity depends on the nature and type of soil and ground acceleration parameters in non-liquefiable soils. In liquefying soils, the buildings on shallow footings may settle and tilt excessively.

These two cases namely the machine foundation case and the foundations for seismic load case are discussed below.

2 COMPARISON OF OBSERVED AND PREDICTED RESPONSE OF TEST BLOCKS

2.1 Tests conducted

2.1.1 Block vibration tests

Vertical vibration tests were conducted on rigid concrete blocks 1.5m x 0.75m x 0.70m and 3.0m x 1.5m x 0.7m cast on level ground. The vibrations of the block were measured with transducers mounted on the top surface of the block which were oriented to sense vertical vibrations. Records of vibrations were obtained for varying frequencies of excitation. The tests were repeated by changing ‘θ’ the angle of setting of the eccentric masses. From the recorded test data, the frequency and the corresponding amplitudes were determined. Values of observed natural frequencies and corresponding amplitudes are given in Tables I and 2.
2.1.2 Tests for determination of dynamic soil properties

The dynamic properties of the soil used in the analysis of machine foundation may be determined by laboratory and in-situ tests. These properties are affected by a number of factors which should be accounted for when selecting the design values. The most important factors which affect these properties are (1) the mean effective confining pressure, (2) the shear strain amplitude, and (3) density of the soil. A good discussion on these corrections has been presented Prakash and Puri (1977,1981,1988). In-situ soil investigations consisted of wave propagation tests, cyclic plate load tests and standard penetration tests. From these tests, the values of dynamic shear modulus $G$ with shear strain were computed.

2.1.3 Computed response of test blocks

The response of the test blocks was calculated by elastic half space method, and impedance function method. In the elastic half space approach the vibrating foundation is treated as resting on the surface of an elastic, semi-infinite, homogenous, isotropic half space (Richart, 1962). The elasticity of the soil and the energy carried into the half space by waves traveling away from the vibrating foundation (geometric damping) are thus accounted for and the response of such a system may be predicted using a mass-spring-dashpot model known as the half space analog (Richart and Whitman, 1967; and Richart, Hall and Woods, 1970). The impedance function method (Gazettas, 1991) gives an approach for calculating soil springs and damping values which are frequency dependent. The dynamic response of the foundation was computed using these above methods and is shown in tables 1 and 2. The details of these methods are not given here.

2.2 Discussion of computed and observed response of test blocks

(1) It is observed from Tables I and 2, that the natural frequencies in vertical vibration for the two test blocks calculated by using the elastic half space analog are generally within about 20% of natural frequencies observed during the vibration tests. These calculated natural frequencies in this case are thus in reasonable agreement with the observed natural frequencies.

(2) The natural frequency of vertical vibrations...
calculated by the impedance function method are somewhat higher for the 1.5m x 0.75m high block as compared to the observed natural frequency. The calculated values of natural frequency by this method are somewhat smaller than the observed values for the 3.0 m x 1.5m x 0.7m high block.

(3) The damped vibration amplitudes as obtained from the elastic half space method do not agree with the observed values. For 1.5 m 0.75 x 0.70m block, the calculated amplitudes are about 2-3 times higher than the observed amplitudes and, for 3.0m x 1.5m x 0.75 m block, the calculated values of vibration amplitude are about 4 times more than the observed values.

(4) The damped amplitudes calculated by the impedance function method for the two concrete blocks used in this study show a somewhat more reasonable agreement with the observed values. The limited data does not justify any general conclusion.

3. FOUNDATIONS FOR SEISMIC LOADS

Shallow foundations in non-liquefiable soils are commonly designed by the equivalent static approach. In liquefiable soils, pore pressure buildup and drainage conditions may result in decrease in strength and considerable damage due to tilting and settlement. Prasad et al (2004) made an experimental investigation of the seismic bearing capacity of sand. A practical method to account for reduction in bearing capacity due to earthquake loading was presented by Richards et al (1993).

3.1 Foundations in non-liquefying soil

The response of a footing 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. Shallow foundations for seismic loads are usually designed by the equivalent static approach. The foundations are considered as eccentrically loaded and the ultimate bearing capacity is accordingly estimated. 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 (Das, B.M 1992). International Building Code generally permit an increase of 33 % in allowable bearing capacity when earthquake loads in addition to static loads are used in design of the foundation. This recommendation may be reasonable for dense granular soils, stiff to very stiff clays or hard bedrocks but is not applicable for friable rock, loose soils susceptible to liquefaction or pore water pressure increase, sensitive clays or clays likely to undergo plastic flow (Day, R.W. 2006).

Richards et al (1993) proposed a simplified approach to estimate the dynamic bearing capacity $q_{ue}$ and seismic settlement $S_{ue}$ of a strip footing for assumed failure surfaces. The seismic bearing capacity ($q_{ue}$) is given by Eq. 1:

$$q_{ue} = cN_{cE} + qN_{qE} + 0.5 \gamma BN_{\gamma E}$$

where, \(\gamma\) = Unit weight of soil

$q = \gamma D_f$ and $D_f$ = Depth of the foundation

$N_{cE}$, $N_{qE}$, and $N_{\gamma E}$ = Seismic bearing capacity factors which are functions of $\phi$ and

$$\tan \psi = k_h / (1-k_v)$$

$k_h$ and $k_v$ are the horizontal and vertical coefficients of acceleration due to earthquake.

