Fig. 4.7 Variation of % increase in penetration resistance with depth
CHAPTER 5

STUDIES, RESULTS AND DISCUSSION ON COMPRESSIVE LOAD RESPONSE OF GRANULAR PILE-ANCHORS

5.1 Introduction

In this chapter, the results of compressive load tests on expansive soil reinforced with granular pile-anchors are presented and the in situ behaviour of granular pile-anchors in expansive clay beds under compressive loading is discussed.

5.2 Compressive load test and test procedure

5.2.1 Loading Frame Arrangement

The loading frame used for the conduct of both compression and pull-out tests has been specially fabricated for this purpose (Fig. 5.1). It consists of two ISMB 300 sections, 5.2 meters long, and laid in parallel such that the distance between them is 2.8 meters.
Two central H-frames are connected to these girders at their centers as shown in the figure. Each H-frame is made of ISMB 200 sections with two vertical beams connected by a horizontal beam as shown in Fig. 5.1. To the top of these two H-frames, two ISMB 300 sections are welded, an ISMB 300 section is placed perpendicular to these H-frames such that the two ends of the girder rest on the horizontal limbs of the H-frames. At the bottom, this ISMB 300 section is connected by bolts and nuts to the horizontal arm of the H-section. It is also connected by bolts and nuts to the top ISMB 300 section. This central portion constitutes the loading frame and the hydraulic jack takes reaction from the ISMB 300 section shown perpendicular to the plane of paper in Fig 5.1. On the side horizontal arms, that is the projected portions on either side of the bottom most ISMB 300 sections, three numbers of ISA 150 sections are placed perpendicular to these arms as shown in the figure, on the top of these ISA 150 sections timber planks are placed and loaded with sand bags to give the desired weight. The hydraulic jack is connected to a pump provided with a bourdon pressure gauge, which gives the actual amount of load applied.

5.2.2 Test Procedure

After compaction, the un-reinforced expansive clay bed or the clay bed reinforced with granular pile-anchor (in the case of single pile-anchor) or granular pile-anchors (in the case of group of pile-anchors) was inundated with water up to the point of saturation. Saturation of the clay beds was confirmed when final heave was attained. Heave was continuously monitored with dial gauges placed on footing plate fastened to the top end
of the mild steel anchor rod. After confirming saturation, compressive load was applied in increments of 1 kN and the corresponding settlement was recorded. Each increment of loading was applied up to the attainment of final settlement under that increment. The load increment of 1 kN was chosen for the sake of convenience.

Fig. 5.1 shows the set up for conducting the field compressive load test. The load was applied through a hydraulic loading jack (Plate 5.1) and the reaction measured from proving ring in the jack. The load was applied through a loading jack in increments of 1 kN in all the load tests. The reaction from the expansive clay bed or composite ground or the granular pile-anchor alone was measured from a proving ring. The settlement of the test plates under each increment of load was measured with dial gauges of sensitivity 0.01 mm at time intervals of 1, 2.25, 4, 6.25, 9, 16 and 25 minutes and at hourly intervals thereafter for 24 hours. The tests were continued until a maximum load equal to twice the allowable bearing pressure had been applied. Loading was applied differently in different cases as detailed below:

(a) **Un-reinforced expansive clay bed:** In the case of un-reinforced expansive clay bed, the diameter of the test plate used for the application of loading was 300 mm. After the attainment of the final heave, the test plate was placed on a thin layer of fine sand laid on the saturated expansive clay bed and loading applied through the hydraulic loading jack. The sand layer was laid for a proper and uniform contact between the test plate and the clay bed (Fig. 3.1a).

(b) **Composite ground (Expansive clay and granular pile-anchor):** In this case, the diameter of the granular pile-anchor was 150 mm and length 1000 mm. Hence,
the length to diameter ratio \((l_{ga}/d_{ga})\) of the pile-anchor was 6.67. For the load test on composite ground, the diameter of the footing plate used was 375 mm. As mentioned above, the footing plate was fastened to the top end of the anchor rod. After inundation and saturation, loading was applied on the footing plate through the loading jack in increments of 1kN. As the diameter of the footing plate was larger than that of the granular pile-anchor, the applied compressive load was transmitted to both the granular pile and the ambient expansive clay through the footing plate, spreading over both the granular pile-anchor and expansive clay (Fig.3.1b).

