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New Approach for Evaluating the Bearing Capacity of Sandy Soil to Strip Shallow Foundation

Zainal Abdul Kareem E.[1, *], Amjad Ibraheem Fadhil[2]

[1]Civil Engineering Department, University of Baghdad, Engineering College, Baghdad, Iraq
[2]Civil Engineering Department, University of Baghdad, Engineering College, Uruk University, Baghdad, Iraq

Abstract

Bearing capacity prediction of shallow foundations is of great importance in shallow foundation design process. Many proposed and implemented methods have been used in this field of soil mechanics for many years and became a standard procedure for these calculations. This work proposes a new method with alternative and simpler approach to predict the ultimate bearing capacity of sandy soil suitable for shallow foundation using strip footing based on the soil surface, which can be further extended for other shapes of shallow foundations, under soil surface, or other types of soils (i.e. c–ϕ) soils. This approach is based on specifying the shape of the failure surface under shallow foundation which is located and defined by a new equation that may describe this surface instead of the multi–relations (i.e. log–spiral curve and a linear relationship proposed by Terzaghi, 1943 [1] and others).This proposed equation is to cover many internal friction angles (ϕ) for sand, ranging from 10º to 50º and normalized for footing width (B); besides being more general and can be directly implemented for this range of internal friction angles of ϕ. Bearing capacity calculation results were found in good agreement with the results obtained from Terzaghi's equation, Meyerhof's [2], and solution based on Rankine wedges method, Lamb 1979 [3], though the values of equivalent N_γ were found more conservative but more realistic and agree with some bearing capacity design codes.

Keywords

Shallow Foundation, Bearing Capacity, Failure Surface, Strip Footing

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1. Introduction

Bearing capacity of soil for foundation had always been one of the most interesting researches subjects in geotechnical engineering, [4]. It is recognized that every foundation problem necessitates the study of ultimate bearing capacity of the soil. The bearing capacity of sand is a function of its inherent resistance to frictional shear which is expressed in term angle of internal friction (ϕ).

Since bearing capacity failure usually results in complete failure of the structure, significant treatment of the subject of bearing capacity of the sand and clay, with the aim developing a true understanding of the factors upon which it depends, is significantly required for practicing soil engineer, [5].

2. Previous work

There were many important studies undertaken by various researchers that were conventionally used in engineering

* Corresponding author
E-mail address: kareem_esmat@yahoo.com (Zainal A. K. E.),

practice with respect to the bearing capacity of soils and are summarized:

2.1. Terzaghi's Bearing Capacity Theory

In 1943 Terzaghi [1] proposed the failure ultimate load under continuous or strip foundation as shown in Fig. 1 The failure area in the soil under the foundation load was divided into three major zones [3]. They are:

i. The triangular zone (I) under the footing base is considered as a part of the footing and penetrates the soil like a wedge because of friction and adhesion between the footing base and the soil.

ii. Zones (II) that are located between zone I and zone III are known as the radial shear zones which contain the shear pattern lines that radiate from outer edge of the base of footing.

iii. Zones (III) are identical of Rankine passive state which respects to shear pattern lines that develop in these zones.

2.2. Meyerhof

In 1951, Meyerhof [2] published a bearing capacity theory which could be applied to rough shallow and deep foundations. The failure surface at ultimate load under a continuous shallow foundation assumed by [2] is shown in Fig. 2. In this Figure *abc* is the elastic triangular wedge, *bed* is the radial shear zone with *cd* being an arc of a log spiral, and *bde* is a mixed shear zone in which the shear varies between the limits of radial and plane shear, depending on the depth and roughness of the foundation. The plane *be* is called an equivalent free surface. The normal and shear stresses on plane *be* are p_0 and s_0, respectively.

2.3. Vesic

In 1973 Vesic, [6] proposed an improved bearing capacity based on [1] theory by introducing different bearing capacity factors and shape factors which were recommended for reliable computation of the bearing capacity. The shape factors proposed by [6] take into account the bearing capacity factors and the footing dimensions.

2.4. Graham et al.

Graham [7] provided a solution for the bearing capacity factor for a shallow continuous foundation on the top of a slope on granular soil based on the method of stress characteristics.

2.5. Jyant Kumar and Priyanka Ghosh

[8] Used the method of stress characteristics to evaluate bearing capacity factor $N\gamma$ for rough circular footing. The failure mechanism considered in the study comprises of a curved non-plastic trapped wedge below the footing being tangential to its base at its edge and inclined at an angle ($\pi/4-\varphi/2$) with the axis of symmetry.

The chosen curved trapped wedge ensures that the angle of interface friction between the footing base and underlying soil mass remains equal to φ at the footing edge. The computed value was found to be significantly smaller than those obtained with the consideration a triangular (conical) trapped wedge below the footing wedge.

