BEARING CAPACITY UNDER SEISMIC LOADING

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SUMMARY

Available methods of bearing capacity for transient loading are examined along with acceptable factors of safety. Relavance of using the methods and input parameters of the methods are discussed in light of recent experimental work. Recommendations to rational approach for evaluation of bearing capacity under seismic loading are presented.

INTORDUCTION

The problem of bearing capacity under seismic loads has been covered in the literature very superficially up to date. Lack of actual cases of failure of foundations during earthquakes, perhaps is the reason for this. The only cases of concern have been liquefaction and dynamic settlement on sands. In this paper an effort has been made to put together, at one place, avaliable information on bearing capacity under seismic loading and provide a rational approach.

METHODS OF B. C. ANALYSIS OF DYNAMICALLY LOADED FOUNDATIONS

The literature deals with three kinds of studies: (1) Studies where the foundation was initially in motion and the soil was at rest; (2) Studies aimed at design of foundations subject to impact loadings; and, (3) Studies where foundation is at rest initially and the foundaiton is in motion. A brief review of related studies follows.

Lamb (1904) was the first to derive a relationship between the vertical harmonic force, applied at a point on the surface of an isotropic, homogeneous, semi-inifinite, elastic medium and the displacement. An analytical investigation was conducted by Sung (1953) to determine the effect of the amplitude of oscillation on the resonant frequency. Richart (1960) outlined Sung's work and presented extensive curves which may be used for analysis and design of a rigid foundation subjected to vibratory loads. Triandafilidis's (1961) analytical work for a very simplified, rigid, plastic material revealed the importance of inertia forces of participating soil mass. Carroll (1963) studied the behavior of square, rigid footings on a clay medium experimentally and analytically. Even with appreciable scatter his conclusions are that predicted results agree with test results using engineering approximations. Lysmer and Richart (1966) developed a method to analyze a vibrating (vertically) rigid circular footing resting on an elastic half-space. Experimental investigations on small rigid footings resting on a bed of sand, under transient loading were conducted by Drenevich and Hall (1966). The magnitude of the load, however, was so small that the response was essentially elastic. Skormin (1970) conducted some bearing tests using a circular plate 60 cm. in diameter, resting on a three meter thick sand layer. His investigations conclude that the streee under

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the foundation was concentrated more than that suggested by the theory of semi-infinite elastic solids. For a detailed review of literature concerning behavior of foundaitons where dynamic loads have been applied directly to the foundations, the works of Selig (1960), Bosvanna et. al. (1974) and Varadhi et. al. (1977) are recommended.

Heller (1964) reports the studies of falure modes of impact loaded footings on sand done by Selig and McKee (1961), and pneumatic hydraulic loading by Shenkman and McKee (1961) and Fisher (1962). Based on the results of the above tests, Vey (1962) commented that when weights are dropped on a footing the failure mode is by punching, while a pneumatic hydraulic loading produced a general shear failure. Analyzing the experimental and analytical work, Heller concluded, "If the inertial bearing capacity of an impact-loaded footing is five or more times the frictional bearing capacity, the punching shear failure mode can occur. The general shear failure mode can occur for lesser values of this ratio." Richart (1965) and Vesić (1973) agree in general with the above conclusions. However, the author believes that to extrapolate the results of impact loaded footings to seismic loading is not a trivial problem.

Very little is available in literature describing studies of dynamic bearing capacity during earthquake type seismic loads. Investigators from Japan (Okamoto, 1973, Shinohara et. al. 1960) have recommended a quantitative determination of bearing power of ground during earthquakes. The bearing capacity is evaluated by either Meyerhof's solution (1953) or Shinohara-Tateshi solution (1960). Both the approaches are psuedo-static where vertical and horizontal accelerations are applied to the center of gravity of the structure and the problem is reduced to a static case of bearing capacity with inclined eccentric loads. According to Meyerhof the bearing capacity for shallow foundations is:

$$q_a = \frac{1}{F} \quad \alpha c N_c' + \gamma_2 D_f (N_q' d_q - 1) + \frac{1}{2}\gamma_1 \beta \, B \, N_\gamma' d_\gamma$$

where q_a = Vertical compenent of allowable bearing capacity ($t/_{m}2$)
F = factor of safety
B = breadth of foundation (m)
D_f = depth of embedment from lowest ground surface adjacent to foundation
c = apparent cohesion of cohsive soil ($t/_{m}2$)
γ_1 = unit weight of soil below the base of foundaiton (for portions below ground water level, use submerged weight) ($t/_{m}3$).
γ_2 = unit weight of soil above the base of foundation (for portions below ground water level, use submerged weight) ($t/_{m}3$)
α, β = shape factors as per Table I
d_q, d_γ = depth factors as per Table II
N_c', N_q' and N_γ' are modified bearing capacity factors, related to N_c, N_q as follows:

$$N_c' = (1 - 2e/B)\,(1 - \alpha/90^o)^2\, N_c$$
$$N_q' = (1 - 2e/B)\,(1 - \alpha/90^o)^2\, N_q$$
$$N_\gamma' = (1 - 2e/B)^2\,(1 - \alpha/\phi)^2\, N_\gamma$$

where α = the angle of obliquity of resultant load (deg.)
e = eccentricity of resultant of load (m)
ϕ = angle of internal friction (deg.)