(c) *Granular pile-anchor alone*: Pertaining to this case, nine tests were performed (Table 5.1). The \(l_{ga}/d_{ga}\) ratio of the granular pile-anchors ranged from 2.5 to 10. The diameter of the footing plate used up to the point of saturation also varied such that the ratio of the diameter of the footing plate to that of the pile-anchor was 2.5. However, after saturation, the footing plate was removed and the test plate was fastened to the top end of the anchor rod. The diameter of the test plate used for the application of compressive load was equal to that of the granular pile-anchor so that pile-anchor alone was loaded in this series of tests (Fig.3.1c).

(d) *Group of granular pile-anchors*: In this test the expansive clay bed was reinforced with 3-group granular pile-anchors installed in the pattern of an equilateral triangle. All the granular pile-anchors were of diameter 150 mm and length 1000 mm. The footing plate used up to the point of saturation was of diameter 375 mm. The soil between the pile-anchors was loaded with a test plate of diameter 300 mm (Fig.3.1d).
Fig. 5.1 Schematic Diagram of the Compressive Load Test Set-up
### Table 5.1 Load test details

<table>
<thead>
<tr>
<th>Description</th>
<th>Length of the pile-anchor (mm)</th>
<th>Diameter of the pile-anchor (mm)</th>
<th>$l_{pl}/d_{pl}$ ratio</th>
<th>Diameter of the test plate (mm) during inundation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Un-reinforced expansive clay bed</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Composite ground</td>
<td>1000</td>
<td>150</td>
<td>6.67</td>
<td>375</td>
</tr>
<tr>
<td>Group of granular pile-anchors</td>
<td>1000</td>
<td>150</td>
<td>6.67</td>
<td>375</td>
</tr>
<tr>
<td>Granular pile-anchor alone:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>10</td>
<td>250</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1000</td>
<td>150</td>
<td>6.67</td>
<td>375</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>5</td>
<td>500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>7.5</td>
<td>250</td>
<td></td>
</tr>
<tr>
<td></td>
<td>750</td>
<td>150</td>
<td>5</td>
<td>375</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>3.75</td>
<td>500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>5</td>
<td>250</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>150</td>
<td>3.33</td>
<td>375</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>2.5</td>
<td>500</td>
<td></td>
</tr>
</tbody>
</table>
Plate 5.1 Compressive load test arrangement
5.3 **Heave characteristics of the granular pile-anchors subjected to compressive load**

Load tests were conducted after saturation or attainment of final heave. Saturation was confirmed by plotting a curve between heave (mm) and time (days). When the curve became asymptotic with the X-axis, saturation was confirmed. Fig. 5.2 shows the rate of heave of un-reinforced expansive clay bed and clay beds reinforced with single granular pile-anchor and 3-group granular pile-anchors. The un-reinforced expansive clay bed attained a maximum heave of 150 mm in 7 months (210 days). Upto 5 months, the increase in heave was noticeable. However, after 5 months, the curve became asymptotical with X-axis, showing no change in the amount of heave (Fig 5.2). The amount of heave decreased in the case of expansive clay bed reinforced with granular pile-anchor because of the effect of anchor. The time required for the attainment of final heave also decreased because of high permeability characteristics of granular materials (Cooper and Rose, 1999; and Wood et al. 2000), which allowed a quick circulation of the inundating water through the system. The amount of final heave was further reduced to 10 mm in the case of expansive clay bed reinforced with 3-group granular pile-anchors. The time required for the attainment of final heave in the clay bed reinforced with 3-group granular pile-anchors was 3 months (90 days). The group effect of granular pile-anchors reduced both the magnitude of final heave and the time required for the attainment of the final heave considerably.
5.4 Stress-settlement behavior

