

# **Seismic Bearing Capacity and Settlements of Foundations**

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**ABSTRACT:** In the context of engineering practice, the problem of the seismic bearing capacity of shallow foundations has been solved indirectly, either due an increase of the static allowable soil pressures related to the probability of occurrence of the design earthquake or by adopting an equivalent pseudo-static approach. However, during last decades, a series of analytical methods that directly address the problem from the seismic point of view has been developed. This paper presents a parametric comparative analysis of different methods for estimating seismic bearing capacity of shallow strip foundations. Analytical methods, developed in the framework of both limit equilibrium and limit analysis theories, and also simplified design procedures typically used in practice were considered. The results obtained show an important decrease of the bearing foundation capacity with increasing of the maximum earthquake acceleration, which highlights the need to obtain a measure of the reliability associated with both calculation methods and safety factors commonly used for seismic design

**KEYWORDS:** limit equilibrium; pseudo-static; seismic bearing capacity; shallow foundation.

## **I. INTRODUCTION**

The bearing capacity of foundations under static loads has been an extensively studied topic in soil mechanics. On the basis of the works of Prandtl (1921) and Terzaghi (1943), several researchers (Caquot & Kerisel, 1953; Meyerhof, 1963; Hansen, 1970; Vesic, 1973; among others) have developed theories for estimating the load capacity of shallow foundations, considering the influence of geometry, embedment depth, and eccentricity and inclination of loads. Despite the inherent limitations of these theories (Bowles, 1996; Lee & Salgado, 2005), they have been successfully used by geotechnical and foundation engineers for many years. However, the problem of the seismic bearing capacity of shallow foundations is far from being considered solved.

first studies related to the seismic effect on the bearing capacity were those carried out by Meyerhof (1951; 1953) and Shinohara et al. (1960), who adopted a pseudo-static approach. By applying horizontal and vertical accelerations to the center of gravity of the structure, the problem is reduced to a static case of bearing capacity with inclined and eccentric loads (Soubra, 1999). Nevertheless, in these methods the inertial effects on the soil mass beneath the foundation are not considered and this could have important effects on the overall system response (Sarma & Iossifelis, 1990; Soubra, 1999; CEN Eurocode 8, 2004).

## **II. RELATED WORK**

Subsequently, researchers such as Sarma & Iossifelis (1990), Richards et al. (1993), Budhu & Al-Karni (1993), Kumar & Kumar (2003) and Choudhury & Subba Rao (2005) have studied the problem of the seismic bearing capacity of shallow strip foundations under vertical loads by using limit equilibrium and considering the inertia solutions which cannot be said to be an upper or a lower-bound one with respect to the exact solution (Soubra, effects in both soil and structure. Nonetheless, it is well known that the limit equilibrium method provides 1997); mainly because this method neglects the stress-strain relationship of the soil and, according to the mechanics of solids, this condition must be

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satisfied for a complete solution (Chen & Scawthorn, 1970). For this reason, authors such as Dormieux & Pecker (1995), Paolucci & Pecker (1997), Soubra (1997; 1999), Zhu (2000) and Ghosh (2008) have addressed this problem in a more rigorously way by using the upper bound theorem of limit analysis theory (Chen & Scawthorn, 1970; Chen, 1975) for an associated flow rule Mohr-Coulomb material. It has allowed obtaining a series of upper envelopes of the exact solution and thus establishing a comparison between the different existing theories. Moreover, during last years the problem of seismic bearing capacity of foundations on slopes or near to them has received attention (Choudhury & Subba Rao, 2006; Yang, 2009; Castelli & Motta 2010; Saada et al., 2011; Farzaneh et al., 2013). However, it has to be mentioned that, although the solutions obtained by using the upper bound theorem of limit analysis theory are rigorous ones and allows limiting the exact solution, they correspond, in general, to an unsafe estimate of the failure load. For this reason, at present the efforts are aimed to improve the existing solutions available in the literature.

