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Bearing Capacity and Settlement Characteristics of Sand Subgrades with Vertical Reinforcement Supporting a Square Footing

J. N. Jha[1] · S. K. Shukla[2]

Abstract The restraining effects in the sand subgrade under the footing can be provided by inserting the vertical reinforcing bars beyond the footing base without disturbing the subgrade below the footing. This technique of soil reinforcement can be quite beneficial for the footings where improvement is necessary. This paper presents the results of model square footing tests with vertical reinforcements inserted into the sand subgrade beyond the footing base only. The variables selected for the investigation were spacing of vertical reinforcement, extent of reinforcement from the edge of footing, length of reinforcement, diameter of reinforcement, width of footing and surface characteristics of the reinforcement. The values of theoretical bearing capacity of reinforced sand subgrade were determined using the method of slices and compared with experimental results. It was found that results were comparable particularly when the length of reinforcement and extent of reinforcement were two times the footing width.

Keywords Bearing capacity ratio · Sand subgrade · Settlement ratio · Vertical reinforcement

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Introduction

The increase in load-bearing capacity of foundation soils with horizontal reinforcement layers has been reported by many investigators [1]. The excavation at foundation site up to the desired depth, followed by compaction of foundation soil, becomes essential after placement of horizontal reinforcement layers. Hence the technique of ground improvement with horizontal reinforcement layer is restricted to newly constructed foundation only. Some preliminary studies using the vertical form of reinforcements have been reported in the literature [2–6]. In most of these studies, the strip footings were used, despite the fact that rectangular and square footings are far more common in practice. In some studies the vertical reinforcements were provided throughout the soil subgrade even directly below the footing base [2, 5]. The arrangement of vertical reinforcement below the footing cannot be treated as soil reinforcement technique in its true sense because buckling of reinforcement below the footing becomes the controlling factor for bearing capacity improvement [7]. Moreover this type of soil reinforcement arrangement below the footing cannot be used for the existing footing.

The restraining effects in the subgrade under the footing can be provided by placing the vertical reinforcing bars beyond the footing base quite easily without disturbing the subgrade just below the footing base. The placement of vertical reinforcement in the subgrade around the footing or beyond the footing base, if found effective, may prove to be quite beneficial for existing footings where improvement is necessary due to the apprehension of danger to the footing. Keeping this in view, the present study has been taken up where the model footing tests were conducted using square footings and the vertical reinforcements were installed beyond the footing without disturbing the

subgrade directly below the footing base. The variables selected for the investigation were spacing of vertical reinforcement, extent of reinforcement from the edge of footing, length of reinforcement, diameter of reinforcement, width of footing and surface characteristics of the reinforcement. A working formula for the factor of safety of slip surfaces based on the method of slices [8] was used and extended in the present investigation to calculate the bearing capacity of reinforced sand subgrade. The theoretical values of bearing capacity thus calculated were also compared with experimental bearing capacity values.

Experimental Investigation

Materials

Sand

The medium to coarse sand with rounded to sub-rounded particles having a specific gravity 2.67 were used in the study. The maximum and minimum unit weights of the sand were 19.87 and 16.26 kN/m³ and the corresponding values of the minimum and maximum void ratios were 0.344 and 0.641, respectively. The effective grain size (D_{10}), uniformity coefficient (C_u), and coefficient of curvature (C_c) of the sand were 0.16 mm, 3.875 and 0.735, respectively. In the tests, the relative density and the unit weight of soil were 56 % and 18.08 kN/m³, respectively. A series of direct shear tests were performed at the same relative density, and the angle of internal friction were found to be approximately 34°. All the laboratory tests were conducted in accordance with relevant Indian standards.

Model Reinforcement

Steel pipes of 50–200 mm diameter are used as micropiles in many cases. A 10 storey building, originally in a precarious condition due to differential settlement was restored to safe condition using micropiles of galvanized steel pipes having diameter and length as 100 mm and 10 m respectively [9]. Therefore in any laboratory model tests, the size of the reinforcements needs to be compatible with the footing size. In the laboratory environment, it is difficult to obtain a vertical reinforcement whose size does not violate the similitude requirement and has the minimum influence on experimental results both qualitatively and quantitatively. However, in the present investigation the criterion for selecting the diameter of vertical reinforcement was that it should not bend while installing in the subgrade. Therefore the minimum diameter of round galvanized mild steel wire was selected after several trials

which could be installed straight within the sand subgrade and was easily available. Literature also reports the use of similar type model reinforcements [10, 11]. The selected average diameters of thin and thick galvanized mild steel wires were 1.7 and 2.51 mm, respectively. The average tensile strength and modulus of elasticity for the two wires were 1.15×10^3 and 2.25×10^9 kPa respectively. Maximum length of reinforcement used in the investigation was kept as 150 mm since the insertion of vertical reinforcement became difficult once the length exceeded 150 mm. The wires of required length were cut and made perfectly straight before pushing them into the sand subgrade. The surface characteristics of reinforcing material were changed by sand coating on plain reinforcements using an adhesive commonly known as araldite.

Footing

The rigid foundation was modelled by a square footing made of good-quality well-seasoned Sal wood. The square model footing of size 100 mm × 100 mm, 75 mm × 75 mm and 50 mm × 50 mm were used for conducting the model tests. The thickness of model footing was kept as 40 mm. The base of the model footing was made rough by cementing a thin layer of sand to it with epoxy glue.

Experimental Procedure

Model footing tests were performed in two different type of test tanks: one a well stiffened square wooden box (1000 mm × 1000 mm × 1000 mm—Type A) and other a rectangular box (955 mm × 480 mm × 100 mm—Type B) with 8 mm thick acrylic glass sheet as frontage. The rectangular test tank was used to observe failure pattern. The sides of the test tanks were braced with stiffeners to avoid lateral yielding during soil placement and loading of the model foundation. Rainfall technique was used to prepare the test beds and the constant height of fall was kept as 300 mm from the top surface. The height of fall was finalized by making several trials in the test tanks prior to the model tests to achieve the desired unit weight. The sand passing through 250 micron sieve and retained on 75 micron sieve were used for the preparation of test beds. Different extents of reinforcements to be placed beyond the footing edge were marked on sand surface layer. The galvanised mild steel wire of required length and diameter were pushed vertically directly into the sand bed manually at the required grid spacing through perforated metallic plate. Utmost care was taken while pushing the wires to maintain its verticality. After placing the reinforcements, level of the sand bed was again checked with spirit level to ensure that the top of each wire remain at the same level

before placing the model footing. A square footing of different sizes was selected for the investigation. The footing was placed on leveled surface at a predefined alignment such that the load from the jack and loading frame would be transferred concentrically to the footing. The load was transferred to the footing through a circular ball placed between the footing and proving ring to eliminate any potential moment transfer from the loading fixtures. The load was applied carefully to the footing through manually operated hydraulic jack at constant rate of settlement 1 mm/min until an ultimate state was reached. The ultimate state was defined as that state at which settlement continued to increase without any further increase in load. The settlement of the footing was recorded by the dial gauges fixed with adapter and resting on two extension plates attached on either side of the footing. The settlements reported are the average of the two dial gauge readings. All the variables selected for study during the investigation were expressed in terms of footing width (B) and are given in Table 1. Reinforcement pattern used in the investigation is shown in Fig. 1, whereas the loading arrangement of the tests in square test tank is shown in Fig. 2.

