Review of Methods to Predict the Bearing Capacity of Shallow Foundations

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ABSTRACT - *Numerous research works have been carried out in the last few decades for the estimation of ultimate bearing capacity of shallow foundations in cohesionless soils through experimental studies on model footings and theoretical analyses. In the recent past, centrifuge modeling and finite element analysis (FEA) are utilised either independently or together to understand the mechanisms of the above problem by simulating all possible conditions which are normally not possible by classical methods and 1g model tests. The objective of this paper is to present some of the rigorous works carried out so far using the above methods and to bring out the limitations of them.*

KEYWORDS: Bearing capacity, Shallow foundations, Limitations, Scale effect.

1. INTRODUCTION

Foundation is a substructure element which provides support for the superstructure and its loads. It includes the soil of the earth's crust and part of the structure which serves to transmit the loads into the soil. A shallow foundation is one in which the structural loads are transmitted to the soil at an elevation required for the function of the structure itself (Leonards 1973) [1]. The load-settlement response of shallow foundations forms one of the important links between the structural and geotechnical engineering since it gives the knowledge of foundation deformation which is required to ensure the serviceability and/or the safety of the structure.

Foundations must be designed to resist not only axial compressive forces, but also uplift (pullout) forces originating from wind load or wave action or overturning moment acting on a structure.

In the design of foundations, it must be ensured that the foundation meets the basic considerations of safety against failure and tolerable settlements. The requirement of safety against failure is centered on the bearing capacity failure of the supporting soils both under axial compression and uplift forces, which occurs as a shear failure of the soil supporting the foundation. Therefore, the shear strength of surrounding soil should be adequate to resist the compressive or tensile force as the case may be, to provide structural stability. The ability of soil to resist compressive force is quantified in terms of ultimate bearing capacity. Many methods by theoretical, empirical and numerical approaches have been formulated for designing foundations against compression forces.

In this paper, the classical bearing capacity theories with their limitations are discussed first. This is followed by investigations using other theoretical methods, model and centrifuge tests and full-scale footings.

2. CLASSICAL BEARING CAPACITY THEORIES AND THEIR LIMITATIONS

Bearing capacity of foundations is generally determined through limit equilibrium, limit analysis and slip-line solutions (method of characteristics) and it has been extensively studied by several researchers, namely Terzaghi (1943) [2], Meyerhof (1950)[3], Caquot and Kérisel (1966) [4] and Zhu et al (2001)[5] using limit equilibrium methods, Caquot and Kérisel (1953)[6], Lundgren and Mortensen (1953)[7], Hansen (1961)[8], Sokolowski (1965)[9] and Bolton and Lau (1993) [10] using slip-line methods, Shield (1954)[11] , Chen (1975)[12], Michalowski and Shi (1995) [13], Michalowski (1997)[14] and Soubra (1999)[15] using limit analysis (as in Silvestri 2003) [16]. The limit equilibrium has been the most widely used method in stability analysis of foundations and slopes owing to its simplicity and reasonably good prediction of failure loads.

The classical theory of plasticity has been widely used to develop a solution for the case of general shear failure, typical of soils possessing brittle-type stress-strain behaviour using theory of plasticity concept. Prandtl (1921)[17] and Reissner (1924)[18] have found that for a rectangular foundation of width B and Length L with depth of embedment D_f and for weightless soil (unit weight of soil γ = 0), the ultimate bearing capacity (q_u) can be calculated by the following expression:

$$q_u = cN_c + qN_q \tag{1}$$

where c is the cohesion and q is the surcharge pressure at the foundation base. N_c and N_q are the bearing capacity factors given by the following expressions:

$$N_q = e^{\pi \tan\phi} \tan^2(\pi/4 + \phi/2) \tag{2}$$

$$N_c = (N_q - 1)\cot\phi \tag{3}$$

For cohesionless soil (c=0), without overburden (q=0), the ultimate bearing capacity is given by the following expression:

$$q_u = 0.5\gamma B N_\gamma \tag{4}$$

The bearing capacity factor N_γ varies sharply with the angle of shearing resistance ϕ. For all the intermediate cases, where $c \neq 0, q \neq 0$ *and* $\gamma \neq 0$, Terzaghi (1943) [2] suggested a superposition method in which the contributions to the bearing capacity (q_u) of a shallow footing of width B subjected to central vertical load from different soil and loading parameters are summed and represented by the following expression.