Shinohara-Tateishi computed the bearing capacity for sandy soil based on following assumptions:

- -The sliding surface has a shape of circular arc
- -A sliding surface starts from one of three points, i.e. both end points of the base slab and intermediate point of the base slab where the distribution of sand reaction greatly changes
- -The bearing capacity is compüted by friction circle method
- -The smallest of the minimum loads, which are computed for the three possible sliding surfaces, will be the ultimate bearing capacity.

Tateishi's experimental work on model tests in laboratory appears to confirm his conclusion that the circular sliding surface and inertia of the sliding section provides sufficiently accurate computation for dynamic bearing capacity. Shinohara et. al. provide the bearing capacity factors based on their approach. Both methods assume that the soil strength in the dynamic case is the same as in the static case, however lower limits of factor of safety are used. A word about factors of safety therefore is in order.

ACCEPTABLE FACTORS OF SAFETY

A survey of major consultants in the U.S.A., designing nuclear power plants, confirmed the obvious conslusion that most consultants use a factor of safety $\geqslant 3$ for static design and $\geqslant 2$ for seismic design. The Japanese Society of Civil Engineers recommends for ordinary structures a factor of safety equal to 1.5 using Meyerhof or Shinorhara-Tateishi Method and not using any dynamic properties of soil nor any dynamic analysis.

The Nuclear Regulatory Commission (NRC) of U.S.A. has never had any occasion to take the bearing capacity problem seriously. The NRC has not raised any objections where a factor of safety of 3 or more is set as a criteria using the psuedo-static approach.

DISCUSSION

The above facts indicate that there does not exist a true seismic bearing capacity method. Dynamic bearing capacity in clayey soils has never been a problem during large earhtquakes. For dry soils, Scott and Schoustra (1974) have referred to another form of instability (other than liquefaction) mentioned commonly in earthquake literature as "lurching" or "cracking" or both. Since the terms are not clearly defined, it was assumed to be surface manifestations of permanent ground displacements caused by earthquake vibrations. An examination based on simple analysis of the equivalent static forces and stresses acting on a column of soil and subjucted to lateral accelerations concluded that the so-called phenomenon of lurching or cracking can be observed only in slopes for dry soils, and then should be analysed on the basis of slope stability. On level sites therr may not be a major concern.

If a psuedo-static approach is used, a formula for eccentric and inclined loads sould be used. Solutions for the latter, yield significantly lower bearing capacities than for central vertical loads. In addition, the peak dynamic forces calculated by dynamic soil-structure interaction should not be used in the psuedo-static approach. This recommendation is based on the rationale that dynamic pressures are in general exaggerated because they neglect inelastic behavior of soils and also act for only a very short interval of time. It may also be pointed out that there does not exist an experimental justification for the validity of psuedo-static approach in clayey soils.

Recent experimental work on foundation response to soil transmitted load at Illinois Institute of Technology (Varadhi et. al., 1977) have disclosed some additional information. The experimental work was conducted on footing placed at surface and also at different depths of burial, the soil used being Ottawa sand. Transient loads were transmitted to the footing through the underlying soil by mechanically induced ground motions. It was found that the size and weight of the footing effect the stress distribution considerably. The attenuation of peak vertical stress was found to be very rapid in a distance equal to the diameter of the footing and at a distance equal to 1.5 times the diameter of the footing from the axis of symmetry, the affect of footing on vertical stress distribution was negligible. The higher the weight of the footing, the lower will be the base contact stress though it will still be many times greater than the static contact pressure. The influence of footing mass on the vertical stress distribution was found to extend down to a depth equal to twice the footing diameter. It was also found that the stress intensities in the vicinity of the footing increased with depth of burial (depth of embankment) of the footing, mainly because the distance from the transient load source decreased. Deceleration amplitude of the footing decreased with the embedment of the footing, thus indicating a greater damping and increased stiffness. (In the experiments the depth of burial was only one footing diameter and the soil above the footing has a negligible contribution to the weight of footing.)

RECOMMENDATIONS

A rational approach of evaluation should consider a dynamic response analysis of the soil structure system giving realist anelastic properties to the soil justified by dynamic laboratory tests and in situ tests. Dynamic properties of sandy soils have been studied in detail, however the dynamic properties of clayey soil are now under investigation. It is well known that the strength of cohesive soils is dependent on rate of loading. Transient loading of impulsive nature yields higher strengths (Taylor, 1967; Thiers and Seed, 1969; Richart, 1965). Cohesive soils under repeated cyclic loading may fail (in magnitude of strain) at cyclic stresses lower than the undrained static strength S_u (Thiers and Seed, 1969). The results generally depend on the ratio of the static deviator stress to the undrained static strength, ratio of dynamic superimposed strength to the undrained static strength, number of cycles and the failure criteria defined in terms of strain. According to Lee (1975), the strength after the cycling will be only slightly reduced, provided the additional dynamic loadings produce less strain than one half the normal static strain to failure.

Medearis (1975) investigated the response characteristics of full scale structures to the transient, earthquake-like ground motions resulted from an underground nuclear detonation. One of the projects involved the detonation of 3 - 30 kiloton nuclear devices more than a mile below ground level in western Colorado in 1973. Five structures were selected for installation of seismic recorders at the base. These structures were located