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Load Bearing Capacity of Footing Resting on the Fly Ash Slope with Multilayer Reinforcements

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ABSTRACT

In several parts of the world, the disposal of waste materials like fly ash is a challenging task. The applications of fly ash as structural fills in foundations is one of the best solutions to disposal problems, because they can be used in large volumes in such application. There may be difficulty due to poor load-bearing capacity of fly ash, especially when footings rest on the top of the fly ash fill slope; but inclusion of polymeric reinforcements as horizontal sheets within the fill may be advantageous in improving the load-bearing capacity of reinforced fly ash slope. The aim of present investigation is to find out the efficacy of multi layer reinforcements in improving the load-bearing capacity when incorporated within the body of fly ash embankment. An increase in load bearing capacity due to the incorporation of reinforcement layers in the model slope was observed in the laboratory tests. The experimental results were compared with the numerical findings obtained from the finite element analysis using commercial software PLAXIS 2D version 9.0.

INTRODUCTION

Use of geosynthetic reinforcements for improving load-bearing capacity of foundation has been extensively reported by researchers. Many times foundations need to be located either on the top of a slope or on the slope itself, for examples, building foundation near river banks, bridge abutment resting on granular fill slope, foundations on embankments, roads constructed on hill slopes, etc. When a footing is constructed on a sloping ground, the load bearing capacity of the footing may be reduced significantly depending upon the location of the foundation with respect to

slope. The improvement of load-carrying capacity of such loaded slopes is therefore one of the important aspects of geotechnical engineering practice because structures are liable to be unsafe due to slope failure. One of the possible solutions to improve the load bearing capacity would be reinforcing the sloped fill with the layers of geogrid. Limited studies on load bearing capacity behaviour of strip footings on reinforced slopes have been reported in the literature (Selvadurai and Gnanendran 1989, Huang et al. 1994, Lee and Manjunath 2000, Yoo 2001, EI Sawwaf 2007, Alamshahi and Hataf 2009, Mittal et al. 2009). A comprehensive overview of geosynthetic-reinforced slopes with some specific recommendations was also reported by Shukla et al. (2011). Due to decreasing availability of good construction site and increasing infrastructural developments throughout the world, the unsuitable sites, especially the low-lying areas are now being used increasingly after filling it with industrial waste whose load bearing capacity is generally low. Industrial wastes particularly the fly ash when locally available and if found suitable can reduce the construction cost apart from encouraging the sustainable development and reducing the environmental problems. In several parts of the world, disposal of fly ash is a great problem and requires a large land area. Fly ash when used as structural fills or as embankments offers several advantages over borrow soils. It is light weight, exerts low overburden pressure on subgrade as a fill material. Compaction curve of fly ash is relatively flat, thus implying that construction is less sensitive to compaction-moisture content than that of the fine grained soils commonly used as structural fill. Fly ash being non-plastic will also solve the problem of dimensional instability as exhibited by plastic soils, which alternatively shrink and swell with seasonal changes in moisture content. Additionally properties of fly ash from a given source are likely to be more consistent as compared to the soil from natural borrow areas. But despite having several advantages of fly ash over borrow soil, there may be difficulty due to poor load-bearing capacity of fly ash, especially when footings rest on the top of the fly ash fill slope. In such case inclusion of polymeric reinforcements as horizontal sheets within the fill may be one of the most viable solutions for improving the load-bearing capacity of fly ash slope. Studies on bearing capacity of shallow foundations on a level and plain fly ash ground have been reported in literature but very limited studies on load carrying capacity behavior of footing resting on a reinforced fly ash slopes are available in the literature (Choudhary and Verma 2001, Choudhary et al. 2010, Jha et al. 2010, Gill et al. 2011). The load bearing capacity determination technique is an important part for proper design of footings on reinforced slope. In recent years, numerical analyses such as finite difference and finite element analyses have become popular in design practices. However despite many attempts, no obvious method for determination of ultimate bearing capacity of strip footing resting on reinforced slope is available and therefore much investigation still remains to be carried out. In the present investigation the laboratory model tests were carried out to

find out the efficacy of multilayer reinforcement in improving the load-bearing capacity when incorporated within a fly ash embankment slope. Numerical analysis was also carried out using commercial software PLAXIS 2D version 9.0 to verify the model test results.

