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Frictional Pullout Resistance and Settlement Criteria of Reinforced soil system

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Abstract

An analytical approach, adopted to find the frictional pullout resistance and settlement of foundations resting on reinforced soil based on the test results of circular footing and site data from plate bearing tests (PLT) for in-situ testing of (PLT) with a diameter of (0.75m) , is summarized. The soil was reinforced using biaxial geomesh. The settlement was determined by considering the compatibility of strain (settlement) between soil and reinforcement element underneath the foundation. Empirical equations were used to estimate the settlement either from the superstructure loads or from in-situ plate load tests on the reinforced soil system, while the frictional pullout resistance of reinforcement calculated and compared with test results based on the new empirical equation. The study concerned two different types of geomesh(CE111 and CE121). It was found that initial horizontal and vertical movement of the reinforcement is needed to mobilize the reinforcing strength. Further, the initial settlement at small loads could be avoided when the reinforcement was placed closer to the base of the footing (U<< B/2) and there was an improvement in the bearing capacity value of the footing. When the reinforcement is placed away from the base of the footing, the initial settlement decreased with a slight improvement in the bearing capacity compared with that of un-reinforced soil. Non-dimensional factors were developed for the settlement and frictional pullout resistance based on the experimental site test results.

الخلاصـــــــة

أقترح في هذه الدراسة تحليل لحساب قوة مقاومة السحب لشرائح التسليح أسفل الأسس المسلحة ترابيا وكذلك الهبوط المتوقع مبنيا على التجارب المختبرية ونتائج فحص تحميل الصفيحة (PLT) لفحوصات عديدة لفحص تحميل الصفيحة بنفس مواصفات نموذج الموديل المختبري ولصفيحة قطر٧٥, . متر ،حيث تم تسليح التربة بواسطة قطع(شبكة) تسليح بلاستيكية. كان مبدأ التحليل قد استند إلى التوافق في الانفعال الحاصل تحت الأسس لكل من التربة وقطع التسليح. أن المعادلات التطبيقية المقترحة هي لحساب الهبوط تحت هذا النوع الأسس سواء بسبب أحمال المنشأ أو من فحص تحميل الصفيحة، في حين قوة مقاومة السحب لشرائح التسليح تم اقتراحها في معادلات أيضا، استخدم نوعين من شرائح التسليح في هذه الدراسة. لوحظ من أجل الحصول على حركة(رأسية/ أفقية) تحت الأسس المسلحة ترابيا، فلابد من وجود أحمال أكبر من تلك المقارنة بالأسس غير المسلحة ترابيا،وأكثر من ذلك أن هذا الهبوط الأولي يمكن تجنبه عندما توضع هذه الشرائح قريبة من الأسس حيث كان تحسن في سعة التحمل لهذه الأسس وكذلك لقرب شرائح التسليح من الأساس ويقل هذا التحسن عند وضع هذه الشرائح بعيدة من الأسس.معاملات لابعدية اقترحت لحساب مقاومة السحب لشرائح التسليح وكذلك الهبوط المتوقع تحت هذه الأسس من النتائج الحقلية.

1. Introduction

Reinforced earth technique is one of the most promising materials that have emerged in the last 30 years from intensive research that has been carried out into alternative construction materials. Reinforced earth technique is not new, the earliest remaining examples of soil reinforcement are ziggurat of ancient city of Dur-Krigatzu in Iraq (6000 B.C.), and the Great Wall of China. It is also known that Romans have used earth reinforcement technique (Ignold 1982) **[1].**

Binquet and Lee (1975 a&b), investigated the mode of failures below the strip footing and a new analysis method using limit equilibrium method were reported in order to calculate the bearing capacity below the strip footing., the shear bands developed beneath the footing with small strains outside the active zone**[2].**

Akinmuusru and Akibolade(1981) investigated laboratory test on square footing resting on reinforced sand; reinforced by rope of fiber material, the study showed that bearing capacity ratio (BCR) increased with increasing number of reinforcement layers up to three layers after that little gain in the value of (BCR) were obtained**[3].**

Sulaiman (1991) investigated the interface between two adjacent footings resting on reinforced sand , the results showed that plain sand both bearing capacity and settlement of adjacent footings are increased when the space separating them was small, the highest improvement was recorded at single layer of reinforcement found to be (1.172)for square footing and (2.5) for strip footing**[4].**

Mekkiyah,H.,M. (1993), studied the behavior of reinforced sand using circular model footing subjected to cyclic loading the results have been shown that the bearing capacity increase with increasing number of reinforcing layers and with decreasing the depth of top most reinforcement layer. Also, the application of varying amplitude on reinforced sand causes a stiffening effect on soil dynamic parameters which depends significantly on the load sequence adopted, while the bearing capacity increased up to three after such cyclic loading**[5]**.

Mekkiyah,H.,M. (2003) investigated the comparisons between the bearing capacities using the dimensionless factors for Circular Footings under static loadings. Non-dimensional factors are adopted (I,Jand M) which were found useful in estimation such comparison. The bearing capacity increased up to 3 rapidly when the value of U is close to the footing base [Top most reinforcement layer depth below the footing] Maximum improvement happened when the umber of layers increased up to 3**[6]**.

