INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

The paper was published in the proceedings of the 7th International Conference on Earthquake Geotechnical Engineering and was edited by Francesco Silvestri, Nicola Moraci and Susanna Antonielli. The conference was held in Rome, Italy, 17 – 20 June 2019.

Earthquake Geotechnical Engineering for Protection and Development of Environment and Constructions – Silvestri & Moraci (Eds)
© 2019 Associazione Geotecnica Italiana, Rome, Italy, ISBN 978-0-367-14328-2

Assessment of pseudostatic methods for the seismic bearing capacity of footings in code-based design

C. Vrettos & E. Seibel
Technical University Kaiserslautern, Kaiserslautern, Germany

ABSTRACT: The common method for assessing the bearing capacity of foundations under seismic loading is based on the assumption of pseudostatic acceleration forces which represent the inertial forces from the superstructure and those in the ground. The components of the classic tripartite bearing capacity equation are provided with suitable reduction factors for this purpose. An alternative representation are dome-shaped 3-D bearing strength surfaces in terms of combinations of vertical load, horizontal load, and bending moment that lead to failure. The bearing capacity equation can also be converted into such surfaces. 3D-surfaces are evaluated and compared for the two limiting cases of purely cohesive or cohesionless soil. Discrepancies observed are analysed. A widely used FEM code with a Mohr-Coulomb constitutive model is applied to reproduce available solutions for typical configurations. Details of the numerical modelling are given. It is concluded that the extended bearing capacity equation is particularly suitable for the application in design, and also offers the advantage of a steady transition from the non-seismic to the seismic loading case.

1 INTRODUCTION

Although the design of shallow foundations is dominated by serviceability aspects and bearing failures are extremely rare (except in the case of liquefaction), the quantification of the bearing capacity provides information on the utilization degree of the soil strength and can thus be used for estimates of the effective stiffness of the non-linear soil in serviceability analyses. Furthermore, in the ongoing evolution of the pertinent Eurocodes EN 1997 and EN 1998 compatibility of designs for the non-seismic and the seismic loading case is of great importance. This was the main motivation for the study presented herein.

Customarily, bearing capacity verification is carried out using the classical three-term equation where the effects of cohesion, foundation width and embedment depth are considered as a linear combination of three components accounting for the effects of cohesion, soil weight, and lateral surcharge. For inclined loading appropriate reduction factors are applied; while load eccentricity is accounted for by a reduced foundation footprint area. A typical example is the set of equations given in EN 1997-1 that is based on a former version of the German Standard DIN 4017. For seismic loading, inertial and kinematic response can be considered within the context of a pseudostatic approach: i) eccentric and inclined inertial forces as imposed by the superstructure are treated as static actions, and ii) acceleration forces in the soil mass are captured by appropriate reduction factors applied to the bearing capacity factors, Paolucci & Pecker (1997). Detailed overviews are given by Cascone & Casablanca (2016), Pane et al. (2016), Conti (2018). For the application in design, partial safety factors for actions and resistances are provided by the pertinent building codes.

An alternative consists in defining 3D bearing strength surfaces (BSS) by means of a single equation comprising combinations of vertical force, horizontal force and moment that induce bearing failure of the foundation. Soil inertia effects are included by appropriate reduction factors. Such equations are often less comprehensive for the user compared to the familiar three-term equation. A typical BSS is the one in EN 1998-5 that has been developed by Pecker

(1997). Inertia effects reflected by this equation have been analysed very recently by Pender (2018). Similar BSS may also be constructed by transformation of the three-term equation using either algebra or numerical analysis by common spreadsheet programs, as shown by Pender (2017) for the classical static case and by Vrettos & Seibel (2017) (2018) for pseudo-static seismic analysis. The aim of this paper is the comparison of the explicit equation for BSS in EN 1998-5 with those derived from the bearing capacity equations in EN 1997-1/DIN 4017 enhanced by appropriate factors accounting for soil inertia effects.