The boundary effects during test in square test tank (Type-A) were avoided by keeping the ratio of the size of the test tank to the size of square footing and the ratio of the depth of sand bed to the size of the footing as 10 or more [12]. Selection of square footing also minimized the dimensional effects during the investigation [13]. Since the effect of depth of embedment was found negligible on the response of footing cell system for circular footing in a confined granular soil [14], no attempt was made in the present investigation to study the effect of depth of embedment on the behavior of footing soil system.

The accuracy and consistency of placement unit weight during the raining process in all the tests were checked by placing small cans of known volume at three different locations in the box. The global unit weight of the sand was calculated by weighing the total sand used for preparing the test bed in the box. The following two conditions were applied for accepted soil beds: the difference in unit weight at three measured locations was less than 1 %, and the difference between the global unit weight of the soil bed and the average unit weight of the three measured values was less than 1.5 %. Repeatability and the consistency of the test data were verified by repeating the tests.

Results and Discussion

The primary objective of this experimental investigation is to evaluate the efficiency of vertical reinforcing element when inserted beyond the footing in improving the load

Table 1 Variables of Study

Type of test → Variables	Type-A		Type-B	
Type of test tank	Square		Rectangular	
Size of test tank (mm)	1000 × 1000 × 1000		955 × 480 × 100	
Type of footing	Square		Square	
Size of footing (mm)	100 × 100	75 × 75	100 × 100	50 × 50
Extent of reinforcement (R)	0.25B, 0.5B, 0.75B, 1.0B, 1.5B & 2.0B	0.25B, 0.5B, 0.75B, 1.0B, 1.5B & 2.0B	0.25B, 0.5B, 1.0B, 2.0B, 3.0B	0.25B, 0.50B, 1.0B, 2.0B
Spacing of reinforcement (S)	0.06B, 0.1B, 0.13B, 0.17B	0.09B, 0.13B, 0.18B, 0.22B	0.1B, 0.13B, 0.17B, 0.26B	0.13B, 0.20B, 0.27B, 0.34B
Length of reinforcement (L)	B, 1.5B	B, 1.5B & 2B	B, 1.5B	B, 1.5B, 2B,3B
Diameter of reinforcement (d)	0.017B, 0.025B	0.022B, 0.033B	0.017B, 0.025B	0.034B, 0.050B
Diameter of reinforcement after sand coating ($d*$)	—	—	0.032B, 0.039B	—

Fig. 1 Reinforcement arrangement
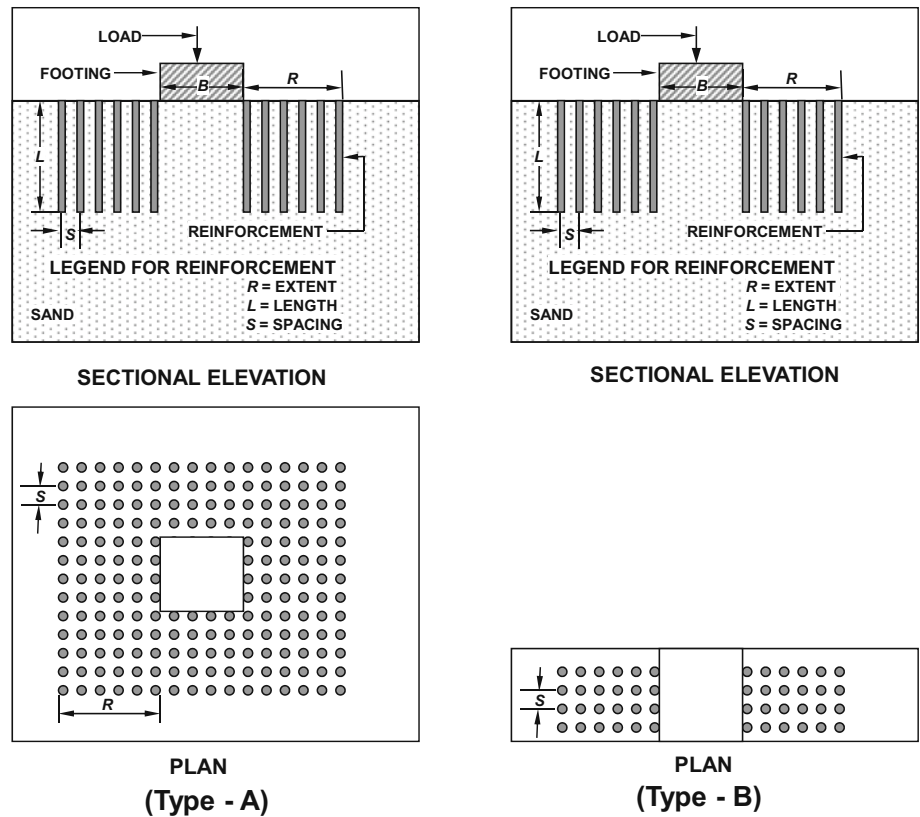
Fig. 2 Loading arrangements for load tests in a square test tank

carrying capacity of the soil. It is always convenient to present the test results of the reinforced system with respect to the corresponding results derived for the footing on an unreinforced soil. For this purpose, the pressure-settlement curves for different cases (Table 1) were obtained by conducting load tests with a square model footing placed on the top of reinforced and unreinforced soil, respectively. Since no definite failure point was observed in the pressure–settlement curve, the ultimate bearing capacity was determined by the double tangent method [13]. The bearing

capacity improvement of soil due to the placement of vertical reinforcement beyond the footing was represented by a non-dimensional factor, called the bearing capacity ratio (BCR), which is defined as the ratio of footing ultimate pressure (q_R) with reinforcement to the footing ultimate pressure without reinforcement (q_0). Thus

$$BCR = \frac{q_R}{q_0} \tag{1}$$

The footing settlement (w) was expressed by another non-dimensional parameter $\delta = (w/B)$ known as the settlement ratio (SR) where B is the footing width. Thus

$$SR = \delta = \left(\frac{w}{B} \times 100\right)\% \tag{2}$$

This BCR was also plotted against other non-dimensional parameters like extent ratio ($X = R/B$) and spacing ratio (S/B), Where R, S and B are the extent of reinforcement, spacing of reinforcement and width of footing respectively.