$$q_u = cN_c + qN_q + 0.5\gamma BN_\gamma \qquad (5)$$

The bearing capacity factors N_c, N_q and N_γ are different functions of ϕ. They represent the effects of soil cohesion c, surface loading q and unit weight γ respectively. In Terzaghi's method (1943) [2], the shear resistance of the soil above the base of the footing is neglected; this soil is considered only as a surcharge imposing a uniform pressure on the horizontal plane at the foundation level. A great variety of proposed solutions are available in literature to the bearing capacity problem. The variations in N_c and N_q values proposed remain insignificant, but there are differences in the reported N_γ values.

Meyerhof (1963) [19] and Hansen (1970)[20] have developed solutions for footings of various shapes. Meyerhof (1963) [19], Hansen (1970) [20], Vesic (1973)[21] and Murff and Miller (1977)[22] have addressed the problem of eccentric and inclined loads on foundation. The bearing capacity equation used for inclined and eccentric loads has the following general form:

$$q_u = cN_c s_c d_c i_c + qN_q s_q d_q i_q + 0.5\gamma BN_\gamma s_\gamma d_\gamma i_\gamma \qquad (6)$$

where q_u is the vertical component of ultimate intensity of load on footing and the factors s, d and i are the footing shape, footing depth and load inclination factors respectively. The shape and depth factors used in Meyerhof (1963)[19], Hansen (1970) [20] and Vesic (1973)[21] methods for cohesionless soils are presented in Table 1 as they are commonly used.

The above methods are quasiempirical methods which assumed that the effects due to soil cohesion, surface loading and soil unit weight are directly superposable whereas the soil behaviour in the plastic region is nonlinear and thus superposition does not hold good for general soil-bearing capacities. The main reason for using the simplified (superposition) method is the mathematical difficulties encountered when using conventional equilibrium methods.

The soil in all these approaches was assumed to behave as a rigid perfectly plastic material obeying the Mohr-Coulomb yield criterion. But, the behaviour of soils shall be more suitably described by elasto-plasticity with hardening than by rigid plasticity. For this type of material, the failure takes place inevitably in a progressive way. The foremost difficulty in finding acceptable solutions lies in the selection of a mathematical model of soil behaviour or its constitutive (stress-strain-time) relationships. In spite of greatly improved capabilities for the solution of boundary value problems of this kind, the theory of bearing capacity is still limited almost exclusively to solutions developed for the rigid-plastic solid of the classical theory of plasticity (Vesic 1975) [23].

The bearing capacity equation (6) is a modification of equation (5) which was formulated for strip footings subjected to vertical axial loads. The expression for depth,

Table-1: The shape and depth factors by Meyerhof, Hansen and Vesic methods

i) Shape factors		
Method	**s_q**	**s_γ**
Meyerhof (1963)	$s_q = 1$ (for $\phi = 0$) $s_q = 1 + 0.1\, N_\phi \frac{B}{L}$ (for $\phi > 10^\circ$) $N_\phi = \frac{1+\sin\phi}{1-\sin\phi}$	$s_\gamma = s_q$
Hansen (1970) as modified by Vesic (1975) and Vesic (1973)	$1 + \frac{B}{L}\tan\phi$	$1 - 0.4\frac{B}{L}$
ii) Depth factors		
Method	**d_q**	**d_γ**
Meyerhof (1963)	$d_q = 1$ (for $\phi = 0$) $d_q = 1 + 0.1\sqrt{N_\phi}\, \frac{D_f}{B}$ (for $\phi > 10^\circ$)	$d_\gamma = d_q$
Hansen (1970) and Vesic (1973)	$1 + 2\tan\phi\,(1-\sin\phi)^2 \frac{D_f}{B}$	1

shape and load inclination factors are largely empirical in nature. Therefore, it is expected that these factors may not satisfactorily account for all foundation and soil behaviour. For instance, the behaviour of a footing under an inclined load having a positive eccentricity should be expected to be different from the same footing subjected to an inclined load with a negative eccentricity (Vivatrat and Watt 1983) [24]. However, the modified bearing capacity equation (6) recognises no such differences (Chen and McCarron 1997) [25].