2 EXPLICIT SOLUTION IN EN 1998-5

In this convenient representation, all actions are normalized with respect to the ultimate bearing force for centric vertical loading $V_{\max}$, and the contributions by the horizontal force H and moment M are given by separate terms. The respective equation in EN 1998-5, Annex F is for strip foundations resting either i) on purely cohesive soil of undrained shear strength s_u, or ii) on cohesionless soil of friction angle φ. A lateral surcharge is not included. That equation reads:

$$\frac{(1-e\cdot\overline{F})^{c_T}(\beta\cdot\overline{H})^{c_T}}{\overline{V}^a\left(\left(1-m\cdot\bar{F}^k\right)^{k'}-\bar{V}\right)^b}+\frac{(1-f\cdot\overline{F})^{c'_M}(\gamma\cdot\overline{M})^{c_M}}{\overline{V}^c\left(\left(1-m\cdot\bar{F}^k\right)^{k'}-\bar{V}\right)^d}-1\leq 0 \tag{1}$$

where

$$\bar{V}=\frac{V}{V_{\max}} \quad , \quad \bar{H}=\frac{H}{V_{\max}} \quad , \quad \bar{M}=\frac{M}{V_{\max}} \tag{2}$$

with

$$V_{\max}=s_u\cdot b\cdot N_c \text{ for cohesive soil} \quad V_{\max}=\frac{1}{2}\gamma\cdot B\cdot N_\gamma \text{ for cohesionless soil} \tag{3}$$

where γ = unit weight of the soil; b = width of the foundation, N_c and N_γ = familiar bearing capacity factors. The fourteen constants in Equation 1 have been determined by curve fitting to the theoretical solution by Pecker (1997) and have no further physical meaning. They are given in EN 1998-5 and are omitted here for the sake of brevity. $\bar{F}$ is a dimensionless inertia force for the soil:

$$\bar{F}=\frac{k_h\cdot\gamma\cdot b}{s_u} \text{ for cohesive soil} \quad \bar{F}=\frac{k_h}{\tan\varphi} \text{ for cohesionless soil} \tag{4}$$

where k_h = pseudostatic horizontal seismic coefficient.

We used a slightly different notation compared to EN 1998-5, set all partial safety factors equal to 1 in order to remove ambiguity, and ignored for simplicity the vertical seismic action component. Further, we do not express Equations 4 in terms of spectral acceleration and soil factors in order to keep the analysis more general. We thus assume that an appropriate value k_h has already been specified.

Note that in the normalized form the exact values of N_c and N_γ are not needed, and their selection is left to the user.

3 DERIVED IMPLICIT SOLUTION

The adaption of the classical bearing capacity equation for a strip footing to include structure and soil mass inertia forces reads:

$$R_n/b' = c \cdot N_c \cdot i_c \cdot e_{c,E} + 0.5 \cdot \gamma \cdot b' \cdot N_\gamma \cdot i_\gamma \cdot e_{\gamma,E} + q \cdot N_q \cdot i_q \cdot e_{q,E} \tag{5}$$

where R_n = component of the bearing load normal to the footing base; b' = modified width of the eccentrically loaded strip footing of width b; γ = unit weight of the soil; c = cohesion; q = lateral surcharge; N_c, N_γ, N_q = bearing capacity factors; i_c, i_γ, i_q = load inclination factors; $e_{c,E}$, $e_{\gamma,E}$, $e_{q,E}$ = soil inertia factors that capture the inertia effects in the foundation soil.

Use of Equation 3 implies that the soil is a cohesive-frictional material with linear Mohr-Coulomb failure envelope, and cohesion and friction mobilized at the same shear strain level. For total stress analyses, c is replaced by the undrained shear strength s_u.

The bearing capacity factors N_q and N_c are known exactly,

$$N_q = \tan^2(45^\circ + \varphi/2) \cdot e^{\pi \cdot \tan\varphi} \quad ; \quad N_c = \left(N_q - 1\right) \cdot \cot\varphi \tag{6}$$

and degenerate for purely cohesive soil to
$N_q = 1 \quad ; \quad N_c = 2 + \pi$ for

$$\varphi = 0 \tag{7}$$

For N_γ on the other hand, numerous expressions have been suggested during the past decades, Orr (2010), Diaz-Segura (2013). The EN 1997-1 equation derives from that given in DIN 4017 and reads:

$$N_\gamma = 2 \cdot \left(N_q - 1\right) \cdot \tan\varphi \tag{8}$$

It comes from extensive large scale tests carried out in the 1960's and 1970's by the Degebo in Germany, Muhs & Weiss (1973). It is much higher than any solution determined either by the slip lines method, e.g. Martin (2005), Smith (2005), or by other numerical continuum methods assuming elasto-plastic soil, Cascone & Casablanca (2016), Vrettos & Seibel (2018). These tests are well documented in a series of detailed reports including investigations on the effects of embedment, load inclination and eccentricity, as well as on the foundation shape. Eventually, the user is free to select the solution he trusts more.

Investigations by Smith (2005) to assess the effects of superposition in the bearing capacity equation in cohesionless soil show that coupling of the two effects (lateral surcharge and soil weight) results in an increase in overall bearing resistance, a fact that provides an additional margin of safety.

Load eccentricity is customarily considered by reducing the strip foundation width b to an effective one b' following the suggestion by Meyerhof (1963):

$$b' = b \cdot (1 - 2 \cdot e/b) \quad , \quad e = M/V \tag{9}$$

The inclination factors given in EN 1997-1 depend on c and φ while in the current edition of DIN 4017 (2006) this dependency has been retained only for i_c. The DIN-equations read:

$$i_c = \frac{i_q \cdot N_q - 1}{N_q - 1} for\ \varphi > 0 \tag{10}$$

$$i_c = 0.5 + 0.5\sqrt{1 - \frac{H}{b' \cdot c}} for\ \varphi = 0 \tag{11}$$

$$i_\gamma = \left(1 - \frac{H}{V}\right)^3 \tag{12}$$

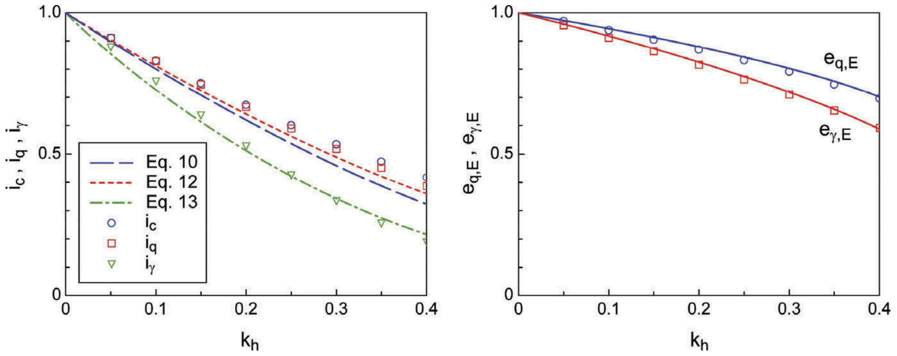

Figure 1. Inclination factors for the structure-inertia (left) and soil inertia factors (right) dependent on the seismic coefficient k_h for φ = 30°. The data points are from Cascone & Casablanca (2016), the lines for Equations 10, 12, 13 and Equations 15, 16, respectively.

$$i_q = \left(1 - \frac{H}{V}\right)^2 \tag{13}$$

The above equations were compared by the authors with the numerical results by Cascone & Casablanca (2016) showing very good agreement, in particular for i_q and i_γ, cf. Figure 1. The adequacy of the simplified incorporation of load eccentricity via Equation 6 has also been verified by several investigations as will be shown in the following.

With regard to the soil inertia factors, it has been demonstrated by various studies including Pane et al. (2016), Cascone & Casablanca (2016) that for all practical purposes

$$e_{c,E} = 1 \tag{14}$$

The other two factors are determined herein by curve fitting from the numerical values by Cascone & Casablanca (2016) that have been also verified by Vrettos & Seibel (2018),

$$e_{\gamma,E} = (1 - \bar{F})^{0.45} \tag{15}$$

$$e_{q,E} = (1 - \bar{F})^{0.3} \tag{16}$$

with $\bar{F}$ from Equation 4. A plot is shown in Figure 1. These values are also in good agreement with the findings by Pane et al. (2016) and Paolucci & Pecker (1997). It should be mentioned that $e_{\gamma,E}$ depends on the contact conditions at the interface footing-soil, the exponent 0.45 closely fitting rough contact that prevails in real-life situations.

