EXPANSIVE SOILS—PROBLEMS AND REMEDIES

B.R. Phanikumar
Professor, Department of Civil Engineering, VIT University, Vellore–632 014, India.
E-mail: phanikumar_29@yahoo.com

ABSTRACT: Expansive soils are one of the examples of tropical soils. These soils pose problems to civil engineers in general and to geotechnical engineers in particular. They cause damage to structures founded in them because of their potential to react to changes in moisture regime. They undergo severe volume changes corresponding to changes in moisture content. Because of their potential to undergo volumetric changes, civil engineering structures such as foundations, retaining walls, pavements, airports, side walks, canal beds and linings are damaged. Various innovative practices have been devised to counteract the volume change problem and to safeguard the structures. This paper discusses the problems and remedies in detail.

1. INTRODUCTION

1.1 Expansive Soils and Problems

Expansive soils pose problems to civil engineers in general and to geotechnical engineers in particular (Chen 1988). They cause damage to structures founded in them because of their potential to react to changes in moisture regime. They undergo severe volume changes corresponding to changes in moisture content. They swell or increase in their volume when they imbibe water and shrink or reduce in their volume on evaporation of water (Chen 1988). Because of their alternate swelling and shrinkage, they result in detrimental cracking of lightly loaded civil engineering structures such as foundations, retaining walls, pavements, airports, side walks, canal beds and linings (Chen 1988).

2. EXISTING FOUNDATION PRACTICES

2.1 Mechanical, Physical and Chemical Alterations

This involves excavation of expansive soil and replacement with non-expansive material, where the depth of active zone (depth from ground surface wherein seasonal moisture fluctuations occur) is small and where a suitable replacement material is available. Sand cushion method and Cohesive Non-Swelling (CNS) layer method are very popular.

2.1.1 *Sand Cushion Method*

In this method, the entire depth of the expansive clay stratum if it is thin, or a part thereof, if it is deep enough, is removed and replaced by a sand cushion compacted to the desired density and thickness. It was reported that swelling pressure varies inversely as the thickness of the sand layer and directly as its density. Hence, sand cushions are formed in their loosest state to avoid the possibility of excessively increasing the swelling pressure without, however, violating the criterion of bearing capacity. The basic philosophy of this method is that, in monsoon, the saturated sand occupies less volume, accommodating some of the heave of underlying expansive soil, and in summer, partially saturated sand bulks and occupies the extra space left by the shrinkage of the soil.

2.1.2 *Cohesive Non-Swelling (CNS) Layer Method*

In this method, about top 1 m to 1.2 m of the expansive soil is removed and replaced by a cohesive non-swelling soil layer. According to Katti (1978), with saturation of expansive soil cohesive forces are developed up to a depth of about 1.0–1.2 m and counteract heave. CNS layer creates an environment similar to that around 1 m deep in an expansive soil with equivalent cohesion to counteract heave.

In physical alteration, granular material is mixed with expansive clay to minimize heave. However, the permeability of the resulting blend would be more than that of the expansive soil resulting in a faster ingress of water into the soil. Chemical alteration involves addition of chemicals to expansive clay to reduce heave by altering the nature of expansive clay minerals (Chen 1988). Lime treatment of expansive soils is the most widely used technique and the most effective technique of chemical alteration to minimize volume changes and to increase the shear strength of foundation expansive soils. The usefulness of the treatment depends on the reactivity of the soil to lime treatment and the extent of dispersion of lime into the soil.

In pavements, a technique called Lime-Slurry Pressure Injection (LSPI) is also used for minimizing swelling of soils. In this technique, lime-slurry is injected into drill-holes under a pressure of 15 kg/cm^2 (Chen 1988). In this technique, lime slurry penetrates into the fissures in the soil mass to a sufficient depth (usually 8 to 10 feet), and the lime-filled seams help control the soil water content, reduce volumetric changes and increase soil strength. However, for LSPI to be an effective technique, the expansive clay soil must contain an extensive network of fissures. Otherwise, lime cannot penetrate into the relatively impermeable soil to an appreciable distance from the injection hole to form a continuous lime seam moisture barrier Lime-soil columns were also tried to stabilize expansive clays in-situ. It was reported that diffusion of lime into the ambient soil is effective up to a radial distance of about 3 times the diameter of the lime-soil column.