LABORATORY MODEL TEST

Materials

Fly ash procured from TISCO (Tata Iron and Steel Company Limited), Jamshedpur, India was used throughout the investigation. The properties reported in this study are specific to the fly ash collected from this source only. As per the particle size distribution, fly ash consists of 68% silt and 28% sand. Maximum dry density and optimum moisture content (OMC) of fly ash were 9.34 kN/m^3 and 48% respectively. Triaxial tests under undrained condition were conducted to determine the value of apparent cohesion (c) and angle of internal friction (ϕ). The values were 20 kPa and 14° respectively. Commercially available polypropylene model geogrids (0.27 mm thick and 300 mm wide) having an average tensile strength of 4.0 kN/m and tie-soil friction angle (ϕ_μ) equal to 35° were used as reinforcing elements. The rigid foundation was modeled by footings made of good quality well-seasoned sal wood. The model footings of size 300mm x 100mm were used for conducting the model tests, since the length (L) to width (B) ratio is 3, it will behave as a strip footing. The thickness of model footings was 70 mm. The base of the model footing was made rough by cementing a thin layer of sand to it with epoxy glue.

Preparation of slope

The required quantity of dry fly ash was mixed with a predetermined amount of water corresponding to the optimum moisture content (OMC). The well mixed fly ash was then spread in the tank in five equal layers, each 180 mm thick, which was finally compacted to 150mm thick layer. Uniform compaction of each layer was achieved by passing a 70 kg smooth towed roller for thirty times over a wooden plank of size 1600 mm (length) x 300mm (width) placed on the fly ash layer. Initially, a few trial foundation beds were made for standardization of compaction and checking of unit weight. In order to verify the uniformity of test bed, undisturbed samples were collected from different locations of the test bed in order to determine the in-situ unit weight and the values were found to be almost same (coefficient of variability within 1.5%). The placement dry unit weight/density achieved by this procedure was 90% of the standard proctor density. To ensure uniform moisture distribution throughout the test, compacted fly ash bed was left for 24 h for moisture equilibration and the top surface was kept covered with wet gunny bags in order to prevent the moisture loss if any. After 24 h; the compacted fly ash bed was cut to desired slope with the help of a

sharp edged trowel. In case of reinforced fly ash slope, the reinforcements were placed at the desired depth within the fill and the compaction was continued in a similar manner until the desired height was reached. At any given position, the location of the reinforcement was such that it extended up to the face of the slope. The experimental set up and test procedure has been reported in detail by Choudhary et al. (2010). The geometry of the test configuration is shown in Figure 1. Different parameters used in the Figure 1 are D_e - edge distance from slope crest, B - width of footing, z - embedment depth and L_r - length of reinforcement respectively and their values used in model test is shown in Table 1.

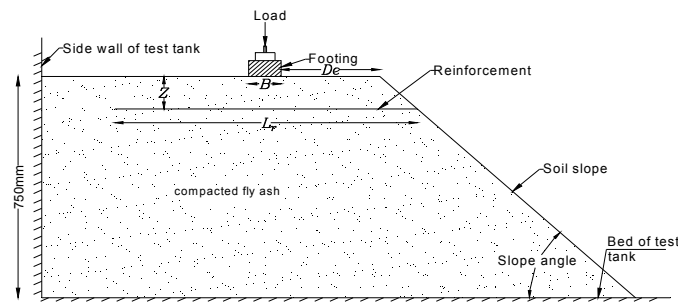


Figure 1. Schematic view of the test configuration

Test parameters

Yoo (2001) reported that for deriving maximum improvement in ultimate bearing capacity the optimum L_r/B ratio (L_r = length of reinforcement, B =Width of footing) should be in the range of 6.0-8.0. Michalowski (1997) reported that the length of reinforcement should be 0.65 times the slope height to prevent collapse of slope due to reinforcement rupture, pullout or direct sliding. The length of the reinforcement (L_r) to be used in the present investigation was decided by conducting a numerical study using Plaxis 2D version 9, explained below for a single layer of reinforced fly ash slope having z/B and D_e/B ratio equal to 1.0. The trend between ultimate bearing capacity (UBC) and length of reinforcement is shown in Figure 2. No significant increase in ultimate bearing capacity was observed when L_r/B ratio increased from 7.0 to 11.0. Therefore the length of reinforcement for the present investigation was also kept constant as $7.0B$ throughout the study.