Neglecting the lateral surcharge by setting $q = 0$, the bearing capacity Equation 3 can be formulated in terms of forces and moments. Incorporating $e_{c,E}$ and $e_{\gamma,E}$, yields:

$$\bar{V} - \frac{1}{2} \cdot \left(1 - 2 \cdot \frac{\bar{M}}{\bar{V}}\right) \cdot \left(1 + \sqrt{1 - \frac{\bar{H} \cdot N_c}{(1 - 2 \cdot \bar{M}/\bar{V})}}\right) \cdot e_{c,E} = 0 \; for \; \varphi = 0 \tag{17}$$

$$\bar{V} - \left(1 - 2 \cdot \frac{\bar{M}}{\bar{V}}\right)^2 \cdot \left(1 - \frac{\bar{H}}{\bar{V}}\right)^3 \cdot e_{\gamma,E} = 0 \; for \; c = 0 \tag{18}$$

The radicand in Equation 17 must be larger than or equal to zero, thus assuring that sliding is prohibited by limiting the horizontal load. The presence of the square root introduces limitations with respect to the maximum allowable moment and horizontal force, cf. Vrettos (2019). Despite the criticism expressed for the incorporation of soil strength into an inclination factor, the square root in Equation 11 and accordingly in Equation 17 ensures equilibrium along the foundation-soil interface.

If one is annoyed by the square root and prefers instead the inclination factors suggested by Meyerhof (1963), the bearing strength surfaces become:

$$\bar{V} - \left(1 - 2 \cdot \frac{\bar{M}}{\bar{V}}\right) \cdot \left(1 - \frac{\arctan(\bar{H}/\bar{V})}{\pi/2}\right)^2 \cdot e_{c,E} = 0 \; for \; \varphi = 0 \tag{19}$$

$$\bar{V} - \left(1 - 2 \cdot \frac{\bar{M}}{\bar{V}}\right) \cdot \left(1 - \frac{\arctan(\bar{H}/\bar{V})}{\varphi}\right)^2 \cdot e_{\gamma,E} = 0 \; for \; c = 0 \tag{20}$$

with φ given in radians. The bound imposed by the sliding resistance in purely cohesive soil has to be applied in any case, yielding an additional inequality to be fulfilled. The same applies of course to Equation 1. It will be shown that at high values of M and H sliding resistance dominates the design.

4 COMPARISON OF BEARING STRESS SURFACES

First of all, for the limiting case of vanishing soil inertia effects, i.e. $\bar{F}$= 0, any seismic design equation should conform to the static solution, if a pseudo-static approach is adopted. Extensive numerical studies carried out recently by Pane et al. (2016), Cascone & Casablanca (2016) confirmed that the use of static load inclination factors in combination with the classical decoupled bearing capacity equation provides an adequate and sufficiently accurate solution.

As pointed out by Pender (2018), due to the requirements of an adequate safety factor for permanent loading situations, the range of values for the vertical load relevant in practice is confined to $\bar{V}$ < 0.4. Even so, we consider in our graphs the entire range 0 < $\bar{V}$ < 1 for the sake of completeness.

We first consider *purely cohesive soil* (undrained case). Figure 2 compares *H-V* and *M-V* sections of the BSS for $\bar{F}$= 0.

In Equation 1 the dependency on $\bar{F}$ is noticeable only for $\bar{V}$ > 0.4 for common cases, and for $\bar{V}$ > 0.3 for very large values $\bar{F}$, cf. Pender (2018). Hence, with a clear conscience one can remove the dependency on $\bar{F}$ in Equation 1 by setting $e, k, k', f, c_M = 0$ for purely cohesive soil. We thus consider merely $\bar{F}$ = 0 in our comparisons for purely cohesive soil. It can be deduced from Figure 2 that there is a considerable deviation between the two solutions, Equation 1 vs. Equation 17, in particular for high values $\bar{M}$ and $\bar{H}$. At a typical value $\bar{V}$ = 0.35 and

