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EFFECT OF PRIMARY AND SECONDARY SETTLEMENTS ON RESPONSE OF GRANULAR PILE REINFORCED GROUND

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ABSTRACT

Creep or secondary compression is a very important phenomenon in clays, organic soils and peat. The paper presents an analysis of granular pile reinforced ground with in situ soil undergoing creep and assuming the swelling or rebound on unloading to be negligible, i.e. Cs = 0. The variations of settlement reduction factor, settlement of the reinforced ground, the load sharing between the GP and the in situ soil, the stress concentration ratio, granular pile – in situ soil interface shear stress, void ratio versus log effective stress, etc., with time, area ratio, and stiffness factor are estimated and presented. Results indicate the in situ soft ground gets unloaded due to creep, another beneficial aspect of treating soft soils with granular piles.

***Keywords**: creep, granular pile, clays, peat, unloading, area ratio, relative stiffness.*

I. INTRODUCTION

Most of the approaches for estimating settlement of the composite ground assume an infinitely large loaded area reinforced with granular piles having constant diameter and spacing. For uniform loading condition and geometry, the unit cell idealization is valid. The unit cell loaded through a rigid plate behaves analogous to one dimension compression as it is laterally confined and the vertical strains on any horizontal plane are uniform. Several methods for estimating the settlement of composite ground are available (Priebe, 1976, Baumann and Bauer, 1974, Hughes et al., 1975, Aboshi et al., 1979, Van Impe and De Beer, 1983, Goughnour, 1983, Alamgir et al., 1994, Shahu et al., 2000, etc.) based on the "equal strain" theory.

Most of the above works restrict the analysis for the response of the soft ground reinforced with granular piles at the end of primary consolidation. Creep or secondary compression is a very important phenomenon in soft clays, organic soils and peat deposits. From consolidation studies on Leda clay, Walker and Raymond (1968) proposed that the laboratory value of $C_{\square\square}$increases linearly with C_c. Mesri (1973) and Mesri et al. (1975) showed linear relationship between $C_{\square}$ and C_c for Chigago blue clay and Mexico city clay, respectively. The concept of constant $C_{\square}$ /C_c ratio was proposed to characterize the compression bahavior of a wide range of natural soils (Mesri and Godlewski 1977). The $C_{\square}/C_c$ concept is based on the assumption that, for any natural soil, the ratio of $C_{\square}$ to C_c is constant for any time, effective stress and void ratio during secondary compression Mesri and Godlewski (1977). The concept has also been used as part of a preloading design method for soft ground Mesri and Choi (1985) and to predict the behavior of the at-rest lateral earth-pressure coefficient (K_0) during secondary compression Mesri and Castro (1987). This technical note present laboratory evidence with regard to the applicability of the $C_{\square}/C_c$ concept to peat compression. Secondary compression behavior of middleton peat with and without surcharging was investigated by Mesri et al. (1997). The range of values of $C_{\square}/C_c$ for different soils are given in Table 1 (Mesri and Vardhanabhuti 2005).

II. FORMULATION AND SOLUTION

The layout and section of soft ground reinforced with granular piles at a spacing, S, are shown in Fig. 1. A unit cell of diameter, d_e, (=1.05S and 1.13S for triangular and square arrangements respectively) and thickness, H, consisting of a granular pile of diameter, d, and with a modulus of deformation, E_{gp}, is shown in Fig 2. Groundwater level is assumed near the top of the soft stratum. The void ratio versus log effective stress relation for the soft in situ normally consolidated soil is shown in Fig 3. The initial void ratio of the soft soil is e_0. Lines OA and AB represent respectively the loading and unloading responses of the soil. The compression and swelling indices of the soft soil are C_c and C_s respectively while the coefficient of creep or secondary compression is C_α. For the present work, the swelling or rebound on unloading (curve AB Fig.3) is considered to be negligible, i.e. C_s=0. The granular pile material is characterized by its deformation modulus, E_{gp}. Granular mat or pad of thickness, H_m, and unit weight γ_m, laid above the soil layer is assumed as incompressible. A uniform load of intensity, q_0, is applied through the rigid granular mat.