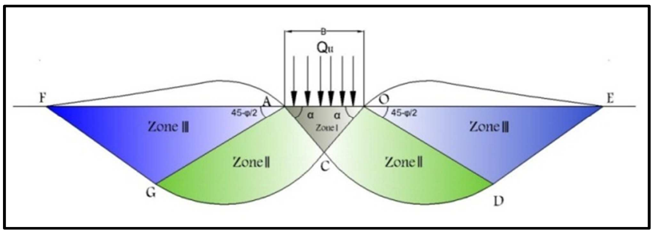


Fig. 1. Bearing capacity failure in soil under a rough rigid continuous foundation (modified after [1] redrawing by Amjad. I .Fadhil.).

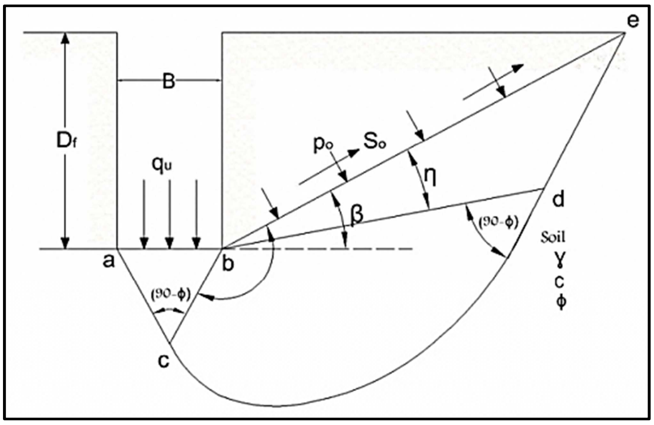


Fig. 2. Slip line fields for a rough continuous foundation after [2], redrawn by Amjad. I. Fadhil.

2.6. Georgiadis

Georgiadis [9] used the finite element analysis based on limit equilibrium or upper bound plasticity calculations to investigate the influence of the various parameters (the distance of the foundation from the slope, the slope height and the soil properties) that affect undrained bearing capacity of strip footings on or near undrained soil slopes. The results of analysis were presented in the form of design charts. A design procedure was also proposed for the calculation of the undrained bearing capacity factor using the undrained shear strength and the bulk unit weight of the soil, the foundation width, the distance of the foundation from the slope, the slope angle and the slope height.

2.7. Lamyaa Najah Snodi

Snodi [10] used the method of characteristics (commonly referred to as the slip line method), to evaluate the values of bearing capacity factor ($N\gamma$) that were computed for rigid surface strip and circular footings with smooth and rough bases. The value of bearing capacity factor Nγ increases significantly with increase in the angle of internal friction. When friction angle φ is less than 25 degree, the computed value of Nγ for circular footing was found smaller than those strip footing and larger values of φ, the magnitude of Nγ for circular footing was greater than those strip footing for both smooth and rough base of footings. On the other hand, the magnitude of Nγ for rough footings was seen to be higher than for footings with smooth base.

The common proposed relation and the factors involved in estimation of ultimate bearing capacity are represented in Table A-1. appendix A.

3. Proposed Method

The proposed method is based mainly on finding a new simpler mathematical model describing the failure surface due to general failure in shallow strip foundation constructed on the surface of sandy soil. This mathematical model is used to calculate the ultimate bearing capacity of the foundation.

The proposed method consists of three stages, which are:

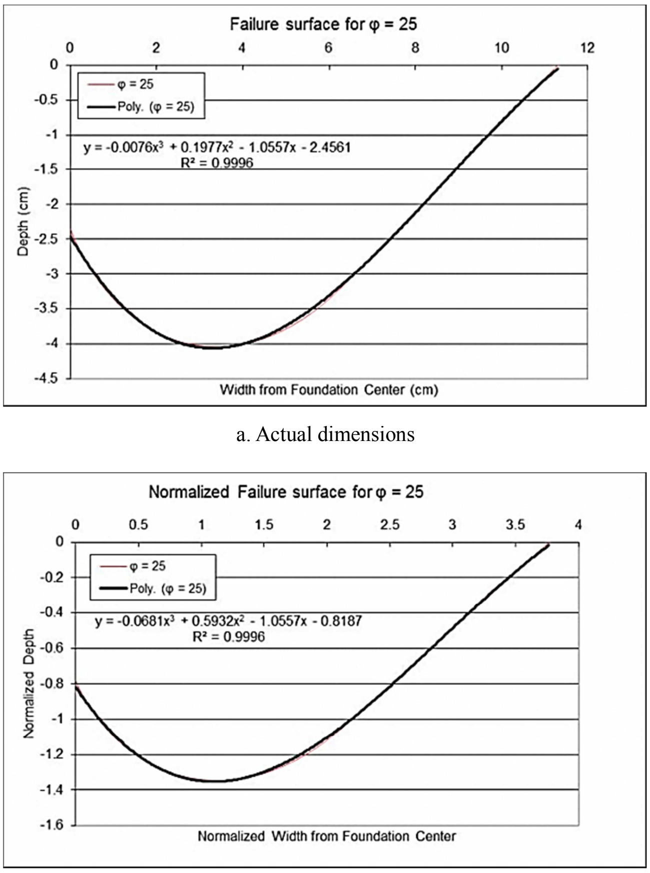
1. Find a new mathematical model for the failure surface related to the foundation width (B) on soil surface, so this mathematical model can be considered general for many values of internal friction angle f and also normalized for the width of the foundation (B).
2. Estimating the shear strength of the soil based on the weight of the soil above the failure surface since the resistance of the soil is gained through stress applied from this load.
3. Calculate the predicted ultimate bearing capacity of the strip foundation.