Addition of calcium chloride ($CaCl_2$) to expansive soil has also proved efficacious in altering the swelling properties of the soil. Calcium chloride is a hygroscopic material and hence, is pre-eminently suited for stabilization of expansive soils, because it absorbs water from the atmosphere and prevents shrinkage cracks occurring in expansive soils during summer. Addition of calcium chloride to expansive soils reduced Plasticity Index (PI), free Swell Index (FSI%), swell potential and ($\Delta H/H$, %) and swelling pressure, p_s, significantly. Another additive that has been found to be quite promising in reducing the swelling characteristics and improving the engineering behavior of expansive soils is fly ash. The efficacy of fly ash as an additive to expansive soils in ameliorating their properties was reported.

2.2 Special Foundation Techniques

The problem of heave or uplift of foundations caused by expansive soils is one of tension developed in the soil due to swelling. Hence, tension-resistant foundations are required for counteracting the heave problem. Various special foundation techniques such as drilled piers, belled piers (Chen 1988) and under-reamed piles are all tension-resistant foundations.

Granular Pile-Anchor (GPA) technique has been a recent innovation over the conventional granular pile, modified into an anchor (Phanikumar 1997). This paper discusses GPA system in detail as it is the latest most successful tension-resistant foundation system. Laboratory testing performed to study the efficacy of Granular Pile-Anchor Foundation (GPAF) system in reducing the amount of heave and improving the engineering behavior of expansive clay beds gave promising results. The effect of length, diameter and relative density of GPAs was studied. In a GPA, the foundation is anchored at the bottom of the granular pile to a mild steel plate through a mild steel rod to render the granular pile tension-resistant by the effect of anchor, which is able to counteract the uplift force exerted on the foundations. In a granular pile-anchor, the resistance to uplift is developed mainly due to, (i) the weight of the granular pile acting in the downward direction and (ii) uplift resistance due to friction mobilized along the cylindrical pile-soil interface, and precludes the possibility of heave of foundations. The upward force acting on the foundation is because of swelling of the expansive soil on imbibition of water. The resisting force acting in the downward direction and counteracting the uplift force is due to friction mobilized along the pile-soil interface, which depends on the shear strength parameters of the interface.

A total of 81 laboratory model tests were conducted for studying the heave behaviour. The relative density of the granular pile was varied as 50%, 60% and 70%. The height of the expansive clay bed and the granular pile-anchor were the same in all the tests. A square mild steel plate of size 100 mm × 100 mm was used as the surface footing in the heave tests (set up not shown here). The rate and amount of heave of the unreinforced clay bed were compared with those of the clay bed reinforced with granular pile-anchor. The unreinforced expansive clay bed attained a final heave of 9% in 9 days. However, the heave of the expansive clay bed reinforced with granular pile-anchor attained a reduced amount of heave of 1.15% in a short period of 3 days (see Fig. 1). As surface area of the GPA increased, heave of the clay bed decreased (see Fig. 2).

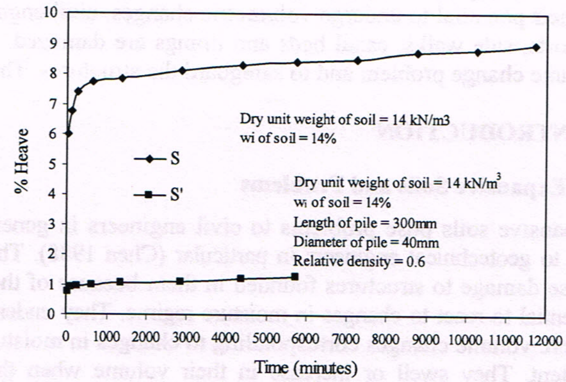

Fig. 1: Rate of Heave

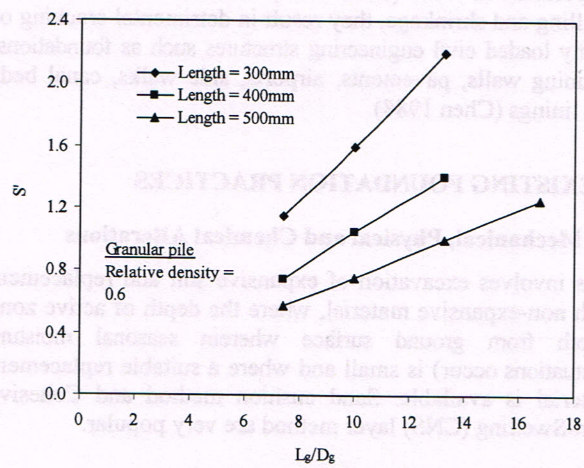

Fig. 2: Effect of l/d Ratio on % Heave for a Given Length

The pull-out capacity of the granular pile-anchors increased with increasing length of the pile-anchor and relative density of the granular material. For a pile of length of 250 mm, the load for 30 mm upward movement was about 250 N, whereas for a pile of length of 350 mm, it was equal to about