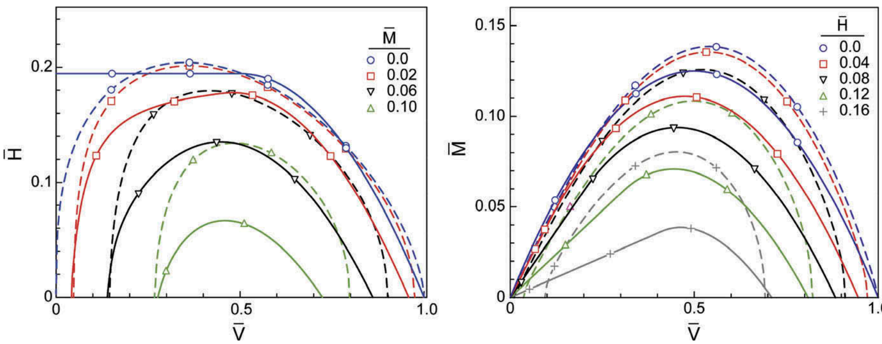

Figure 2. Contour lines in the $\bar{V}$-$\bar{H}$ and $\bar{V}$-$\bar{M}$ sections of the BSS for purely cohesive soil (undrained case) for $\bar{F}$= 0. Solid lines are for Equation 17 from EN 1997-1/DIN 4017, dashed lines are for Equation 1 from EN 1998-5.

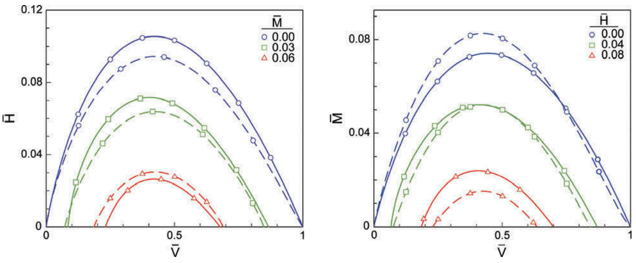

Figure 3. Contour lines in the $\bar{V}$-$\bar{H}$ and $\bar{V}$-$\bar{M}$ sections of the BSS for cohesionless soil (drained case) for $\bar{F}$= 0. Solid lines are for Equation 18 from EN 1997-1/DIN 4017, dashed lines are for Equation 1 from EN 1998-5.

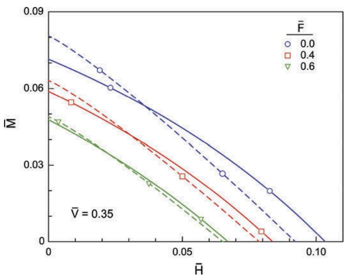

Figure 4. $\bar{H}$-$\bar{M}$-relationship at various$\bar{F}$ for fixed $\bar{V}$and for the drained case. Solid lines are for Equation 18 from EN 1997-1/DIN 4017, dashed lines are for Equation 1 from EN 1998-5.

for $\bar{H}$= 0.08, for example, the deviation is 24% on the unsafe side. Hence, Equation 1 needs improvement in order to comply with the static solution of the familiar bearing capacity equation. Unless, Equation 1 is adopted for static routine design as well.

Cohesionless soil (drained case) is considered next. Figure 3 compares Equation 1 and Equation 18 for$\bar{F}$= 0 showing satisfactory agreement. The effect of $\bar{F}$ on the relationship$\bar{M}$ vs. $\bar{H}$at given $\bar{V}$ as expressed by Equation 1 and Equation 18, respectively, is reproduced in Figure 4 also showing an acceptable agreement.

For completeness, we compare in Figure 5 the $\bar{M}$ vs. $\bar{V}$sections of the BSS defined by Equation 1 and Equation 19 by including for Equation 19 the limitation of the horizontal

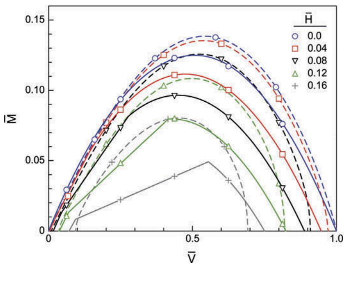

Figure 5. Contour lines in the $\bar{V} - \bar{M}$ sections of the BSS for the undrained case for $\bar{F}$= 0. Solid lines are for Equation 19 derived from Meyerhof (1963), dashed lines are for Equation 1 from EN 1998-5.

force imposed by the horizontal equilibrium (sliding) requirement at the foundation-soil interface. Since the line defined by the latter requirement shall apply to all solutions, it constitutes a bound also for Equation 1. A comparison of Figure 5 with Figure 2 reveals that Equation 17 and Equation 19 yield similar results, whereas differences are observed between Equations 17, 19 and Equation 1.