These stages use an alternative method for the derivation of the ultimate bearing capacity compared to the original method by Terzaghi, [1]; nevertheless maintaining the same shape of the failure surface.

4. Equation Derivation

Figure 1 describes the failure surface under strip foundation on sand where the area under and besides the shallow foundation is divided into three zones as shown in the Figure. The curve CDE represents the failure surface from one side and CGF from the other side, both of these curved lines exhibit relative movement between their particles and subjected to shear failure and considered as a slip surface with very large strains. For different internal friction angle (ϕ) the dimensions of the failure surface changes maintaining the same shape governed by equation of the logarithmic spiral $r = r_o e^{\theta tan\phi}$ giving the curve CD (Figure 1), and a straight line DE making angle of (45 – ϕ/2) with the soil surface.

The proposed method starts by drawing the failure surface CDE for each internal friction angle (ϕ) according to the logarithmic spiral equation mentioned in addition to the straight line Terzaghi, [1]) for an arbitrary foundation width of (e.g. 3 cm), a minimum of 337 points were obtained to draw the failure surface for ϕ =10º and a maximum of 2780 points were obtained to draw the failure surface for ϕ =50º.



a. Actual dimensions

b. Normalized dimensions

Fig. 3. A) Actual and b) Normalized dimensions.

A sample of ϕ = 25º failure surface points are shown in (Figure 3); in addition to the normalized failure surface that was obtained by dividing each dimension by the foundation width (B), also a best fit curve was obtained for each case, and since the normalized case will be considered in this study as it is more general case, then the normalized best fit equations will be used, the original dimensions then can be restored by multiplying each dimension by the foundation width B. A third degree polynomial was found to best describe these best fit equations with very high correlation coefficients as shown in Figure 4.

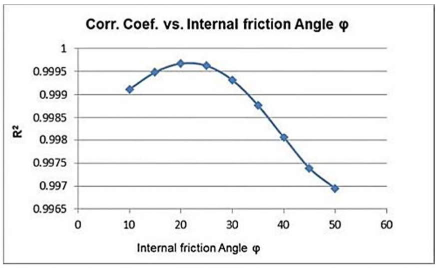


Fig. 4. Correlation coefficient vs. Internal Friction angle Ø.

The general form of the equation describing the variation of the failure surface for all internal friction angles (ϕ) (10º to 50º) is found to be:

$$D_N = a(H_N)^3 + b(H_N)^2 + c(H_N) + d \qquad (1)$$

where:

D_N = Normalized Depth = $\frac{y}{B}$,

H_N = Normalized Horizontal Distance from center of the foundation = $\frac{x}{B}$,

y = vertical dimension of failure surface depth,

x = horizontal dimension of the failure surface from foundation center,

B = Foundation Breadth, and

a, b, c, and d = constants that vary in value depending on the value of the internal friction angle of the sand.

The values of the constants a, b, c, and d were obtained for each internal friction angle as shown in Table 1. Best fit curves equations obtained from Figure (5, a, b, c, and d) were also determined describing the variation of coefficient a, b, c, and d with the internal friction angle ϕ as shown:

$$a = (4.65124 \times 10^{-7}\emptyset^3 - 1.78058 \times 10^{-4}\emptyset^2 + 1.39649 \times 10^{-2}\emptyset - 0.314319) \qquad (2)$$

$$b = 6.05232 \times 10^{-6}\emptyset^3 - 2.38304 \times 10^{-4}\emptyset^2 - 2.89221 \times 10^{-2}\emptyset + 1.37237 \qquad (3)$$

$$c = -2.30369 \times 10^{-6}\emptyset^3 + 4.31239 \times 10^{-4}\emptyset^2 - 1.01146 \times 10^{-2}\emptyset - 1.0362 \qquad (4)$$

$$d = -2.00929 \times 10^{-5}\emptyset^3 + 1.01386 \times 10^{-3}\emptyset^2 - 3.15048 \times 10^{-2}\emptyset - 0.348137 \qquad (5)$$

Table 1. Values of constants a, b, c, and d.

