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# Behaviour of footings on reinforced sloped fills

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ABSTRACT: A series of bearing capacity tests with footings placed on unreinforced and reinforced slope were performed to investigate the performance of footing located at the crest of a sloped fill influenced by the presence of a reinforcing layer within the body of the fill. Fly ash; a waste product coming out of the thermal power plants was used as fill material. The results of the investigation indicate that fly ash can be satisfactorily used for such geotechnical applications. An analysis has also been proposed to obtain the ultimate bearing capacity of the footing on reinforced sloped fill. Comparison of the theoretical analysis with test results shows good agreement.

## 1 INTRODUCTION

In some engineering practice, heavy structures such as bridge abutments or other traffic facilities have to be constructed close to the crest of a slope. The estimation of the ultimate load capacity of such foundation is important for safe and efficient design of such foundations (Meyerhof 1963; Winterkorn and Fang 1975, Shields et al. 1977). The positioning of the footing in relation to the edge of the embankment fill is another important aspect; which has implications not only on the safety but also on the economy and efficiency of the overall design of such structures. In view of the escalating cost of construction with the use of conventional materials, utilization of industrial wastes for such major geotechnical application has been highly emphasised during last few decades to bring economy in construction. To overcome difficulties in getting conventional material; fly ash has been successfully used as a structural fill or embankment material in a number of construction projects throughout the world. Difficulty in using fly ash is that it has low bearing capacity. In recent years, reinforced earth technique has been looked upon as a cost-effective solutions to many such soil and foundation engineering problems. Reinforced fly ash embankments shall, therefore; have promising potential in the days to come, where the fly ash will provide the bulk of the mass and the reinforcement will provide the necessary strength to the mass of the geotechnical system. Reinforcement of the embankment in such situation will not only offer better performance in terms of their load supporting capacity but also can be made stable under an externally applied load at relatively steeper slope resulting in minimum of material requirement and land acquisition.

The beneficial effects of incorporating tensile reinforcement in soil fills has been described by several researchers since the pioneering work by Vidal. As of now; reinforced earth technique has become a powerful construction technique for civil engineers that has demonstrated its value in wide variety of practical applications such as steep reinforcement slopes, retaining walls, shallow slips in clay embankment, embankments on soft soils, working platform and unpaved roads etc. The advent of improved polymer materials has further added a new dimension to the efficiency of design and construction of such reinforced earth structures. The reinforcing techniques concentrate on the use of geogrids in improving the performance (i.e. the load-carrying capacity and settlement) of the abutment fill. Geogrids are essentially synthetic reinforcing mesh structures which have been used to enhance the bearing capacity of soils. Examples such applications are given by Binquet and Lee (1975 a, b), Basset and Last (1978), Akinmusuru and Akinbolade (1981), Fragaszy and Lawton (1984) and Verma and Char (1986).

Although reinforced soil has become popular in recent years, the problem of the behaviour of a footing loaded in the vicinity of the crest of reinforced slope has received only limited attention (Gnanendran & Selvadurai 1989, Huang & Tatsuoka 1994, Manjunath et al. 1994). The scope of the investigation in all these studies was particular to the specific problem in hand and purely experimental in nature. Therefore; in the present investigation, after conducting many model tests, theoretical analysis of the problem has been done and based on theoretical analysis, equations have been suggested for determination of ultimate bearin capacity of the reinforced slopes.