The present paper makes up the deficiency by considering bearing capacity and settlements directly. First, the standard static bearing capacity formulation is reviewed briefly in order to develop a comparable dynamic limit-analysis model. This, in turn, allows the formulation of seismic bearing capacity factors  $N_y$ ,  $N_q$ , and  $N_c$  to compare to static values. At the same time, the model will depict how the pattern of slip surfaces changes with increasing acceleration to explain the shear flow and settlements observed in both the laboratory and the field. Computation of settlements for any earthquake trace is then possible using a sliding-block approach that will allow the engineer both to check existing foundations designed statically or, for new ones, to use a seismic design procedure to eliminate settlements or reduce them to an acceptable level.

### III. STATIC BEARING CAPACITY

The static bearing capacity or ultimate load that a footing can sustain is usually calculated by superposition where the contributions from soil cohesion  $c$ , soil unit weight  $\gamma$  and surcharge loading  $q$  are added together. Each contribution can be expressed as a bearing-capacity factor  $N_y$  to give the total capacity (Terzaghi 1943)

$$PL = cN_c + qN_q + \frac{1}{2} \gamma B N_\gamma$$

The factors are themselves functions only of the internal friction angle  $\phi$  which is assumed constant, and are derived using upper-bound limit analysis in which the same failure mechanism is assumed throughout. The classic two-dimensional slip-line field obtained by Prandtl (1921) as shown in Fig. 1 is the traditional failure mechanism used. The homogeneous soil of effective unit weight  $\gamma$  has Mohr-Coulomb shear strength characteristics  $c$  and  $\phi$  and can be considered as a rigid plastic body. For shallow foundations,  $d < B$ , the overburden is idealized as a surcharge  $q = \gamma d$ , thus neglecting any shear resistance on planes EF and AE. This slip field is theoretically correct only if the weight of the soil below the footing base is neglected, in which case, without a surcharge, the footing could support no load on cohesionless soil.

The basic Prandtl failure pattern has three regions: (1) An active or captured wedge of soil (region I) that moves downward; (2) a passive wedge in region III that moves upward, and (3) a logarithmic radial-fan transition zone in region II moving laterally as it rotates about A. For  $\phi = 0$ , region II becomes the quadrant of a circle and the active and passive wedge angles  $\theta_A$  and  $\theta_P$  are both  $\pi/4$ . For static equilibrium, the dimensionless Prandtl bearing-capacity factors for a weightless soil ( $\gamma = 0$ ) and smooth base are found to be (Jumikis 1962)



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### IV. SEISMIC BEARING CAPACITY

This Coulomb failure mechanism can be directly extended to the dynamic earthquake situation. Referring to Fig. 3, inertial body forces  $k_h'y$  and  $k_v'y$  due to accelerations  $k_{hg}$  and  $k_{vg}$ , which were set equal to zero for the static case, are now included. As in the static case, the density or unit weight  $\gamma$  and friction angle  $\phi$  are assumed constant. Force equilibrium gives the seismic counterpart .

where  $K_{AE}$  and  $K_{PE}$  are seismic earth pressure coefficients are

$$K_{AE} = \frac{\cos^2(\phi - \theta)}{\cos\theta \cos(\delta + \theta) \left\{ 1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta)}{\cos(\delta + \theta)}} \right\}^2} \dots\dots\dots(1)$$

$$K_{PE} = \frac{\cos^2(\phi - \theta)}{\cos\theta \cos(\delta + \theta) \left\{ 1 - \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta)}{\cos(\delta + \theta)}} \right\}^2} \dots\dots\dots(2)$$

where  $P_{AE}$  and  $P_{PE}$  the critical angles of rupture are

$$P_{AE} = a + \tan^{-1} \left\{ \frac{\sqrt{(1 + \tan^2 a)[1 + \tan(\delta + \theta)\cot a]} - \tan a}{1 + \tan(\delta + \theta)(\tan a + \cot a)} \right\} \dots\dots\dots(3)$$