	a	b	c	d
10	-0.190647	1.0628	-1.09687	-0.588274
15	-0.14565	0.909368	-1.0983	-0.651763
20	-0.103184	0.74874	-1.0839	-0.727261
25	-0.0680654	0.593208	-1.05569	-0.818688
30	-0.0416114	0.450501	-1.01426	-0.932197
35	-0.0233838	0.326292	-0.961192	-1.07697
40	-0.0119021	0.223557	-0.898048	-1.26785
45	-0.00536951	0.143253	-0.82704	-1.53005
50	-0.00208747	0.0846815	-0.75235	-1.90856

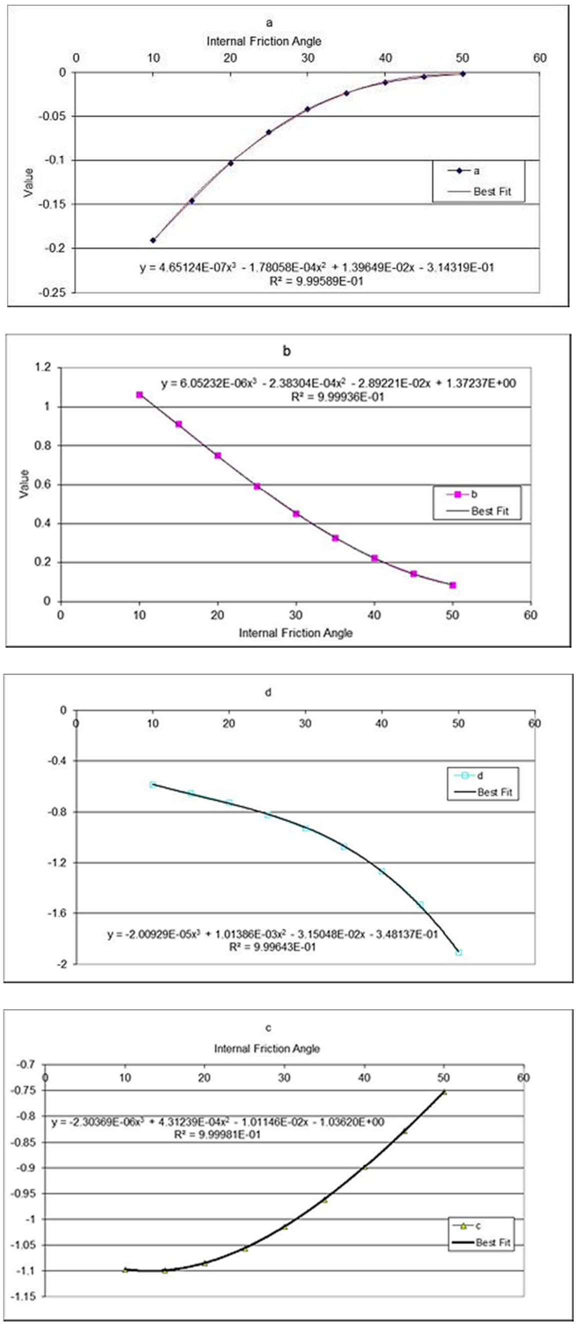


Fig. 5. a, b, c, and d Constants variations with ϕ.

Equations 1, 2, 3, 4, and 5 could be implemented to find the equation of the failure surface of any sand of known internal friction angle ϕ as demonstrated below.

e.g. for soil of ϕ = 10º, substituting the value of ϕ in equations 2, 3, 4, and 5 we get:

a = –0.19201, b = 1.0628, c = –1.0965, and

d = –0.58772.

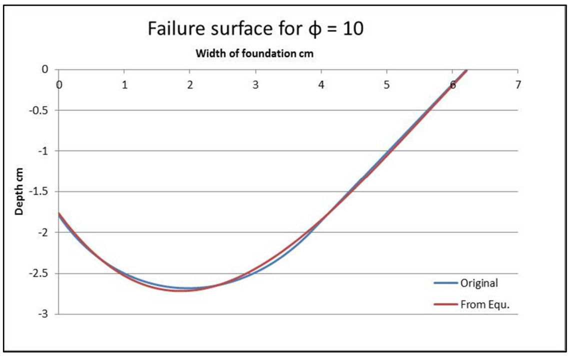
So equation 1 will read for normalized dimensions:

$$D_N = -0.19201(H_N)^3 + 1.0628(H_N)^2 - 1.0965(H_N) - 0.58772 \qquad (6)$$

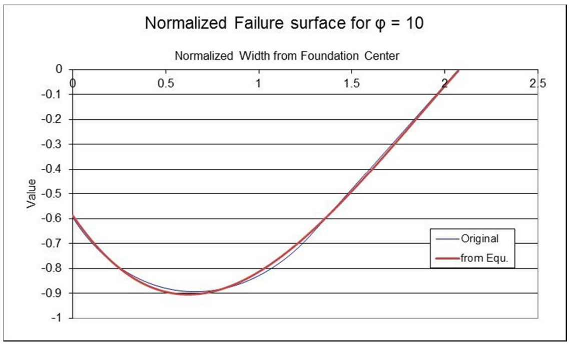
And for real dimensions when both sides of equation 1 are multiplied by (B=3), then

$$y = -0.021334(x)^3 + 0.35427(x)^2 - 1.0965(x) - 1.76316 \quad (7)$$

Equations 6 and 7 were drawn against predicted values as shown in Figure 6.