Mekkiyah,H.,M. (2007), investigated to calculate the modulus of elasticity of reinforced soils and the settlement from empirical equations, it was founded that the modulus of elasticity can be increased by a range of an average of six time the value of unreinforced soil and the settlement reduced in values compared from different site tests results**[7].**

Further, there are a limited number of studies in the literature on the possibility of using analytical developed equations to estimate the frictional pullout resistance of reinforcement bellows the footing and its settlement. This paper reports the initial Findings of such a study and attempts to provide a relatively simple approach to estimate the frictional pullout resistance and settlement of reinforced soil system.

2. ANALYTICAL AND EXPERIMENTAL APPROACH

The use of reinforcement (Geosynthetics) to improve the bearing capacity of footings and to reduce settlement has been proven to be cost-effective for foundation system. A reinforced soil-foundation system consists of one or more layers of geosynthetics and a control soil placed below the footing. The reinforcement is usually placed horizontally. However, there are cases in which vertical or sloped reinforcement may be used below the footing. Further, the reinforcement placed within the tensile arc of strain field causes realignment of the strain field which improves performance in both stiffness and load carrying capacity (Jones 1985) **[8]**. The ideal reinforcement pattern for the direction of the principal tensile strain is shown in Figures (1) and (2). As shown in these figures, the ideal pattern has a reinforcement placed horizontally below the footing and becomes progressively more vertical further from the footing (Bassat and Last 1978) [9].

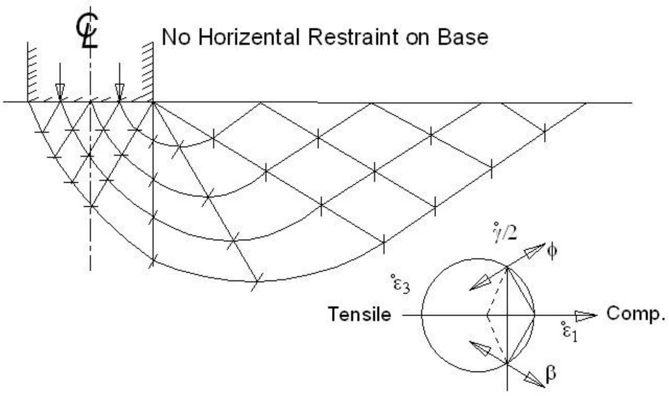

Figure (1) Zero extension characteristics for dilating soil (After Bassat and Last, 1978) [9].

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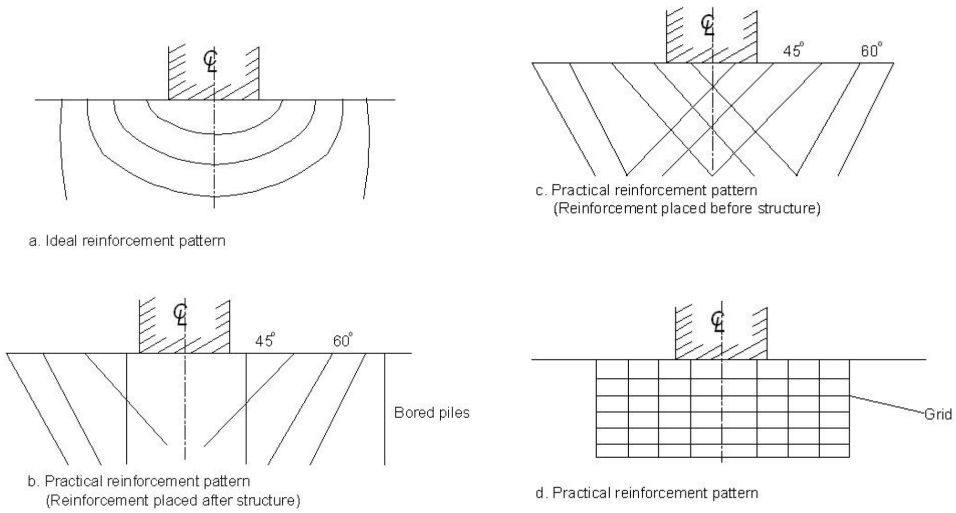

Figure (2) Different reinforcement orientations below the footing (After Bassat and Last, 1978) [9].

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The calculation of footing immediate settlement for different soil types are estimated on the basis of elasticity, provided that the elastic properties of the soil (modulus of elasticity E, and Poisson's ratioυ) are known. These two parameters can be evaluated in the lab from soil samples obtained during site investigation processes for cohesive soils. However, for granular soils, it is much more difficult, if not impossible in most cases. The in-situ testing on granular soils may not accurately give these soil properties which are needed for the calculation of settlement. In the case of reinforced soil systems, it seems to be difficult to use traditional investigation methods such as borings, or to use other traditional techniques such as pressure meter tests or cone penetrometer tests. Such methods and techniques require drilling to various depths which will deform the reinforcement mesh below the footing. Plate bearing test on reinforced foundation systems resting on homogeneous sand to a sufficient depth, on the other hand, can be used as an economical alternative. From the plate bearing tests data which can be used to estimate the overall modulus of the soil which provides a representative parameter for use in conventional settlement estimation.