## 2 EXPERIMENTAL PROGRAMME

The model tests were conducted in an open ended masonry tank having dimensions of 1.80 m x 1.20 m x 0.30 m with its frontage made of 12 mm thick perspex sheet, which facilitated the observation of failure surface at the end of each experiment. The rigid footing was modelled by a seasoned teak wood section measuring 300 mm x 100 mm (L x B) in plan area. A schematic view of the test facility alongwith the equivalent loading diagram for calculating the load on the footing at failure is shown in Figure 1. The fly ash used for the model tests were procured directly from the electrostatic precipitators of Tata Iron & Steel Company Limited, Jamshedpur. The standard Proctor's density and the corresponding O.M.C. was found to be 9.34 $kN/m^3$ and 48% respectively. The value of apparent cohesion(c) and the angle of internal friction ($\phi$) were about 20 kPa and $14^0$ respectively. Commercially available polypropylene model geogrids, 0.27 mm thick and 300 mm wide having tensile strength equal to 4.0 kN/m and tie-soil friction angle ($\phi_\mu$) equal to $35^0$ were used as reinforcing elements. The models were prepared by compacting the fly ash at O.M.C. in layers of 150 mm thickness. The placement dry density was about 90% of the Standard Proctor's density. The compacted fly ash was first built upto the desired height and the reinforcement was placed at the specified location and the filling was then continued upto a height of 750 mm. The longitudinal sides of the test tank were covered with thin and transparent sheets coated with white grease in order to reduce friction between the soil and the sides of the test tank and to induce a state near plane strain in the tested soil mass. The fly ash was compacted to the required level with reinforcing elements buried at specified locations. Extra soil was cut out from one side of the fill with the help of a sharp edged trowel to form the slope of $45^0$. The model footing was then placed in position at the surface of the compacted fill and then loaded gradually until failure.The corresponding settlements were recorded by using two dial guages placed diagonally opposite.

The following parameters were chosen in the present study :

1. The ratio of the distance of the edge of the footing from the crest of the slope ($D_e$) to the width of the footing (B) i.e. $D_e/B$.
2. The ratio of embedment depth of the reinforcing layer (Z) to the width of the footing (B) i.e. Z/B.

The ratio De/B was varied from 1.0 to 3.0 while the ratio Z/B was varied in the range of 0.25 to 3.0.

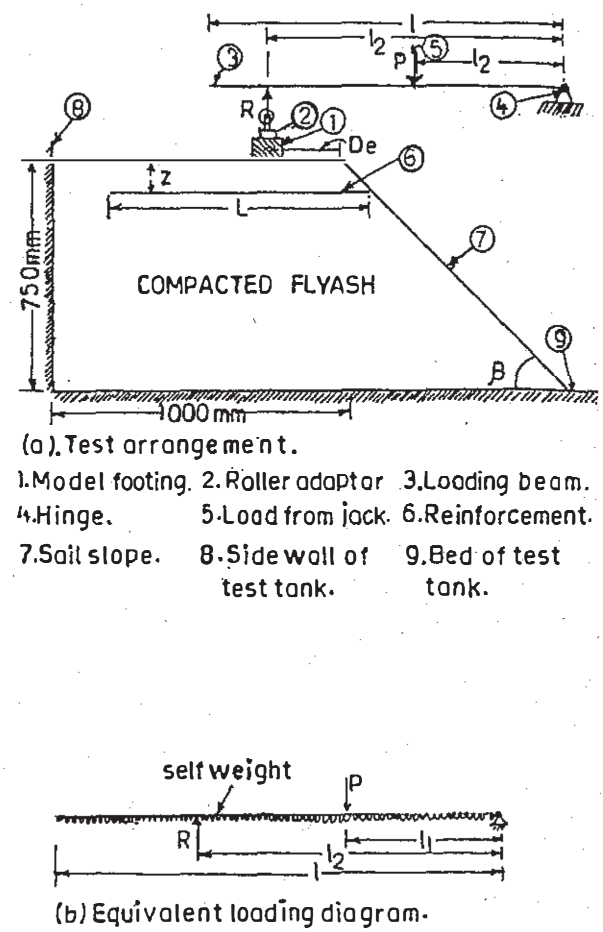


Figure 1. Test arrangement & equivalent loading diagram.

The length of reinforcing element was kept constant as L = 10B throughout the test programme.