Yoo (2001), EI Sawwaf (2007), Alamshahi and Hataf (2009) reported that for deriving maximum improvement in load bearing capacity of reinforced slope, the vertical spacing between reinforcements should be in the range of $0.50B$ - $0.75B$. Thanapalasingam and Gnanendran (2008) advocated embedment depth for the first and second layer as $0.25B$ and $0.50B$ from the ground surface of sloped fill for multiple reinforcement layers ($N = 4$, where N is the number of reinforcement layers). Based on these reports the embedment ratio (z/B) for reinforced slope ($N = 1-7$) were selected from 0.25 to 3.0 in the present investigation depending on number of geogrid

layers used as reinforcement $[(z_j/B)_{j=1 \text{ to } N} = 0.25, 0.50, 1.0, 1.50, 2.0, 2.50, 3.0]$. The depth of first and second layer of reinforcement from the top surface of bed was kept as $0.25B$ and $0.50B$ respectively and thereafter the vertical spacing between the consecutive layers was maintained as $0.50B$. The details of various test parameters considered during the investigation are given in Table 1.

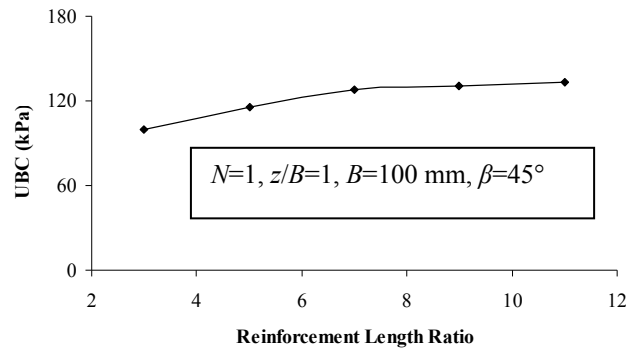


Figure 2 Variation of ultimate bearing capacity with geogrid length ratio

Table 1. Variables of the Study (Experimental and Numerical Study)

Type of test	Constant parameter	Variable parameter
Reinforced slope	$B = 100 \text{ mm}$ $\beta = 45^\circ$ $L_r = 7B$	$(z_j/B)_{j=1 \text{ to } 7} = 0.25, 0.50, 1.0, 1.5, 2.0, 2.50, 3.0$
		$N = 1, 2, 3, 4, 5, 6, 7$
		$D_e/B = 1.0, 2.0, 3.0$

NUMERICAL APPROACH

A series of two-dimensional finite element analyses (FEA) on a reinforced fly ash footing-slope system was performed in order to verify the laboratory model tests results and understand the deformations trends within the soil mass. The analysis was performed using the finite element program Plaxis 2D (professional version 9). Plaxis is capable of handling a wide range of geotechnical problems such as deep excavations, tunnels and earth structures such as retaining walls and slopes. The analysis does, however, require the generation of initial effective stresses by means of K_0 -procedure. The geometry of the reinforced fly ash footing-slope system is the same as that of laboratory model test. It was decided to use the non-linear Mohr-Coulomb criteria to model the fly ash for its simplicity, practical importance and the availability of the parameters needed. The parameters used for numerical analysis are c (Cohesion), ϕ (Friction angle), ψ (Dilatancy angle), E (Young's modulus), μ (Poisson's ratio); they have been obtained from basic test on fly ash samples. Angle of dilatancy (ψ) is generally zero if value of friction angle (ϕ) is less than 30° (Bolton 1986). The other parameters used for numerical analysis are well defined in PLAXIS 2D manual and are given in Table 2. A refined mesh was adopted to minimize the effect of mesh dependency on the finite element modeling for cases involving

changes in the number, length, and the location of geogrid layers. In the finite element modeling, as the slope surface is not horizontal, the initial stress condition of the slope was established first by applying the gravity force due to soil in steps with the geogrid reinforcements in place. A prescribed footing load was then applied in increments (load control method) accompanied by iterative analysis up to failure. For convergence to occur the desired minimum and maximum number of iteration selected was 4 to 10. By default the maximum number of trial calculation step was set as 250 and in all cases the ultimate level achieved before this step and thereafter calculation stopped (www.plaxis.nl). The modeled boundary conditions were assumed such that the vertical boundaries are free vertically and constrained horizontally while the bottom horizontal boundary is fully fixed. The analyzed slope geometry, generated mesh and the boundary conditions of a typical finite element model verified for the optimum case with scale is shown in Figure 3. The optimum condition is obtained when edge distance (D_e) and number of reinforcement layer (N) are equal to footing width (B) and 04 (four) respectively where maximum improvement was achieved. The soil parameters assigned for the top and bottom fly ash layers were assumed to maintain the same in all the finite element analyses for the unreinforced system. The interaction between the geogrid and soil was modeled at both sides by means of interface elements. For the reinforced case, reinforcement layers were assigned at the required depth with suitable strength reduction factors between the contact surfaces and stiffness of the reinforcement entered as additional parameters, which were introduced in interface section. The software enables the automatic generation of 15 node triangle plane strain elements for the soil. The number of element used in reinforced tests for optimum case is 308 element while in unreinforced tests the number is 233. Figure 4 shows a typical effective stress distribution for the optimum condition.