The improvement in the modulus of subgrade reaction from different studies and site data as a result of reinforcement is in the range of 2 to 10 times that of unreinforced soils. It was assumed that the modulus of elasticity of reinforced soil (E_R) will be increased by the same ratio (i.e., E_R=(2-10)E_S), where E_S is modulus of elasticity for unreinforced soil and E_R can be estimated from equations (1)[Mekkiyah(2007)] [7].

$$E_R = (FI) * Ksun * B\,(1-\upsilon^2) \text{ -------------------(1)}$$

Where:
E_R: Modulus of elasticity for reinforced soil.
FI: Improvement factor (FI = 2 and 10 for 1 and 3 reinforcement layers respectively)
Ksun: The subgrade reaction value of unreinforced soil.
B: Footing width (for an equivalent square).
υ: Passion's ratio (recommended ranges are between 0.28 and 0.34 for 3 and 1 reinforcement layers respectively).

While the settlement below a reinforced soil system can be estimated from equation (2) [Mekkiyah (2007)] [7]; which should be used with the following limitations in mind:

- Best estimation for base contact pressure (q) should be used.
- For the circular footing it is better to convert the footing width to equivalent square.
- The sand layer depth can cause settlement to a depth of Z= 1.5 to 2 times B or to a depth where a hard stratum is encountered below the base.

$$\text{---------------------------- (2)}\ \delta_{FlP} = 0.8\frac{q \times B}{E_R}$$

Where:
: Footing and/or plate settlement (cm). δ_{FlP}
q: Load from (superstructure) on footing and/or plate(Kg/cm^2).
B: Footing width (an equivalent square) (cm).

When the previous limitations are considered, the settlement estimated from the above equation gives good correlation with the test results. Another method of analysis was proposed for settlement estimation by adopting a non-dimensional factor for any size of footing or plate bearing dimensions. The value of α factor that will provide a settlement of 25 mm is used in equation (3) [Mekkiyah(2007)] [7]:

$$\delta_F = \frac{2\delta_P}{\left(B_P / B_F\right)^{\alpha}} \quad (3)$$

Where:

: Footing settlement (mm) δ_F

: Settlement from footing and/or plate bearing test (mm) δ_P

α : Non dimensional factor as shown in Figures (6-13) [Mekkiyah (2007)] [7].

B_P: plate size (m).

B_f: footing size (m).

By using the plate load-settlement curve for δ_F of 25mm, the value of the corresponding bearing pressure can be found from the curve of the computed value of δ_P from equation (3). This bearing pressure is the safe pressure for a given permissible settlement (δ_F) which can not make any distortion for the reinforcement in the site area from the plate bearing test, or one can run a reverse calculation to find out the safe pressure for the settlement criterion. If the footing is allowed to settle for (50 mm) then the value of (α) obtained from Figs. 6-13 should be increased by 20-25%.

3. YIELD CRITERION IN REINFORCED FOUNDATION SYSTEMS

The yield stress is defined when permanent deformation initiates. The yield stress which is a boundary to separate the elastic and plastic deformation for soils is usually not clearly defined and is not a constant value. The locus of the stress at which a soil yields is called yield surface. The stresses smaller than yield stresses cause the soil to respond elastically, and stresses larger than yield stresses cause the soil to respond in an elastoplatic way. The yield stress for soil continuously increases or decreases as the soil hardens or softens. The load settlement curves for reinforced soil systems were found to be elastic when the reinforcement is placed close to the base of the footing (i.e., $U \leq B/2$). The previous studies verify this behavior, and higher yield stresses were obtained at failure due to reinforcement location in this zone (when U is smaller than or equal to B) which is due to the inclusion of additional confining stresses in the soil. The additional confining stresses are the result of the placement of the reinforcement in the soil. The failure criterion in the medium dense reinforced sand have been defined as the bearing capacity at which the settlement is twice the settlement at 60-75% of the safe bearing pressure for the case of $U \leq B/2$ (Fig. 3), Further, the failure criterion in the medium dense reinforced sand has been defined as the bearing capacity at which the settlement is twice the settlement at 80-90% of the safe bearing pressure for the case of $U \geq B$ (Fig. 4). From the tests results it was found that δ_1 is clearly smaller than δ_2, which clearly shows the benefit of reinforcement inclusion in the zone of tension arc, where the zone of high tensile stresses exists. Figure (5) shows the general load settlement trends for both cases.

Additionally, the footing on a reinforced foundation system is more likely to experience a gradual failure curve than a plunging failure. This clearly shows that the settlement is highly reduced when reinforcement is placed closer to the base of footing, while it is improved in a lesser degree when reinforcement is placed further from the footing (Figures 3 and 4).

The value of δ_p obtained from equation (3) represents the value $2\delta_1$ and /or $2\delta_2$ in figures (3) and (4) in order to verify the safe pressure in the proposed yield failure criterion for reinforced footing systems. The plate load tests should not be used to determine the ultimate bearing pressure of footings resting on sandy soils because scale effects in such a case give misleading results.