2.1 Analysis For Primary Consolidation

The modulus of deformation, E_{gp}, of the granular pile and the compression index, C_c, of the soft soil are assumed constant with depth. Consequently, the constrained modulus, D_s, of the soft soil (Lambe and Whitman 1969), increases with effective stress, σ', and thus with depth as

$$D_s = \frac{(1 + e_0)\sigma'}{0.434\ C_c} \tag{1}$$

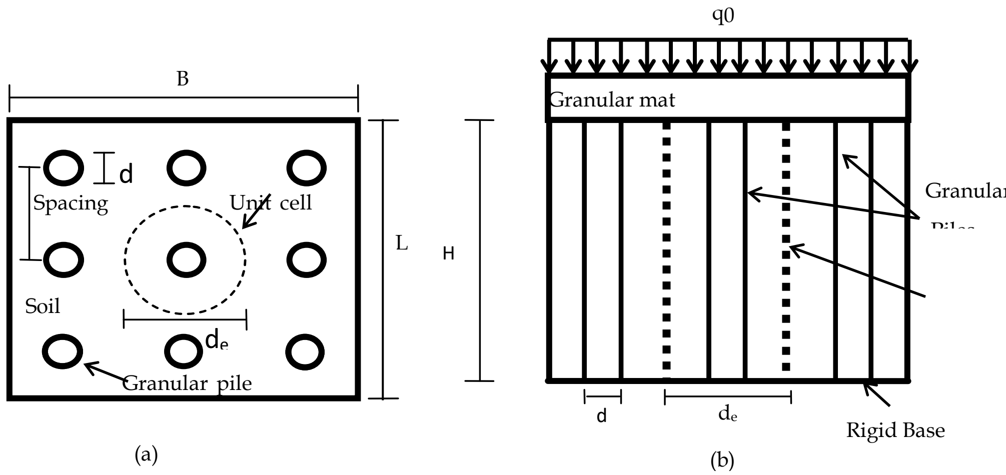
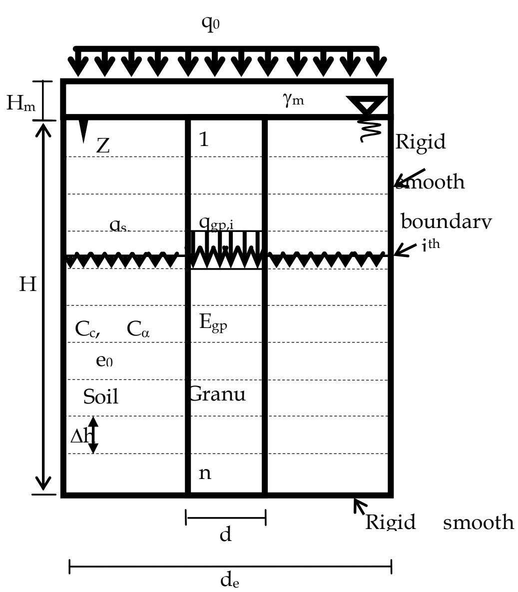


Figure 1 a) Layout and b) Section of Soft Ground Reinforced with Granular Piles



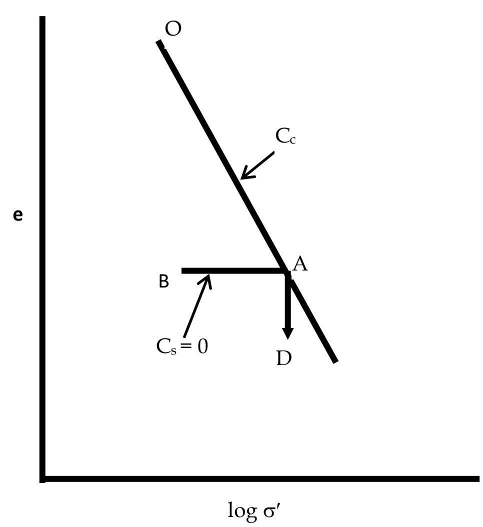


Figure 2 Unit Cell

Figure 3 Void Ratio vs Effective Stress-OA - Loading; AD - Creep; - AB (Cs=0) - Unloading