510 N, indicating an increase of about 100%. The % increase in the pull-out load when the relative density was increased from 50 to 70% was about 30% for an upward movement of 30 mm (see Figs. 3 and 4). It was found that the load-carrying capacity of the expansive clay bed also increased by the installation of granular pile-anchors. The loading intensity for a settlement of 25 mm in the case of the reinforced clay bed increased by 2½ times over that for the same settlement in the case of the unreinforced clay bed (see Fig. 5).

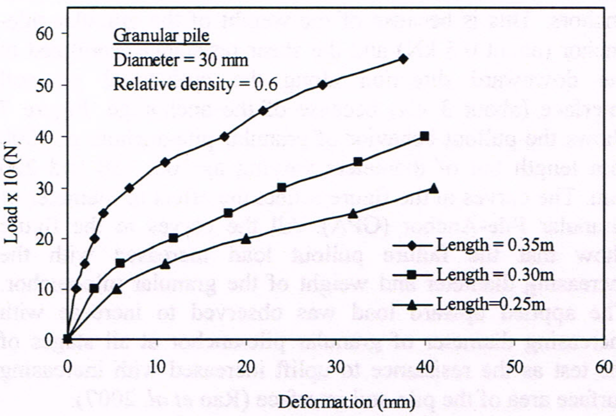

Fig. 3: Influence of Length of Granular Pile-Anchor on the Pullout Behavior

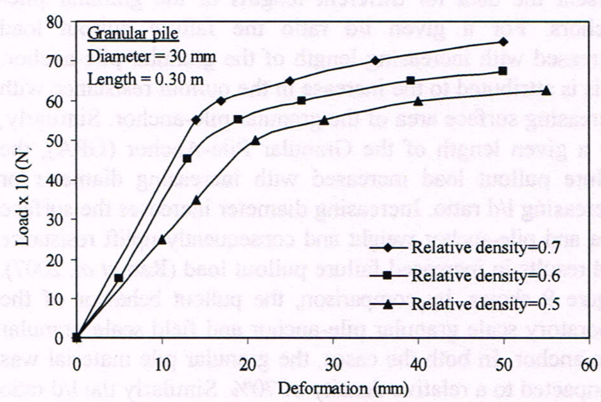

Fig. 4: Effect of Relative Density on the Pull-Out Behavior

3. FULL SCALE TESTING ON GPA SYSTEM

Granular pile-anchor system was also tested in field conditions by performing in situ tests (Rao *et al.* 2004). It was found that heave reduction and the rate of heave were similar to those obtained in the laboratory model tests.

An extensive test program was conducted in situ for establishing the efficacy of granular pile-anchors in expansive clay beds. Amount of heave and rate of heave were observed both in the case of un-reinforced expansive clay beds and clay beds reinforced with granular pile-anchors. The expansive soil used in this investigation was collected from a depth of 1.5 m below the ground level in Amalapuram, Andhra Pradesh, India. The soil was a highly swelling soil with Free Swell Index (FSI) of 180%. It was a CH soil. Table 1 shows the index properties of the expansive soil. The maximum dry unit weight was 14 kn/m^3 and the optimum moisture content was 27%. The granular material used for the installation of granular piles was a blend of 20% stone chips and 80% coarse sand. The particle sizes of stone chips and coarse sand were the same as the granular material used in laboratory testing. It may be mentioned here that all the granular piles in the test program were compacted at a relative density of 70%.