Bearing Capacity Behaviour

Figure 3 shows a relationship between pressure (p) and settlement ratio (δ) with or without reinforcement at different spacing ratio (S/B) for a typical Type- A test. The footing width, length of reinforcement, extent of
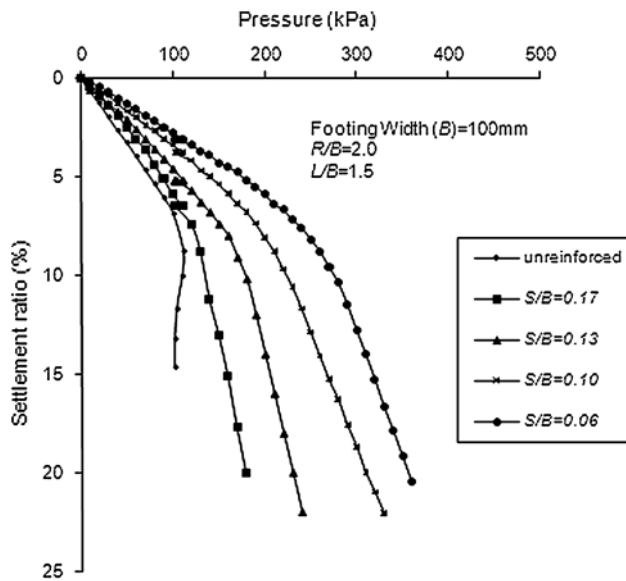
Fig. 3 Variation of pressure with settlement ratio (δ) for different values of spacing ratio (S/B)

Fig. 4 Variation of BCR with settlement ratio for different values of spacing ratio (S/B)

reinforcement and diameter of reinforcement for this typical case were 100 mm, 1.5B, 2B, and 1.7 mm, respectively. It can be seen from the Fig. 3 that the installation of vertical reinforcement appreciably improved the ultimate bearing pressure which increased with the decrease in spacing ratio. Comparing the curves of Fig. 3, it can be observed that the placement of vertical reinforcements beyond the footing up to an extent of 2B (B is footing width) with spacing ratio (S/B) of 0.68 improved the bearing pressure from 110 kPa for unreinforced case to 456 kPa for the reinforced case at 8.6 % settlement ratio (δ). Therefore, it can be concluded that when an excessive settlement is a controlling factor in determining the allowable bearing capacity, placement of vertical reinforcements beyond the footing may significantly reduce the settlement ratio for the same level of bearing load. The inclusion of vertical reinforcements actually resists the lateral displacement of the soil particles underneath the footing thus confining the soil leading to decrease in vertical settlements and improvement in bearing capacity. The results of Fig. 3 were presented again in a non-dimensional form in Fig. 4 showing the variation between BCR and settlement ratio (SR). It can be observed that the introduction of vertical reinforcements made the subgrade more compressible in the initial stages of ultimate bearing pressure. The BCR decreases initially and then attained a minimum value at around 5–7 % of settlement ratio (δ) and after that again increased with an increase in settlement ratio (δ). The BCR finally reached to a constant value at approximately 18–20 % of settlement ratio (δ). Comparing the curves of Fig. 4, It is noticed that the maximum initial decrease in BCR occurred for the case where the spacing of
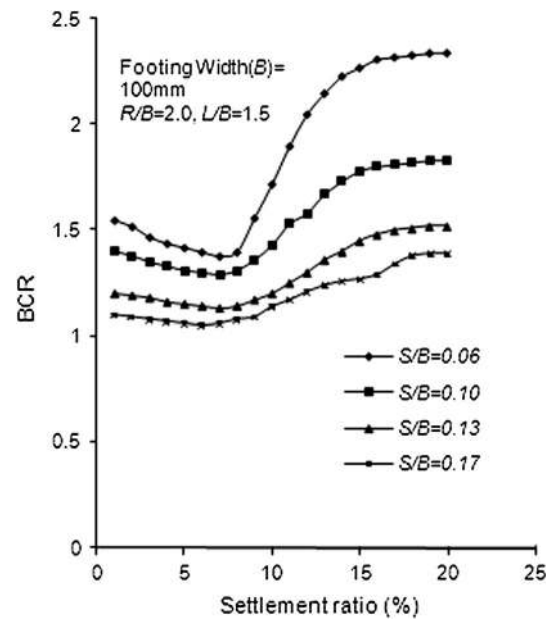
reinforcement ($S = 0.06B$) was minimum. This may be due to the fact that the reinforcements initially were in temporary compression due to the application of vertical confining stress. For any increase in the strength of soil foundation system, the initial compressional prestress must be overcomed and this is only possible through sufficient shear distortion in reinforcements. If this inference is correct, then an increase in stiffness of reinforcement or an increase in number of reinforcements (which also results in stiffening of subgrade) will also result in greater compressional prestress loss, thus decreasing the initial BCR at low settlement ratio. This trend is in fact exhibited by the curves of Fig. 4. Similar trends were observed in other cases as well.

Effect of Spacing of Reinforcing Element (S)

Figure 5 shows a typical plot of a test results (Type-A) between BCR and spacing ratio (S/B) at different extent ratio ($X = R/B$). The footing width (B), length ratio (L/B) of reinforcement and diameter of reinforcement (d) for this typical case were 100 mm, 1.5 and 1.7 mm, respectively. Comparing the curves of Fig. 5 it can be observed that at a given extent ratio ($X = 2.0$), when the spacing ratio of reinforcement (S/B) was increased from 0.06 to 0.17, the values of BCR decreased from 3.98 to 1.68. Similar trends were observed for all other tests considered in the study. Therefore it can be concluded that at a given extent ratio, BCR increases with the decreases in spacing ratio (S/B) and vice versa.
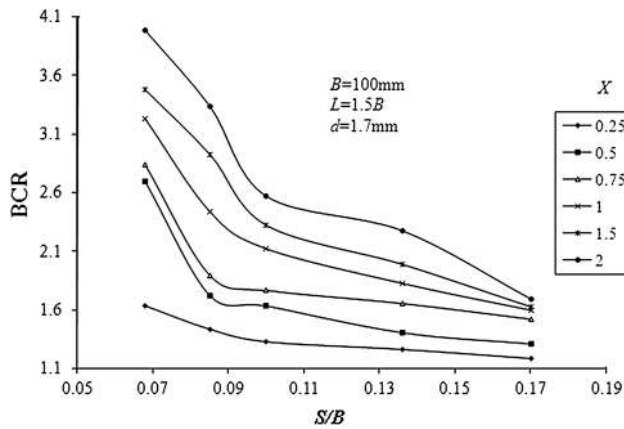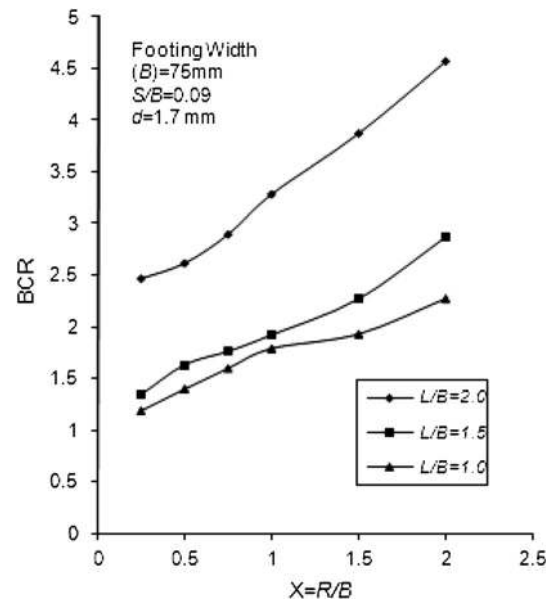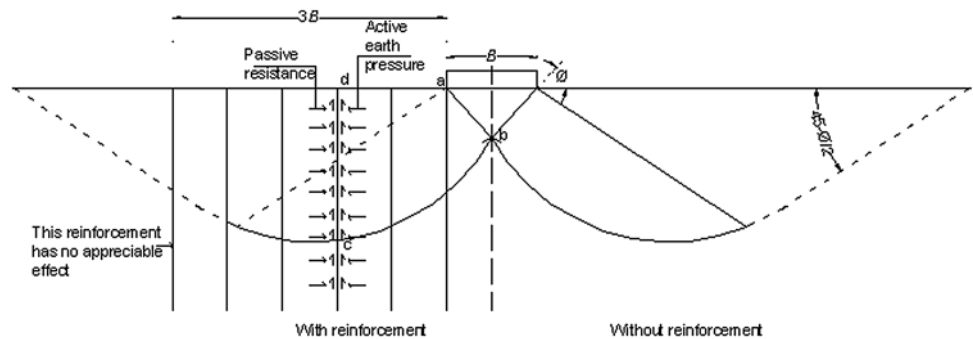
Fig. 5 Variation of BCR with spacing ratio (*S/B*) for different values of extent ratio (*X*)

