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ORIGINAL PAPER

Behaviour of Prestressed Geosynthetic Reinforced Granular Beds Overlying Weak Soil

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Abstract This paper presents an analytical model for predicting the improvement in bearing capacity of geosynthetic reinforced granular beds overlying weak soil due to prestressing the reinforcement. A punching shear failure mechanism is envisaged in the analytical model. Results obtained from a series of laboratory scale bearing capacity tests and finite element analyses are used to validate the model. The addition of prestress to the reinforcement results in significant improvement of the settlement response and the load-bearing capacity of the foundation. The parameters studied are strength of underlying weak soil, thickness of granular bed, magnitude of prestressing force, direction of prestressing forces, type of reinforcement and number of layers of reinforcement. The bearing capacity ratios (BCRs) values predicted by the analytical model are found to be in good agreement with the experimentally obtained BCR values. Finite element analysis are carried out using the FE program PLAXIS to study the effect of prestressing the reinforcement. Results obtained from finite element analysis are found to be in reasonably good agreement with the experimental results.

Keywords Geogrid · Geotextile · Granular bed · Prestress · Model test · Finite element analysis

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Introduction

The decreasing availability of proper construction sites has led to the increased use of marginal ones, where the bearing capacity of the underlying deposits is very low. In marginal ground conditions, geosynthetics enhance the ability to use shallow foundations in lieu of the more expensive deep foundations. In this technique, one or more layers of geosynthetic reinforcement and a controlled backfill material are placed underneath foundations to improve the strength and to reduce deformations of the foundation system.

A number of studies have expanded the knowledge on the failure mechanisms and the potential benefits of soil reinforcement on the bearing capacity and settlement of shallow foundations [3, 7]. Several experimental and analytical studies were conducted to evaluate the bearing capacity of footings on reinforced soil [1, 5, 7, 8, 10].

Geosynthetics are extensible reinforcements and require some strain for mobilizing the required tensile stress. The strain in reinforcement occurring due to initial settlement is not sufficient to mobilize the required tensile stress in it. Hence geosynthetics demonstrate their beneficial effects only after considerably large settlements, which is not a desirable feature for shallow foundations. Prestressing the reinforcement is a promising technique to increase the load bearing capacity of a geosynthetic reinforced soil without the occurrence of large settlements. Lovisa et al. [4] conducted laboratory model studies and finite element analyses on a circular footing resting on sand reinforced with geotextile to study the effect of prestressing the reinforcement. It was found that the addition of prestress to reinforcement resulted in significant improvement in the load bearing capacity and reduction in settlement of foundation.

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In this paper an analytical model is proposed to predict the improvement in bearing capacity due to prestressing the reinforcement in the granular bed. The proposed analytical model is formulated by improvising the model of Shivashankar et al. [7]. The results of the analytical model are validated by conducting laboratory scale bearing capacity tests and finite element analyses using the FE program PLAXIS version 8. The parameters considered in the study are strength of the underlying weak soil, thickness of granular bed, magnitude of prestress, direction of prestress, number of layers of reinforcement and type of reinforcement.

Laboratory Model Tests

The experimental programme involved a series of laboratory scale plate load tests on model footings resting on prestressed geosynthetic reinforced granular beds (PRGB). Details of the experimental programme, test procedures and analysis of test results are presented below.

Materials

The material used for granular bed is poorly graded medium sand and locally available soil termed as 'Shedi soil' is used as weak soil and properties of both soils are given in Table 1. The values of c and Φ are obtained from direct shear test under UU conditions and verified from plate load tests on the weak soils or sand. Particle size distribution curve of both soils are shown in Fig. 1. The Shedi soil is used in two conditions namely moist condition (termed as moist soil or weak soil 1) and also used in submerged condition (termed as submerged soil or weak soil 2). The shear parameters of weak soil and sand are determined by conducting direct shear test, since preparation of sample for conducting triaxial compression test is difficult. The values of shear parameters obtained are verified from the results of laboratory scale bearing capacity tests conducted on weak soil and sand. As per Unified soil classification system the weak soil used is classified as SP-ML.

Shedi soils are predominantly found in the western coast of peninsular India, which receives heavy rainfall during monsoon. They are dispersive soils and their strength reduces drastically under saturation condition. Many foundation and slope stability problems are reported wherever this soil is encountered [2, 9]. The reinforcements used are geogrid and geotextile and their properties are given in Tables 2 and 3 respectively.

Test Setup

The load tests are conducted in a combined test bed and loading frame assembly. The dimensions of the tank are 750 mm length × 750 mm width × 750 mm depth. The model footing is a rigid mild steel plate of 100 mm × 100 mm size and 20 mm thickness. The footing is loaded by a hand operated Jack of 10 kN capacity supported against a reaction frame. The load is measured using a proving ring and deformation using two dial gauges placed diametrically opposite to each other. Some of the tests were repeated to check the consistency of the experimental results. The figure of the test set up is shown in Fig. 2 and photograph of the same in Fig. 3.

Preparation of Test Bed and Testing Procedure

At first the weak soil is filled in the ferrocement tank to the required level with compaction done in layers, to achieve the pre-determined density. Then sand is filled up to the bottom level of reinforcement and compacted. The reinforcement is then placed with its centre exactly beneath the jack, and the prestress is applied. Then sand above the reinforcement is placed and compacted to the pre-determined density. The densities to which the soils are compacted are indicated in Table 1. The compactive effort required to achieve the required density of both the soils is determined by trial and error. Preparation of underlying soil in all the tests involved compaction of soil using a rammer. In the preparation of granular bed, the sand was compacted using a small plate vibrator.

Tests are carried out with single layer and double layer of reinforcement. In the literature, it is reported that optimum depth of placement of the first layer of reinforcement is 0.2B to 0.5B (B is the width of footing) [6]. The depth of reinforcement from the base of footing is adopted as 0.5B for all the tests with single layer reinforcement. In case of double layer reinforcement, the depth of top layer is 0.25B from the base of footing and that of bottom layer is 0.5B from the base of footing.

The prestress applied is equal to 1, 2 and 3 % of the tensile strength of the reinforcement and is distributed over three pulleys. In uniaxial prestressing the prestress is applied only in the X-direction whereas in biaxial prestressing it is applied in both X and Y directions as shown in Figs. 4 and 5 respectively.

After preparing the bed, the surface is leveled, and the footing is placed exactly at the centre of the loading jack to avoid eccentric loading. The footing is loaded by a hand-operated hydraulic jack supported against a reaction frame. A pre-calibrated proving ring is used to measure the load transferred to the footing. The load is applied in small increments. Each load increment is maintained constant until the footing settlement is stabilized. The settlement is measured using two dial gauges and their average value is adopted. The settlement at the interface between two soils is determined by measuring the levels at specified points at

Table 1 Properties of sand and weak soils used in the model tests

Property	Value		
	Sand	Weak soil 1 (moist soil)	Weak soil 2 (submerged soil)
Specific gravity	2.61	2.32	2.32
Average dry unit weight during model test (kN/m^3)	16.60	16.00	16.00
Void ratio during model test	0.54	0.42	0.42
Water content during model test (%)	0	10	31.5
Effective grain size D_{10} (mm)	0.50	0.11	0.11
D_{60} (mm)	1.30	0.155	0.155
D_{30} (mm)	0.80	0.125	0.125
Coefficient of uniformity C_u	2.60	1.41	1.41
Coefficient of curvature C_c	1.00	0.92	0.92
Friction angle Φ (°)	31.0	12	6
Cohesion (kPa)	0	10	5.5