(a)



(b)

Fig. 6. a) Actual and b) Normalized dimensions.

4.1. Estimating the Shear Strength of the Soil

The ultimate bearing capacity of the strip foundation constructed on the surface of a sandy soil depends mainly on two factors:

- The area of the failure surface that resist the stresses applied by the foundation, and
- The shear strength of the sand along this surface.

Assuming one unit length of the strip footing perpendicular to the cross section shown in Figure (7), the resisting area is the length of the curve of the failure surface (2–3) multiplied by (1) unit of strip length. Implementing equation (1), the length of the curve for one side is:

$$L = \int_0^{x_{max}} \sqrt{1 + \left(\frac{dy}{dx}\right)^2} = \int_0^{x_{max}} \sqrt{1 + \left(\frac{dD_N}{dH_N}\right)^2} \quad (8)$$

where:

L=length of the curve

x_{max}= value of x where the curve intersect the soil surface(y = 0, point 3 in Figure 7)

The derivative of equation 1 is:

$$\frac{dD_N}{dH_N} = 3a(H_N)^2 + 2bH_N + c \quad (9)$$

$$L = \int_0^{x_{max}} \sqrt{(3a(H_N)^2 + 2bH_N + c)^2 + 1} \quad (10)$$

$$L = \int_0^{x_{max}} \sqrt{\begin{array}{c} 9a^2(H_N)^4 + 12ab(H_N)^3 + \\ (4b^2 + 6ac)(H_N)^2 + 4bcH_N \\ +c^2 + 1 \end{array}} \quad (11)$$

The value of x_{max} can easily be found by equating equation 1 to zero and solve for H_N

$$a(H_N)^3 + b(H_N)^2 + c(H_N) + d = 0 \quad (12)$$

Or in terms of real dimensions, the equations 9, 10, and 11 can be multiplied by (B).

The shear strength can be estimated as$\tau = \sigma' tan\emptyset'$, the stress can be estimated as a result of the weight of the soil above the failure surface acting on that surface, as shown in Figure (7).

Approximate methods can be used to predict this stress as:

a Equivalent stress method: Find an equivalent rectangular with the same area of the curve having the same width W, then by dividing the area over the width we can find the height H where

$$\text{Area}_{\text{Rect.}} = H \times W = \text{Area}_{\text{above Curve}}$$

Or

$$H = \frac{\text{Area}_{\text{above Curve}}}{x_{max}} = \frac{\int_0^{x_{max}} D_N}{x_{max}} = \frac{\int_0^{x_{max}} a(H_N)^3 + b(H_N)^2 + c(H_N) + d}{x_{max}} \quad (13)$$

- The stress then can be calculated as $\sigma' = \gamma_{Soil} \times H$
- The shear strength $\tau_f = \sigma' tan\emptyset'$,
- Multiplying the shear strength τ by the length of the curve for two sides gives the ultimate force that can be carried by the soil.

$$F_{Total} = 2\tau_f L \quad (14)$$

The ultimate bearing capacity can be calculated as:

$$q_{ult} = \frac{F_{Total}}{B} \quad (15)$$

And for real dimensions, the equation of the length of the curve must be multiplied by the width B, as shown:

$$q_{ult} = \frac{F_{Total}}{B} = \frac{2B\tau_f L}{B} = 2L\sigma' tan\emptyset' \quad (16)$$

$$q_{ult} = 2L\gamma_{Soil}Htan\emptyset' \quad (17)$$

$$q_{ult} = \frac{2L\gamma_{Soil}tan\emptyset' \int_0^{x_{max}} a(H_N)^3+b(H_N)^2+c(H_N)+d}{x_{max}} \quad (18)$$

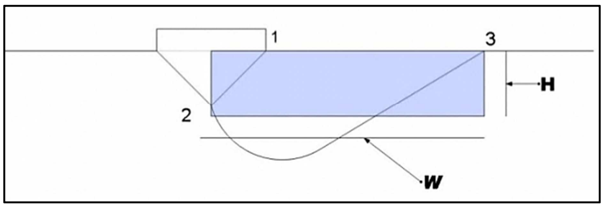


Fig. 7. Representing the equivalent area above the curve.

b Slices Method: An alternative method can be used to calculate the shear strength of the soil by dividing the area above the curve into slices (Figure 8) then the shear force for each slice can be calculated and summed together, then q_{ult} can be calculated as:

$$q_{ult} = \frac{2\gamma_{Soil}tan\emptyset' \sum_{i=1}^{n} ShearForce_i}{B} \quad (19)$$

where

n = the number of slices

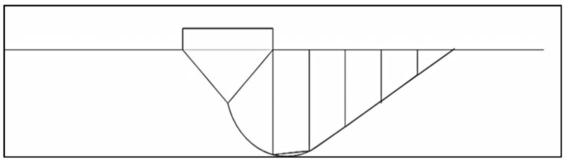


Fig. 8. Slices Method.