## 3 THEORETICAL ANALYSIS

### 3.1 *Bearing capacity of unreinforced slopes*

Of the various theories available for determination of bearing capacity of slopes, the theory presented by Meyerhof (1957) with certain modifications in context of compacted fly ash slopes seems to be in good agreement with experimental results. The bearing capacity is generally computed for a footing either on top of a slope or on face of slope as

$$q_{ult} = cN_{cq} + \tfrac{1}{2}\gamma BN_{\gamma q} \quad (1)$$

Theoretical bearing capacity calculated with the above equation gave results higher than those obtained experimentally. Therefore, the above equation needs to be modified keeping in view the compressibility of fly ash. In order to take the effect of compressibility of fly ash into acocunt, it is proposed to modify equation (1) by incorporating the soil compressibility factor in equation (1) as suggested by Vesic (1973) from the anology of expansion of cavities. The Meyerhof's equation in its modified form for bearing capacity of footings on slopes may therefore, be written as

$$q_0 = cN_{cq}F_{cc} + \tfrac{1}{2}\gamma B N_{\gamma q} F_{\gamma c} \qquad (2)$$

where:

$N_{cq}$, $N_{\gamma q}$ are Meyerhof's bearing capacity factors and are functions of $D_e/B$ for different inclinations of slope. The variation of $N_{cq}$ and $N_{\gamma q}$ are presented in the form of curves (Meyerhof 1957). $\gamma$ is the unit weight of the soil. $F_{cc}$, $F_{\gamma c}$ are soil compressibility factors as proposed by Vesic (1973) and are functions of soil friction angle ($\phi$) and rigidity index ($I_r$). $q_0$ is the ultimate bearing capacity for unreinforced fly ash slope.

### 3.2 *Bearing capacity of reinforced slopes*

Since the primary objective is to evaluate the efficiency of the reinforcement, it is convenient to present the results for the reinforced soil system with respect to an identical unreinforced system. The most comprehensive work relating to the bearing capacity of reinforced subgrades is presented by Binquet and Lee(1975b) and the same has been extended for the case of footings resting on the top of a slope by modifying some of the assumptions to make it more realistic. The prominent among these assumptions are :

1. At the plane separating the downward and lateral movements of soil mass, the geogrids are assumed to undergo two right angle bend around two frictionless roller.
2. The geogrid-soil friction co-efficient is assumed to vary with respect to depth as per the following relation :

$$f_e = mf \qquad (3)$$

where :

$f$ = Geogrid–soil friction coeffcient = tan $\phi_\mu$
$f_e$ = Mobilized geogrid-soil friction co-efficient
m = Mobilization factor given by;

$$m = (1.0 - Z/B)\,0.15 + 0.85 \ (4.a) \qquad (4a)$$
for $Z/B \le 1.0$

$$m = (3.0 - Z/B)\,0.25 + 0.20 \ (4.b) \qquad (4b)$$
for $Z/B > 1.0$

Incorporating the above mobilization factor into the expression for tie pull out frictional resistance of any tie placed at a depth Z (Binquet & Lee 1975b), the mobilized tie pull out frictional resistance ($F_B$) is given by

$$F_B = 2\,f_e(LDR)[A_3 B L q_0(q_R/q_0) + \gamma\, L(L_0 - X_0)Z] \qquad (5)$$

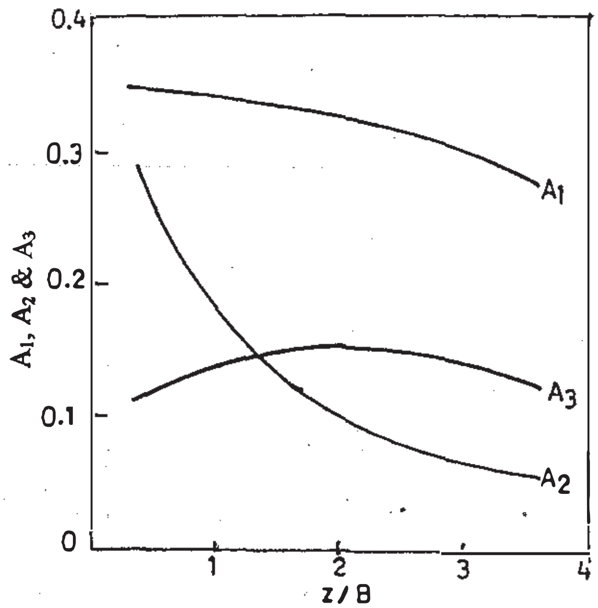

Figure 2a. Variation of $A_1$, $A_2$ & $A_3$ with Z/B.