As a result, the sharing of load between GP and the soft soil becomes a function of depth, with deeper layers of soft clay able to carry larger stresses than those at shallower depths. The unit cell (Fig. 2) is discretized into 'n' number of elements each of thickness, $\Delta h=H/n$, to analyse the distribution of stresses in the GP and the soft soil at various depths. The average stresses in the granular pile and the soft soil at the end-of-primary consolidation at mid-height of the i^{th} layer are $q_{gp,eop,i}$ and $q_{s,eop,i}$ respectively. Equilibrium of vertical forces at any depth, z, inside the unit cell is expressed as

$$q_0 = q_{gp,eop,i} A_r + q_{s,eop,i} (1 - A_r) \tag{2}$$

where the area ratio, $A_r=(d/d_e)^2$

2.1.1 Stress on Soil and Granular Pile (GP) during EOP

The compression of the i^{th} element of the granular pile $\Delta S_{gp,eop,i}$, is

$$\Delta S_{gp,eop,i} = \frac{q_{gp,eop,i}}{E_{gp}} \Delta h \tag{3}$$

where E_{gp}= modulus of deformation of the granular pile; and $\Delta h=H/n$ thickness of the element.

The compression at the end of primary consolidation of i^{th} element of the normally consolidated soil surrounding the granular pile in the unit cell, $\Delta S_{s,eop,i}$ is

$$\Delta S_{s,eop,i} = \frac{C_c}{(1+e_o)} \Delta h \log\left(1 + \frac{q_{s,eop,i}}{\sigma'_{0i}}\right) \tag{4}$$

where $\sigma'_{0i}= \gamma_m H_m+\gamma_{sub} z_i$, effective overburden stress at the middle of the i^{th} element in the soil, γ_m and H_m are the unit weight and thickness of the granular mat laid on top of the soft in situ soil respectively, γ_{sub} is the submerged unit weight of the soil and z is the depth of the center layer of the i^{th} element.

The compatibility of displacements for the i^{th} element at the end-of-primary (EOP) consolidation, is

$$\Delta S_{gp,eop,i} = \Delta S_{s,eop,i} \tag{5}$$

Substituting Eqs. (3) and (4) in Eq. (5), one gets

$$q_{gp,eop,i} = \frac{C_c}{(1+e_o)} E_{gp} \log\left(1 + \frac{q_{s,eop,i}}{\sigma'_{0i}}\right) \tag{6}$$

For convenience, all the above stress parameters are normalized with σ'_{av} where $\sigma'_{av} = \gamma_{sub} H/2$.

Using the above normalized parameters, Eqs. (2) and (6) may be rewritten as

$$q^*_0 = q^*_{gp,eop,i} A_r + q^*_{s,eop,i} (1 - A_r) \tag{7}$$

and
$$q^*_{gp,eop,i} = \mu \left[\log\left(1 + \frac{q^*_{s,eop,i}}{\sigma^*_{0i}}\right)\right] \tag{8}$$

where $\mu = \frac{C_c}{(1+e_0)} \frac{E_{gp}}{\sigma'_{av}}$ - a relative stiffness parameter.

Eqs. (7) and (8) are solved iteratively to obtain $q^*_{s,eop,i}$ and $q^*_{gp,eop,i}$ for all layers for the applied stress q^*_0. The results obtained are once again normalized with applied stress, q^*_0, in the form $q'_{s,eop,i}$ and $q'_{gp,eop,i}$, as

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where $$q'_{s,eop,i} = \frac{q^{*}_{s,eop,i}}{q^{*}_{0}} = \frac{q_{s,eop,i}}{q_{0}} \tag{9}$$

$$q'_{gp,eop,i} = \frac{q^{*}_{gp,eop,i}}{q^{*}_{0}} = \frac{q_{gp,eop,i}}{q_{0}} \tag{10}$$

The above procedure is repeated and $q'_{s,eop,i}$ and $q'_{gp,eop,i}$ obtained for all the elements.