$$P_{PE} = -a + \tan^{-1} \left\{ \frac{\sqrt{(1 + \tan^2 a)[1 + \tan(\delta - \theta)\cot a]} + \tan a}{1 + \tan(\delta + \theta)(\tan a + \cot a)} \right\} \dots\dots\dots(4)$$

where  $\theta = \tan^{-1} \frac{K_h}{(1 - K_v)}$  and  $a = \phi - \theta$

where  $P_{LE} = cN_{CE} + \gamma dN_{qE} + \frac{1}{2} \gamma B N_{\gamma E}$  is seismic bearing capacity

where  $N_{ce}$ ,  $N_{qE}$ ,  $N_{\gamma E}$  are seismic bearing- capacity factor

$$N_{\gamma E} = \tan p_A \left( \frac{K_P}{K_A} - 1 \right)$$

$$N_{qE} = \frac{K_P}{K_A}$$

$$N_{ce} = (N_q - 1) \cot \phi$$

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$\theta$	$\phi$	$\delta$	$K_{AE}$
10	10	5	1.05
10	10	10	1.08
10	10	15	1.12
10	10	20	1.17
10	10	25	1.23
10	10	30	1.32
10	10	35	1.43
10	10	40	1.57

TABLE 1: values of  $K_{AE}$ , when  $\Phi$  and  $\theta$  are constant .

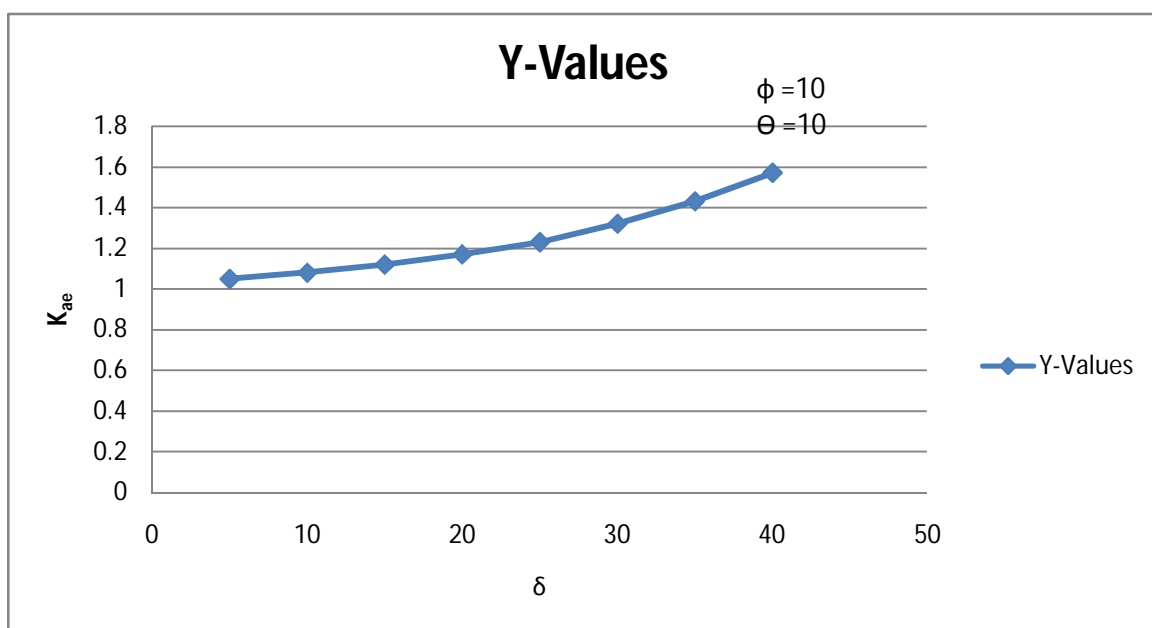


Fig. 1 : Graph Between  $K_{ae}$  &  $\delta$

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$\delta$	$\theta$	$\phi$	$K_{AE}$
10	10	5	0
10	10	10	1.08
10	10	15	0.7472
10	10	20	0.6136
10	10	25	0.5162
10	10	30	0.4334
10	10	35	0.3628
10	10	40	0.3019

TABLE 2 : values of  $K_{AE}$ , when  $\delta$  and  $\theta$  are constant .