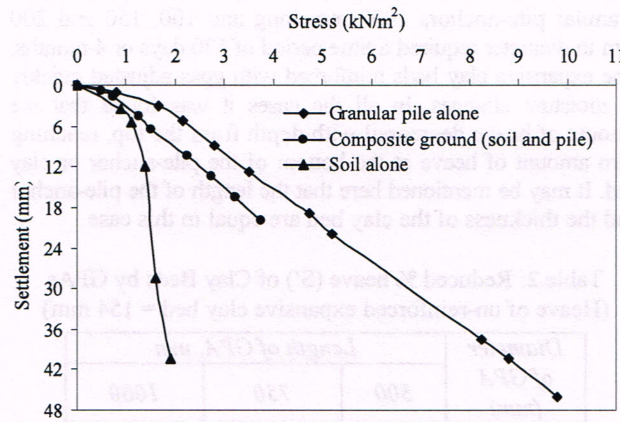

Fig. 5: Stress-Settlement Curves

Table 1: l_{gp}/d_{gp} Ratios of Granular Pile-Anchors

Diameter of the pile, mm	*Length of the pile, mm*		
	500	*750*	*1000*
100	5	7.5	10
150	3.33	5	6.67
200	2.5	3.75	5

In all the tests the expansive clay beds were compacted at a placement water content of 15% and dry unit weight of 14 kN/m^3. The thickness of expansive clay beds was fixed at 1000 mm. the length of the granular pile-anchor (l_{gp}) was varied as 500, 750 and 1000 mm. for each length, the diameter of granular pile-anchors (d_{gp}) was varied as 100, 150 and 200 mm. Hence, the range of the length to diameter ratio (l_{gp}/d_{gp}) of granular pile-anchors varied from 2.5 to 10. Table 1 shows the l/d ratios of the GPAs tested. Heave of the clay bed at different depths and radial distances from the center of the granular pile-anchor was also continuously monitored as the clay bed was inundated.

The un-reinforced expansive clay bed attained a final amount of heave of 154 mm or final % heave of 15.4%. As there was no technique provided in the clay bed for arresting heave, the clay bed heaved to its fullest extent. However, in the case of clay beds reinforced with GPAs in situ, heave was reduced considerably. The respective amounts of reduced heave of clay beds reinforced with GPAs of length 1000 mm and diameters 100,150 and 200 mm were 48, 27 and 15 mm. Hence, the final % of heave was 4.8%, 2.7% and 1.5% in the above cases (Phanikumar *et al.* 2007). The amount of heave was reduced quite effectively by reinforcing the clay beds with GPAs due to the effect of anchorage and shear resistance mobilized along the pile-soil interface (see Table 2). Regarding the rate of development of heave, the un-reinforced expansive clay bed required about 210 days or 7 months of continuous wetting to attain the final amount of heave (154 mm), whereas the clay bed reinforced with granular pile-anchors required a much less time. For example, the clay bed reinforced with granular pile-anchors 1000 mm long and 100, 150 and 200 mm in diameter required a time period of 120 days or 4 months. The expansive clay beds reinforced with gpas adjusted quickly to moisture changes. In all the cases it was found that the amount of heave decreased with depth from the top, reaching zero amount of heave at the bottom of the pile-anchor or clay bed. It may be mentioned here that the length of the pile-anchor and the thickness of the clay bed are equal in this case.

Table 2: Reduced % heave (S′) of Clay Beds by GPAs (Heave of un-reinforced expansive clay bed = 154 mm)

Diameter of GPA (mm)	*Length of GPA, mm*		
	500	*750*	*1000*
100	75	64	48
150	63	47	27
200	46	38	15

However, in the case where the length of the pile-anchor was less than thickness of the clay bed, some amount of heave was recorded at the bottom of the pile-anchor. This was because the expansive clay beneath the base of the pile-anchor swelled. Heave increased with increasing radial distance from the center of the granular pile-anchor in curvilinear fashion.

Further, a test program was conducted in the field on granular pile-anchors embedded in expansive clay beds. The reinforced clay beds were saturated and the amount and rate of heave were observed and compared with those of the unreinforced clay beds. After complete saturation, pullout load was applied on the granular pile-anchor and the upward movement noted. In total, nine field pullout tests were conducted on single granular pile-anchors varying the length and diameter of granular pile-anchor. One pullout test was conducted to study the group behavior also (Rao *et al.* 2007).