RESULTS AND DISCUSSION

The comparison of experimental and numerical results has been shown through Figures 5-7. A typical variation of pressure and settlement ratio with and without soil reinforcement for different layer of reinforcement ($N = 1-7$) is presented in Figure 5 for the case when edge distance, $D_e = 1.0B$ and slope angle, $\beta = 45^\circ$. It can be seen from Figure 5 that the ultimate bearing pressure of the footing increases with an increase in number of reinforcement layers up to certain value and thereafter any further increase in number of reinforcement layers (N) does not enhance the ultimate bearing capacity of the footing. This trend of variation remains the same both for experimental and numerical results. Similar results were obtained also for other edge distance considered in the investigation, such as $De = 2.0B$ and $3.0B$. Maximum improvement in ultimate bearing capacity occurs when number of reinforcement is four or more for all the edge distance considered in the investigation. In order to study the effect of number of reinforcing layers (N) on the bearing capacity improvement, studies were carried out by changing only the number of reinforcement layers (N). The embedment ratio (z/B) for multilayer reinforcement were varied from 0.25 to 3.0 [$(z_j/B)_{j=1 \text{ to } 7} = 0.25, 0.50, 1.0, 1.50, 2.0, 2.50, 3.0$] depending on number of geogrid reinforcement layers ($N = 1-7$) used. Figure 6 shows the comparison for

experimental and numerical results of ultimate bearing capacity with number of reinforcing layers (N) for different edge distance ratio ($EDR= De/B=1.0, 2.0, 3.0$).

Table 2. Parameters used in Numerical analysis

Parameters	Fly ash	Parameters	Fly ash	Parameters	Wooden Footing	Geogrid
γ_{unsat} [kN/m ³]	13.82	c_{ref} [kN/m ²]	20	EA [kN/m]	88200.0	4.0
E_{ref} [kN/m ²]	8000.0	ϕ [°]	14	EI [kNm ² /m]	36.01	
μ	0.380	Ψ	0.0	M_p [kNm/m]	1E15	
G_{ref} [kN/m ²]	2900.0	R_{inter}	0.55	N_p [kNm/m]	1E15	
E_{oed} [kN/m ²]	14976	Interface permeability	Neutral			

γ_{unsat} : Unit weight, G_{ref} :Shear modulus, E_{oed} :Oedometer modulus, R_{inter} :Strength reduction factor, EA:Axial stiffness,EI:Flexural rigidity, M_p :Maximum bending moment, N_p :Axial force

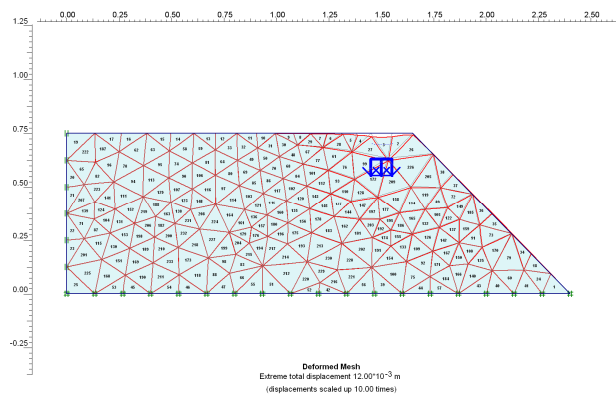


Figure 3. Deformed mesh for the optimum condition

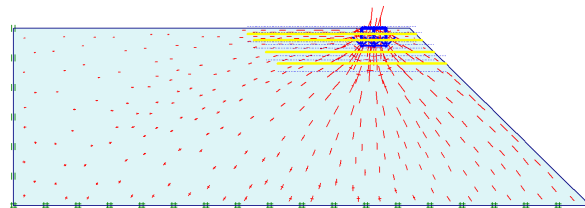


Figure 4. Effective Stresses for the optimum condition

It can be observed that for a given edge distance, bearing capacity, in general, increases with the increasing number of geogrid layers (N) within the fill; however,

the rate of improvement in bearing capacity becomes less significant when the number of geogrid layers incorporated in fly ash fill is four or more. The increase