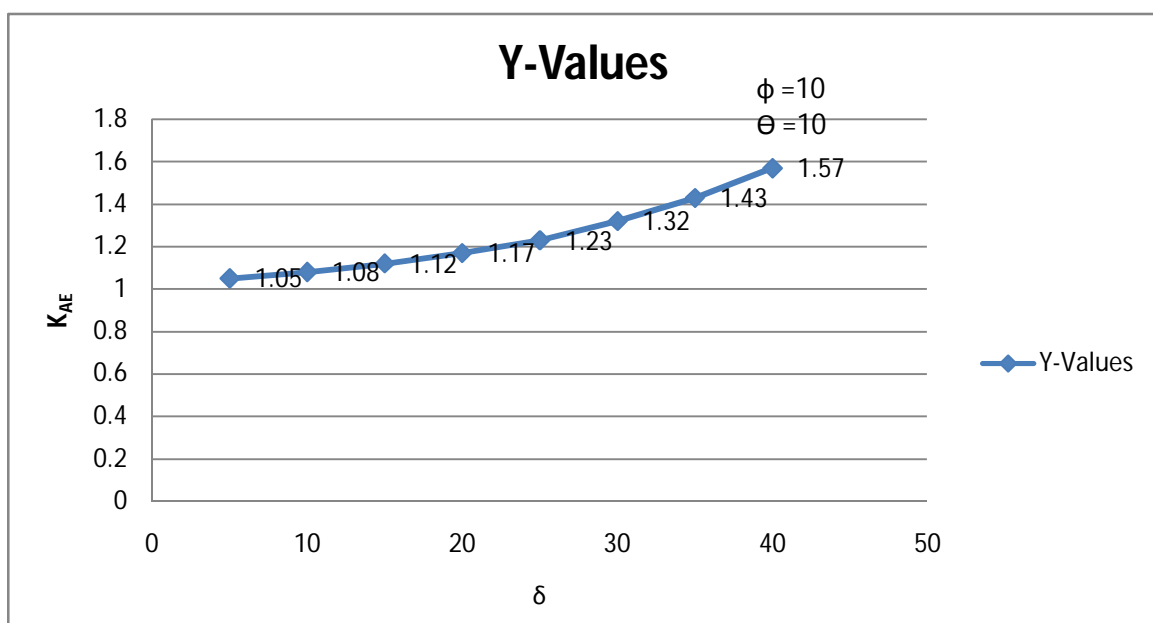


Fig. 2 : Graph Between  $K_{ae}$  &  $\delta$

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$\delta$	$\theta$	$\phi$	$K_{PE}$
5	10	5	0
5	10	10	1.08
5	10	15	1.66
5	10	20	2.16
5	10	25	2.77
5	10	30	3.57
5	10	35	4.66
5	10	40	5.05

TABLE 3 : values of  $K_{PE}$ , when  $\delta$  and  $\theta$  are constant .