The results of load tests conducted on an un-reinforced expansive clay bed and clay bed reinforced with granular pile-anchors are discussed in terms of stress-settlement behavior. As the diameter of the test plate changed in different tests, the results are plotted in terms of stress. Fig. 5.3 shows the stress-settlement curves for the un-reinforced expansive clay bed, the composite ground (granular pile-anchor and expansive clay bed together) and granular pile-anchor alone. The data plotted in the figure show that the clay bed reinforced with granular pile-anchor gave an improved compressive load response in comparison to the un-reinforced clay bed as reflected in the stress-settlement curve for composite ground. For example, the stress required to cause a settlement of 25 mm in the un-reinforced expansive clay bed was 200 kN/m$^2$ whereas the stress required to cause the same amount of settlement in the case of composite ground was 500 kN/m$^2$ (Table 5.2). This indicates that the load-carrying capacity of the clay bed reinforced with granular pile-anchor improved by 150%. The applied load was effectively resisted by the granular material by virtue of high friction angle. The stress required to cause the same amount of settlement of 25 mm further increased when granular pile-anchor alone was loaded (Fig. 5.3).

When the granular pile-anchor alone was loaded, the applied load was entirely resisted by densely compacted granular material alone which showed an angle of internal friction of 40° at a relative density of 70% as determined from direct shear tests. The stress required to cause a settlement of 25 mm in this case was 675 kN/m$^2$ (see Table
5.2). This indicates an improvement of 240% in applied stress with reference to the unreinforced expansive clay bed (Table 5.2).

Fig 5.3 also shows a comparison of the results of load tests on laboratory-scale and field-scale granular pile-anchors (Phanikumar, 1997; Phanikumar et al. 2004) in terms of stress-settlement behaviour. The dotted lines indicate the stress-settlement behavior of laboratory-scale granular pile-anchors. The data shown in the figure pertain to the cases of composite ground and granular pile-anchor alone. It may be mentioned here that the stress-settlement curve for the laboratory-scale un-reinforced expansive clay bed tallies closely with that for field-scale clay bed. The stress-settlement curve in the figure indicates that the pattern of compressive load response of the laboratory-scale granular pile-anchors was similar to that of the field-scale granular pile-anchors. But, the load response in the case of in situ granular pile-anchors was better than that in the case of laboratory-scale granular pile-anchors. The stresses required to cause a settlement of 25 mm in laboratory-scale cases of composite ground and granular pile-anchor alone was 450 and 600 kN/m² respectively, whereas the stresses for the same amount of settlement in the field-scale cases was 500 and 675 kN/m² respectively (Table 5.2). This clearly establishes the efficacy of granular pile-anchor foundation (GPAF) system in expansive clay bed in situ with regard to compressive load response also.

Fig 5.4 shows the stress-settlement behavior of un-reinforced expansive clay bed and clay bed reinforced with a group of granular pile-anchors. The figure also shows the stress-settlement curve for composite ground for comparison. The compressive load response for the expansive clay bed reinforced with a group of granular pile-anchors
showed a significant improvement in load-carrying capacity over the un-reinforced clay bed. Though the test plate was placed on saturated clay alone in both the cases, the resistance to compressive load offered by the clay reinforced with a group of granular pile-anchors was higher than that by un-reinforced clay. Heave was reduced significantly in the expansive clay bed reinforced with granular pile-anchors. This means that the decrease in dry unit weight or the increase in void ratio of the clay bed owing to heaving of clay was arrested by granular pile-anchors. Hence, the reinforced clay bed could resist the compressive load better than the un-reinforced clay bed wherein complete heave occurred leading to a significant reduction in dry unit weight or increase in void ratio. The stress required to cause a settlement of 25 mm in un-reinforced expansive clay bed was 200 kN/m², whereas it increased to 330 kN/m² (Table 5.2) in the case of expansive clay bed reinforced with a 3-group of granular pile-anchors showing an improvement of 65% (Table 5.2). The stress-settlement curve for composite ground lay above that for a clay bed reinforced with a group of granular pile-anchors indicating an improved compressive load response in the case of composite ground. Table 5.2 summarizes the stress values required to cause a settlement of 25 mm in different cases. The table also shows the percentage increase in stress for different cases with reference to un-reinforced expansive clay bed.