2.2 Analysis For Creep Or Secondary Consolidation

The conventional approaches for the analyses of granular pile reinforced ground have been carried out considering only the primary consolidation settlements. However, in situ soft soils especially highly plastic clays and peat experience significant creep or secondary settlements over long periods of time. The succeeding analysis considers the response of the granular pile reinforced ground undergoing creep. Creep settlements (Line AD in Fig. 3) are estimated from

$$S_{cr} = \frac{C_{\alpha}}{\left(1+e_{p}\right)} H \log\left(\frac{t}{t_{0}}\right) \tag{11}$$

where C□ is the secondary compression index, e_p is the void ratio at the end-of-primary (EOP) for i^{th} layer, H is the thickness of the in situ soil, ‘t’ is time and t_0 is the time for the EOP consolidation, usually taken as 0.1 year. Mesri et al. (2005) express C□ in terms of the ratio C□□C_c (Table 1). The creep rates for granular soils are much less compared to those for plastic clays and peat and hence the creep of granular pile material is assumed to be zero in this paper.

2.2.1 Stress Transfer during Creep

The soft in situ soil tends to creep under the stresses transferred to it at the end of primary consolidation. As the GP material does not creep, part of the stress acting on each soft clay layer gets transferred to GP. The increase in stress on the GP due to creep is □$q_{gp,cr,i}$ while □$q_{s,cr,i}$ is the decrement or reduction of stress on the i^{th} layer of soft in situ soil undergoing creep.

Equilibrium of vertical forces at any depth inside the unit cell during creep is expressed as

$$\Delta q_{gp,cr,i} A_r - \Delta q_{s,cr,i} (1 - A_r) = 0 \tag{12}$$

2.2.2 Stress on Soil and Granular Pile (GP) during Creep

The net unloaded stress, □$q_{s,i}$, on any element, i, of the in situ soil is the sum of the unloaded stresses up to that level, i.e.

$$\Delta q_{s,i} = \sum_{j=1}^{i} \Delta q_{s,cr,j} \tag{13}$$

Net compression, give the term of the i^{th} element of a normally consolidated fine-grained soil surrounding the granular pile in the unit cell is the difference in settlement due to creep and rebound due to unloading as

$$\Delta S_{s,cr,i} = \left[\frac{C_{\alpha}}{(1+e_{p})} \Delta h \ \log\left(\frac{t}{t_{0}}\right) - \frac{C_{s}}{(1+e_{p})} \Delta h \ \log\left(\frac{q^{*}_{s,eop,i}}{q^{*}_{s,eop,i} - \Delta q_{s,i}} \right) \right] \tag{14}$$

For $C_s = 0$ Eq. (26) reduces to

$$\Delta S_{s,cr,i} = \frac{C_{\alpha}}{(1+e_{p})} \Delta h \ \left[\log\left(\frac{t}{t_{0}}\right) \right]$$

or $$\Delta S_{s,cr,i} = \frac{C_c}{(1+e_p)}\Delta h\left[\frac{C_\alpha}{C_c}\log\left(\frac{t}{t_0}\right)\right] \tag{15}$$

where $\Delta S_{s,cr,i}$ = compression of the ith element of the soil due to creep; $C_\alpha = \Delta e/\Delta\log(t)$ - the secondary compression index, e_p -the void ratio at EOP.

Similarly, the net increase of stress, $\Delta q_{gp,i}$, on GP due to transfer of stresses from all the elements up to element i, is

$$\Delta q_{gp,i} = \sum_{j=1}^{i}\Delta q_{q,cr,j} \tag{16}$$

Compression, $\Delta S_{gp,cr,i}$, of the ith element of the granular pile due to the net increase in the stress on the GP is

$$\Delta S_{gp,cr,i} = \frac{\Delta q_{gp,i}}{E_{gp}}\Delta h \tag{17}$$

Compatibility displacements for the any element during creep, is

$$\Delta S_{gp,cr,i} = \Delta S_{s,cr,i} \tag{18}$$

Substituting Eqs. (17) and (19) in Eq. (18), $\Delta q_{gp,cr,i}$ is obtained as

$$\Delta q_{gp,j} = \frac{C_c}{(1+e_p)}E_{gp}\left[\frac{C_\alpha}{C_c}\log\left(\frac{t}{t_0}\right)\right] \tag{19}$$