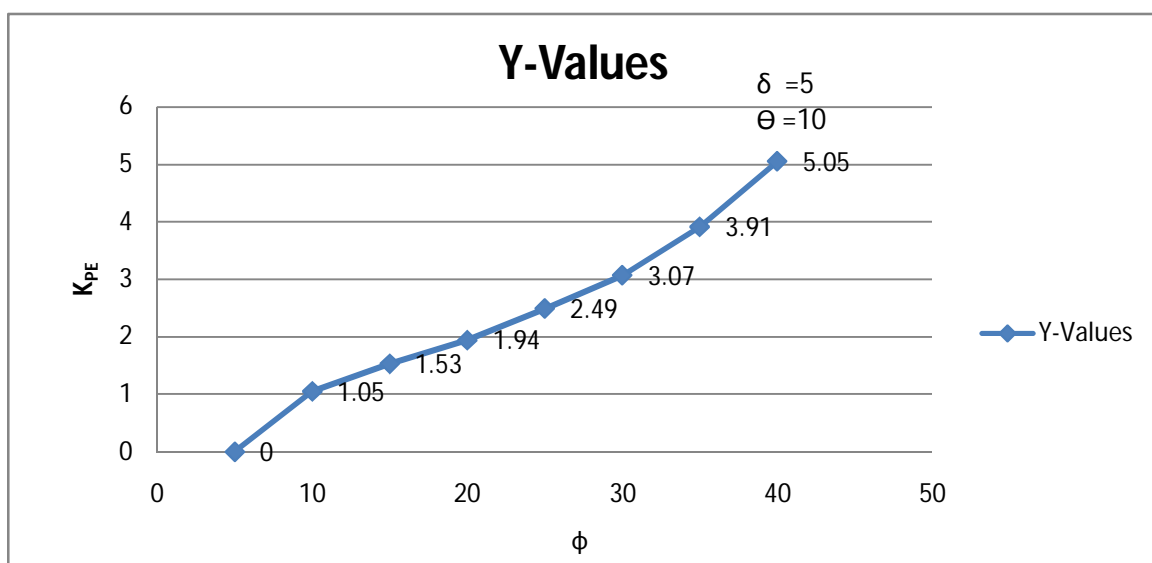


Fig. 3 : Graph Between  $K_{PE}$  &  $\delta$

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## V. SEISMIC SETTLEMENTS AND DESIGN

Analogous to the seismic movement of walls laterally, foundations will settle in a similar fashion in an earthquake whenever the acceleration ratio exceeds the critical value  $k_h$ . Even at moderate acceleration levels, the cost of foundations designed to stay below this critical value and avoid seismic settlement may be excessive and for larger earthquakes it may be prohibitive. In such cases, a displacement-governed design may be preferable or required.

The active-passive Coulomb wedge mechanism adopted in the present analysis allows a straightforward adaptation of the sliding-block approach used by Richards and Elms (1979) for retaining walls to calculate seismic displacements of foundations. (in which the movements are greatly exaggerated), the simple assumption is made that movement will take place only when sideways at the angle  $P_{Ae}$ , pushing the passive wedge sideways as required by the Coulomb mechanism. The usual sliding-block assumption is made that once movement starts, it continues at a constant acceleration until the relative velocity between the moving wedges and nonsliding soil reaches zero. Integration of the relative velocity gives the displacement along the active slip surface for a particular seismic pulse. Thus, displacement in an actual earthquake takes place in an incremental manner. The footing works its way downward, as it also moves backwards and forwards, in a series of small slip movements that are calculable for given record. The seismic settlements will be the computed displacement multiplied by the tangent of  $P_{Ae}$  calculated. The critical acceleration level is exceeded in either direction. Once sliding is initiated, the active wedge moves downward. The sliding-block displacements have been calculated for a number of standardized earthquake records by Newmark (1965), Franklin and Chang (1977), and others, leading to various approximate expressions for displacements (Whitman 1990). The straight-line relation suggested by Richards and Elms (1979) is appropriate for footing settlements, as the expected displacements are relatively small.

## VI. CONCLUSION

A seismic limit-analysis mechanism for computing bearing capacity and settlement by the classical upper-bound approach has been presented and explored in some detail. While this Coulomb active-passive wedge mechanism is not exact, it allows the straightforward development of seismic bearing-capacity factors directly related to their static counterpart. The comparison of the two depicts clearly the rapid deterioration of foundation strength with increasing acceleration. This, in turn, explains observations both in the field and in the laboratory of seismic bearing failures and excessive settlements that are not attributable to either liquefaction or other changes in soil properties. Within the stated assumptions, the seismic bearing-capacity factors derived and the simple sliding-block procedure for computing settlements (or for design to limit settlements) are both considered conservative. Thus, while many aspects of this solution can be refined, the straightforward approach presented can serve to give both better fundamental insight into this aspect of earthquake engineering and to develop better procedures for the design of foundations in seismic zones.