3. EXPERIMENTAL STUDIES AND RELATED THEORETICAL METHODS

Observations of decreasing bearing capacity factor N_γ with increasing width of the foundation were termed the 'scale effect' by De Beer (1963) [26]. The scale effect phenomenon was shown as early as 1935 by Berry (1935) [27], who presented the results that showed the bearing capacity of the model circular footings increased disproportionately with increasing footing sizes, viz. 0.0508 m, 0.0718 m, 0.1016 m and 0.1437 m (as reported by Cerato and Lutenegger 2007) [28].

The actual scale effect observed in small-scale footings is partly related to the mean stress felt underneath a footing i.e. the larger the footing, the higher the mean stress and therefore, the lower the friction angle. Mean stress increases with increasing footing width and from the curvature of the Mohr-Coulomb failure envelope theory, friction angle decreases as mean stress increases. Hence, the observed scale effect between N_γ and footing width can be related directly to the friction angle experienced beneath different footing sizes i.e. as footing width increases, mean stress increases and friction angle decreases (Meyerhof 1950[3]; De Beer 1963 [26], 1965a [29], 1965b[30]; Vesic 1975)[23]. The Mohr-Coulomb strength envelope is usually approximated by a straight line in the fairly limited stress range used in most laboratory shear testing programs. When the stress range is larger, the envelope is not straight but curved with low ϕ values at high stresses. De Beer (1963)[26] stated that the scale effect was due to the nonlinear Mohr-Coulomb failure envelope. The amount of curvature depends on the relative density of sand, with dense sands possessing more curvature than loose sands (Vesic and Barksdale 1963[31]; Bishop 1966[32]; Lee and Seed 1967[33]).
A more fundamental explanation for scale effect, however, is that the bearing capacity of a footing on sand is affected by both peak and critical state strengths. Strength (or friction angle) is a function of dilation and would not be uniform underneath a loaded foundation, and a complicated equation would be necessary to take this into account. Therefore, De Beer (1963)[26] suggested that an easier approach would be to use the curved failure envelope in a stress characteristic calculation.

Muhs (1963) [34]and De Beer (1965 a, 1965b)[29,30] stated that the shear failure in soil under the footing was a phenomenon of progressive rupture at various stress levels. Consequently when the footing load is gradually increased, the shear strength is not simultaneously mobilised at all points on the slip surface. At first it is mobilised at the points of highest shear strains, progressing then to other points in the soil body along the developing slip plane. The average shear strength mobilised along a slip line under the foundation decreases with the foundation size. Vesic (1975)[23] stated that there are three independent reasons for this decrease of strength with footing size: (a) the curvature of Mohr envelope; (b) progressive rupture along the slip line; (c) presence of zones or seams of weakness in all soil deposits.

From the tests performed on real size footings at DEGEBO in Berlin and small size model footings performed at the University of Ghent, for sand, De Beer (1965a)[29] has concluded that for high density sands, the values of N_γ found with large footings could be smaller than those found with small footings, and the influence of progressive rupture under a large footing could be higher than that under a small footing and the scale effect is partially due to the different influences of progressive rupture phenomenon.

Ovesen (1975) [35] carried out two series of concentric loading centrifuge tests on circular footings embedded in sands, varying the combination of the diameter of footing and acceleration. The first series of tests were conducted on a prototype diameter of 123 cm. The second series of tests were conducted with footings of constant diameter of 2.5 cm, varying acceleration ng (n=10 to 85), thus representing prototypes of different sizes. Similar studies have been undertaken by Mikasa and Takada (1973) [36] and Cherkasov et al (1970) [37]. From the results of centrifuge tests in his study and those of other researchers, Ovesen (1975) [35] concluded that for a given prototype footing, the centrifuge models on sand built to scale $1/n_1$ and $1/n_2$ yielded the same bearing capacity for at least $1 \le \frac{n_1}{n_2} \le 3$, for the range of n between 30 and 80.