Fig. 7 Variation of BCR with extent ratio (*X*) for different values of length ratio (*L/B*)

Placement of reinforcements actually interrupted the propagation of failure plane (Fig. 6) resulting in an improvement of bearing capacity of the footing and may be explained as follows. As the spacing between the reinforcing elements increases, the number of reinforcing elements per unit length of footing width decreases thus increases the clear distance between the reinforcement. This allows more soil to migrate between the reinforcing elements and a larger movement of soil occurs under the footing. Consequently, the shear force developed along these element decreases. But when spacing between vertical reinforcing elements decreases, an arching effect of soil is developed between the reinforcing elements leading to a decrease in lateral movement of soil under the footing which results in complete soil confinement and ultimately increasing the bearing capacity.

Effect of Length of Reinforcing Element (*L*)

Figure 7 shows the variation between *BCR* and the extent ratio (*X*) at different length of reinforcement (*L*) for a typical case of Type-A test. The footing width (*B*), spacing ratio (*S/B*) and diameter of reinforcement (*d*) selected for this case were 75 mm, 0.09 and 1.7 mm, respectively. Comparing the curves of the Fig. 7, it can be observed that

BCR improved from 2.86 to 4.56 when length of reinforcement (*L*) increased from *B* to 2*B* at an extent ratio (*X*) of 2. Similar observations were obtained for other tests considered in the study. Thus it is quite clear that at any given value of extent ratio (*X*) of reinforcement as the length of reinforcement increases *BCR* also increases.

Embedded part of reinforcements in the stable underlying soil increases as the length of reinforcing elements increases (Fig. 6), thus offering a greater resistance to the lateral movement of soil under the footing. This improved resistance enhances the confinement of soil which significantly decreases the vertical settlement thus improving the bearing capacity.

Effect of Extent of Reinforcement (*R*) from the Edge of Footing

To know the effect of extent of reinforcement on bearing capacity improvement, *BCR* have been plotted against extent ratio (*X* = *R/B*) at constant length of reinforcement (*L*) for different spacing ratio (*S/B*). Figure 8 shows a

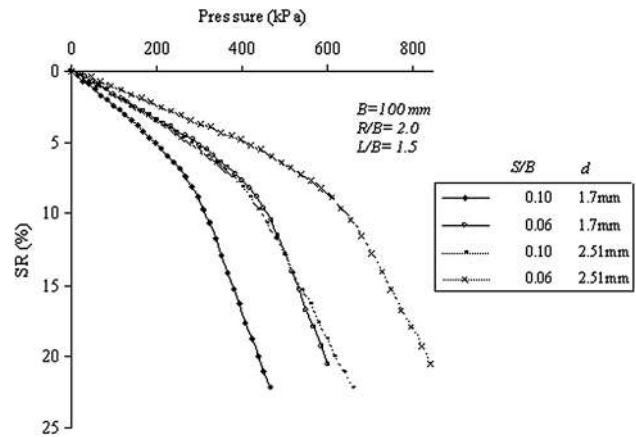**Fig. 6** Intersection of anticipated failure surface with and without reinforcement

Fig. 8 Variation of BCR with extent ratio (X) for different values of spacing ratio (S/B)

Fig. 9 Variation of pressure with SR having different diameters of reinforcement

typical variation of BCR with X at different S/B ratio for a Type-A test. The footing width (B), length ratio (L/B) and diameter of reinforcement for this particular case were 75 mm, 2 and 1.7 mm, respectively. Comparing the curves of Fig. 8, it is quite clear that when extent ratio ($X = R/B$) increased from 0.25 to 2.0, BCR increased from 2.44 to 4.5 at a spacing ratio (S/B) equal to 0.09. Therefore it can be concluded that at a given spacing ratio (S/B) when extent of reinforcement (R) increases, BCR also increases. Similar observations were obtained for all the other tests considered in the study. It appears that increase in lateral extent of reinforcement enhances the resistance against the lateral displacement of soil underneath the footing thus developing an arching effect of soil between the reinforcements. Consequently this arching effect created a complete sand confinement situation and this finally results an increase in the bearing capacity.

Effect of Rigidity of Reinforcing Element (d)

To know the effect of rigidity of reinforcement on bearing capacity improvement, the tests were carried out with reinforcing wire having different diameter of 1.7 and 2.51 mm, respectively. Being the same material, 2.51 mm diameter wire was more rigid than 1.70 mm wire. Figure 9 shows typical test results (Type-A) indicating the variation between pressure (p) and settlement ratio (δ) for different diameters. The footing width (B), the length of reinforcement (L) and extent of reinforcement (R) selected for the above case were 100 mm, 1.5B and 2B, respectively. Two

typical values of spacing ratio (S/B) of reinforcements considered for this particular case were 0.06 and 0.10, respectively. Comparing the curves of the Fig. 9, it can be observed that the ultimate bearing pressure (p) and the corresponding settlement ratio (δ) were 456 kPa and 8.6 %, respectively at $S/B = 0.06$ and $d = 1.7$ mm, which increased to 643 kPa at $\delta = 8.6$ % when the diameter of the reinforcement changed from 1.7 to 2.51 mm. It can also be seen from Fig. 9 that $p-\delta$ curve with spacing ratio (S/B) and diameter (d) equal to 0.06 and 1.7 mm, respectively almost coincided with another $p-\delta$ curve having spacing ratio (S/B) and diameter (d) equal to 0.10 and 2.51 mm, respectively. From the curves of Fig. 9, it is clear that for the same spacing ratio and test conditions, reinforcing wire having thicker diameter gave higher value of BCR. The increase in diameter of reinforcing elements increases the stiffness and resistance to the lateral movement of soil under the footing, thus ultimately improving the bearing capacity. It can be said that for the same range of improvement in bearing capacity, number of reinforcements required per unit footing width will be less for thicker reinforcing element.