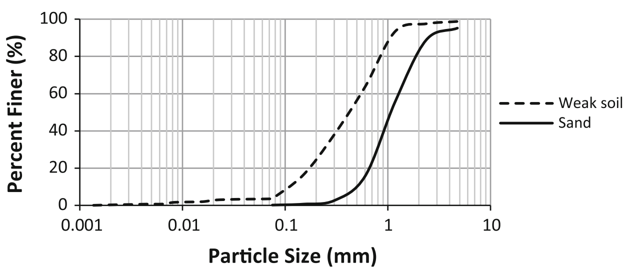


Fig. 1 Particle size distribution of sand and weak soil used

Table. 2 Properties of geogrid used in the model tests

Property	Value
Mass per unit area (gm/m^2)	730.00
Aperture Size (mm)	8 × 6
Thickness (mm)	3.30
Tensile Strength (kN/m)	7.68
Extension at maximum load (%)	20.20
Color	Black
Polymer	HD-polyethyelene

Table. 3 Properties of geotextile used in the model tests

Property	Value
Mass per unit area (gm/m^2)	206.00
Thickness (mm)	0.58
Breaking Strength—Warp (5 × 20 cm) (Kg)	257.7
Breaking Strength—Weft (5 × 20 cm) (kg)	181.90
Extension at Break (%)—Warp	36.90
Extension at Break (%)—Weft	30.20
Interfacial friction angle with sand(°)	30.50
Style (quality no:)	P.D. 381
Material	Polypropylene

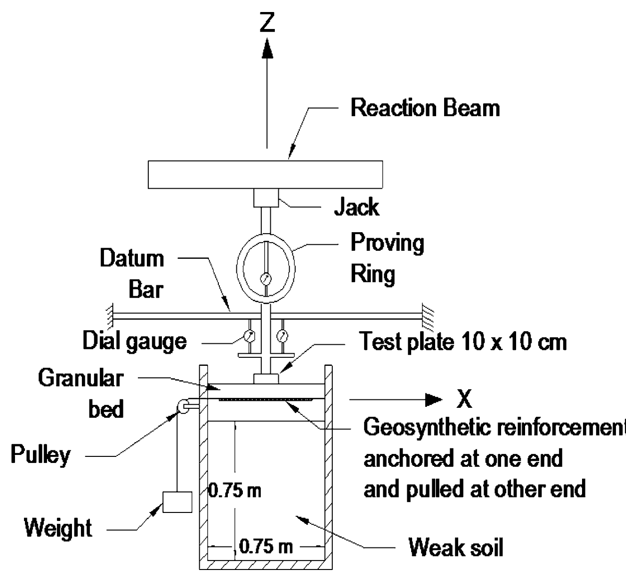


Fig. 2 Test set up

regular intervals on the surface of weak soil before and after each test. The test tank is emptied and refilled for each test to ensure that controlled conditions are maintained throughout the investigation. The details of testing programme are given in Table 4. When shedi soil is in the submerged condition, the level of water table is monitored by installing four peizometers in the test tank.

Analytical Modeling

In the present study, all laboratory scale plate load tests conducted (i.e., unreinforced, reinforced and prestressed reinforced granular beds) are modeled analytically by using the original or improvised model proposed by Shivashankar et al. [7]. They proposed a punching shear failure

Fig. 3 View of test set up

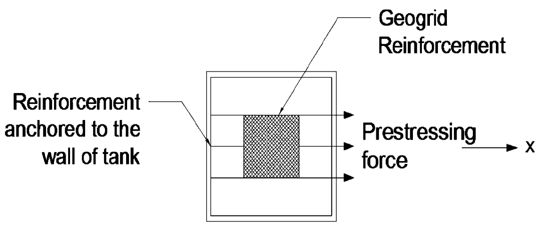


Fig. 4 Uniaxial prestressing

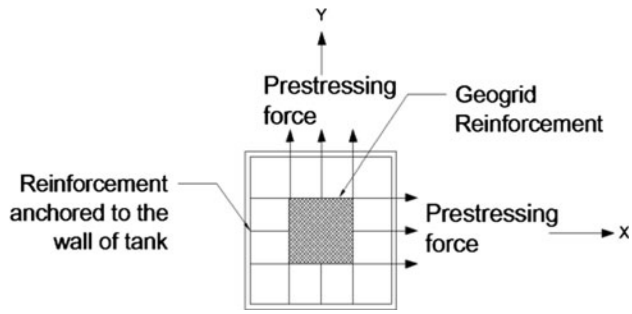


Fig. 5 Biaxial prestressing

mechanism for reinforced granular bed (RGB) [without prestressing the reinforcement], in which both the footing and the portion of the RGB directly beneath the footing are envisaged to act in unison to punch through the soft soil underneath. The improvement in bearing capacity of a RGB is considered to comprise of three components namely Shear layer effect, Confinement effect and Surcharge effect. These effects are represented in Figs. 6, 7 and 8 respectively. They proposed the following equations for computing Bearing Capacity Ratio

$$\mathrm{BCR} = 1 + \Delta\mathrm{BCR}_{\mathrm{SL}} + \Delta\mathrm{BCR}_{\mathrm{CE}} + \Delta\mathrm{BCR}_{\mathrm{SE}} \tag{1}$$

where BCR = Bearing Capacity Ratio $\Delta\mathrm{BCR}_{\mathrm{SL}}$, $\Delta\mathrm{BCR}_{\mathrm{CE}}$, $\Delta\mathrm{BCR}_{\mathrm{SE}}$ = Improvement in bearing capacity ratio due to shear layer, confinement and surcharge effects respectively.

Shear Layer Effect

In shear layer effect, the shear stress mobilized along the failure surface due to the passive pressure developed in soil is considered (Fig. 6). The equation proposed for strip footings is

$$\Delta\mathrm{BCR}_{\mathrm{SL}} = 2\tau_1/\mathrm{Q} \tag{2}$$

$$\tau_1 = P_{\mathrm{p}}\tan\phi_{\mathrm{s}} \tag{3}$$

$$\Delta\mathrm{q}_{\mathrm{SL}} = 2\tau_1/\mathrm{B} \tag{4}$$

where Q is the bearing capacity of underlying weak soil, τ is total vertical force along the punching shear failure plane due to shear layer effect, P_p is force due to passive pressure developed on the sides of failure surface, acting normally, per unit length, ϕ_s is the angle of internal friction of granular material.

The above equation was developed for a strip footing. In the present study, since a square footing is used, the equation is modified as given below

$$\tau_1 = P'_{\mathrm{p}}\tan\phi_{\mathrm{s}} \tag{5}$$

$$\Delta\mathrm{BCR}_{\mathrm{SL}} = 4\tau_1/\mathrm{Q} \tag{6}$$

$$\Delta\mathrm{q}_{\mathrm{SL}} = 4\tau_1/\mathrm{B}^2 \tag{7}$$

where P_p is the passive pressure developed on each of four sides of square column of granular soil beneath the square footing, B is the width of the square footing

Confinement Effect

The tensile stress mobilized in the reinforcement will provide a confinement effect to the soil beneath the footing. The shear stress developed along the failure surface due to this confining stress is considered here (Fig. 7).