Yamaguchi et al (1977)[38] performed 1g model and centrifuge tests on dense sand (D_r = 0.87) with foundation model widths (B) of 2, 3 and 4 cm and depths of embedment (D_f) as 0, 0.5B and 1B. The centrifuge experiments were conducted at centrifuge field accelerations of 10, 20 and 40g. Their study showed that the influence of grain size on centrifuge tests for bearing capacity estimation was insignificant. The scale effect existed with respect to the bearing capacity factor N_γ, but it vanished at around 90 cm of footing width. Further, it was found that the shearing strains showed considerable variation along the slip line. The average mobilised angle of shearing resistance along the slip line was smaller than the maximum value of the angle of shearing resistance ($\phi_{p\max}$) obtained by plane strain shear tests and it decreased with the increase in footing width, the extent of decrease increased with the increase in D_f/B ratio. The presented results indicated that the progressive failure became more prominent with increasing footing width or increasing soil confinement, for a soil of a given relative density.

Ingra and Baecher (1983) [39] evaluated N_γ and load inclination and eccentricity factors from the results of bearing capacity data from model footing tests using statistical analysis. The linear regressions of $\ln(N_\gamma)$ on ϕ yielded the following equations.

$\ln(N_\gamma)_{(L/B=1)} = -2.107+0.173\,\phi$ (for square) (7)

$\ln(N_\gamma)_{(L/B=6)} = -1.667+0.173\,\phi$ (for strip) (8)

The N_γ values for strip footings obtained from equation (8) compare reasonably well with the experimental results of Selig and Mckee (1961) [40] , De Beer and Ladanyi (1961)[41], Verghese (1972) [42], Vesic (1963) [31] and De Beer (1970) [43].

The main conclusions arising from the studies are as follows: Model test and theoretical results are characterised by a certain scatter, but they can be grouped visibly into two separated not overlapping intervals. The N_γ values of classical methods are lower than those obtained from model tests. The N_γ values of Ingra and Baecher (1983) [39] for strip footings on sands of different densities compare reasonably with the experimentally predicted N_γ values of other investigators.

Zadroga (1994)[44] analysed the influences of eccentricity and inclination of load, subsoil surface inclination, depth and shape of foundation in noncohesive soils, on bearing capacity using Polish, Finnish and Japanese model test results, and compared with classical and other theoretical methods (Balla 1962 [45]; Garber and Baker 1977[46]; Lewandowska and Dembicki 1991[47] ; Narita and Yamaguchi 1992 [48]; Saran and Agarwal 1991 [49]; Ingra and Baecher 1983) [39]. A significant quantitative difference was found to occur for each factor. The analysis of model test results and the comparison of results with the theoretical methods of other investigators showed considerable under estimation of results for classical methods and reasonable agreement for other theoretical methods. The results of model tests for strip foundations, and square and circular footings were statistically evaluated. The expected values of N_γ i.e. $E(N_\gamma)$ proposed are as follows:

For footings:

$E[N_\gamma] = 0.096\exp[0.188\,\phi]$ (9)

For strip foundations:

$E[N_\gamma] = 0.657\exp[0.141\,\phi]$ (10)

Hettler and Gudehus (1988) [50] proposed a method for finding a weighted average of angle of shearing resistance (ϕ'_m) to be used in Terzaghi's (1943) [2] bearing capacity equation and calculating N_γ as a function of footing width, taking into consideration the scale effect in terms of stress dependency of ϕ'. This method, however, requires an iterative procedure to find ϕ'. Kutter et al (1988) [51] suggested an alternative method to use the best straight-line fit to the curved envelope over the range of stress of interest without considering the variation of ϕ' with stress level.