Effect of Width of Footing (B)

In order to study the effect of footing width on bearing capacity, Type A tests were conducted on reinforced sand subgrade for three different sizes of footing width ($B = 50$, 75, 100 mm). Figure 10 shows a typical plot between BCR and extent ratio ($X = R/B$) for three different footing width ($B = 50$, 75, 100 mm) at constant length of reinforcement (L), area ratio (A_r/A_s) and diameter of reinforcement (d), respectively. Area ratio (A_r/A_s) is defined as the ratio of area of reinforcement (A_r) to the corresponding area of soil (A_s) in plan (Fig. 1). The area ratio, length and diameter of reinforcement for this particular case considered were 49.084×10^{-3}, 1.5B and 1.7 mm, respectively.
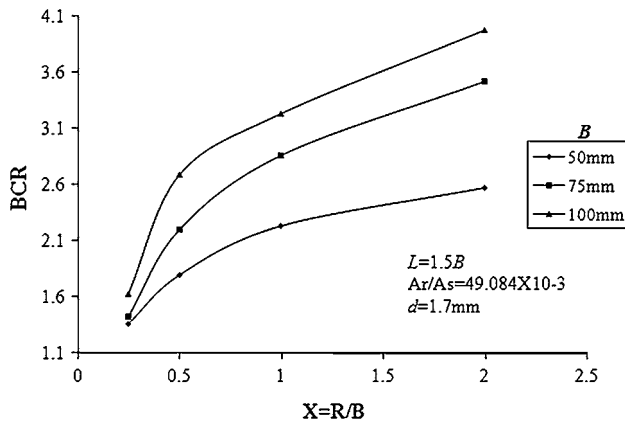
Fig. 10 Variation of BCR with extent ratio (X) for different footing widths (B)

Comparing the curves from the Fig. 10, it can be observed that *BCR* value improved from 2.57 to 3.98 at extent ratio of 2, when footing width increased from 50 to 100 mm. Therefore it is quite clear that *BCR* increases with the increase in the footing width (B) and this trend is unaffected by the value of extent ratio (X). Although results reported earlier in the literature [15, 16] cites similar trend for unreinforced cases, and this trend remained unchanged even when sand subgrades were reinforced with vertical reinforcements beyond the footing base.

## Effect of Surface Characteristics of Reinforcing Element (*d**)

The surfaces of reinforcements having diameters 1.70 and 2.51 mm were made rough by coating a sand layer. After sand coating the diameter of the sand coated wires (d_s) were 3.12 mm (diameter of plain wire $d_{1p} = 1.7$ mm) and

3.96 mm (diameter of plain wire $d_{2p} = 2.51$ mm), respectively. Figure 11 shows the typical variation between *BCR* and extent ratio (X) for different diameters having different surface characteristics. The footing width (B), spacing ratio (S/B) and length of reinforcement (L) for this typical test were 100 mm, 0.13 and 1.5B, respectively which were conducted in a rectangular test tank having same width as equal to the size of the square footing (Type-B). It can be observed from Fig. 11 that sand coated wires representing rough reinforcements were far more effective in improving the *BCR* than the uncoated or plain reinforcements. For example BCR is equal to 1.99, when extent ratio and diameter of plain reinforcement are 2.0 and 1.7 mm, respectively. But the replacement of plain reinforcement ($d_{1p} = 1.7$ mm) with rough or sand coated reinforcement ($d_{1s} = 3.12$ mm) in the above case enhanced the *BCR* from 1.99 to 3.40. The sand coating increased the diameter, roughness and stiffness of reinforcing elements that ensured greater mobilization of friction between soils and reinforcements which ultimately improved the bearing capacity.

Comparison of Experimental Results with Theoretical Values

Soil-reinforcement interaction responsible for improvement in load bearing capacity of subgrade can be explained through failure mechanism. In the present investigation since the boundary effects for square test tanks (Type A) tests are negligible [12], hence failure surface for these cases cannot be drawn. Therefore to draw failure surface, the model tests with square footing were conducted in a rectangular test tank of size 955 mm × 480 mm × 100 mm box with 8 mm thick acrylic glass sheet as frontage. Special care was taken

Fig. 11 Variation of BCR with extent ratio (X) having different surface characteristics
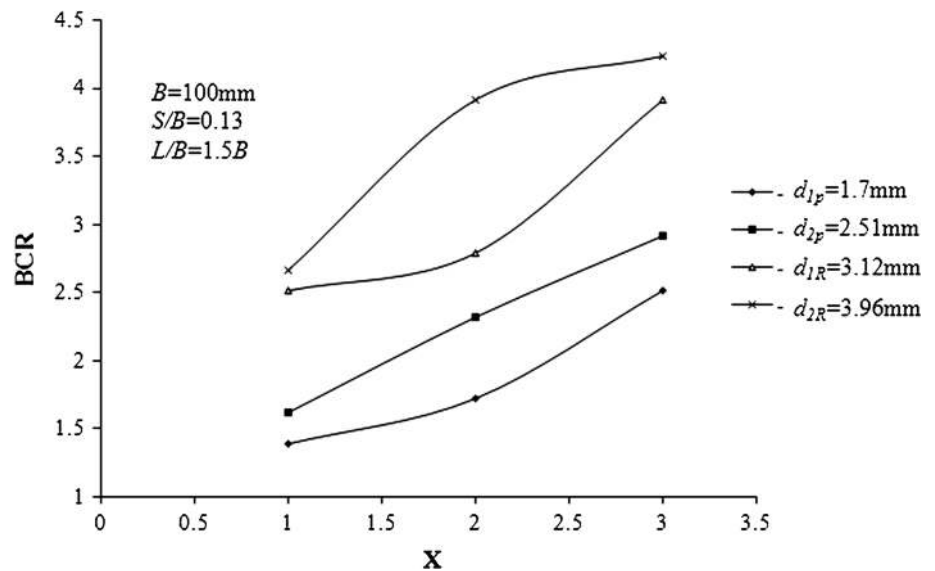
to make the test tank as rigid as possible. The inside wall of the test tank was polished to reduce the side friction. The sand beds were prepared by pouring local sand in layers of 20 mm through rainfall techniques held at a fixed height 300 mm above the surface. After pouring each layer of sand, coloured sand was spread by the side of the glass sheet to obtain a colour band of 1 mm thickness. Final compacted test bed thus showed alternate layers of coloured and ordinary sand. This arrangement facilitated the observation of developed failure surface through the side of acrylic glass sheet. One such test bed along with the loading arrangement has been shown in Fig. 12. After the completion of each test the stiffeners were removed and the position of footing was traced first and then failure surfaces were obtained by joining the kinks of the coloured bands. Position of reinforcing wires after failure was also traced out by gradual removal of sand from outside the reinforced zone on either side of the footing. After drawing the typical failure surfaces, the sand was removed gradually from the acrylic glass sheet side of the test tank and the profile of the final position of reinforcing wires were drawn on the glass sheet with glass marking pencil. The position of reinforcement was finally traced on tracing paper along with the final position of the footing and failure surfaces drawn earlier. One such typical failure surface and the elementary condition of equilibrium for a slice are also shown in Figs. 13 and 14. Working formula as given in Eq. (3) for the factor of safety of slip surfaces due to presence of vertical reinforcement by the method of slices [17] has been extended in the present investigation.