The equation proposed for strip footing was

$$\Delta\mathrm{BCR}_{\mathrm{CE}} = 2\tau_2/\mathrm{Q} \tag{8}$$

$$\tau_2 = T_R\tan\phi\mathrm{s} \tag{9}$$

$$\Delta\mathrm{q}_{\mathrm{CE}} = 2\tau_2/\mathrm{B} \tag{10}$$

where τ_2 is the total vertical force along the punching shear failure plane due to confinement.effect of reinforcement, T_R is tensile stress mobilized in the reinforcement is $2L\sigma_{\upsilon}\tan\delta$ L is length of reinforcement beyond the failure surface, σ_V the vertical stress at the level of reinforcement, δ is the angle of friction between reinforcement and soil is the ϕ_s for geogrid

The above equation was developed for a strip footing. In the present study, since a square footing is used, the equation is modified as given below

$$\tau_2 = T_R' \tan\phi_s \tag{11}$$

$$\Delta BCR_{CE} = 4\tau_2/Q \tag{12}$$

$$\Delta q_{CE} = 4\tau_2/B^2 \tag{13}$$

where T_R' is the tensile stress mobilized in reinforcement beyond each of the four sides of square column of granular soil beneath the square footing, B is width of the square footing.

In the case of PRGB, if the friction on reinforcement (on one side of the square prism, along plane of reinforcement) is less than the applied prestress, value of T_R' is taken as equal to the value of applied prestress. If the friction in reinforcement is more than applied prestress, the value of T_R' is taken as equal to value of frictional resistance over the reinforcement.

Additional Surcharge Effect

The vertical stresses along the punching shear failure surface due to shear layer effect and confinement effect are envisaged to act as a surcharge stress on the underlying soft soil. There will be an improvement in bearing capacity due to this surcharge stress. The distribution of this surcharge stress was assumed to be exponential for a strip footing [7], as shown in Fig. 8. The improvement in bearing capacity due to this surcharge stress is given by

$$q_o = 0.84(\Delta q_{SL} + \Delta q_{CE}) \tag{14}$$

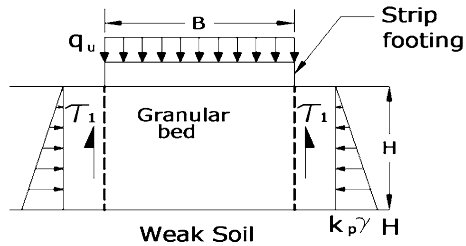


Fig. 6 Shear layer effect for GB, RGB and PRGB [7]

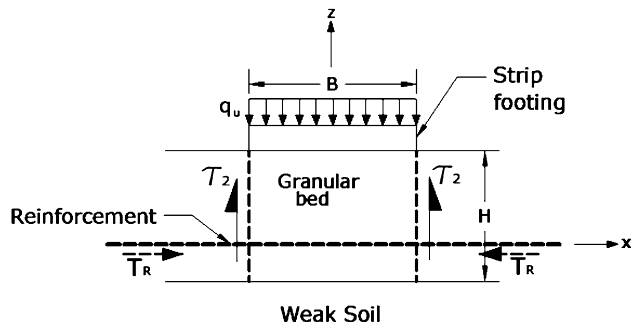


Fig. 7 Confinement effect for GB, RGB and PRGB [7]

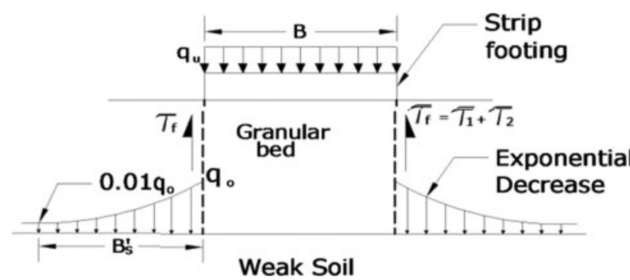


Fig. 8 Surcharge effect for GB and RGB [7]

where q_o is the Intensity of surcharge stress at the edge of the failure plane due to shear layer and confinement effects.

Surcharge stress was envisaged to decrease exponentially from q_o at edge of footing to $0.01q_o$ at end of reinforcement.

Table 4 Testing Programme

Series	Type	Number of layers of reinforcement	Reinforcement type	Thickness of granular bed	Direction of prestress	Magnitude of prestress
A	Weak soil 1 (moist soil)	–	–	–	–	–
	Unreinforced GB on weak soil 1	–	–	B and 2B	–	–
B	Reinforced GB on weak soil 1	1 and 2	Geogrid and geotextile	B and 2B	–	–
C	Prestressed RGB on weak soil 1	1 and 2	Geogrid and geotextile	B and 2B	Uniaxial and biaxial	1, 2 and 3 %
D	Weak soil 2 (Submerged soil)	–	–	–	–	–
	Unreinforced GB on weak soil 2	–	–	B and 2B	–	–
E	Reinforced GB on weak soil 2	1 and 2	Geogrid and geotextile	B and 2B	–	–
F	Prestressed RGB on weak soil 2	1 and 2	Geogrid and geotextile	B and 2B	Uniaxial and biaxial	1, 2 and 3 %

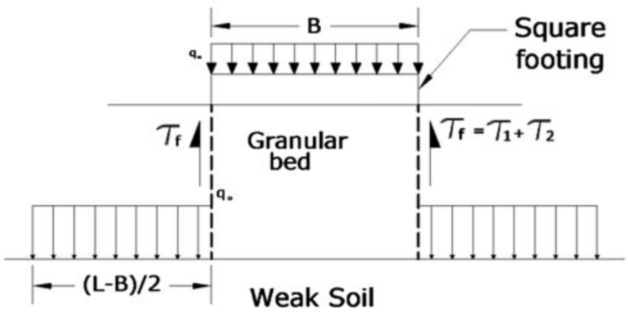


Fig. 9 Surcharge effect for PRGB proposed in the present study

In the present study, in case of PRGB, the additional surcharge stress is envisaged to be uniform over the reinforcement in the direction of prestressing (Fig. 9). This is also justified from the measured (more or less uniform) settlements. In case of Uniaxial prestressing, the surcharge stress is considered to decrease exponentially in the cross direction. Average surcharge stress is considered around the square footing and accordingly ΔBCR_{SE} is estimated.

$$\Delta q_{SE} = q_{avg} \times N_q \tag{15}$$

$$\Delta BCR_{SE} = \Delta q_{SE}/Q \tag{16}$$

Finite Element Analysis

In the present study, loading tests on Reinforced Granular beds are also simulated numerically using the program PLAXIS (version 8) which is a finite element software package. Due to symmetry of the soil-footing-reinforcement system, an axisymmetric model is used to carry out the finite element analysis. The settlement of the rigid footing is simulated using non zero prescribed displacements.

The soil is modeled using 15-noded triangular elements. The reinforcement is modeled using the 5-noded tension element. The prestress is applied as a horizontal tensile load to the reinforcement (Fig. 10). Medium mesh size is adopted in all the simulations. To simulate exactly the testing procedure in the laboratory, staged construction procedure is adopted in the calculation phase. In the first stage, weak soil up to its top level is simulated. In the second stage, sand up to the bottom level of reinforcement is simulated. In the third stage the reinforcement with prestress is simulated and in the fourth stage sand above the reinforcement is simulated. In the final stage the footing with prescribed displacement is simulated. Such a staged construction procedure is necessary because the reinforcement should be prestressed before filling soil above it, otherwise the friction between soil and reinforcement will prevent the elongation of reinforcement due to prestressing. The deformed shape of soil is shown in Fig. 11.