## REFERENCES

1. Terzaghi, K. & Peck, R. B. (1967). *Soils mechanics in engineering practice*. J. Wiley, New York.
2. Hansen, J. B. (1970). A revised and extended formula for bearing capacity. *Danish Geotech. Inst. Bulletin*, No.28, Denmark.
3. De Beer, E. E. (1970). *Experimental determination of shape factors and the bearing capacity factors of sands*. *Géotechnique*, Vol. 20, No. 4, pp. 387-411.
4. Vesic, A. S. (1973). *Analysis of ultimate loads of shallow foundations*. *J. of Soil Mech. and Fndn Div., ASCE*, Vol. 99(SM1), pp. 45-73.
5. Golder, H. Q. (1941). *The ultimate bearing pressure of rectangular footings*. *J. Instn Civ. Engrs*, Vol. 17, No. 2, pp. 161-174
6. Shield, R. T. & Drucker, D. C. (1953). The application of limit analysis to punch-indentation problems. *J. of Appl. Mech., ASCE*, pp. 453-460.
7. Shield, R. T. (1955). On the plastic flow of metals under conditions of axial symmetry. *Proc. Roy. Soc. of London A*, Vol. 233, pp. 267-287
8. Cox, A. D., Eason, G. & Hopkins, H. G. (1961). Axially symmetric plastic deformations in soils, *Phil. Trans. Roy. Soc. of London A*, Vol. 254, No. 1036, pp. 1-45.
9. Nakase, A. (1981). Bearing capacity of rectangular footings on clays of strength increasing linearly with depth. *Soils and Foundations*, Vol. 21, No. 4, pp. 101-108.
10. Ugai, K. (1985). Bearing capacity of square and rectangular footings on nonhomogeneous clays. *J. of JSSMFE*, Vol. 25, No. 4, pp. 179-185.
11. Narita, K., Yamaguchi, H. (1992). Three-dimensional bearing capacity analysis of foundations by use of a method of slices. *Soils and Foundations*. Vol. 32, No. 4, pp. 143-155
12. Michalowski, R. L. (2001). Upper-bound load estimates on square and rectangular footings. *Géotechnique*, Vol. 51, No. 9, pp. 787-798



# International Journal of Innovative Research in Science, Engineering and Technology

(An ISO 3297: 2007 Certified Organization)

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13. Michalowski, R. L., Dawson, E. M. (2002). Three-dimensional analysis of limit loads on Mohr-Coulomb soil. *Fndn of Civ. and Inv. Eng.*, Vol. 1, pp. 137-147.
14. Richards, R., Elms, D. G., and Budhu, M. (1990). "Dynamic fluidization of soils." *J. Geotech. Engrg., ASCE*, 116(5), 740-759
15. Sarma, S. K., and Iossifelis, I. S. (1990). "Seismic bearing capacity factors of shallow strip footings." *G-otechnique*, London, England, 40(2), 265-273.
16. Sokolovsky, V. V. (1960). *Statics of soil media*. Translated by D. H. Jones and A. N. Schofield, Butterworths, London, England.
17. Terzaghi, K. (1943). *Theoretical soil mechanics*. John Wiley & Sons, Inc., New York, N.Y.
18. Whitman, R. V. (1990). "Seismic design and behavior of gravity walls," *Design and Performance of Earth Retaining Structures*, Geotech. Spec. Pub. No. 25, Proc., ASCE Spec. Conf., ASCE, New York, N.Y., 817-842
19. Terzaghi, K. (1943). *Theoretical soil mechanics*. John Wiley & Sons, Inc., New York, N.Y..