Normalizing the above stress parameters with σ'_{av}, one gets

$$\Delta q^*_{s,i} = \frac{\Delta q_{s,i}}{\sigma'_{av}} \tag{20}$$

$$\Delta q^*_{gp,i} = \frac{\Delta q_{gp,i}}{\sigma'_{av}} \tag{21}$$

Using the above normalized parameters, Eqs. (24) and (31) may be rewritten as

$$\Delta q^*_{gp,i}A_r - \Delta q^*_{s,i}(1-A_r) = 0 \tag{22}$$

$$\Delta q^*_{gp,i} = \mu\left[\frac{C_\alpha}{C_c}\log\left(\frac{t}{t_0}\right)\right] \tag{23}$$

$$\Delta q^*_{s,i} = \frac{\Delta q^*_{gp,i}\,A_r}{(1-A_r)} \tag{24}$$

The values of $\Delta q^*_{s,i}$ and $\Delta q^*_{gp,i}$ are obtained from Eqs. (35) and (36) for all the layer of the soil and the granular pile. The results obtained are expressed once again in terms of $\Delta q'_{s,i}$ and $\Delta q'_{gp,i}$ as

where $$\Delta q'_{s,i} = \frac{\Delta q^*_{s,i}}{q^*_0} = \frac{\Delta q_{s,i}}{q_0} \tag{25}$$

$$\Delta q'_{gp,i} = \frac{\Delta q^*_{gp,i}}{q^*_0} = \frac{\Delta q_{gp,i}}{q_0} \tag{26}$$

The final stresses on the ith layers of soil, $q_{s,cr,i}$, and granular pile, $q_{gp,cr,i}$, at any time during the creep are respectively

$$q'_{s,cr,i} = q'_{s,eop,i} - \Delta q'_{s,i} = q_{s,eop,i} - \Delta q_{s,i} \tag{27}$$

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$$q'_{gp.cr.i} = q'_{gp,eop,i} + \Delta q_{gp,i} = q'_{gp,eop,i} + \Delta q_{gp,i} \tag{28}$$

2.3. Settlement (S) During Creep

Reinforced Ground (S_t)

Since heave due to unloading is zero (C_s=0), the normalized settlement, $\Delta S_{t,i}/H$, of the i[th] element of the soft soil around GP during creep is

$$\frac{\Delta S_{t,i}}{H} = \frac{\Delta h}{H}\left[\frac{C_c}{(1+e_0)}\log\left(1+\frac{q_{s,eop,i}}{\sigma'_{oi}}\right) + \frac{c_\alpha}{(1+e_p)}\log\left(\frac{t}{t_0}\right) - \frac{C_s}{(1+e_p)}\log\left(\frac{q_{s,eop,i}}{q_{s,eop,i} - \Delta q_{s,i}}\right)\right] \tag{29}$$

As was indicated in Eq. (39), the stresses acting on each of the elements of soft soil get transferred to the GP during creep. At a particular time, t_1, element 1 gets completely unloaded with respect of the increment in stress, $q_{s,eop,1}$, corresponding to the end of primary. Once stress on the soil within element 1 is completely unloaded, no further creep settlement can occur. Similarly, elements 2 to n will not undergo creep beyond times, t_2, t_3, and so on. Thus creep settlements for t >t_i, correspond to those for t=t_i. Thus for t>t_i, t=t_i is substituted for all the elements in Eq. (29).