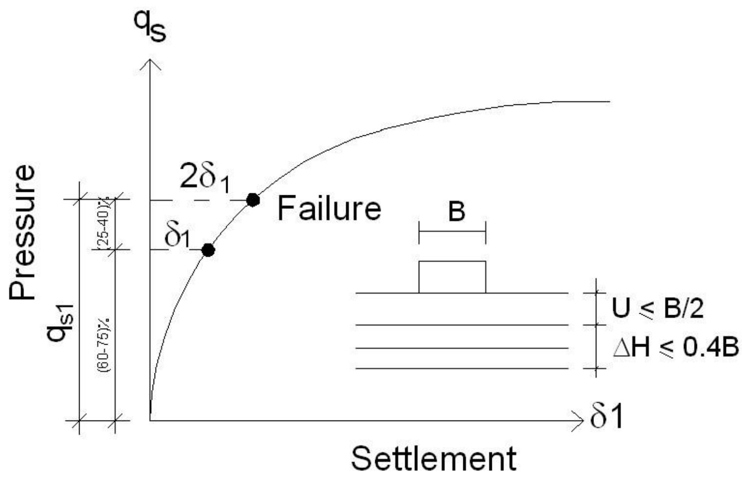
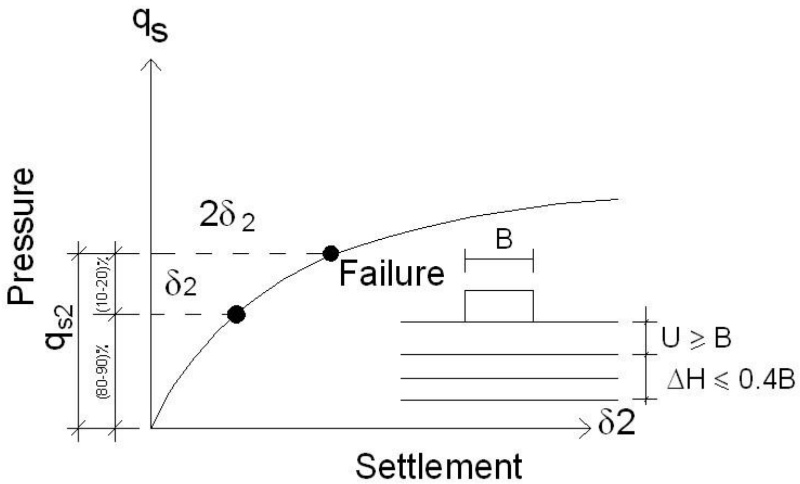


Figure (3) Safe bearing capacity (qs) for the settlement criterion of circular footing resting on reinforced subgrads (U ≤ B/2) [Mekkiyah (2007)] [7].



Figure(4) Safe pressure (qs) for the settlement criterion of circular footing resting on reinforced subgrads (U ≥ B)[Mekkiyah(2007)] [7].

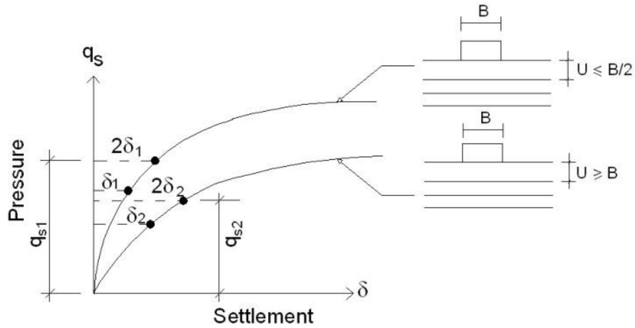


Figure (5) Safe bearing capacity (qs) for the settlement criterion for RFS *(U≤ B/2 and U≥ B)* reinforced subgrads) [Mekkiyah (2007)] [7]*.*

It was also noted that, when the reinforcement was placed in the zone of maximum soil shear, it acted to significantly inhibit the development of a classical bearing failure. The results in the next figures (6-13) [Mekkiyah(2007)] [7]. Clearly demonstrate that reinforcement below the shallow footing on sand can reduce the amount of the settlement, especially differential settlement under the four corners of footings. Footings resting on unreinforced sandy soil settled unevenly, while footings on reinforced soil settled evenly with no tipping of any corners during the observation for the settlement values at the corners after ending the plate bearing test.