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

$$q_s = cN_c + qN_q + 0.5 \gamma BN_{\gamma}$$

in which $N_c$, $N_q$ and $N_\gamma$ are the static bearing capacity factors. Figure 1 shows plots of $N_{cE}/N_c$, $N_{qE}/N_q$ and $N_{\gamma E}/N_\gamma$ with $\phi$ and $\tan \psi$.

![Figure 1. Variation of $N_{cE}/N_c$, $N_{qE}/N_q$ and $N_{\gamma E}/N_\gamma$ with $\phi$ and $\tan \psi$ (After Richards et al 1993)](image-url)
3.2 Seismic Settlement of foundations

Bearing capacity-related seismic settlement takes place when the ratio $k_v/(1 - k_v)$ reaches a critical value $(k_v/1 - k_v)_c$. If $k_v = 0$, then $(k_v/1 - k_v)_c$ becomes equal to $k_v$. Figure 2 shows the variation of $k_v$ (for $k_v = c = 0$; granular soil) with the static factor of safety (FS) applied to the ultimate bearing capacity Eq., for $\phi = 10^\circ, 20^\circ, 30^\circ, \text{and} 40^\circ$ 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,

$$S_{eq}(m) = 0.174 \frac{V^2}{A g} \left[ \frac{k_{h*}}{A} \right]^{-4} \tan \alpha E \quad (3)$$

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$^2$).

Figure 2. Critical acceleration $k_h*$ (After Richards et al, 1993)

Suppose a typical strip foundation is supported on a sandy soil with $B =$ 2 m, and $D_f =$ 0.5 m, and $\gamma =$ 18 KN/m$^3$, $\phi =$ 34$^\circ$, 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$^2$ (Eq. 2). The reduced ultimate bearing capacity for vertical load is calculated as 290 KN/m$^2$ (Eq 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)

3.3 Shallow foundations in liquefying soil

Gazezas et al (2004) studied tilting of buildings in 1999 Turkey earthquake. Detailed investigation of the “Adapazari failures” showed that significant tilting and toppling were observed only in relatively slender buildings (with aspect ratio: $H/B > 2$), provided they were laterally free from other buildings on one of their sides. Wider and/or contiguous buildings suffered small if any rotation, for the prevailing soil conditions and type of seismic shaking: most buildings with $H/B > 1.8$ overturned, whereas buildings with $H/B < 0.8$ essentially only settled vertically, with no visible tilting. Figure 3 shows a plot of $H/B$ to tilt angle of building.

Soil profiles based on three SPT and three CPT tests, performed in front of each building of interest, reveal the presence of a number of alternating sandy-silt and silty-sand layers, from the surface down to a depth of at least 15 m with values of point resistance $qc \approx (0.4 - 5.0)$ MPa (Gazetas 2003). Seismo-cone measurements revealed wave velocities $Vs$ less than 60 m/s for depths down to 15 m, indicative of extremely soft soil layers. Ground acceleration was not recorded in Tıgıcilar. Using 1-D wave propagation analysis, the EW component of the Sakarya accelerogram (recorded on soft rock outcrop, in the hilly outskirts of the city) leads to acceleration values between 0.20 g -0.30 g, with several significant cycles of motion, with dominant period in excess of 2 seconds. Even such relatively small levels of acceleration would have liquefied at least the upper-most loose sandy silt layers of a total
thickness 1–2 m, and would have produced excess pore-water pressures in the lower layers Gazetas et al (2004).

3.4 Recent development in shallow foundations under seismic loading

Seismic bearing capacity factors for a strip footing resting on cohesionless soil were determined by Dormieux and Pecker (1995) using the upper bound theorem of yield. Using the classical Prandtl like mechanism, it was established that the reduction in bearing capacity was mainly caused by load inclination (Dormieux and Pecker; 1995).

Choudhury and Subba Rao (2005) determined seismic bearing capacity factors for shallow strip footing using the limit equilibrium approach and pseudo-static method of analysis. The reduction in bearing capacity under the combined effect of vertical and horizontal forces was explained by using smaller failure surface (Figure 6) compared to case when only static vertical loads are applied.

Figure 6. Failure Surfaces under static and Seismic Loading (Choudhury and Rao 2005)

Gajan and Kutter (2009) proposed ‘contact interface model’ for analysis of shallow foundations subjected to combined cyclic loading. The rigid footing and the soil beneath the footing in the zone of influence are modelled as a single macroelement. The non-linear relationship between the cyclic loads and displacement at the footing soil interface are accounted for.

4. CONCLUSIONS

(1) For the case of machine foundations, reasonable predictions of frequencies and amplitudes can be made using available approaches provided soil parameters are rationally chosen.

(2) Method to estimate seismic bearing capacity and settlements of strip foundations has been reviewed. Analytical solution need validation on model, full scale and/or centrifuge tests.

(3) Further research on model and field tests on settlement and tilt of shallow foundations and their analysis are needed to develop reasonable design procedures for earthquake resistant design. Several researchers are investigating this problem.

REFERENCES