Figure 6 shows the pullout load-upward movement curves for single granular pile-anchors of a uniform diameter of 200 mm with lengths varying as 500, 750 and 1000 mm. The curves indicate the influence of length of granular pile-anchor. All the granular pile-anchors were tested up to failure. The curves presented in the figure show that the failure pullout load increased with increasing length of the granular pile-anchor. The curves also indicate that, at all stages of loading, the upward load required to be applied on the granular pile-anchor to cause a given upward movement increased with increasing length of granular pile-anchor. However, it may be observed that, up to an applied uplift load of 4 kN, there was no significant upward movement in the granular pile-anchors. This is because of the weight of the granular pile-anchor (about 0.5 kN) and the shear resistance mobilized in the downward direction along the cylindrical pile-soil interface (about 3 kN) because of the anchorage. Figure 7 shows the pullout behavior of granular pile-anchors of 1000 mm length but of diameters varying as 100, 150 and 200 mm. The curves in the figure reflect the effect of diameter of Granular Pile-Anchor (GPA). All the curves in the figure show that the failure pullout load increased with the increasing diameter and weight of the granular pile-anchor. The applied upward load was observed to increase with increasing diameter of granular pile-anchor at all stages of the test as the resistance to uplift increased with increasing surface area of the pile-soil interface (Rao *et al.* 2007).

Figure 8 shows the variation of failure pullout load (kN) with the l/d ratio of Granular Pile-Anchors (GPA). The curves present the data for different lengths of the granular pile-anchors. For a given l/d ratio the failure pullout load increased with increasing length of the granular pile-anchor. This is attributed to the increase in the pullout resistance with increasing surface area of the granular pile-anchor. Similarly, for a given length of the Granular Pile-Anchor (GPA), the failure pullout load increased with increasing diameter or decreasing l/d ratio. Increasing diameter increases the surface area and pile-anchor weight and consequently uplift resistance and results in increased failure pullout load (Rao *et al.* 2007). Figure 9 shows, by comparison, the pullout behavior of the laboratory scale granular pile-anchor and field scale granular pile-anchor. In both the cases, the granular pile material was compacted to a relative density of 70%. Similarly the l/d ratio of granular pile-anchor in both the cases was equal to 10. While the diameter and length of laboratory scale granular pile-anchor were 30 and 300 mm, those of the field scale granular pile-anchor were 100 and 1000 mm, respectively. The thickness of the expansive clay bed was equal to the length of the Granular Pile-Anchor (GPA) in both the cases. The curves in the figure show that the uplift load (kN) required to be applied for any given upward movement (mm) of the granular pile-anchor was significantly higher in the field scale Granular Pile-Anchor (GPA) than that in the case of laboratory scale Granular Pile-Anchor (GPA).