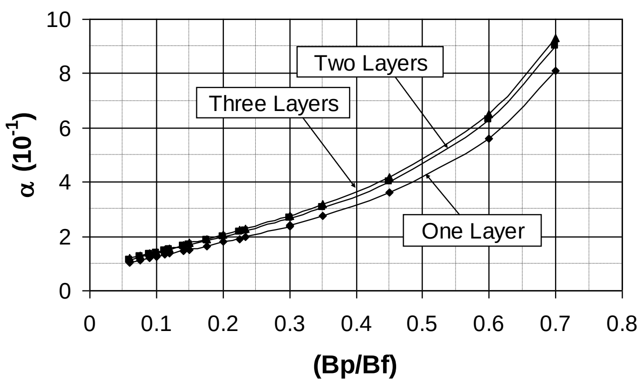


Figure (6) α **- (Bp/Bf) relationships for (U=B/6) CE111.**

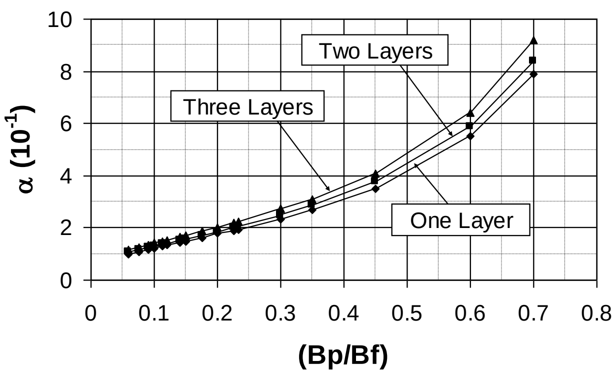


Figure (7) α **- (Bp/Bf) relationships for (U=B/3) CE111*.***

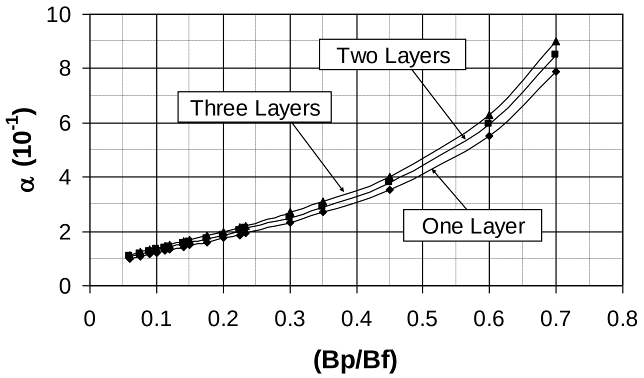


Figure (8) α - (Bp/Bf) relationships for (U=B/2) CE111.

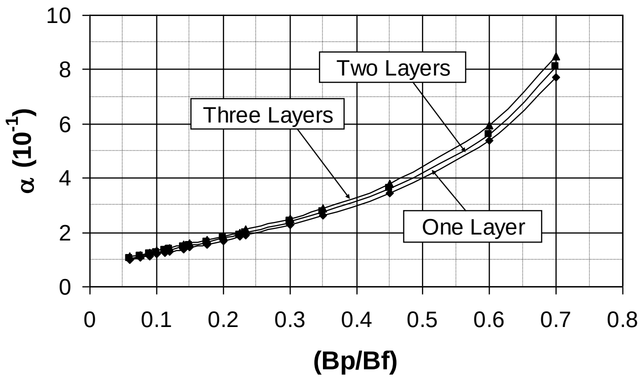


Figure (9) α - (Bp/Bf) relationships for (U=B) CE111.

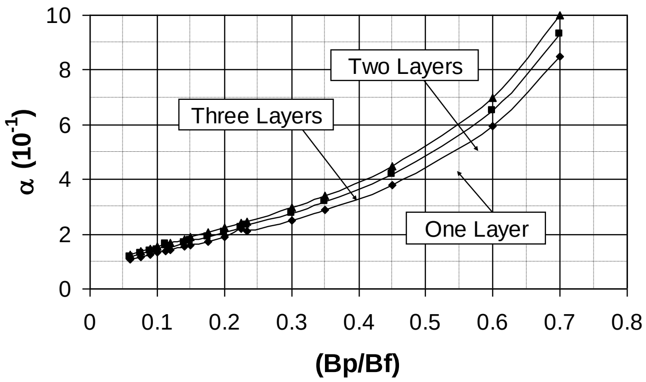


Figure (10) α - (Bp/Bf) relationships for (U=B/6) CE121.

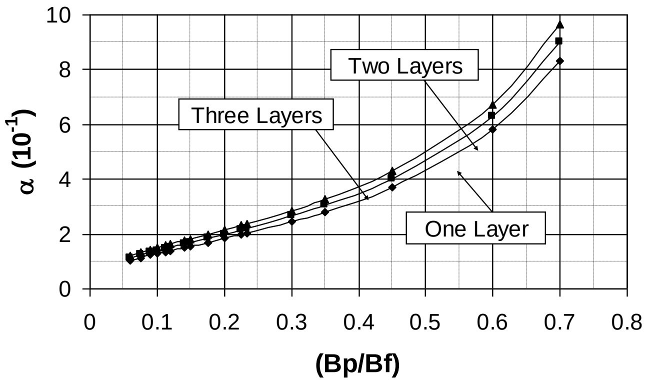


Figure (11) α - (Bp/Bf) relationships for (U=B/3) CE121.

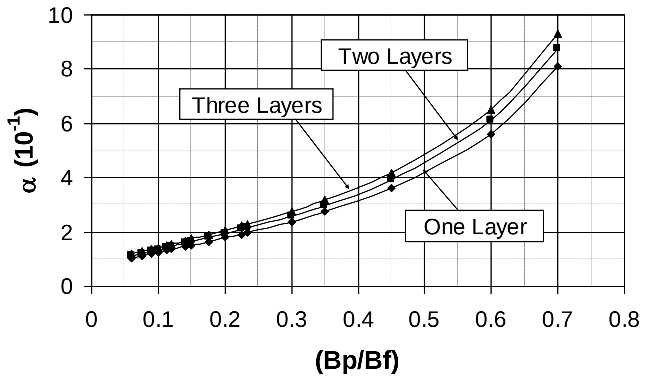


Figure (12) α - (Bp/Bf) relationships for (U=B/2) CE121.