As an example of the implementing the proposed method, an example is taken from Lambe, 1977 where a strip foundation is on the soil surface. Fig (9)

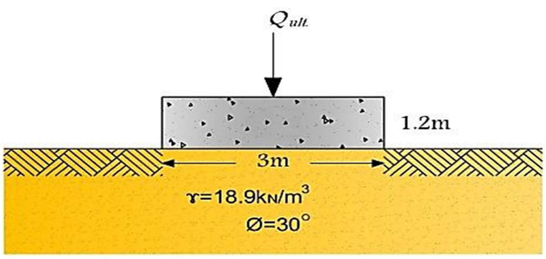


Fig. 9. Bearing Capacity Calculations, after Lambe, 1977.

Solution:

1. Find the appropriate equation for ϕ=30

$$a = 4.65124 \times 10^{-7}30^3 - 1.78058 \times 10^{-4}30^2 + 1.39649 \times 10^{-2}30 - .314319 = -0.043066$$

$$b = 6.05232 \times 10^{-6}30^3 - 2.38304 \times 10^{-4}30^2 - 2.89221 \times 10^{-2}30 + 1.37237 = 0.45108$$

$$c = -2.30369 \times 10^{-6}30^3 + 4.31239 \times 10^{-4}30^2 - 1.01146 \times 10^{-2}30 - 1.0362 = -1.0137$$

$$d = -2.00929 \times 10^{-5}30^3 + 1.01386 \times 10^{-3}30^2 - 3.15048 \times 10^{-2}30 - .348137 = -0.93096$$

2. Equation 1 will read

$$D_N = -0.043066(H_N)^3 + 0.45108(H_N)^2 - 1.0137(H_N) - 0.93096 \quad (20)$$

To obtain read dimensions, we need to multiply both sides by B=3m as in the example

$$\therefore y = -0.0047851x^3 + 0.15036x^2 - 1.0137x - 2.79288 \quad (21)$$

This is the equation of the failure surface for ϕ = 30°, dimensions are in (m)

3. To find x_{max} then

$$-0.0047851x^3 + 0.15036x^2 - 1.0137x - 2.79288 = 0$$

Solving for x, then x = 14.398 m

Length of the curve L=15.908 m

Area above the curve = 47.097 m^2

H = 47.097 / 14.398 = 3.271 m

σ = 18.9 × 3.271 = 61.8219 kN/m^2

τ = σ × tan(ϕ) = 61.8219 × tan(30) = 35.693 kN/m^2

F_{Total} = 2 × 35.693 × 15.908 = 1135.608 kN

$$q_{ult} = \frac{1135.608}{3} = 378.536 \text{ kN/m}^2$$

Or by using slices method, solving will give:

$$q_{ult} = 340.833 \text{ kN/m}^2$$

Comparing this value to the values obtained by Kumbhojkar Table (A-3)

q_u = 0.5×19.8×3×19.13 =568.2 kN/m^2.

Or by Meyerhof Table (A-4)

q_u = 0.5×19.8×3×15.67 =465.4kN/m^2.

Or by Hansen Figure 11

Q_u = 0.5×19.8×3×14= 415kN/m^2.

4.2. Equivalent Nγ

Based on the proposed equation, a new expression of Nγ can be proposed as follows, from Terzaghi, 1943:

$$q_{ult} = 0.5 \times \gamma_{soil} \times B \times N_\gamma \quad (22)$$

Then

$$0.5 \times \gamma_{soil} \times B \times N_\gamma = \frac{2L\gamma_{Soil}\tan\emptyset' \int_0^{x_{max}} a(H_N)^3+b(H_N)^2+c(H_N)+d}{x_{max}} \quad (23)$$

Substituting equation 10 for L and multiplying by B to obtain real dimensions:

$$\therefore N_\gamma = \frac{\int_0^{x_{max}} \sqrt{(3a(H_N)^2+2bH_N+c)^2+1} \int_0^{x_{max}} a(H_N)^3+b(H_N)^2+c(H_N)+d}{Bx_{max}} \quad (24)$$

$$\therefore N_\gamma = \frac{4\tan\emptyset' \int_0^{x_{max}} \sqrt{(3a(H_N)^2+2bH_N+c)^2+1} \int_0^{x_{max}} a(H_N)^3+b(H_N)^2+c(H_N)+d}{x_{max}} \quad (25)$$