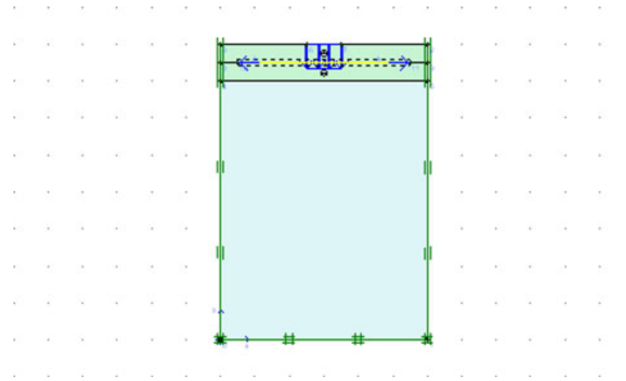


Fig. 10 Geometric Model of reinforced granular beds

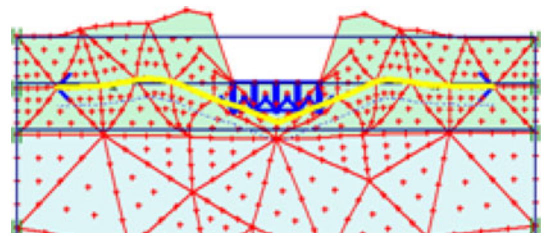


Fig. 11 Deformed shape after loading

Results and Discussions

Analytical Modeling

The results obtained from the 'improvised model' of Shivashankar et al. [7], proposed in this paper, are presented in Figs. 12, 13, 14, 15, 16 and 17. The BCRs obtained experimentally and those predicted by the model for single layer geogrid reinforcement, double layer geogrid reinforcement and single layer geotextile reinforcement are shown graphically in Figs. 12, 13 and 14 respectively. Comparisons between the bearing capacity ratios predicted by the analytical model and by FE analysis for single layer geogrid reinforcement, double layer geogrid reinforcement and single layer geotextile reinforcement are shown graphically in Figs. 15, 16 and 17 respectively. It is observed, from these figures, that the model predicts the BCR with fairly good accuracy.

It is observed that the values of BCR are higher for submerged soil (weak soil 2) compared to that of moist soil (weak soil 1) i.e. improvement is more for weak soil 2. This is because in the case of submerged soils overlain by granular bed, the pore water in weak soil escapes into the granular above it, and shear strength of the weak soil is increased somewhat and overall load carrying capacity of the layered soils increases. Therefore, prediction of BCR by the analytical model is better for moist soil than for submerged soil. The results therefore imply that punching

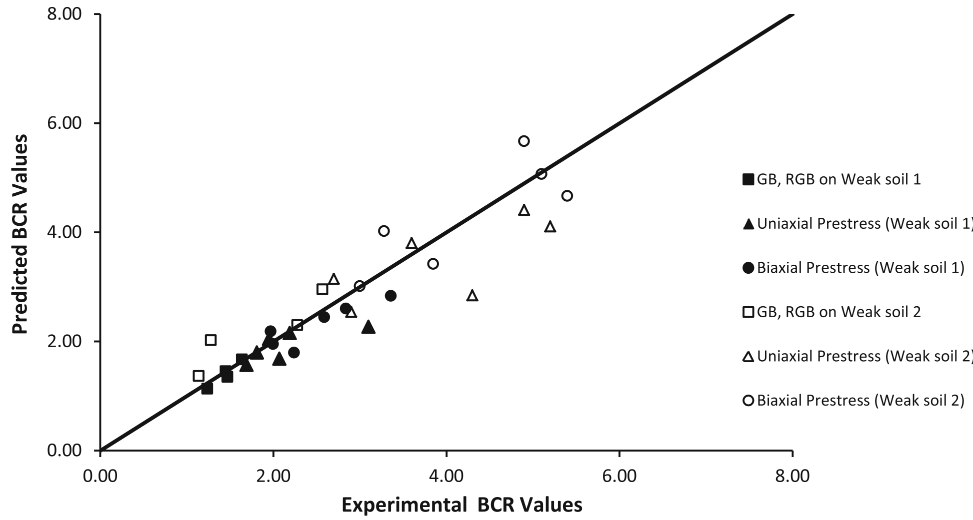


Fig. 12 Comparison between observed and predicted values of bearing capacity ratios (BCRs*)* for GB, RGB and PRGB with single layer geogrid reinforcement overlying (moist) weak soil 1and (submerged) weak soil 2

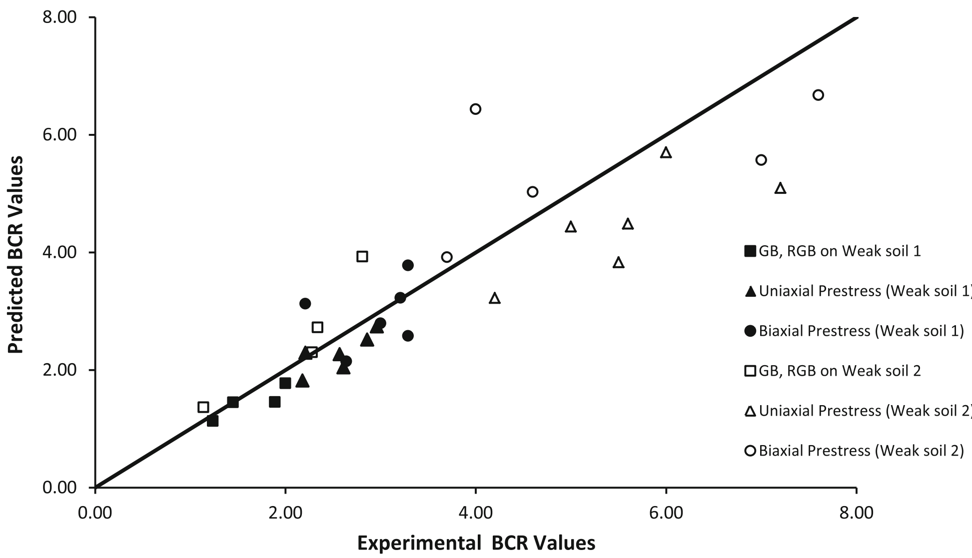


Fig. 13 Comparison between observed and predicted values of bearing capacity ratios (BCRs) for GB, RGB and PRGB with double layer geogrid reinforcement overlying (moist) weak soil 1and (submerged) weak soil 2

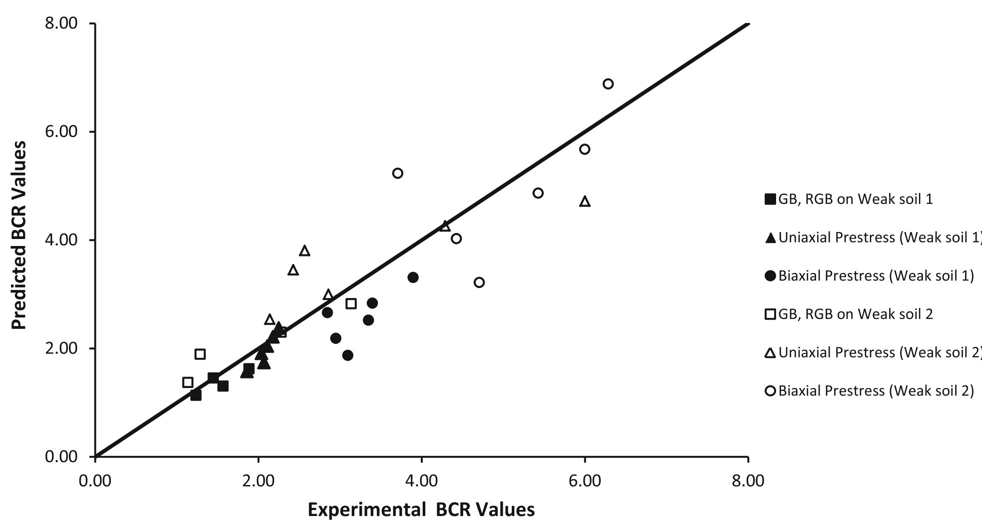


Fig. 14 Comparison between observed and predicted values of bearing capacity ratios (BCRs) for GB, RGB and PRGB with single layer geotextile reinforcement overlying (moist) weak soil 1 and (submerged) weak soil 2