5 EFFECT OF LOAD ECCENTRICITY

The numerical procedure adopted by the authors to validate the analytical solutions for N_c und N_q and the numerical slip lines solutions for N_γ is described in Vrettos & Seibel (2018) and follows the suggestions by Loukidis & Salgado (2009) and Cascone & Casablanca (2016). The widely used FEM code PLAXIS 2D (2017) with an elasto-plastic Mohr-Coulomb constitutive model is applied. An associated flow rule is assumed; an assumption that is also implied by the analytical

plasticity solutions. The pseudostatic action within the soil mass is modelled by a horizontal volume force $k_h \cdot \gamma$. The bearing load is determined by successively increasing the force exerted on the footing. Load step size is automatically adjusted by the algorithm. The FE-mesh is 30 m wide and 10 m deep. The surface strip footing is 2 m wide with a bending stiffness $E \cdot I$ = $2.11 \cdot 10^6$ kNm^2/m. A modulus of elasticity E_{soil} = 30 MPa and a Poisson's ratio ν = 0.2 are assigned to the soil. The contact between soil and footing is assumed perfectly rough.

The three components of the bearing resistance have been considered separately. Table 1 displays selected results for φ = 30°. Equation 6 is compared against the numerical FEM solution. For this purpose, we determine from the numerical analyses the ratio $R_{n,e>0}/R_{n,e=0}$ where R_n denotes the resistance normal to the footing base, as used in Equation 3. For the cohesion and lateral surcharge components $R_{n,e>0}/R_{n,e=0}$ = b'/b, whereas for the component due to soil weight $R_{n,e>0}/R_{n,e=0} = (b'/b)^2$. For example, for φ = 30° and e/b = 0.25 the simplified Equation 6 yields b'/b = 0.50, while from the FEM analyses we obtain for the three bearing resistance components b'/b = 0.54/0.52/0.51. The results confirm the validity of the simplified approach according to Equation 6. Similar findings are reported by Krabbenhoft et al. (2012).

Table 1. Influence of load eccentricity for the particular bearing capacity components; ultimate limit load R_n in kN/m, b = 2 m, φ = 30°.

	Eq. 9	$q = 0, \gamma = 0$		$c = 0, \gamma = 0$		$c = 0, q = 0$	
e/b	b'/b	R_n	b'/b	R_n	b'/b	R_n	b'/b
0.00	1.0	803.0	1.00	628.4	1.00	775.5	1.00
0.125	0.75	638.6	0.80	506.5	0.81	493.8	0.80
0.250	0.50	432.5	0.54	329.3	0.52	202.5	0.51
0.375	0.25	226.3	0.28	55.7	0.09	50.2	0.26

6 CONCLUSIONS

Equation 1 presently included in EN 1998-5 needs improvement at least for the case of purely cohesive soil in order to adequately match the static solution provided by the companion code EN 1997-1 and other design codes that adopt the classical three-term bearing capacity equation. The large number of constants is disproportional to the nature of the problem and the range of situations covered. It is evidently related to the inherent difficulty in expressing the BSS by two independent terms accounting for the effects of bending moment and horizontal shear, respectively. Although this separation is very instructive, the application of Equation 1 will always need some kind of elementary numerical implementation making the need of decoupling at the cost of a large number of constants questionable. Equations 17 and 18 on the other hand, derived from the bearing capacity equation with appropriate inclination factors corrected for the effects of soil inertia have a much more condensed form and can be implemented very easily numerically. They possess the major advantage of a smooth transition from the non-seismic to the seismic case in code-based routine design. This feature is essential for regions of moderate seismicity. Furthermore, general BSS derived from the bearing capacity equation by simple numerical means may incorporate foundation geometry, lateral surcharge and cohesive-frictional soil.