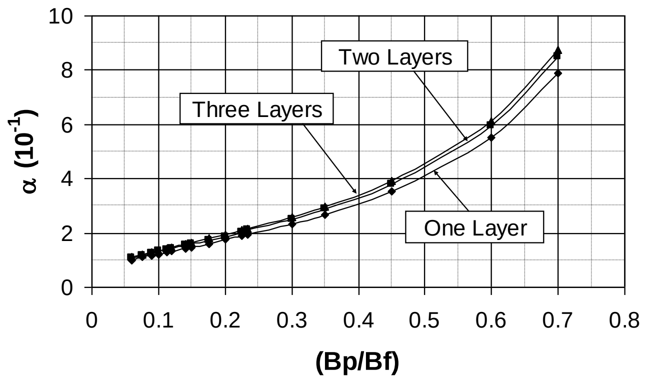


Figure (13) α - (Bp/Bf) relationships for (U=B) CE121.

4. MODIFIED DESIGN METHOD FOR THE BEARING CAPACITY OF SURFACE FOOTING

Based on the test values of bearing capacity of strip footings resting on reinforced sand proposed by Binquet and Lee (1975) [2], new relationships were developed in this study to obtain the frictional pullout resistance of model circular footings resting on reinforced sand. The dimensionless factors proposed by Binquet and Lee (I, J, and M in Fig 14) were modified to new dimensionless factors for circular footings. It is noted that the output of applying the modified equations gives higher values of bearing capacity for circular footings unless a reduction factor (η_1 and/or η_2) is applied to get values closer to the actual bearing capacity as obtained from the model tests. This reduction factor was developed from the data analysis of test results using a computer program. The output of the analysis is shown in Figs. (15) and (16). The figures show the relationship between η_1 and η_2 with U/D for different number of layers. Further, additional tests were performed at U/D of 1/3 to get the experimental bearing capacity of the circular footing resting on one, two, and three layers of reinforcement. The resulted values of bearing capacity from the tests compared well with the values obtained from the modified equations after using the reduction factors. If the reduction factor is not applied, the newly proposed equations for circular footings will give values of bearing capacity that are discordant with the actual expected values of bearing capacity (Fig.(17) Shows these differences).

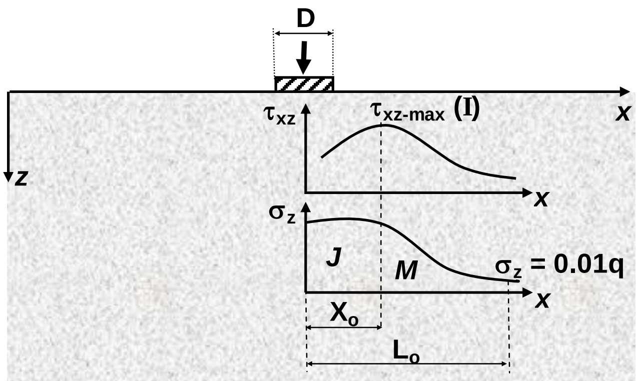


Figure (14) Stress Distribution Below the Circular Footing [Mekkiyah(2003)] [6].

4-1 The Modified Equations

The modified equations developed for surface circular footings on reinforced sands are [Mekkiyah(2003)] [6]:

…………………………..(4)

$$TD(Z,N)=\frac{1}{N}\left[\frac{\int_{0}^{x_o}\sigma_{z(z/a)}dx}{q}D-\frac{\tau_{xz(\max)}}{q}\Delta H\right](\eta q-q_0)$$

$$Tf=2f(LDR)\left[\frac{\int_{X_o}^{L_o}\sigma_{z(z/a)}dx}{q}D|1-\eta|q+\gamma(L_0-X_0)z\right] \quad \text{………………………… (5)}$$

Where: *TD(Z,N)* is the developed reinforcement stress in any layer of reinforcement and depth, Tf is the frictional pullout resistance of the reinforcement layer, *D* is the diameter of circular footing, *a* is the radius of footing, X_0 is the distance from the center line of footing to

the location of maximum shear stress (Lambe and Whitman, 1979) [10], L_0 is the distance from the center line of footing to the location when σz is equal to 0.01q , σz(z/a) is the vertical stress in soil at any depth (z) and distance away from centerline, *f* is the soil layer coefficient of friction which is defined as (tan (ϕ_f)/FS), ϕ_f is the soil-layer friction angle, FS is the factor of safety for the layer pullout, *LDR* is the linear density ratio for the reinforcement, *ΔH* is vertical spacing between reinforcement layers τ xz(max) is the maximum shear stress in soil at depth z and distance away from the centerline of the footing, η, |1-η|is the reduction factor (η_1 and/or η_2) in Equations 4 and 5 and the absolute value of the reduction factor .
q_o is the bearing capacity of circular footing on unreinforced soil, *q* is the bearing capacity of circular footing on reinforced soil, and γ is the soil density (kg/cm^3).It is important to mention that the vertical spacing between reinforcement layers (*ΔH*) was not tested at values larger one third of the size of the footing. Further, the number of reinforcement layers was limited between 1 and 3 and the size of the footing (*D*). The soil layer coefficient of friction (*f*) was calculated using a factor of safety of 3. The results reported in this study are based on the given limits only and the effect of changing the limit of any variable on the results should be examined by running new tests with the new limits. Intensive calculations were done to calculate the stress below circular footings (vertical stress and the location of maximum shear stresses are as defined in Fig. 14). The results are presented in Figs 18, 19, and 20 where numerical integration were carried out using 1/3 Simpson rule for the zones (J and M).