Fig. 15 Comparison between predicted values of bearing capacity ratios using analytical model and FE analysis for GB, RGB and PRGB with single layer geogrid reinforcement overlying (moist) weak soil 1 and (submerged) weak soil 2

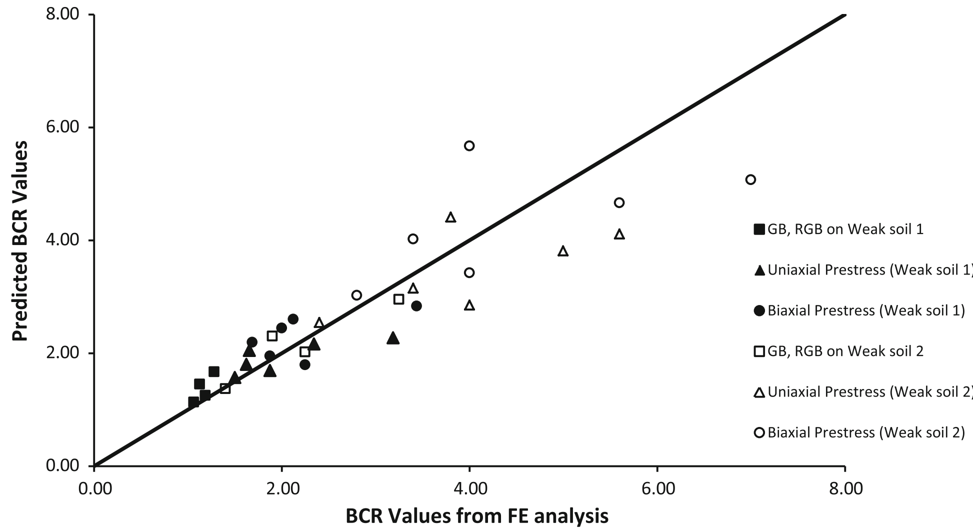


Fig. 16 Comparison between predicted values of bearing capacity ratios using analytical model and FE analysis for GB, RGB and PRGB with double layer geogrid reinforcement overlying (moist) weak soil 1 and (submerged) weak soil 2

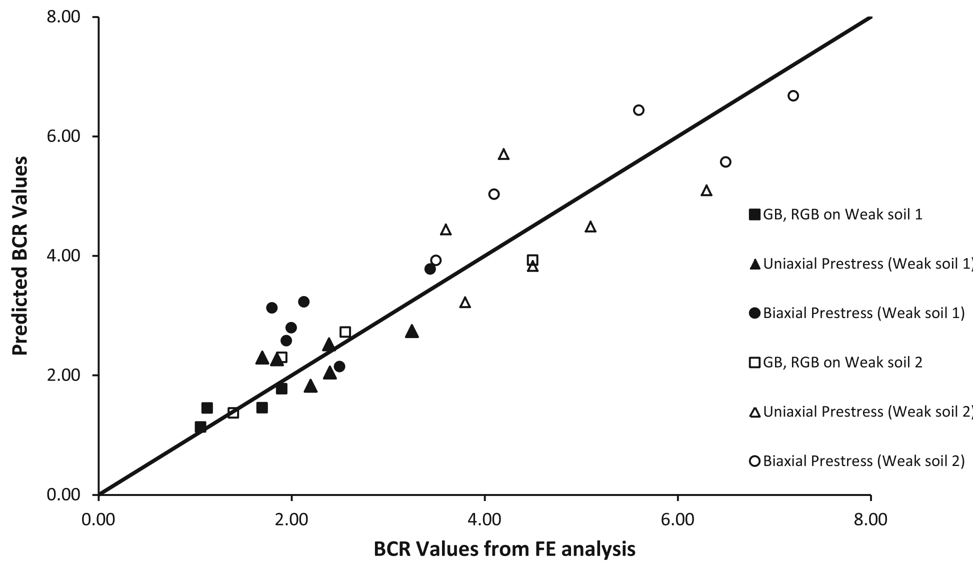
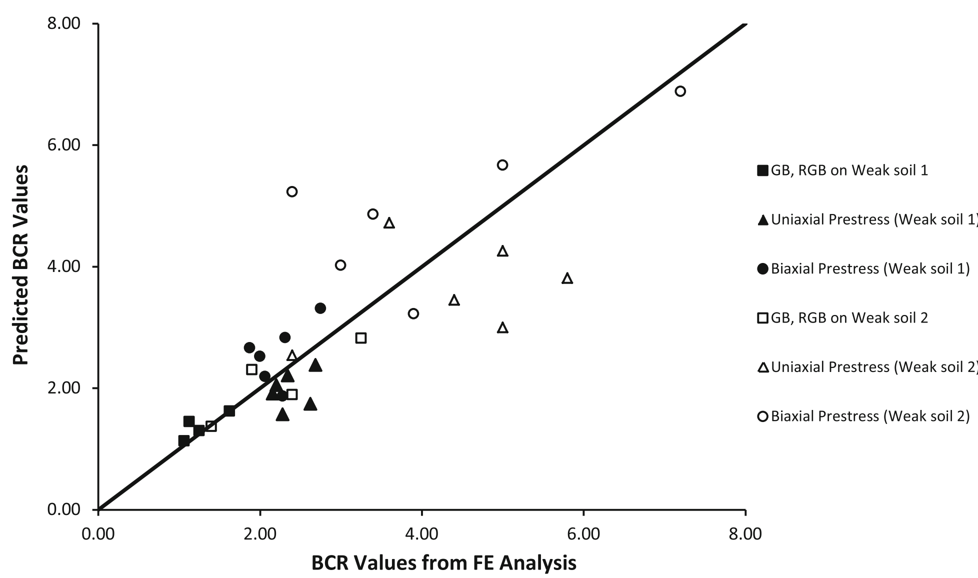


Fig. 17 Comparison between predicted values of bearing capacity ratios using analytical model and FE analysis for GB, RGB and PRGB with single layer geotextile reinforcement overlying (moist) weak soil 1 and (submerged) weak soil 2



shear failure mechanism is the predominant failure mechanism in case of moist weak soil (weak soil 1).

Finite Element Analysis

Vertical stress versus normalized settlement curves from experimental studies and finite element analyses for PRGB with single layer geogrid reinforcement overlying (moist) weak soil 1 are presented in Figs. 18, 19, 20 and 21. It can be seen from these figures that there is a reasonably good agreement between experimental and finite element analysis results.