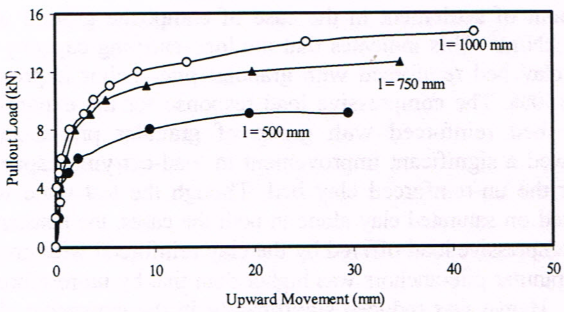

Fig. 6: Pullout Behavior of Granular Pile-Anchors of Diameter 200 mm

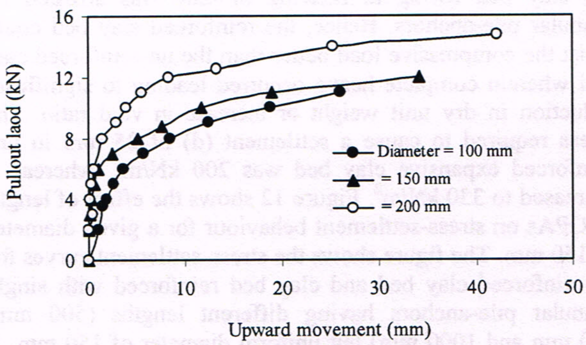

Fig. 7: Pullout Behavior of Granular Pile-Anchors of 1000 mm Length

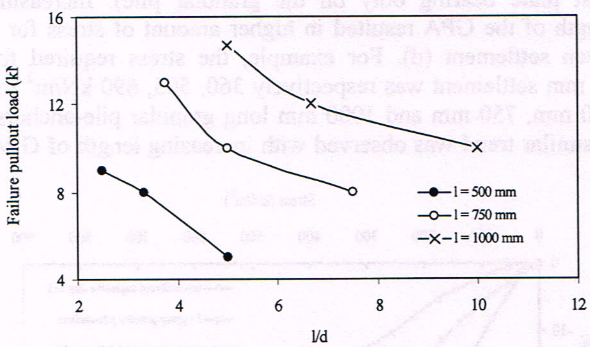

Fig. 8: Variation of Failure Pullout Load with l/d Ratio of Granular Pile-Anchor

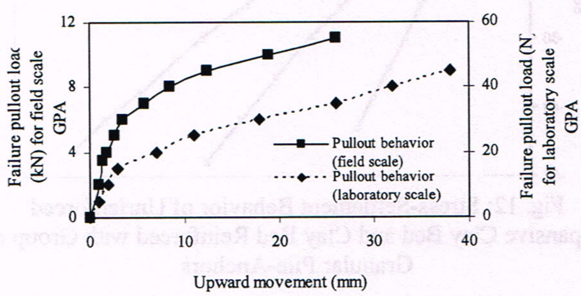

Fig. 9: Comparison of Pullout Behavior of Laboratory Scale and Field Scale Granular Pile-Anchors ($l/d = 10$; $D_r = 70\%$)

The large variation in the behavior of the Granular Pile-Anchors (GPAs) in the laboratory study and the field study was due to the scale effect (Rao *et al.* 2007).

Figure 10 shows, by comparison, the variation of failure pullout load with l/d ratio of granular pile-anchors in laboratory scale study and field scale study. The data pertain to three field scale Granular Pile-Anchors (GPA) of length 1000 mm (diameter = 100, 150 and 200 mm) and three laboratory scale Granular Pile-Anchors (GPA) of length 500 mm (diameter = 30, 40 and 50 mm). The expansive clay beds in both the cases were compacted at placement water content of 15% and dry unit weight of 14 kN/m^3. All the granular piles were compacted to a relative density of 70%. Irrespective of the l/d ratio the failure pullout load for field scale granular pile-anchors was much higher than that for laboratory scale granular pile-anchors. This was because of the increased surface area, frictional resistance and the weight of granular material in the case of field scale granular pile-anchor and the consequent increase in uplift resistance. Moreover, the lateral swelling pressure of expansive clay bed in situ, which confines the Granular Pile-Anchor (GPA) is also significantly high compared to that in the laboratory scale expansive clay bed (Rao *et al.* 2007).

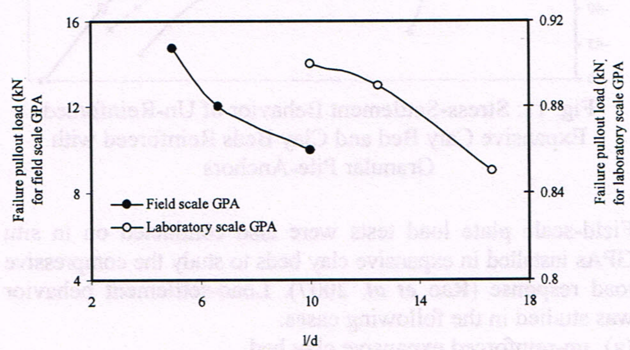

Fig. 10: Variation of Failure Pullout Load with l/d in Field and Laboratory Scale Granular Pile-Anchors

Figure 11 shows, by comparison, the pullout behavior of the granular pile-anchor when tested single and when tested under group effect. The granular pile-anchor under group effect resulted in increased uplift load for a given upward movement in comparison to that of a single granular pile-anchor tested. This is because of the influence of the granular pile-anchors in the group on the test granular pile-anchor. As heave of the expansive clay bed was reduced significantly on the installation of group of granular pile-anchors, which also act as tension members, there was not much reduction in the dry unit weight of the expansive clay bed. As a result the pile-soil interface friction for the group would be much more than in the case of the single granular pile-anchor. This also resulted in a higher lateral swelling pressure (confining the test granular pile-anchor radially) than in the case of a single granular pile-anchor. There is an arching action between the granular pile-anchors, which offers more resistance to uplift. The uplift load required to be applied on the granular pile-anchor to cause an upward movement of 25 mm was 13.7 kN when tested under group effect as against an uplift load of 11.25 kN for the same amount of upward movement of 25 mm when tested single. This indicated a percentage increase of 22.22% in the applied uplift load when the granular pile-anchor was tested under group effect. The failure pullout load of the granular pile-anchor when tested under group was 18 kN as against a failure pullout load of 12 kN for the granular pile-anchor when tested single, indicating an improvement of 50% in the failure pullout load (Rao *et al.* 2007).