Or simply

$$0.5 \times \gamma_{soil} \times B \times N_\gamma = 2L\gamma_{Soil} H\tan\emptyset' \quad (26)$$

$$N_\gamma = \frac{4LH\tan\emptyset'}{B} \quad (27)$$

We can express the value of N_γ in simple chart as shown in Figure (10) where Figure (11) shows the comparison of (N_γ) values obtained from different approaches (i.e. Terzaghi, Meyerhof, Hansen, Rankine wedges), we can observed that the value of N_γ for new approach is located between Rankine wedges and Hansen's, also the values of N_γ according to (Terzaghi and Meyerhof) is shown to be overestimated in comparison with Rankine wedges method.

Table A-2 in appendix A illustrate the methods used by some countries, where it is clearly shown that Hansen approach is most likely to be adopted for estimating the ultimate bearing capacity than using Terzaghi and Meyerhof method to predict value of ($N\gamma$) factor.

5. Conclusions

The new approach proposed in this work is found to have a very good agreement in trend with the other methods of predicting the ultimate bearing capacity of sandy soils; and many conclusions can be mentioned.

1. The method depends on simpler mathematical model using only one general equation to describe the failure surface applying a polynomial of the third degree that gave a very high correlation coefficient.
2. The required parameters for prediction can be easily obtained by simple mathematical operations (i.e. curve length, area above the curve, etc.).
3. Shear strength prediction is made by simple approximations that gave more conservative and realistic values especially on high values of internal friction angle ϕ.

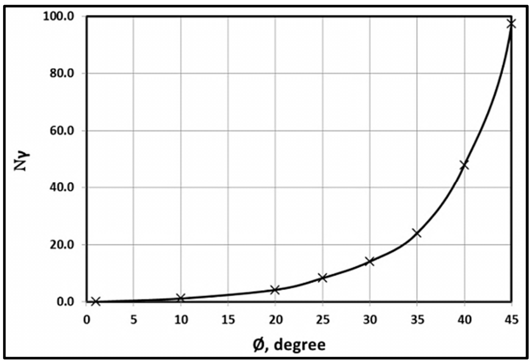


Fig. 10. Proposed values of Nγ.

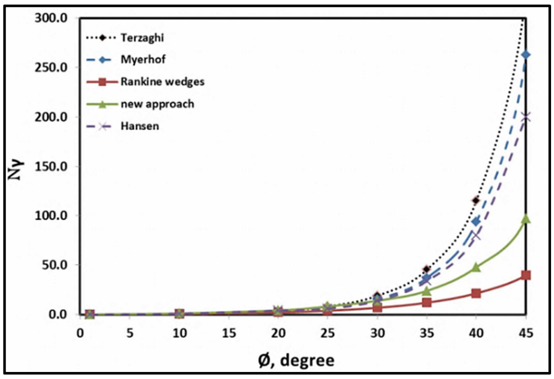


Fig. 11. Comparison of Nγ values with other values obtained from some approaches.

Appendix A

Table A-1. Classical formula of bearing capacity factors, after Sieffert, J.G., y Bay-Gress, Ch. (2000)[11].

Author	N_γ	N_c	N_q
Terzaghi[1]	$\frac{\tan\varphi}{2}\left(\frac{k_{p\gamma}}{cos^2\varphi}-1\right)$ $k_{p\gamma}$ is given in table	(N_q-1) cotφ	$\frac{a^2}{2cos^2\left[\left(\frac{\pi}{4}\right)+\left(\frac{\varphi}{2}\right)\right]}$ $with\ a = \exp[\left(\frac{3\pi}{4}-\frac{\varphi}{2}\right)tan\varphi]$
Meyerhof[2]	(N_q-1) tan (1.4φ)	N_q-1) cotφ(	$tan^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)\exp(\pi\tan\varphi)$
Hansen [12]	N_q-1) tan φ(1.5	N_q-1) cotφ(	$tan^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)\exp(\pi\tan\varphi)$
Vesic [6]	N_q+1) tan φ(2	N_q-1) cotφ(	$tan^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)\exp(\pi\tan\varphi)$
Eurocode 7 [13]	N_q-1) tan φ(2	N_q-1) cotφ(	$tan^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)\exp(\pi\tan\varphi)$

Table A-2. Values of Nq, Nc, and Nγ, after Sieffert, J.G., y Bay-Gress, Ch. (2000).