Fig 5.5 shows the effect of length of granular pile-anchors on stress-settlement behavior for a given diameter of 150 mm. The figure shows the stress-settlement curves for un-reinforced clay bed and clay bed reinforced with single granular pile-anchors having different lengths (500 mm, 750 mm and 1000 mm) but uniform diameter of 150
Table 5.2 Comparison of stresses for a settlement of 25 mm for different cases

<table>
<thead>
<tr>
<th>Description</th>
<th>Stress (kN/m²)</th>
<th>Increase in stress with respect to un-reinforced expansive clay bed (%)</th>
<th>Increase in stress with respect to group effect (%)</th>
<th>Increase in stress with respect to composite ground (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Un-reinforced expansive clay bed</td>
<td>200</td>
<td>0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Group effect</td>
<td>330</td>
<td>65</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Composite ground</td>
<td>500</td>
<td>150</td>
<td>49</td>
<td>0</td>
</tr>
<tr>
<td>Granular pile-anchor alone</td>
<td>675</td>
<td>240</td>
<td>97</td>
<td>33</td>
</tr>
</tbody>
</table>
mm. It may be mentioned here that the data shown in Fig 5.5 pertain to load tests performed on granular pile-anchors alone (test plate bearing only on the granular pile). Increasing length of the granular pile-anchor resulted in higher amount of stress for a given settlement. For example, the stress required for 25 mm settlement was respectively 360, 505, 690 kN/m2 for 500 mm, 750 mm and 1000 mm long granular pile-anchors. A similar trend was observed with increasing length of granular pile-anchor for other diameters also.

Of the three granular pile-anchors mentioned above (Fig 5.5), one was an end-bearing pile-anchor and the other two were floating. As the diameter of the granular pile-anchors was the same at 150 mm, the term \( \frac{1}{2} \gamma B N_r \) would remain the same in the equation for ultimate bearing capacity, where \( N_r \) is the bearing capacity factor. However, the resistance to compressive loading offered by friction generated over the cylindrical surface area of the granular pile-anchors increases with increasing length of the pile-anchor. Hence, the increase in the stress required to be applied on the granular pile-anchor for a given settlement of 25 mm.

Another important phenomenon to be considered here is bulging (discussed in detail in the following section), or the lateral flow of the granular material into the surrounding clay under the applied compressive loading (Plate 5.2). Bulging is one of the mechanisms of failure of granular piles. Granular piles (either end-bearing or floating), which are greater than three diameters in length fail in bulging. All the three pile-anchors
discussed in Fig 5.5 are longer than 3 pile diameters. Hence, they underwent bulging under the applied compressive load (Plate5.3). It was observed that both the diameter and length of bulge increased with increasing length of the pile-anchor for a given diameter (Table 5.3). The larger the diameter and length of the bulged portion of the granular pile-anchor, the higher will be the resistance to the applied compressive loading. Hence, the improvement in the load-carrying capacity of the granular pile-anchors with increasing length for a given diameter.

Granular piles under compressive loading deform laterally also apart from undergoing settlement in the vertical direction (Plate5.2). In this phenomenon, the granular material of the pile flows radially outward into the surrounding expansive clay. In a long granular pile with lengths greater than 3 pile diameters, this phenomenon occurs up to a certain depth from the top of granular pile. Thus, the deformed top portion of the pile has a larger diameter than the bottom un-deformed portion. This is called bulging. The length and diameter of the bulged portion of the granular pile depend upon various factors such as the relative density of granular material, the un-drained shear strength of clay surrounding the granular pile and the intensity of applied loading.

After the load tests on granular pile-anchors alone, the diameter of the deformed pile or bulge diameter (d_b) was measured at different depths (z) from the top of the pile-
### Table 5.3 Diameter and length of bulge