REFERENCES

Cascone, E. & Casablanca, O. 2016. Static and seismic bearing capacity of shallow strip footings. *Soil Dynamics and Earthquake Engineering* 84: 204–223.

Conti, R. 2018. Simplified formulas for the seismic bearing capacity of shallow strip foundations. *Soil Dynamics and Earthquake Engineering* 104: 64–74.

Diaz-Segura, E.G. 2013. Assessment of the range of variation of N_γ from 60 estimation methods for footings on sand. *Canadian Geotechnical Journal* 50: 793–800.

DIN 4017:2006-03. Baugrund – Berechnung des Grundbruchwiderstands von Flachgründungen.

EN 1997-1:2004. Eurocode 7 – Geotechnical design, Part 1: General rules. Brussels: European Committee for Standardization.

EN 1998-5:2004. Eurocode 8 – Design of structures for earthquake resistance, Part 5: Foundations, retaining structures and geotechnical aspects. Brussels: European Committee for Standardization.

Krabbenhoft, S., Damkilde, L. & Krabbenhoft, K. 2012. Lower-bound calculations of the bearing capacity of eccentrically loaded footings in cohesionless soil. *Canadian Geotechnical Journal* 49: 298–310.

Loukidis, D. & Salgado, R. 2009. Bearing capacity of strip and circular footings in sand using finite elements. *Computers and Geotechnics* 36: 871–879.

Martin, C.M. 2005. Exact bearing capacity calculations using the method of characteristics. In *11th International Conference IACMAG, Torino* 4: 441–450.

Meyerhof, G.G. 1963. Some recent research on the bearing capacity of foundations. *Canadian Geotechnical Journal* 1(1): 16–26.

Muhs, H. & Weiss, K. 1973. Inclined load tests on shallow strip footings. In *Proceedings of the VIII International Conference on Soil Mechanics and Foundation Engineering*, Moscow, 1.3: 173–179.

Orr, T.L.L. 2010. Discussion of White et al. 2008. A comparison of the bearing capacity of flat and conical circular foundations on sand. *Géotechnique* 58(10): 781–792.

Pane, V., Vecchietti, A. & Cecconi, M. 2016. A numerical study on the seismic bearing capacity of shallow foundations. *Bulletin of Earthquake Engineering* 14(11): 2931–2958.

Paolucci, R. & Pecker, A. 1997. Seismic bearing capacity of shallow strip foundations on dry soils. *Soils and Foundations* 37(3):95–105.

Pecker, A. 1997. Analytical formulae for the seismic bearing capacity of shallow strip foundations. In P. Sêco e Pinto(ed.), *Seismic Behavior of Ground and Geotechnical Structures*: 261–268. Rotterdam: Balkema.

Pender, M.J. 2017. Bearing strength surfaces implied in conventional bearing capacity calculations. *Géotechnique* 67(4): 313–324.

Pender, M.J. 2018. Earthquake inertia effects on shallow foundation bearing strength. *Géotechnique* 68 (7): 640-645.

PLAXIS 2D, Reference Manual, 2017.

Smith, C.C. 2005. Complete limiting stress solutions for the bearing capacity of strip footings on a Mohr-Coulomb soil. *Géotechnique* 55(8): 607–612.

Vrettos, C. 2019. Discussion of Pender 2017. Bearing strength surfaces implied in conventional bearing capacity calculations. *Géotechnique* (in print). doi: 10.1680/jgeot.18.D.005.

Vrettos, C. & Seibel, E. 2017. Bearing capacity of strip footings in regions of medium seismicity: Reappraisal of the pseudostatic approach in code-based design in light of recent computational results. In *Proceedings of the 3rd International Conference on Performance-Based Design in Earthquake Geotechnical Engineering*, Vancouver, Canada, Paper 204.

Vrettos, C. & Seibel, E. 2018. Bewertung von pseudo-statischen Methoden zum Grundbruchwiderstand von Fundamenten unter seismischer Beanspruchung. *Bautechik* 95(12): 859–871.