countries	N_γ	N_c	N_q	Formulae	Curves	Table
Austria (A)	Specific	Specific	Specific	No	Yes	Yes
Czech Republic (CZ)	Meyerhof	Meyerhof	Hansen	Yes	Yes	No
Germany (D)	Meyerhof	Meyerhof	E7[13]	Yes	Yes	Yes
France (F)	Meyerhof	Meyerhof	Giroud	No	No	Yes
Finland (FIN)	Meyerhof	Meyerhof	Hansen	Yes	--	--
Ireland (IRL)	Meyerhof	Meyerhof	Hansen	No	Yes	No
Norway (N)	Meyerhof	Meyerhof	Hansen	No	No	No
Portugal (P)	Terzaghi Meyerhof	Terzaghi Meyerhof	Terzaghi Meyerhof Hansen Vesic	Yes	Yes	Yes
Sweden (S)	Meyerhof	Meyerhof	Specific	Yes	No	No
Slovenia (SLO)	-----	Meyerhof	E7	No	N_c-N_γ	No
Eurocode 7	Meyerhof	Meyerhof	Specific	Yes	No	No

Table A–3. Bearing capacity factors used in the research study, N_c and N_q from Terzaghi (1943) and N_γ from Kumbhojkar (1993) [14].

Ø'	N_c	N_q	N_γ	Ø'	N_c	N_q	N_γ	Ø'	N_c	N_q	N_γ
1	6.00	1.10	0.01	18	15.12	6.04	2.59	35	57.75	41.44	45.41
2	6.30	1.22	0.04	19	16.57	6.70	3.07	36	63.53	47.16	54.36
3	6.62	1.35	0.06	20	17.69	7.44	3.64	37	70.01	53.80	65.27
4	6.97	1.49	0.10	21	18.92	8.26	4.31	38	77.50	61.55	78.61
5	7.34	1.64	0.14	22	20.27	9.19	5.09	39	85.97	70.61	95.03
6	7.73	1.81	0.20	23	21.75	10.23	6.00	40	95.66	81.27	115.31
7	8.15	2.00	0.27	24	23.36	11.40	7.08	41	106.81	93.85	140.51
8	8.60	2.21	0.35	25	25.13	12.72	8.34	42	119.67	108.75	171.99
9	9.09	2.44	0.44	26	27.09	14.21	9.84	43	134.58	126.50	211.56
10	9.61	2.69	0.56	27	29.24	15.90	11.60	44	151.95	147.74	261.60
11	10.16	2.98	0.69	28	31.61	17.81	13.70	45	172.28	173.28	325.34
12	10.76	3.29	0.85	29	34.24	19.98	16.18	46	196.22	204.19	407.11
13	11.41	3.63	1.04	30	37.16	22.46	19.13	47	224.55	241.80	512.84
14	12.11	4.02	1.26	31	40.41	25.28	22.65	48	258.28	287.85	650.87
15	12.86	4.45	1.52	32	44.04	28.52	26.87	49	298.71	344.63	831.99
16	13.68	4.92	1.82	33	48.09	32.23	31.94	50	347.5	415.14	1072.80
17	14.60	5.45	2.18	34	52.64	36.50	38.04				

Table A–4. Variation of Meyerhof's Bearing Capacity Factors N'_q, N'_c and N'_γ (Meyerhof 1951).

Ø'	N_c	N_q	N_γ	Ø'	N_c	N_q	N_γ	Ø'	N_c	N_q	N_γ
1	5.14	1.00	0.00	18	13.10	5.26	2.00	35	46.12	33.30	37.15
2	5.38	1.09	0.002	19	13.93	5.80	2.40	36	50.59	37.75	44.43
3	5.63	1.2	0.01	20	14.83	6.40	2.87	37	55.63	42.92	53.27
4	6.19	1.43	0.04	21	15.82	7.07	3.42	38	61.35	48.93	64.07
5	6.49	1.57	0.07	22	16.88	7.82	4.07	39	67.87	55.96	77.33
6	6.81	1.72	0.11	23	18.05	8.66	4.82	40	75.31	64.20	93.69
7	7.16	1.88	0.15	24	19.32	9.60	5.72	41	83.86	73.90	113.99
8	7.53	2.06	0.21	25	20.72	10.66	6.77	42	93.71	85.38	139.32
9	7.92	2.25	0.28	26	22.25	11.85	8.00	43	105.11	99.02	171.14
10	8.35	2.47	0.37	27	23.94	13.20	9.46	44	118.37	115.31	211.41
11	8.80	2.71	0.47	28	25.80	14.72	11.19	45	133.88	134.88	262.74
12	9.28	2.97	0.60	29	27.86	16.44	13.24	46	152.10	158.51	328.73
13	9.81	3.26	0.74	30	30.14	18.40	15.67	47	173.64	187.21	414.32
14	10.37	3.59	0.92	31	32.67	20.63	18.56	48	199.26	222.31	526.44
15	10.98	3.94	1.13	32	35.49	23.18	22.02	49	299.93	265.51	674.91
16	11.63	4.34	1.38	33	38.64	26.09	26.17	50	266.89	319.07	873.84
17	12.34	4.77	1.66	34	42.16	29.44	31.15				

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