Fig. 12 Loading arrangements for load tests in a rectangular test tank

$$F_R = \left(\sum \tau_f \cos^{-2}\alpha \Delta x + E \right) \Big/ \left(\sum (p+t)\Delta x \tan\alpha \right) \quad (3)$$

where, E is the total resistance offered by all reinforcements $= \sum (E_s N_r/b)$, E_s is the lateral force per reinforcement $= 1.5\gamma dL^2 K_p$, [18], N_r is the total number of reinforcing element on one side of the center line of the footing, b is the width of the sand box, γ is the unit weight of the soil, d is the diameter of the reinforcing element, L is the length of reinforcing element, K_p is the coefficient of Rankine passive earth pressure $= (1 + \sin\phi)/(1-\sin\phi)$, ϕ is the angle of internal friction of soil, τ_f is the shearing resistance along the element of slip surface $= [(p + t)\tan\phi]/[1 + (\tan\alpha \tan\phi)/F]$, p is the $dW/dx + dP/dx$, t is the dT/dx.

It is important to realize that the pressure bulb as well as the failure wedge at the stage of bearing capacity form in shorter direction (Analogous to the bending of a one way slab in shorter direction). Therefore the analysis for continuous footing cases can be extended to hold good without much error for rectangular footing by taking 'B' as width (the smaller of two plan dimension) or for a square footing by taking 'B' as the side of square [19]. Square or rectangular footings are commonly used in practice therefore, the same working formula derived above has been extended to square footing placed in a square test tank (Type-A) as well, where values from both x and y directions have been considered. Since the square footing was used in the present study, the value of the working formula both in X and Y direction will be same due to symmetry of footing, hence added to obtain the final value of factor of safety for the unreinforced case.

$$F_{SUR} = 2\left(\sum \tau_f \cos^{-2}\alpha \Delta x \right) \Big/ \left(\sum (p+t)\Delta x \tan\alpha \right) \quad (4)$$

Now the total resistance (E') offered by vertical reinforcement in the present case is different from that of the formula given in Eq. (2) because of the presence of more number of reinforcing wire/elements around the footing. Therefore the factor of safety in the present case can be given as

$$F_{SR} = 2\left(\sum \tau_f \cos^{-2}\alpha \Delta x + E' \right) \Big/ \left(2\sum (p+t)\Delta x \tan\alpha \right) \quad (5)$$

The theoretical ultimate bearing capacity of unreinforced case of a square footing [20] was calculated using the following equation

$$q = 0.5\gamma B \xi_\gamma N_\gamma \quad (6)$$

Substituting the value of shape factor (ξ_γ) of square footing and bearing capacity factor (N_γ) the theoretical bearing capacities of square footing of different footing width were calculated. The theoretical ultimate bearing capacities of the unreinforced and reinforced subgrades were

Fig. 13 Failure surface of reinforced sand subgrade
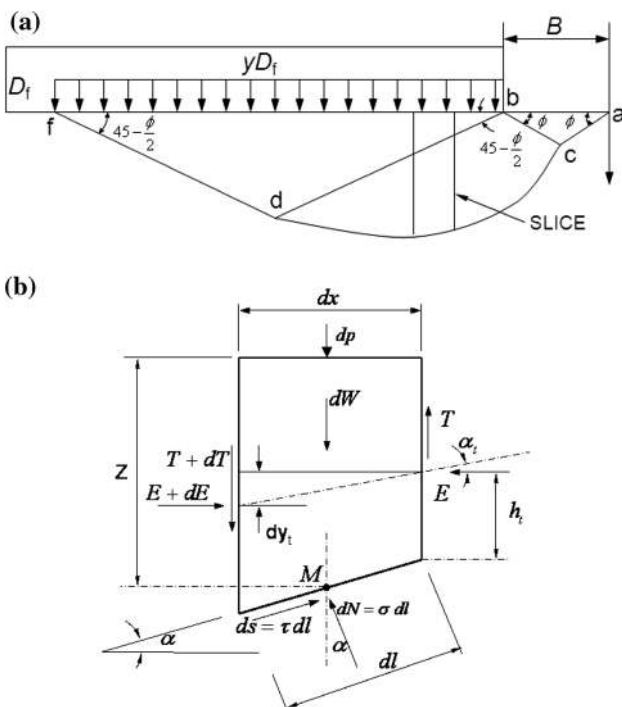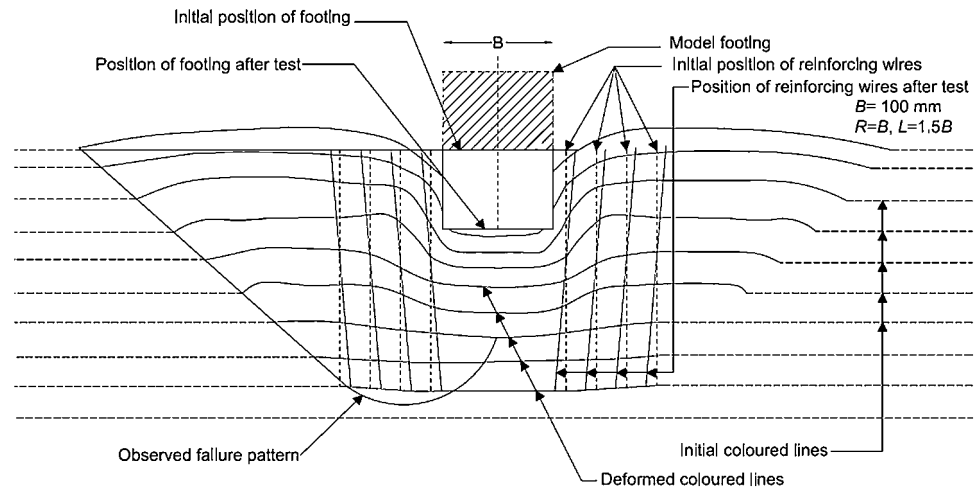
Fig. 14 a Slip surface showing the elementary slice, **b** forces on the **e**lementary slice for equilibrium

obtained by multiplying the initial theoretical bearing capacity load calculated from Eq. (6) with the minimum factors of safety obtained from Eqs. (4) and (5), respectively. A theoretical *BCR* was obtained by taking the ratio of ultimate bearing capacity of reinforced and unreinforced cases, respectively. Now this theoretical *BCR* was compared with experimentally obtained values of *BCR* and typical results are presented in subsequent paragraph for comparison.