From Fig. 18 which represents the variation of bearing pressure with footing settlement of uniaxially prestressed

Fig. 18 Stress versus normalized settlement curves for granular bed of thickness B with uniaxial prestressing overlying (moist) weak soil 1

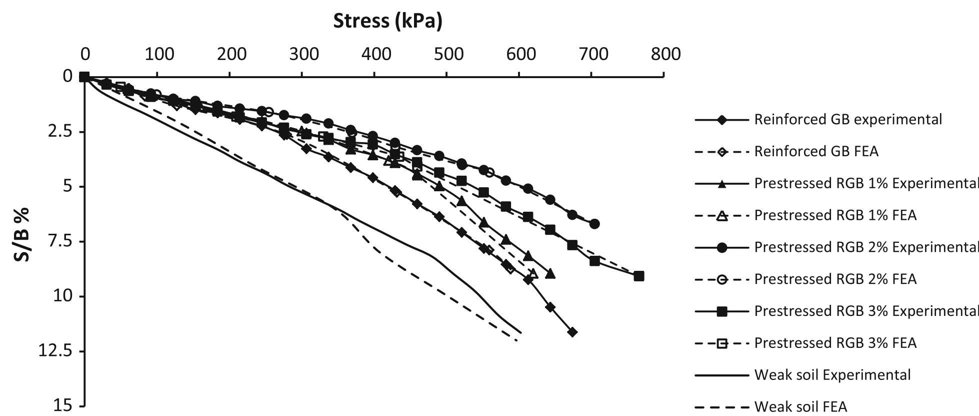


Fig. 19 Stress versus normalized settlement curves for granular bed of thickness B with biaxial prestressing overlying (moist) weak soil 1

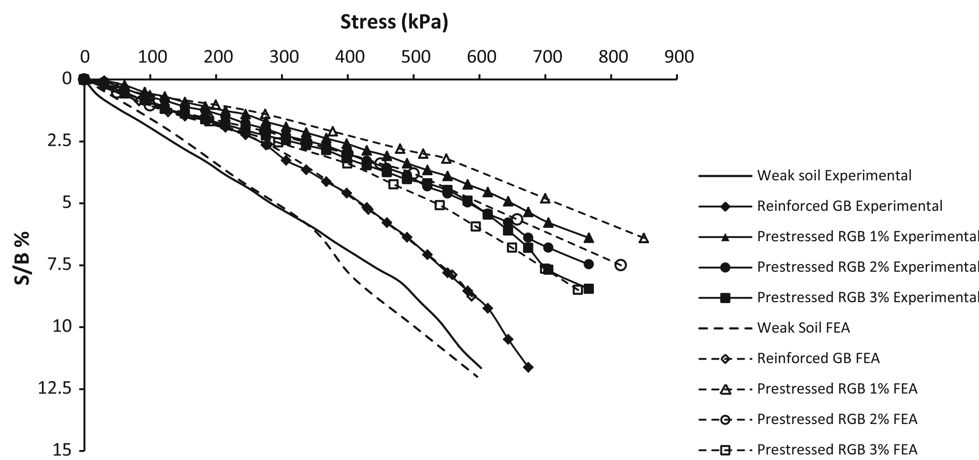
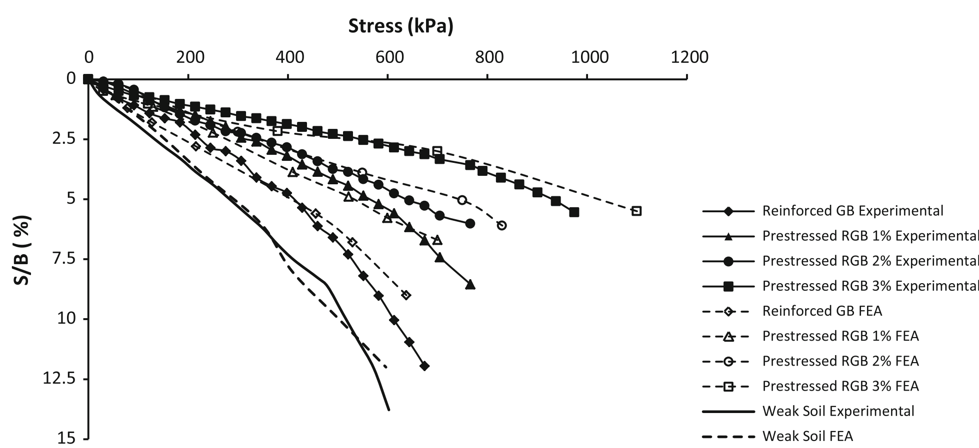


Fig. 20 Stress versus normalized settlement curves for granular bed of thickness 2B with uniaxial prestressing overlying (moist) weak soil 1



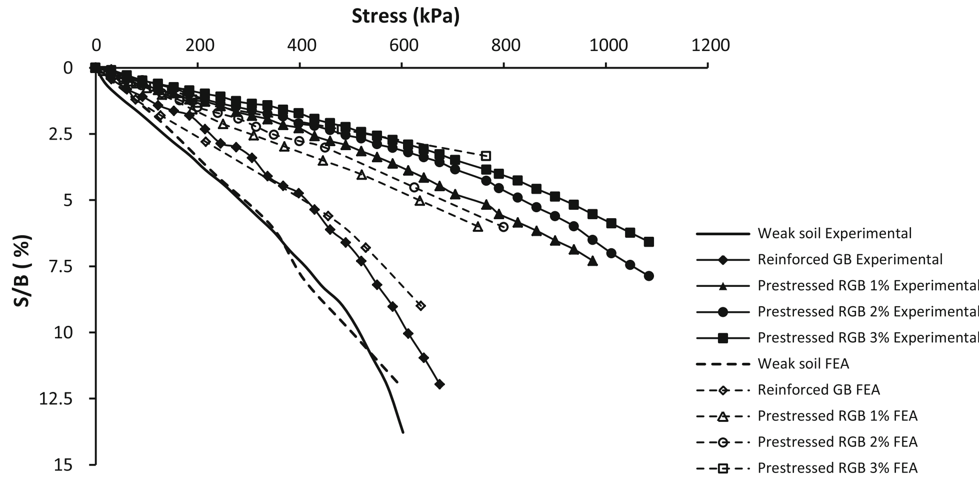


Fig. 21 Stress versus normalized settlement curves for granular bed of thickness 2B with biaxial prestressing overlying (moist) weak soil 1

granular bed of thickness B overlying (moist) weak soil 1, it can be seen that maximum improvement is observed when the magnitude of prestress was equal to 2 % of the tensile strength of reinforcement. Further addition of prestress is not beneficial. However for a granular bed of thickness B with biaxial prestressing overlying (moist) weak soil 1, it is observed that the maximum improvement in settlement behaviour occurred when the magnitude of prestress was equal to 1 % of the tensile strength of reinforcement. Further increase in prestress is not beneficial (Fig. 19).

The results obtained from a granular bed of thickness 2B with uniaxial prestressing overlying (moist) weak soil 1 is shown in Fig. 20. It is observed that the maximum improvement is when the magnitude of prestress is equal to 3 % of the tensile strength of reinforcement. The results obtained from a granular bed of thickness 2B with biaxial prestressing overlying (moist) weak soil 1 indicates that maximum improvement is got also when the magnitude of prestress is equal to 3 % of the tensile strength of reinforcement (Fig. 21).

Bearing Capacity Ratio

The ratio of bearing capacity of improved soil to that of original soil is termed as BCR. The BCR values at 5 mm settlement (S/B = 5 %) are determined for various cases from the stress versus normalized settlement curves. The variations of BCR with the magnitude of prestress for various cases are plotted in Figs. 22 and 25.