<table>
<thead>
<tr>
<th>Length of the granular pile-anchor, ( (l_{gp}) ) mm</th>
<th>Diameter of the granular pile-anchor, ( (d_{gp}) ) mm</th>
<th>Maximum bulge diameter ( (d_b) )</th>
<th>Maximum bulge length ( (l_b) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000</td>
<td>100</td>
<td>130 mm or 1.3( d_{gp} )</td>
<td>240 mm or 0.24 ( l_{gp} )</td>
</tr>
<tr>
<td></td>
<td>150</td>
<td>204 mm or 1.36 ( d_{gp} )</td>
<td>272 mm or 0.27 ( l_{gp} )</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>284 mm or 1.42 ( d_{gp} )</td>
<td>310 mm or 0.31 ( l_{gp} )</td>
</tr>
<tr>
<td>750</td>
<td>100</td>
<td>126 mm or 1.26 ( d_{gp} )</td>
<td>200 mm or 0.27 ( l_{gp} )</td>
</tr>
<tr>
<td></td>
<td>150</td>
<td>194 mm or 1.3 ( d_{gp} )</td>
<td>240 mm or 0.32 ( l_{gp} )</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>268 mm or 1.34 ( d_{gp} )</td>
<td>260 mm or 0.35 ( l_{gp} )</td>
</tr>
<tr>
<td>500</td>
<td>100</td>
<td>118 mm or 1.18 ( d_{gp} )</td>
<td>175 mm or 0.35 ( l_{gp} )</td>
</tr>
<tr>
<td></td>
<td>150</td>
<td>180 mm or 1.20 ( d_{gp} )</td>
<td>194 mm or 0.39 ( l_{gp} )</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>254 mm or 1.27 ( d_{gp} )</td>
<td>232 mm or 0.46 ( l_{gp} )</td>
</tr>
</tbody>
</table>
Plate 5.2 Compression failure of the granular pile-anchor

Plate 5.3 Bulging of the granular pile-anchor
anchor. The increase in diameter of the deformed granular pile-anchor was measured from the center of the granular pile-anchor. The length of the deformed portion of the pile-anchor was also measured and is indicated as bulge length ($l_b$). Bulge characteristics such as bulge profile, maximum bulge diameter and bulge length are discussed in terms of radius of deformed pile-anchor and corresponding depth from the top of the pile. Fig 5.6 shows the non-dimensional bulge profiles for granular pile-anchors of 1000 mm and diameter 100 mm, 150 mm and 200 mm. The profiles show the variation of non-dimensional radius of the deformed pile ($r/r_o$) with variation in the non-dimensional bulge length ($z/l_{gp}$), where $r_o$ is the radius of the un-deformed granular pile-anchor, $r$ is the radius of the deformed granular pile-anchor and $l_{gp}$ is the total length of the granular pile-anchor. The bulge profiles indicate the geometry of the lateral deformation of the pile. The bulge profiles show that bulging or bulge diameter increased with increasing diameter of the granular pile-anchor. Bulge length also increased with increasing diameter of the granular pile-anchor.

Table 5.3 shows the maximum bulge diameter and bulge length in terms of diameter ($d$) and length ($l$) of the un-deformed granular pile-anchor respectively. The maximum bulge diameter increased from $1.27d_{gp}$ to $1.42d_{gp}$ for granular pile-anchors of diameter 200 mm when length of pile-anchor increased from 500 to 1000 mm. Hence, the percentage increase in diameter was augmented from 27% to 42%. The maximum bulge diameter increased with increasing diameter for a given length of granular pile-anchor. For example, the maximum bulge diameter increased from $1.3d_{gp}$ to $1.42d_{gp}$ for granular pile-anchors of length 1000 mm when diameter increased from 100 mm to 200 mm. This
shows that the percentage increase in diameter changed from 30% to 42% when the
diameter increased from 100 mm to 200 mm. Hence, the effect of diameter of granular
pile-anchor was more pronounced on bulge diameter than that of length of granular pile-
anchor. In this series of tests, the range of maximum bulge diameter observed was 1.18
d_{gp} to 1.42 d_{gp}.

The maximum bulge length observed was 310 mm in the case of granular pile-
anchor of length 1000 mm and diameter 200 mm. In terms of length and diameter of
granular pile-anchor the maximum bulge length, therefore, was 0.31 l_{gp} and 1.55 d_{gp}
respectively. The range of maximum bulge lengths observed was 175 mm to 310 mm
respectively in granular pile-anchors of diameter 100 to 200 mm and lengths 500 and
1000 mm. The maximum bulge length increased with increasing diameter of granular
pile-anchor for a given length. It also increased with increasing length of a granular pile-
anchor for a given diameter. However, it is interesting to observe that the effect of length
of granular pile-anchor is more pronounced on increase in maximum bulge length than
the effect of diameter. For example, the percentage increase in maximum bulge length
was 29% in granular pile-anchor of 1000 mm when the diameter increased from 100 to
200 mm. In contrast to this, the percentage increase in maximum bulge length was 37%
in a granular pile-anchor of 100 mm diameter when its length was increased to 500 to
1000 mm.
5.6 Conclusions