Figure 15 shows a typical case of comparison of theoretical and experimental *BCR* for a Type-A test, where the variation of *BCR* with 'X' has been reported at different spacing ratio (S/B). The footing width (B), length of

reinforcement (L) and diameter of reinforcement (d) for this typical case were 75 mm, 2B and 1.7 mm, respectively. It can be observed from the figure that the trend in the variation of theoretical and experimental *BCR* with extent ratio (X) is similar, that is, *BCR* increases with the increase in 'X'. At a given value of extent ratio (X), the value of theoretical *BCR* is always less than that of the experimental *BCR* with few exceptions, particularly, for the cases when the soil is sparsely reinforced or extent ratio (X) $\leq$ 0.5. It can also be observed from the figure that the difference between theoretical and experimental *BCR* value gets reduced considerably when the extent ratio (X) of the reinforcement $\geq$ 1.5. This may be attributed to the fact that the installation of large number of reinforcements around the footing increases the unit weight of soil [21] which also changes the stress distribution within the soil. But when $S/B \geq 0.18$ or $X \leq 0.5$, the soil were considered sparsely reinforced and in all such cases the disturbance caused in initial in situ unit weight of the soil during the installation of vertical reinforcements is not being compensated fully by the confinement produced by less number of reinforcements present in the soil.

Another typical plot of Type-A test showing the variation of *BCR* (theoretical and experimental) with extent ratio (X) for different length of reinforcement (L) has been given in Fig. 16. The spacing ratio (S/R), width of footing (B) and diameter of reinforcement (d) for this particular case were 0.09, 75 and 1.7 mm, respectively. In this case also both theoretical and experimental *BCR* increases with increase in extent ratio (X). Again it is being observed from the figure that the theoretical *BCR* is always less than the experimental *BCR* with few exceptions, particularly for the cases when the $X \leq 0.5$ and $L = B$. The displacement of vertical reinforcements is the most important parameter which significantly affects the result. Therefore any small displacement in reinforcement can generate sufficient friction at the soil reinforcement interface, which in turn may
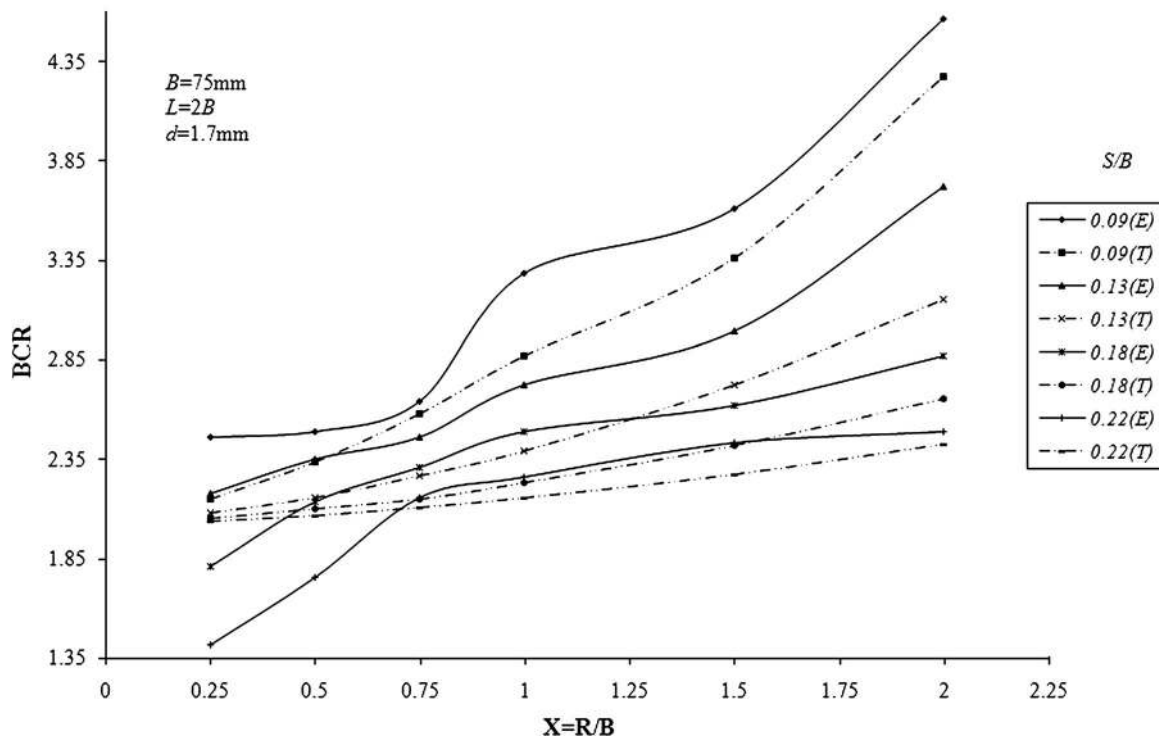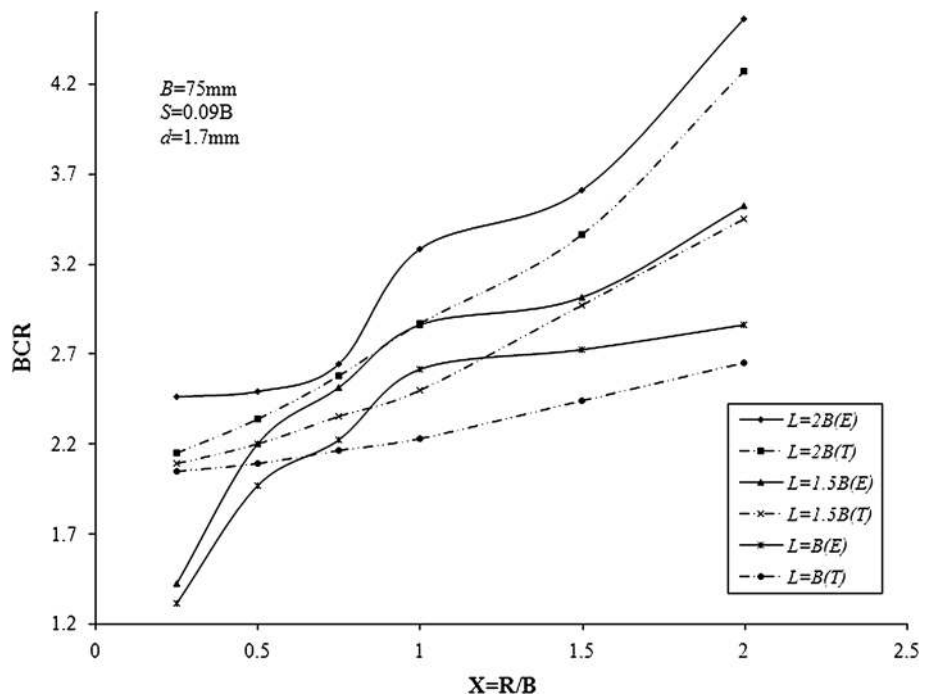
Fig. 15 Comparison of BCR (*E* and *T*) with extent ratio (*X*) for different values of spacing ratios (*S/B*)