The experimental results obtained from granular beds of thickness B and 2B with single and double layer geogrid reinforcement overlying (moist) weak soil 1 are presented in Fig. 22. It is observed that for granular beds of thickness B with uniaxially prestressed single layer reinforcement, the BCR increases till the prestress is increased to 2 %. A further increase in prestress reduces the BCR. In case of biaxial prestressing, 1 % prestress is giving maximum BCR. At 2 and 3 % prestress, the BCR values attained due to uniaxial prestressing and biaxial prestressing are nearly equal. In granular beds of thickness 2B, the BCR increases with prestress in both uniaxial and biaxial prestressing. In general the improvement attained with granular beds of

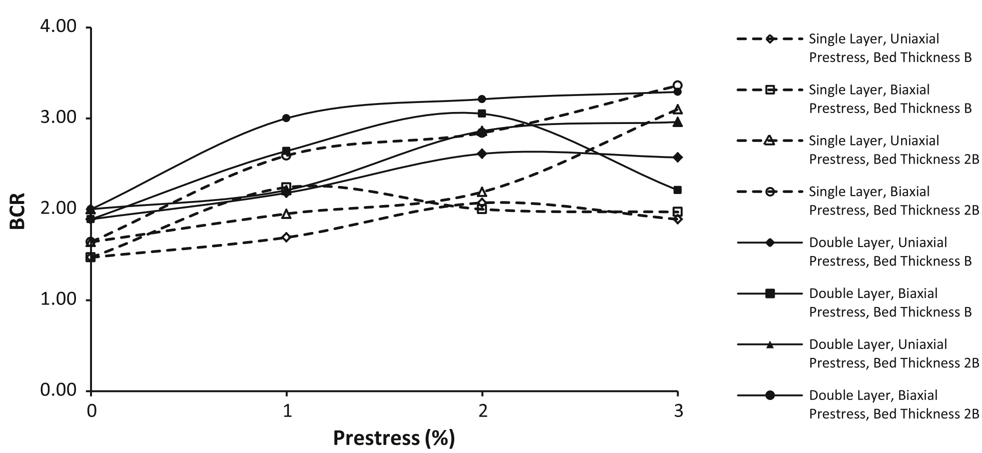


Fig. 22 Variation of BCR with prestress in single and double layer geogrid reinforced granular beds overlying (moist) weak soil 1

thickness 2B is more than that with granular beds of thickness B. However at 1 % prestress, the BCR observed in granular bed of thickness B with biaxially prestressed single layer reinforcement is more than that of granular bed of thickness 2B with uniaxially prestressed single layer reinforcement. In granular beds of thickness B with two layers of reinforcement, the maximum BCR is attained at a prestress of 2 % for both uniaxial and biaxial prestressing. But for granular beds of thickness 2B, maximum BCR is attained at a prestress of 3 % and the improvement attained when the prestress is increased from 2 to 3 % is very less.

In general double layer reinforcement gave more improvement than single layer reinforcement and biaxial prestressing gave more improvement than uniaxial prestressing. It is observed that at 1 and 2 % prestress, granular bed of thickness B with biaxially prestressed double layer reinforcement gives more improvement than granular bed of thickness 2B with uniaxially prestressed double layer reinforcement. It is also observed that granular bed of thickness 2B with biaxially prestressed single layer reinforcement gives more improvement than granular bed of thickness 2B with uniaxially prestressed double layer reinforcement.

Figure 23 presents the experimental results obtained from geogrid reinforced granular beds overlying (submerged) weak soil 2. In general the BCR values attained in (submerged) weak soil 2 due to prestressing the reinforcement is much higher than that attained in (moist) weak soil 1. BCR is the ratio of the load carried by improved ground (with granular bed/Reinforced granular bed/prestressed reinforced granular bed) to the load carried by the unimproved ground (with only the weak soil). Weak soil 2 being submerged soil is weaker soil than weak soil 1. Improvement with weak soil 2 is more than with weak soil 1 which is reflected in higher BCR values.

In granular beds of thickness B with both single and double layer reinforcement, the improvement attained due to uniaxial prestressing is more than that with biaxial prestressing. This is contrary to that of (moist) weak soil 1 where biaxial prestressing gave more BCR than uniaxial prestressing. In granular beds of thickness 2B, for (submerged) weak soil 2, biaxial prestressing gives more BCR than uniaxial prestressing, which is similar to (moist) weak soil 1. It is observed that for granular beds of thickness 2B the BCR is reaching a peak value between a prestress of 1 and 2 % and then reducing. This is contrary to that of (moist) weak soil 1 where BCR increased with prestress.

Comparison between the BCRs obtained for single layer geogrid and geotextile reinforcement overlying (moist) weak soil 1 is shown in Fig. 24 and that for overlying (submerged) weak soil 2 is shown in Fig. 25. In moist soil, the bearing capacity ratios obtained with geotextile reinforcement are slightly higher than that with geogrid reinforcement for almost all the cases. In submerged soil, geogrid gives better BCR during uniaxial prestressing whereas during biaxial prestressing, geotextile is giving a higher value of BCR.

From experimental studies as well as finite element analyses, it is observed that after a certain percentage of prestress, the BCR decreases with the increase in prestress. The improvement in bearing capacity depends upon the stress at the interface between reinforcement and granular soil. The tensile stress gets mobilized in the reinforcement due to the applied prestress and due to the friction developed between the reinforcement and surrounding granular soil. Results of finite element analysis indicated that in most of the cases, as the prestress increases, the normal stress at the interface between reinforcement and granular soil decreases. Initially as the prestress is applied, the BCR increases due to an increase in the tensile stress in reinforcement and due to an increase in the interface stress. But as the applied prestress is further increased, the stress transfer between reinforcement and surrounding granular soil reduces resulting in a reduction of BCR.

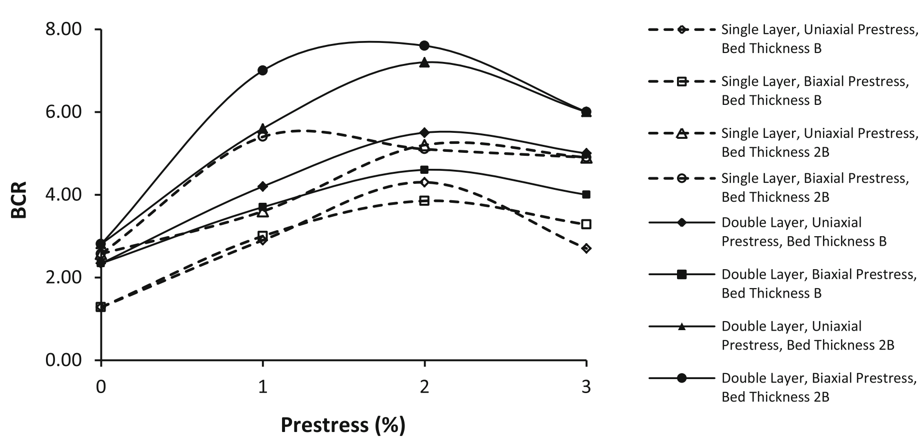


Fig. 23 Variation of BCR with prestress in single and double layer geogrid reinforced granular beds overlying (submerged) weak soil 2