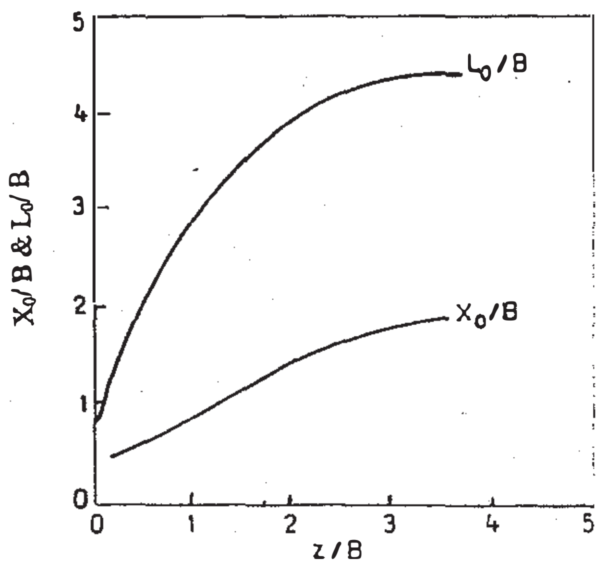

Figure 2b. Variation of $L_0/B$ & $X_0/B$ with Z/B (after Binquet & Lee, 1975 b).

The corresponding value of tie force ($T_{rj}$) may be obtained by substituting the value of ultimate bearing capacity for unreinforced slope ($q_0$) into the expression for tie force i.e.

$$T_{rj} = q_0[(q_R/q_0) - 1]\,(A_1B - A_2Z) \qquad (6)$$

Considering the limiting equilibrium condition and equating the developed tie force (Eq.6) to the mobilized tie pull out frictional resistance (Eq.5) and on rearranging, we get:

$$q_R = q_0 + [2f_e(LDR)L\{A_3 B q_R + \gamma\,(L_0 - X_0)Z\}] \,/\, (A_1B - A_2Z) \qquad (7)$$

where :

$A_1$, $A_2$ and $A_3$ are dimensionless forces and $L_0$ and $X_0$ denotes the locii of the points of maximum shear stress and the point where the vertical stress intensity is only 1% of the applied bearing pressure. All these quantities are functions of depth ratio (Z/B) and their variation with depth ratio are presented in the form of curves as shown in Figure 2a. and Figure 2b. The term (LDR) refers to linear density ratio and represents the total width of reinforcement ties per unit length of the footing. On substitution of the various values, Equation (7) can be solved for the desired bearing pressure $q_R$. The theoretical bearing capacity values, $q_{th}$ for the various cases are tabulated in Table 1.

crease in the load carrying capacity is primarily due to improved redistribution of load provided by the geogrid. While at greater depth of embedment ((Z/B)>2.0)), the presence of the geogrid reinforcement does not lead to a significant improvement in the load carrying capacity as major portion of failure surface passes through unreinforced portion and consequently, frictional resistance offered by the geogrid is less. The detailed outline of the results are presented in Table 1. From Table 1 it can also be seen that for a given depth ratio, there is a marked increase in the bearing capacity as the edge distance is increased from B to 3B. However, for a given depth ratio there is marginal decrease in the BCR values as the edge distance is increased.