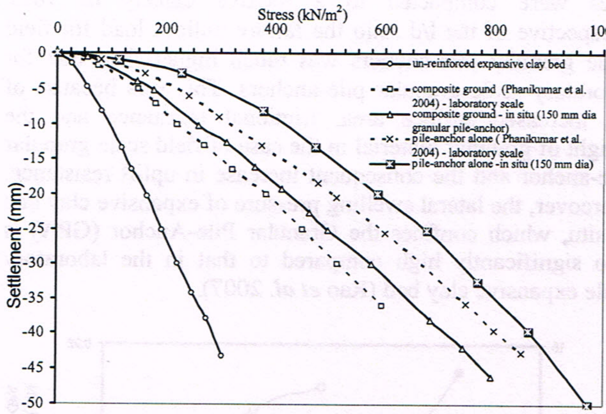

Fig. 11: Stress-Settlement Behavior of Un-Reinforced Expansive Caly Bed and Clay Beds Reinforced with Granular Pile-Anchors

Field-scale plate load tests were also conducted on in situ GPAs installed in expansive clay beds to study the compressive load response (Rao *et al.* 2007). Load-settlement behavior was studied in the following cases:

(a) un-reinforced expansive clay bed
(b) composite ground with the test plate spreading on both the granular pile-anchor and the expansive clay bed
(c) Granular Pile-Anchor (GPA) alone with the test plate bearing only on the granular pile-anchor, and
(d) expansive clay bed reinforced with a group of granular pile-anchors.

Figure 11 shows the stress-settlement curves for the un-reinforced expansive clay bed, the composite ground (granular pile-anchor and expansive clay bed together) and GPA 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. This indicates that the load-carrying capacity of the clay bed reinforced with granular pile-anchor improved by 150%. The compressive load response for the expansive clay bed reinforced with 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 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 significant reduction in dry unit weight or increase in void ratio. The stress required to cause a settlement (d) of 25 mm in un-reinforced expansive clay bed was 200 kN/m^2, whereas it increased to 330 kN/m^2. Figure 12 shows the effect of length of GPAs on stress-settlement behaviour 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 mm. It may be mentioned here that the data shown in Figure 12 pertain to load tests performed on granular pile-anchors alone (test plate bearing only on the granular pile). Increasing length of the GPA resulted in higher amount of stress for a given settlement (d). For example, the stress required for 25 mm settlement was respectively 360, 505, 690 kN/m^2 for 500 mm, 750 mm and 1000 mm long granular pile-anchors. A similar trend was observed with increasing length of GPA

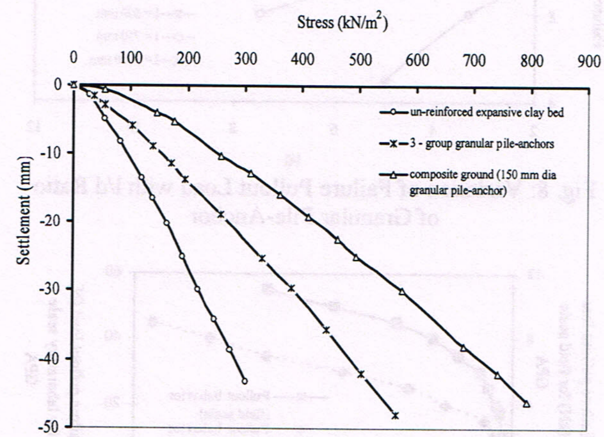

Fig. 12: Stress-Settlement Behavior of Unrienforced Expansive Clay Bed and Clay Bed Reinforced with Group of Granular Pile-Anchors

for other diameters also. Figure 12 shows the stress-settlement behavior of un-reinforced expansive clay bed and clay bed reinforced with group of GPAs.

4. CONCLUSION

Expansive soils, being problematic soils with their innate potential for severe volume changes upon changes in moisture content, need to be treated with special techniques. Sand cushion, CNS layer, chemical and physical alteration techniques have been successful. Granular Pile-Anchor (GPA) technique has been a recent technique developed with some innovation introduced into granular piles. GPA has met with considerable success as observed in both laboratory and field scale tests. Heave was reduced significantly and the surrounding soil was found improved. Results of pullout tests and compressive load tests indicated that length, diameter and relative density of GPA govern heave and engineering behaviour of GPA system.

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