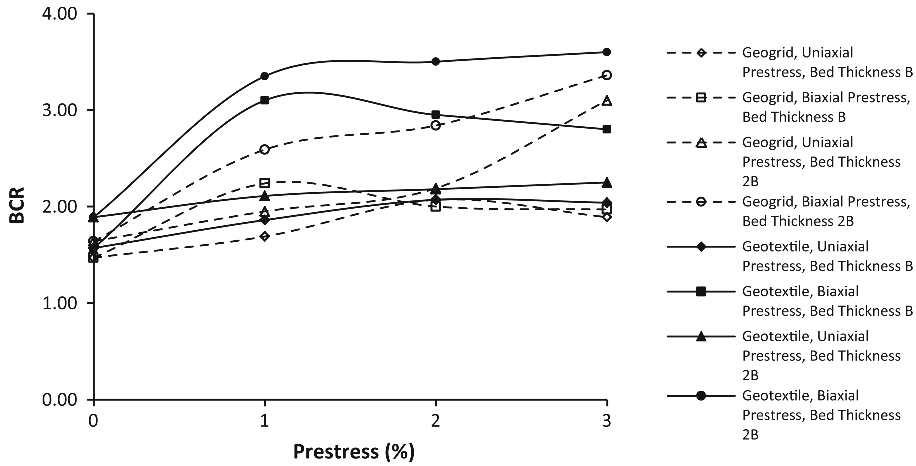


Fig. 24 Comparison between observed bearing capacity ratios for GB, RGB and PRGB with single layer geogrid and geotextile reinforcement overlying (moist) weak soil 1

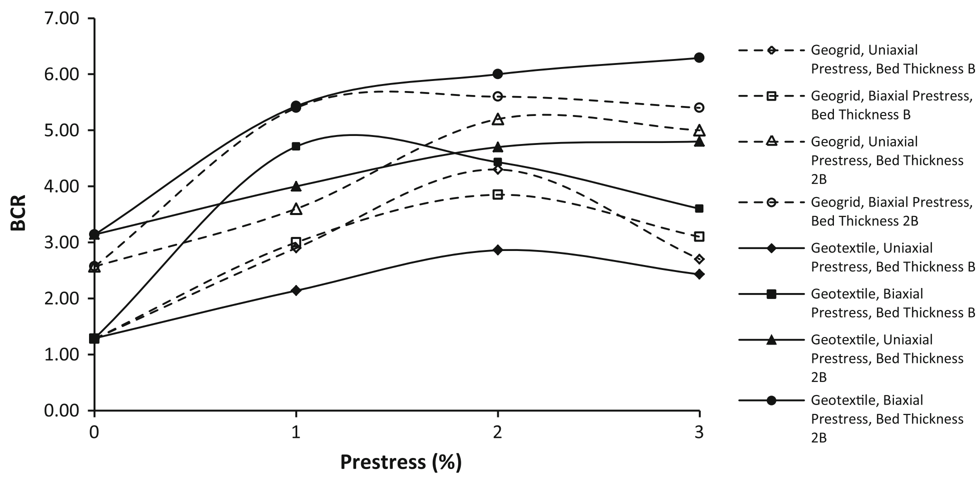


Fig. 25 Comparison between observed bearing capacity ratios for GB, RGB and PRGB with single layer geogrid and geotextile reinforcement overlying (submerged) weak soil 2

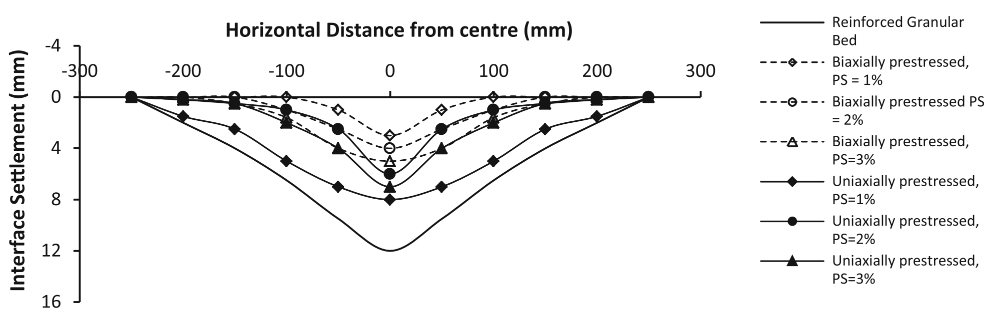


Fig. 26 Distribution of settlement at the interface between RGB and (moist) weak soil 1 when thickness of granular bed is B

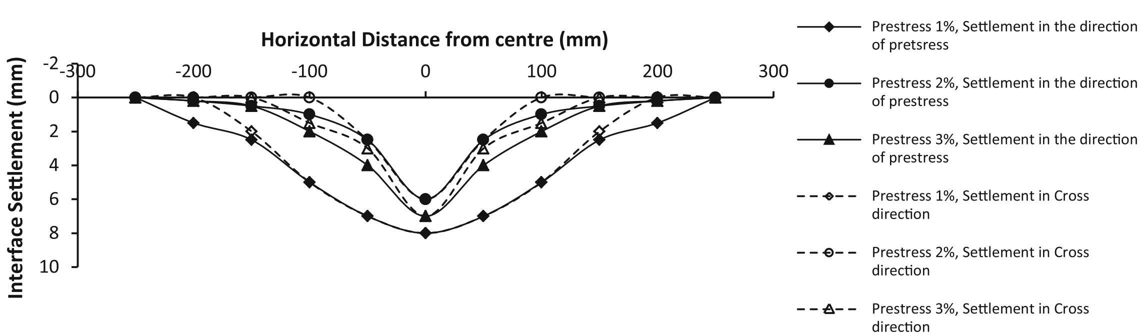


Fig. 27 Distribution of settlement at the interface between RGB and (moist) weak soil 1, in the direction of prestress and in its perpendicular direction, for granular bed of thickness B, when prestress is Uniaxial

Settlement Measurement at the Interface After Test in Case of (Moist) Weak Soil 1

The distribution of settlement at the interface between sand and (moist) weak soil 1, in a granular bed of thickness B, with single layer geogrid reinforcement, subjected to uniaxial and biaxial prestressing is given in Fig. 26. In general the settlement of the underlying weak soil is greatly reduced due to prestressing of reinforcement.

The interface settlement along the direction of prestress and along its perpendicular direction, during uniaxial prestressing of a granular bed of thickness B, with single layer reinforcement, is presented in Fig. 27. It is observed that along the direction of prestress, the interface settlement, is distributed on a wider area than in the cross direction.

Conclusions

Based on the results obtained from experimental work, analytical modeling and finite element analyses, the following conclusions can be made on the behaviour of prestressed reinforced granular beds overlying weak soils.

1. The addition of prestress to geosynthetic reinforcement significantly improves the bearing capacity and settlement behaviour of the soil. Prestressing the geosynthetic reinforcement results in increased load bearing capacity of soil without the occurrence of large settlements, as compared to geosynthetics without any prestress.
2. The improvement in bearing capacity depends upon the thickness of granular bed, magnitude of prestress, direction of prestress, number of layers of reinforcement and type of reinforcement. The improvement in bearing capacity increases with the thickness of granular bed.
3. The results obtained from finite element analyses are in reasonably good agreement with the experimental results.
4. The proposed analytical model predicts the bearing capacity ratios for granular beds overlying weak soil with reasonably good accuracy.
5. Prediction is better for moist soil than for submerged soil, which implies that the punching shear failure mechanism is predominant failure mechanism in case of moist soil.

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