An extensive field-test program was performed to study the compressive load response of expansive clay beds reinforced with granular pile-anchor foundations (GPAF) in situ. Granular pile-anchors, which are chiefly a tension-resistant foundation technique devised for counteracting heave of expansive clay beds, are also subjected to compressive loading from superstructure. Hence, field-scale compressive load tests were conducted in situ to establish the technique of GPAF as an efficacious foundation practice in expansive soils. The test program included the study of stress-settlement characteristics and bulge characteristics such as length and diameter of bulge of granular pile-anchors embedded in expansive clay beds. The main conclusions drawn from the field tests are as follows:

1. Reinforcing expansive clay beds with granular pile-anchors reduced the amount of final heave and also improved the rate of heave. The un-reinforced expansive clay bed of 1000 mm thickness attained a maximum heave of 150 mm in 7 months (210 days), whereas the clay bed reinforced with granular pile-anchors attained a final amount of heave of 28 mm in a short time of 4 months (120 days). The reduction in final heave in the case of expansive clay bed reinforced with granular pile-anchor was due to the resistance to uplift mobilized along the pile-soil interface. The time required for the attainment of final heave also decreased because of high permeability characteristics of granular materials. The amount of final heave was further reduced to 10 mm in the case
of expansive clay bed reinforced with 3 group granular pile-anchors in 3 months (90 days). This is in accordance with the observations made in the laboratory-scale model study on granular pile-anchors (Phanikumar, 1997; Phanikumar et al. 2004 b).

2. The expansive clay bed reinforced with granular pile-anchor (composite ground) gave an improved compressive load response in comparison to the un-reinforced clay bed. The stress required to cause a settlement of 25 mm in the case of composite ground was 500 kN/m² as against 200 kN/m² in the case of un-reinforced expansive clay bed, showing an improvement in the load-carrying capacity of 150%.

3. The compressive load response of granular pile-anchor alone further improved. When the granular pile-anchor alone was loaded, the applied load was resisted by compacted granular material alone. The stress required to cause a settlement of 25 mm in the case of granular pile-anchor alone was 675 kN/m², indicating a percentage improvement of 240% in load-carrying capacity with reference to un-reinforced expansive clay bed. The observations made with regard to compressive load response in the case of granular pile-anchors in situ are in agreement with those in the case of laboratory-scale model study.

4. The compressive load response of the expansive clay bed reinforced with a group of granular pile-anchors also showed a significant improvement in load-carrying capacity over the un-reinforced clay bed. As the decrease in dry unit weight or the increase in the void ratio of the clay bed owing to heaving of clay was arrested by granular pile-anchors, the compressive load response was better than the un-reinforced clay bed. A percentage improvement of 65 was observed in load-carrying capacity in the case of the clay bed reinforced with 3-group granular pile-anchors over the un-reinforced expansive clay bed.
5. The maximum bulge diameter increased with increasing diameter and length of the pile-anchor. The maximum bulge diameter increased from $1.27d_p$ to $1.42d_p$ when length of pile-anchor increased from 500 to 1000 mm. Similarly, the maximum bulge diameter increased from $1.3d_p$ to $1.42d_p$ when the diameter of the pile-anchor increased from 100 mm to 200 mm. The effect of diameter of granular pile-anchor was more on bulge diameter than that of length of granular pile-anchor.

6. The maximum bulge length increased with increasing diameter and length of granular pile-anchor. The effect of length of granular pile-anchor is more pronounced on increase in maximum bulge length than that of diameter. The percentage increase maximum bulge length was 29% in granular pile-anchor of 1000 mm when the diameter increased from 100 to 200 mm, whereas the percentage increase in maximum bulge length was 37% in a granular pile-anchor of 100 mm diameter when its length was increased to 500 to 1000 mm.
Fig. 5.2 Rate of heave of clay bed reinforced with 1000 mm long granular pile-anchor
Fig. 5.3 Stress-settlement behavior of un-reinforced expansive clay bed and clay beds reinforced with granular pile-anchors.
Fig. 5.4 Stress-settlement behavior of unreinforced expansive clay bed and clay bed reinforced with groups of granular pile-anchors.
Fig. 5.5 Effect of length of granular pile-anchors on stress-settlement behavior (diameter = 150 mm)