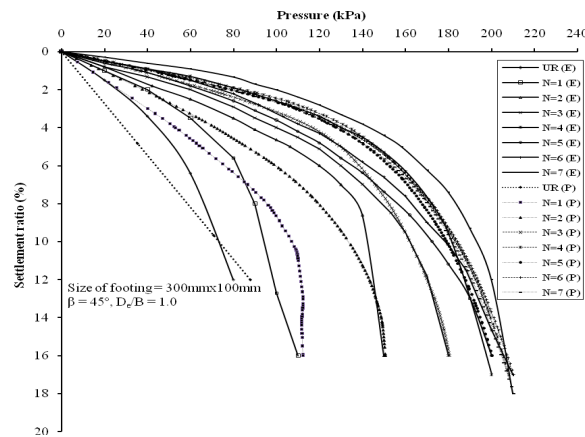


Figure 5. Pressure vs Settlement ratio (Experimental and Numerical results)

in ultimate load carrying capacity of fill slope with increasing number of reinforcing layers can be attributed to reinforcement mechanism derived from the passive resistance, interlocking of the transverse member and adhesion between the longitudinal/transverse geogrid members and the fly ash. When restraining force exerted by reinforcement is imposed on soil elements, the reorientation of strain characteristics associated with the restraint of minor principal strain of soil element

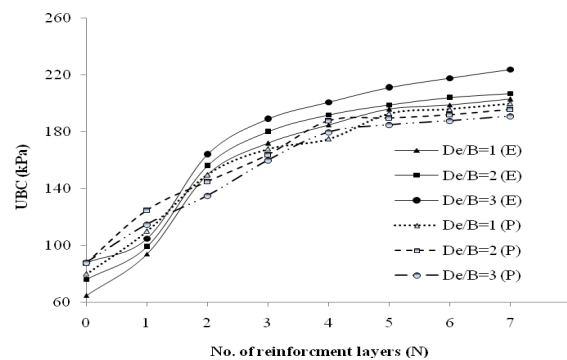


Figure 6. Ultimate bearing capacity vs No. of reinforcing layers

occurs in the vicinity of the reinforcement. A part of the reinforced zone where relatively large reinforcement force has developed, behaves like a part of the rigid footing and transfers a major part of the footing load into a deeper zone. This mechanism can be described as *deep footing effect*. The mobilized passive earth resistance of soil column confined in the geogrid apertures along with the interlocking, limits the spreading of slope and lateral deformations of soil particles.

The mobilized tension in the reinforcement enables the geogrid to resist the imposed horizontal shear stresses built up in the soil mass beneath the loaded area and transfer them to adjacent stable layers of soil leading to a wider and deeper failure zone. Therefore, soil–geogrid interaction not only plays a role in increasing the bearing capacity due to developed longer failure surface but also results in broadening the contact area between soil and rigid bottom surface of the footing. A typical plot of distribution of effective stress for optimum condition shown in Figure 4 also confirms the above analogy. The results shown in Figure 6 have also been presented in terms of a non-dimensional parameter as bearing capacity ratio (BCR) and shown in Figure 7. Bearing capacity ratio (BCR) is defined as a ratio of the load carrying capacity of a footing resting on the reinforced slope to that of the footing resting on the unreinforced slope. This figure shows the comparison of experimental and numerical BCR for all the edge distance ratio ($EDR = D_e / B$) considered in the investigation. It can be noticed that the experimental bearing capacity ratio coincides closely with the numerical bearing capacity ratio, thus confirming the validity of numerical analysis.

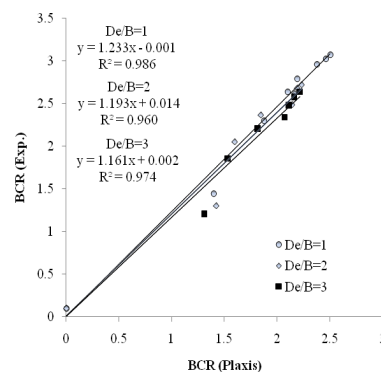


Figure 7. Comparison of experimental and numerical BCR

CONCLUSIONS

The following conclusions may be drawn from the present study from the present study and they are applicable to the situations considered here only.

1. Fly ash can be used successfully as an embankment fill material.
2. Insertion of a geogrid reinforcement layer at a suitable location within the slope fill considerably improves the load carrying capacity of footings located on such slopes.
3. The load carrying capacity increases with an increase in number of reinforcement layers within the slope.
4. The maximum improvement occurs when the number of reinforcement layers is four or more. Any increase in reinforcement layers thereafter does not affect the result significantly.
5. Experimental observations are found to be in good agreement with numerical results.

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