Tatsuoka et al (1991)[52] reported that two factors which may be responsible for footing size effect on the bearing capacity are the stress level dependency of mechanical properties of sand and the change of the ratio B/d_{50} (B = width of footing and d_{50} = particle size finer than 50%). The second factor may be considered as a particle size effect. Tatsuoka et al (1991) used 1g model and centrifugal test results as well as Siddiquee's (1991) [53] computer simulation of the 1g and centrifuge tests and showed that decrease in N_γ in 1g tests was due to stress level effect as observed in the centrifuge tests for a constant width B_o and the particle size effect. The particle size effect was shown as the difference in results between 1g and centrifuge tests for an identical width $B = nB_o$, where B_o = width of 1g model footing and n = acceleration in the centrifuge.

From the results of model tests on sand, the threshold values of B/d_{50} above which the particle size effect becomes insignificant have been reported. The scale effect due to particle size occurred for $B/d_{50} < 33$ (Bätcke 1982)[54] or $B/d_{50} < 100$ (Steenfelt 1977 [55]; Jarzombek 1989)[56]. Kusakabe (1995)[57]] recommended model testing with $B/d_{50}>100$ to avoid the particle size effect.

In the literature, research results regarding the effect of footing size on the shape factor (s_γ) are contradictory. The value of s_γ calculated from an expression proposed by Meyerhof (1963) [19] decreased with the friction angle of soil. As discussed earlier, the mobilised friction angle of soil under foundations decreases with footing size. This implies that the value of s_γ calculated from Meyerhof's formula decreases with footing dimension. Kusakabe et al (1991)[58] showed similar result from the centrifuge tests on footings with aspect ratios (L/B) of 7, 5, 3 and 1. It was found that the scale effect was more pronounced when the footing geometry became more of three-dimensional, i.e. smaller aspect ratio (close to unity), and showed that the shape factor decreased from 0.6 to 0.4 as the prototype footing width increased up to 3 m (B_o = 100 mm, n = 30g) for an aspect ratio of 7.

The studies of Zhu et al (2001) [5] on footings indicated that s_γ increases with footing size. Their work involved a numerical and centrifuge experimental study of the scale effect on the bearing capacity and shape factor of strip and circular footings resting on dry dense sand. A new wedge failure mechanism was proposed for the analysis using the method of characteristics. The circular model footing was 44 mm in diameter and it was tested in the centrifuge at acceleration levels of 1, 10, 40, 100 and 160g corresponding to prototype diameters of 0.044, 0.44, 1.76, 4.4 and 7 m respectively. The strip model footing was 15 mm in width and 75 mm in length and was tested at acceleration levels of 1, 10, 20, 40, 80, 120 and 160g. The results indicated that a tenfold increase of footing size resulted in an approximately 55% reduction in the bearing capacity factor N_γ. Consistent results have been obtained from centrifuge tests. Numerical analysis and experimental modeling showed that the shape factor (s_γ)

increased with footing dimension. The value of s_γ for circular footings from numerical analysis is 22% lower than the traditional value of s_γ = 0.6. The experimental results indicated that using the traditional value of 0.6 is unconservative for small footings by about 25% but may be conservative for large foundations by about 5%.

Michaelowski (1997)[14] estimated the bearing capacity factor N_γ for strip footing using the kinematical approach of limit analysis. A procedure where the three terms in bearing capacity formula are consistent with one failure mechanism was developed. The bearing capacity factors obtained through this approach was found to provide the best upper bound of bearing capacity to the true limit load.

The variability of N_c and N_q with the changes in $c/\gamma B$ and $q/\gamma B$ was found to be small and therefore, the exact solution found by Prandtl (1921) [17] and Reissner (1924) [18] are justified. However, N_γ increased significantly with the increase in both $c/\gamma B$ and $q/\gamma B$. Further, N_γ takes higher values than those suggested by any widely used method when all the terms of bearing capacity are consistent with one collapse mechanism.