4-2 Comments on the Method Proposed by Binquet and Lee

The following points include some comments on the method proposed by Binquet and Lee:

4-2-1 Effect Number of Layers

Binquet and Lee (1975) [2] assumed that:

$$TD(Z,N) = TD(Z,N=1)/N \quad \ldots\ldots\ldots\ldots\ldots\ldots.. (6)$$

Where the developed tension force in the reinforcement elements per layer varies inversely with the number of layers (N). For example, TD in both cases A and B (Fig. 21) are the same based on the assumption of Binquet and Lee. However, in reality TD is not the same for cases A and B because of the difference in U value, where ΔH_1 is constant for both cases.

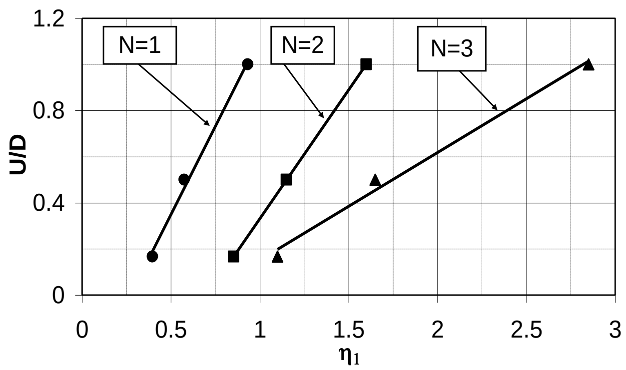


Figure (15) U/D Versus Reduction Factor (η_1) for Netlon CE111[Mekkiyah(2003)] [6].

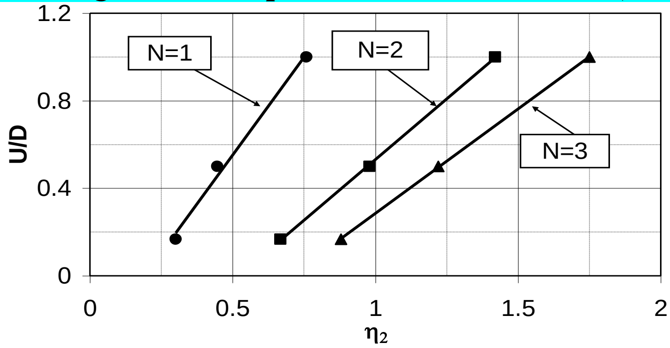
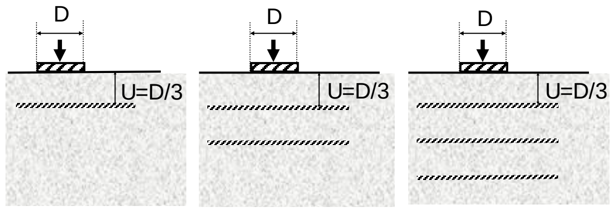


Figure (16) U/D versus Reduction Factor (η_2) for Netlon CE121 [Mekkiyah (2003)] [6].

4-2-2 Effect of Tensile Strength

Tensile strength of reinforcing element has no effect in the equation of Binqunt and Lee as shown in Fig.21 for both cases C and D, the bearing capacity calculated by the equations of Binquet and Lee is the same. However, it is expected that the bearing capacity will be different in each case because the tensile strength of the reinforcement used in C is different than that used in D. As a result the reduction factor adopted (η_1 and/or η_2) take into consideration the effect of U on the bearing capacity of circular footing in addition to the reinforcement strength which is reflected finally on the value of TD.



q_{Exp}= 0.5	q_{Exp}= 0.58	q_{Exp}= 0.66
$q_{mod\text{-}Equ}$= 0.58	$q_{mod\text{-}Equ}$= 0.54	$q_{mod\text{-}Equ}$= 0.62
$q_{B\&L}$= 0.27	$q_{B\&L}$= 0.66	$q_{B\&L}$= 1.22
η_1= 0.48	η_1= 0.98	η_1= 1.33

The units of q_{EXP}, $q_{mod\text{-}equ}$, and $q_{B\&L}$ are kg/cm^2
q_{Exp}: expected bearing capacity from test results.
$q_{mod\text{-}Equ}$: expected bearing capacity by the modefied equation.
$q_{B\&L}$: expected bearing capacity as suggested by Binquet and Lee along with dimensionless factors I, J, and M.

Figure (17) Comparisons between the Bearing Capacities Using the New Dimensionless Factors for Circular Footings I, J, and M [Mekkiyah (2003)] [6].

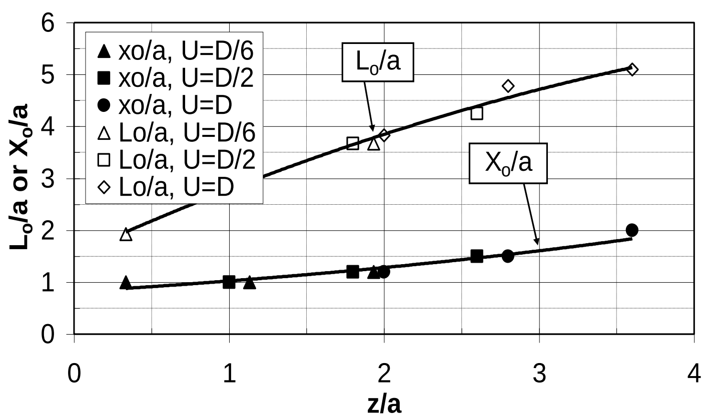


Figure 18 (Z/a)-(Lo/a) or (Xo/a) Relationship [Mekkiyah (2003)] [6].