Table 1. Outline of the results.

| Sl.No | Z/B | Experimental B.C. $q_{ex}$ ($kP_a$) | | | Theoretical B.C. $q_{th}$ ($kP_a$) | | |
|---|---|---|---|---|---|---|---|
| | | $D_e = B$ | $D_e = 2B$ | $D_e = 3B$ | $D_e = B$ | $D_e = 2B$ | $D_e = 3B$ |
| 1 | U.R | 65 | 76 | 88 | 72.75 | 77.71 | 81.16 |
| 2 | 0.25 | 94 | 99 | 107 | 87.18 | 93.11 | 97.23 |
| 3 | 0.50 | 122 | 128 | 140 | 93.63 | 99.93 | 104.30 |
| 4 | 0.75 | 128 | 140 | 154 | 103.12 | 109.88 | 114.57 |
| 5 | 1.0 | 131 | 137 | 149 | 113.80 | 121.01 | 126.03 |
| 6 | 1.50 | 114 | 126 | 139 | 109.73 | 116.43 | 121.08 |
| 7 | 2.0 | 104 | 115 | 128 | 106.77 | 113.18 | 117.64 |
| 8 | 2.50 | 91 | 105 | 115 | 99.52 | 105.46 | 109.60 |
| 9 | 3.0 | 82 | 94 | 106 | 94.30 | 99.97 | 103.92 |

Note : U.R = Unreinforced, B.C = Bearing capacity

## 4 RESULTS AND DISCUSSIONS

It is convenient to present results for the reinforced slope with respect to the corresponding results obtained for its unreinforced counterpart. A term bearing capacity ratio(BCR) has, therefore, been introduced to analyse the test data and is defined as follows :

$$BCR = q_R / q_0$$

in which; $q_R$ and $q_0$ are bearing capacity for the reinforced and unreinforced cases respectively. With the help of proposed analysis, the bearing capacities of footings on reinforced/unreinforced slopes were calculated for varying edge distances and depth ratios. The results obtained analytically are presented in terms of BCR. Finally, the results were compared with the experimentally obtained BCR values for the various cases. The variation of BCR with depth ratio for varying edge distance, are presented in Figure 3. From Figure 3, it can be seen that for any given edge distance, there is a considerable increase in the ultimate bearing capacity of the footing resting on reinforced slope as compared to that of unreinforced slope. However, the increase in load carrying capacity is significant upto a depth ratio of 1.0 and the increase becomes marginal beyong depth ratio of 2.0. At lesser depth of embedment ((Z/B)<1.0), the in-

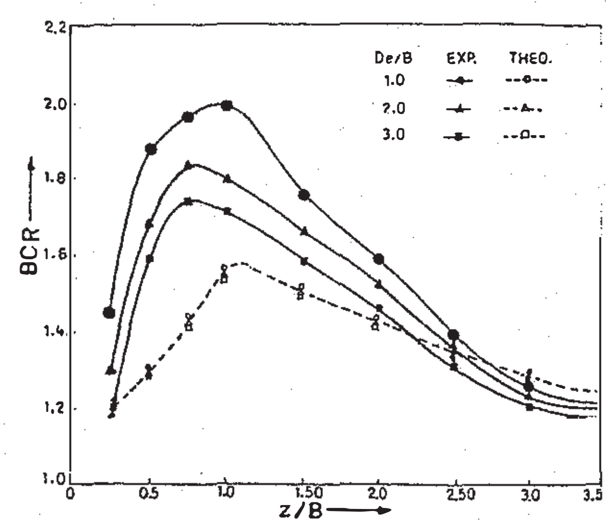

Figure 3. Variation of BCR with Z/B.

## 5 CONCLUSIONS

1. Fly ash can be successfully used even in steep faced embankments and the load carrying capacity of such embankments can be enhanced by 100% by incorporating a geogrid reinforcement
2. The edge distance affects the load carrying capacity of unreinforced as well as reinforced slopes

and load carrying capacity increases with increase in edge distance for both the cases.

3. The optimum location of the geogrid reinforcement is at a depth between 0.5 and 1.0 times the width of the footing.

4. The location of the geogrid layer at a depth greater than twice the width of the footing does not lead to significant improvement in the load carrying capacity.

5. Comparison of the analytically obtained results with model test data shows a good agreement and equation (7) can be used to evaluate bearing capacity of footings resting on reinforced slopes.

The results of the present investigation are sufficiently encouraging and may be extended to prototype.

## 6 ACKNOWLEDGEMENT

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