A closed form approximation of the minimum solution to N_γ was obtained when c = 0 and q = 0. The N_γ values thus obtained were a reasonable estimate of soil weight influence, but it became increasingly conservative with increase in $c/\gamma B$ and $q/\gamma B$. On comparison with other theoretical methods, it was found that the N_γ (when c = 0 and q = 0) values obtained in this study appeared to match with the values of Vesic (1973) [21] and considerably higher than the values of Meyerhof (1963) [19] and Hansen (1970) [20].

Soubra (1999) [15] considered two failure mechanisms for the analysis of the static and seismic bearing capacity factors using the upper bound method of the limit analysis theory. Two kinematically admissible failure mechanisms M1 and M2 were considered in the framework of the upper bound method of the limit analysis theory. The M1 mechanism shown in Figure 1 was symmetrical, and composed of a triangular active wedge under the footing and two radial shear zones composed of a sequence of rigid triangles. It permitted the calculation of the bearing capacity in the case of no-seismic loading. The M2 mechanism was nonsymmetrical and was composed of a single radial shear zone. It permitted the calculation of the bearing capacity in the presence of seismic loading.

The N_γ factors given by the M1 mechanism were compared with those of Caquot and Kérisel (1953) [6], Meyerhof (1965) [59] and Vesic (1963) [31]. It was reported that the proposed values compared well with the values of Caquot and Kérisel and Vesic with a maximum difference of 20% at $\phi = 30°$ and at $\phi = 20°$ respectively, and were found to be higher than those of Meyerhof theory with a maximum difference of 43% at $\phi = 30°$. It was also found that for the static case, both M1 symmetrical and the M2 nonsymmetrical mechanisms gave the exact solution of the

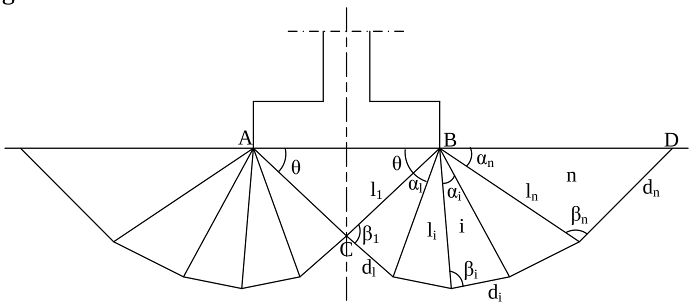

Fig-1: Failure mechanism M1 for static bearing capacity analysis (Soubra 1999)

static N_c and N_q factors. For the N_γ factor, the M2 mechanism gave greater upper bound solution than the M1 mechanism. However, the maximum difference did not exceed 4% for $\phi \geq 20°$.

Perkins and Madson (2000) [60] proposed an approach based on relative density for shallow foundations on sand and described the effect of progressive failure on ultimate bearing capacity in terms of the relative dilatancy index inherent in strength-dilatancy relationships. Competing with the notion of progressive failure offered by Yamaguchi et al (1977) [38], the authors observed from the shear tests that the potential for progressive failure is more acute for low confinement conditions or for smaller footing widths and stated that two counteracting mechanisms occur: (i) The physical observation that progressive failure, being defined in terms of the nonuniformity of shear strain and mobilised friction angle in the soil at peak footing load, is more significant as footing width increases, and (ii) the potential for progressive failure, being defined by the difference between the peak and residual strength of the soil, is more significant as the footing width decreases. The authors have postulated that the combination of these two effects can be described in terms of strength-dilatancy characteristics of the soil, which are dictated by the soil type, relative density and footing geometry.

Dewaikar and Mohapatro (2003) [61] developed a procedure using the concept of force equilibrium condition coupled with Kötter's equation for the evaluation of N_γ with Terzaghi's mechanism. Application of Kötter's equation made the analysis statically determinate in which the unique failure surface was identified using force equilibrium conditions.
The following equation was proposed for the estimation of N_γ.

$$N_\gamma = \frac{4P_{p\gamma}}{\gamma B^2} - \frac{\tan\phi}{2} \tag{11}$$

where $P_{p\gamma}$ = Passive thrust on retaining wall, γ = unit weight of soil, ϕ = angle of internal friction of soil and B = width of footing.
The computed N_γ values were found to be higher than Terzaghi's values in the range of 0.25 – 20%, with a diverging trend for higher values of angle of internal friction of soil.