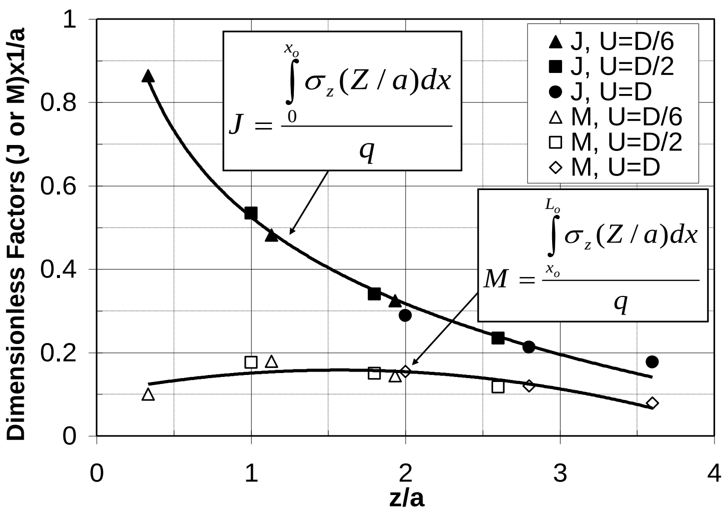


Figure (19) (Z/a)-J or M Relationship for the Effect of Vertical Stress [Mekkiyah (2003)] [6].

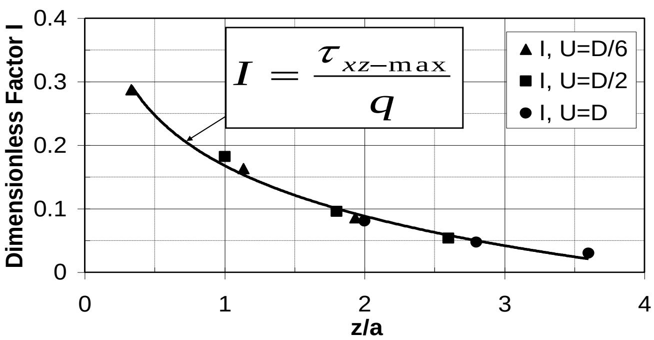


Figure (20) (Z/a)-I Relationship for the Location of Maximum Shear Stress [Mekkiyah (2003)] [6].

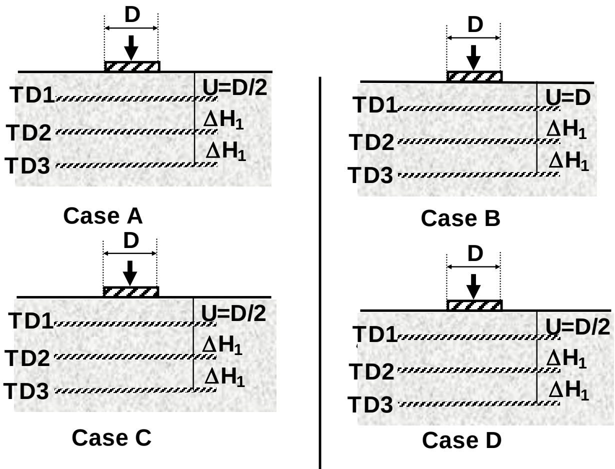


Figure (21) Developed Stress (TD) in the Reinforcement layers Under the Footing Based on the Method of Binquet and Lee [Mekkiyah (2003)] [6].

5. Conclusions

The following main conclusions are drawn from the test results.

- The depth of top most reinforcement layer is found to be more effective when it is located near the base of the footing and the bearing capacity increased up to 3 rapidly when the value of U (top most reinforcement layer) is close to the size of the base of the footing, and the number of layers of reinforcement is three.
- It is found that bearing capacity increased when the number of layer increased up to 3, after that there is little improvement in the bearing capacity.
- The settlement is smaller when a stiff geo-grid is used below the footing. (i.e. high pullout tensile resistance to carry the loads).
- The failure criterion in the medium dense reinforced sand have been defined as safe bearing capacity at which settlement is twice the settlement at 60%-75% of q_s for the case of ($U \leq B/2$), while the reinforced layer at depth of ($U \geq B$), the failure criterion can be defined also near to that of un-reinforced and medium sand at 80%-90% percentage of q_s. This amount of reduction in settlement are shown from that the value of $\delta 1 << \delta 2$.
- The safe bearing pressure for footing rest on reinforced soil can be estimated with (Fs=3) from equation (2) after getting (δp) from equation (3); in condition that a plate load test should be achieved.

- The new modified equations are derived from the equations of Binquet and Lee (1975) **[2]** for strip footings were modified as follows: First, the dimensionless parameters (I, J, and M) were developed based on elasticity theory of stress below circular footings. Second, the reduction factor (η_1 and/or η_2) was introduced to reflect the effect of the depth of top most reinforcement layer along with the tensile strength of reinforcement. Further, two types of figures (Figs. 15 and 16) were introduced to reflect the effect of the pullout tensile strength of reinforcement on the value of reduction factor. The resulted values of bearing capacity from the tests compared well with the values obtained from the modified equations after using the reduction factors.

6. References

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