Fig. 16 Comparison of BCR (*E* and *T*) with extent ratio (*X*) for different values of length ratio (*L/B*)

mobilize the tensile force in the reinforcing elements [22]. The tangential components of this tensile force directly resist the shear and the normal component increases the confining stress on the shear plane. During failure it is being observed that the displacement of reinforcement depends on the anchoring of the reinforcement, which in turn depends on the length of reinforcing element. As the length of reinforcement increases, the larger portion get anchored in lower soil mass

Fig. 17 Comparison of BCR
(E and T) with extent ratio
(X) for different values of
spacing ratio (S/B)
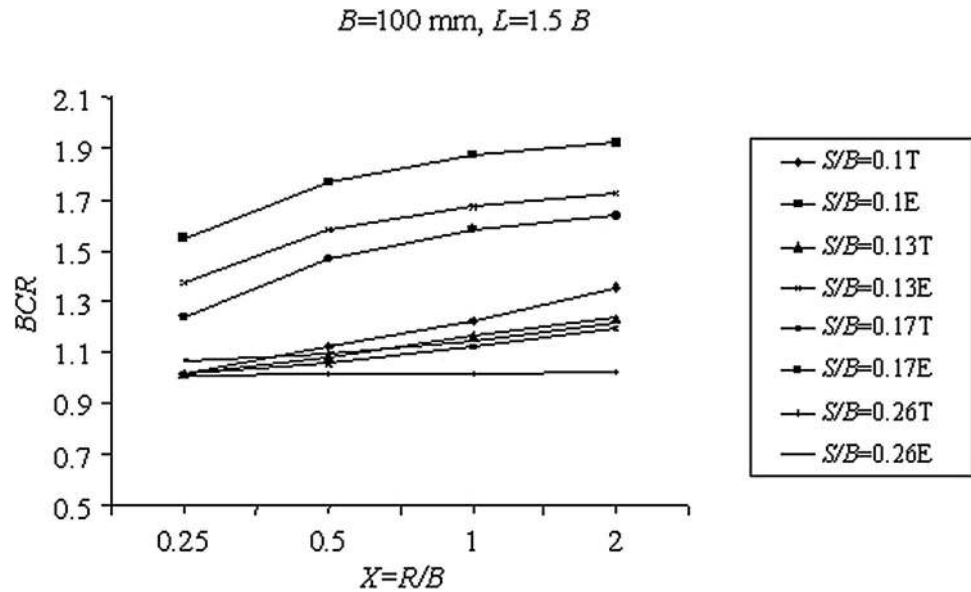
which do not fail and hence do not move along with the sliding slip surface. Though the length of reinforcing element has been considered during the calculation of the passive resistance but the anchoring effect and displacement of reinforcement were not taken into account in the theoretical analysis. When $L = B$ and $X \leq 0.5$, the soil were again considered sparsely reinforced and the reinforcements move along the failure plane without offering any resistance to the flow of soil due to insufficient anchorage which resulted in reduction of experimental BCR.

Theoretical and experimental BCR have also been compared for square model footing tests conducted in a rectangle test tank having the same width as equal to the size of the square footing (type-B). A typical plot giving the comparison of theoretical and experimental BCR with extent ratio (X) at different spacing ratio (S/B) has been shown in Fig. 17. In this case also the theoretical BCR was always less than experimental BCR and is due to the fact that the soil movements were restraint in one direction during the tests.

Limitations

The test results reported in the present paper are based upon the small scale model footing tests while problem encountered in the field are of prototype footing. Use of a small scale model to investigate the behaviour of a full scale foundation is a widely used technique [23] but due to scale effects especially in granular soils it may not play the same role in the laboratory models as in the prototype. The differences occur primarily because of the differences in stress level between the model tests and the field tests and due to the influence of the ratio of footing width to the particle size. The stress level under the small scale model

footing is much smaller than that under a full scale footing. Since in granular soils, angle of internal friction is high at low stress level when compared with the angle of friction at higher stress level, hence the average shear strength mobilized along a slip line under a foundation decreases with an increase in foundation size. Another factor which should be considered is the effect of adjacent footing and the intersection of the stresses under the prototype footing. The scale effect due to footing width/particle size ratio vanishes provided the size of the particle is small compared to the footing dimension. However, the scale effects due to variation in stress level will occur in 1 g (earth gravity) modeling. Though it is of limited use in predicting the behaviour of a particular prototype, the use of 1 g models can be useful in predicting trends of general behaviour. Further investigation using full scale tests or centrifugal model tests or numerical analysis is therefore necessary before quantifying actual design parameters.

Conclusions

The beneficial effect of using vertical reinforcing elements in improving the bearing capacity and settlement characteristics of sand subgrades have been demonstrated through a series of small scale square footing tests in the laboratory. Based upon the results and discussion following general conclusions can be drawn:

1. Soil confinement has a significant effect on improving the behaviour of square footing. The BCR increased by a factor of 4.57 as compared to the unreinforced case.
2. The soil confinement by vertical reinforcement laterally around the footing without disturbing the subgrade

directly below the footing base could be considered as a method to improve the bearing capacity of isolated footing on medium sand. Confinement of soil around the individual footing could easily be achieved by placing the vertical reinforcement with different length, spacing, diameter and extent leading to a significant improvement in their load settlement response.

3. In cases where structures are very sensitive to settlements, soil confinements by vertical reinforcements laterally around the footing can be used to obtain the same allowable bearing pressure at a much lower settlement.

4. Substantial improvement in bearing capacity can be derived if the spacing of reinforcement (S), length of reinforcement (L) and extent of reinforcement (R) lie in the range of (0.06 to 0.09)B, (1.5 to 2.0)B and (1.5 to 2.0)B, respectively.

5. Thickness of reinforcement affects bearing capacity of sand subgrade significantly. Based on the experimental results it was found that by changing the diameter of reinforcing wire from 1.7 to 2.51 mm, BCR increased by 40 %.

6. Surface characteristics of reinforcing wire also affect bearing capacity of sand subgrade significantly. By changing the surface of reinforcement from plain to rough by sand coating, the BCR improved by 50–70 %. Though rough reinforcements are far more effective in improving the ultimate bearing capacity as compared to the plain reinforcement but its installation posed problem during placement particularly when the length of reinforcing elements exceeded 150 mm.

7. For the same area ratio and diameter of reinforcement, the BCR decreases with the decrease in footing width or vice versa. By changing the size of the model footing from 50 to 100 mm, the BCR improved by 54 %.

8. Considering the experimental evidence estimated theoretical BCR seems to be reasonable, but the computed values are invariably lower than the experimental value. Improved co-relation may be achieved through a more refined analysis if substantial data under different conditions are available.

9. The ratio of diameter of reinforcement to the particles for a field installation is likely to be different, further studies on bigger model or numerical analysis is necessary to quantify the parameters for actual design conditions.

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