Gandhi (2003) [62] carried out laboratory load tests on model footing of strip (L/B=6), square and circular shaped, with a width of footing ranging from 5.85 cm to 15.2 cm. Sands of different densities with relative density varying from 20% to 80% (loose to dense conditions) have been used. From the study, it has been concluded that the rate at which N_γ decreased with increase in width of footing was higher than that for N_q and that the laboratory values of N_γ were 8.3% to 30.8% higher while those of N_q were 47.8% to 99.5% lower than the corresponding values of the Indian Standard code of practice. Empirical equations have been proposed for the shape factors s_q and s_γ in terms of angle of shearing resistance. It was also found that the relative settlement at failure (= settlement at failure/width of the footing) was higher for a smaller size footing than that for a larger size footing, at any given relative density.

From the results of model scale square and circular footing tests ranging in width from 0.025 m to 0.914 m on two compacted sands, Cerato and Lutenegger (2007) [28] showed that the relative density has a pronounced influence on N_γ than grain size and that the scale effect is more important for dense sands. A small footing (small mean stress) would act, as if it was on a denser state of a soil than a large footing, even if they were tested on sand with the same void ratio. Therefore, the model scale test must be performed at lower density sand than a corresponding prototype footing in order to correctly predict the behaviour. It was also shown that model scale footing test results produced higher values of N_γ than theoretical equations and therefore should not be used for the design of full-scale footings without a reduction.

Dhiraj Raj and Bhrathi (2013) [63] have reviewed the methods available for the estimation of bearing capacity of shallow foundation on slope and near slope and extracted the following conclusions. 1) All the approaches used by different researchers for the evaluation of bearing capacity of shallow foundation on slope or near the slope have their own sets of assumptions and hence corresponding weaknesses also, 2) Some investigations show that , in case of noncohesive soils, the bearing capacity is always governed by foundation failure, while in cohesive soil the bearing capacity of the foundation Is dictated by the stability of slope , 3) Hybrid methods (viz. combination of FE method with Limit analysis or FE method with Limit equilibrium) are giving the most satisfactory results for bearing capacity calculation, 4) The method which gives the minimum bearing capacity for shallow foundation on slope is considered for conservative design.

Keshavarz and Kumar (2017) [64] have numerically evaluated ultimate bearing capacity of circular and strip footings, placed over rock mass using the method of stress characteristics for both smooth and rough footing-rock interface. The modified HB failure criterion was used in the analysis. The bearing capacity has been presented in the form of nondimensional bearing capacity factors as a function of different input parameters for rock mass. The authors have noted that an increase of GSI and mi leads to an increase in the values of $N\sigma$ and $N\sigma0$ and that the factor $N\sigma$ has been found to increase continuously with a decrease in the value of $\sigma c/(\gamma b)$. The roughness of the footing has been found to have more significant effect for a circular footing as compared with a strip footing. The results obtained from the present study have been found to compare quite well with the different solutions available from literature.

4. LARGE-SCALE EXPERIMENTAL STUDIES

Muhs et al (1969)[65] conducted load tests with rigid foundation (B = 0.6 m and L =1.2 m) at depths of embedment 0 and 0.2 m. Six tests were conducted at the test field covering an area of 150 m^2 of DEGEBO in Berlin in loose (porosity n = 41% and relative density D_r = 0.25), medium dense (n = 38% and D_r = 0.45) and dense sand (n = 35% and D_r = 0.65). These tests were conducted in submerged condition to eliminate the effect of apparent cohesion.. In case of loose and medium dense sands, below the respective top layer of sand of about 1.6 m thick is medium dense sand laid to a thickness of 1.4 m, thus making the total thickness of sand bed as 3 m. From the measurement of the normal stresses in the base of the footing, the authors have shown that under the rigid footings in the lower zone of the load with a nearly linear load-settlement characteristic, the resulting stress distribution always has stress concentration at the foundation edges, independent of the density of the sand and embedded depth of the footing. With increasing load, the stresses at the edges increased only a small amount or decreased to values even below those already obtained, while in the central zone of the footing, mobilisation of reserves of bearing capacity of the soil increased up to the failure. The maximum angle of base friction was found to depend on roughness of the base only and not on the angle of shearing resistance of soil or its density. In spite of practically smooth surface, the measured average angle of base friction amounted to 0.51 to 0.67 times the angle of friction of sand.

Weiß (1970)[66] conducted two series of large-scale experiments on footings of width 0.5 m and length ranging from 0.5 m to 3.5 m, in sand of medium density with and without embedding depths. From the results of the above tests and that of all the tests in sands of different densities with central loading, carried out at DEGEBO since 1951, it was concluded that with the increase in angle of shearing resistance, the bearing capacity factors derived from the tests were lower than the theoretical values according to DIN 4017, part 1. The reason for this difference was attributed to the phenomenon of the progressive rupture in the dense sand. The authors have found that there exists a direct relation between the bearing capacity factors (N_q and N_γ) and the cone resistance of the deep sounding apparatus measured in a depth ranging from 1 to 1.5 m, for foundations of different sizes in sands with different relative densities and grain size distribution. The grain size distribution did not affect the influence of the shape of the footing on the bearing capacity.

Muhs and Weiß (1971)[67] examined the ultimate bearing capacity and the settlement behaviour of a nonuniform gravelly soil, with three different densities. A total of 46 large-scale experiments with footings of a width of 0.5 m and a length of 0.5 m to 2 m were carried out partly in moist soil and partly in submerged soil. They found that for small angles of internal friction (ϕ), N_γ values from the tests approached the DIN values and for large values of ϕ, the N_γ values approached those of Hansen. The comparison made by the authors with the results of earlier studies on uniform fine and medium dense sand showed that in spite of greater dry density of gravelly soil compared to uniform sand, it did not produce greater bearing capacity.

Muhs and Weiss (1973) [68] continued their earlier study and investigated the influence of load inclination on the ultimate bearing capacity of shallow strip foundations through large-scale load tests in cohesionless soils with a foundation width of 1 m and length of 3 m. All the tests were carried out with a water level of 1 to 2 cm above the surface of the fill to eliminate the influence of apparent cohesion on the failure load. They concluded that the decrease of the vertical component of failure load resulting from load inclination parallel to the short side of a footing was larger than in the case where the inclined load was applied parallel to the long side of the footing.

From the load tests carried out on five large spread footings ranging in size from 1 to 3 m and embedded to a depth of 0.75 m, into a medium dense, fairly uniform, silty silica sand, Briaud and Gibbens (1999) [69] concluded that when the load settlement curves of the five footings are plotted as pressure versus settlement over width (s/B) ratio, the five curves collapse into one and the apparent scale effect disappears. New correlations connecting the bearing capacity and pressure at working loads to pressuremeter limit pressure, cone penetrometer point resistance and the standard penetration test (SPT) blow count have been proposed. From the experiments, it was found that 78% of the settlements occur within a depth of 1B and 97% occur within a depth of 2B. The creep settlement, effect of cyclic loading and preloading on creep rate, zone of influence under the footing, mode of deformation of the soil mass and the volume change observations have also been studied.

5. CONCLUSION

It is seen from the above review that the problem of bearing capacity of shallow foundations on cohesionless soils was studied for many years. However, an accurate solution capable of predicting peak load carrying capacity for a wide range of soil relative densities, effective stress conditions and foundation shapes within a practical context remains elusive.
The classical methods were developed based on small-scale foundation experiments (Meyerhof 1950[3], 1963[19]; Hansen 1970 [20]; De Beer 1970 [43]; Vesic 1973) [21], whereas large-scale tests at that time indicated the inability of these solutions to predict the actual field behavior (Muhs 1963[34], De Beer 1965a [29], 1965b [30]). Centrifuge experiments conducted over the past 20 years have demonstrated similar problems while providing for an advanced understanding of the problem (Ovesen 1975[35]; Yamaguchi et al 1977[38] ; Kusakebe et al 1991)[58].

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