The normalized total settlement for ground reinforced with GP, S_t/H, is obtained by summing the settlements of all the elements as

$$\frac{S_t}{H} = \sum_{i=1}^{n} \frac{\Delta S_{t,i}}{H} \tag{30}$$

2.3.1 Unreinforced Ground (Sunt)

The applied stress, q_0, gets directly transferred to the soft un-reinforced ground. The normalized settlement of i[th] element for un-reinforced ground at any time, t, during creep is obtained as

$$\frac{\Delta S_{unt,i}}{H} = \frac{C_c}{(1+e_0)}\frac{\Delta h}{H}\log\left(1+\frac{q_0}{\sigma'_{oi}}\right) + \frac{C_\alpha}{1+e_p}\frac{\Delta h}{H}\log\left(\frac{t}{t_0}\right) \tag{31}$$

The normalized total settlement for un-reinforced ground, S_{unt}, is obtained by adding the settlements of all the elements as

$$\frac{S_{unt}}{H} = \sum_{i=1}^{n} \frac{\Delta S_{unt,i}}{H} \tag{32}$$

2.3.2 Settlement Reduction Factor during Creep

The settlement reduction factor, β, is defined as the ratio of settlements of treated and untreated ground as

$$\beta = \frac{S_t}{S_{unt}} \tag{33}$$

TABLE 1 Values of C_α/C_c for Geotechnical Materials (after Mesri and Vardhanabhuti 2005)

1	Granular soils including rock fill	0.02 ± 0.01
2	Shale and mudstone	0.03 ± 0.01

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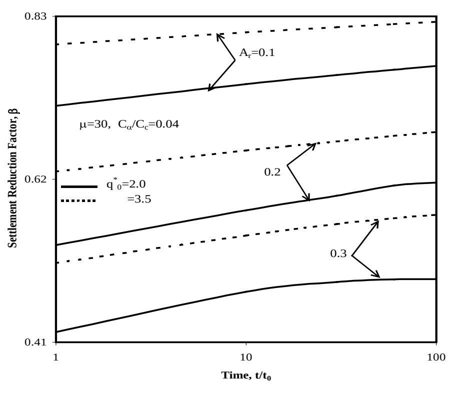
3	Inorganic clay & Silt	0.04 ± 0.01
4	Organic clay & Silt	0.05 ± 0.01
5	Fibrous & Amorphous Peat	0.06 ± 0.01

III. RESULTS AND DISCUSSION

The effect of secondary compression/creep on the response of granular pile reinforced ground is evaluated for representative values of different input parameters of reinforced ground. Effect of number, n, of elements into which the unit cell is discretized on reinforced ground response is studied by varying n from 10 to 50. The results converge for $n \geq 20$.

The response of the GP reinforced ground in terms of the variations of the normalized stresses on the soil and the granular pile, the shear stress at the soil-granular pile interface, the stress concentration factor, the settlement reduction factor, □□□and the e - log □' responses for treated and untreated ground, are discussed for the following ranges of parameters:
Area ratio, A_r: 0.1, 0.2, 0.3 and 0.4
Stiffness factor, □□□10, 30 and 60
Creep ratio, $C_{□}/C_c$: 0, 0.02, 0.04 and 0.06 and
Normalized applied stress, q^*_0: 2, 2.5, 3.0 and 3.5.

3.1 Effect of the Area Ratio (Ar)



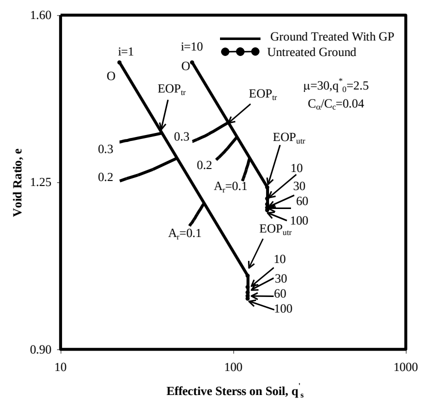


Figure 13 Settlement Reduction Factor, □ vs Time,

Figs. 3.1 and 3.2 present the effect area ratio, (A_r□=0.1, 0.2 and 0.3) for normalized applied stress, q^*_0, of 2.0, 2.5, and 3.5, stiffness factor, □=30, creep ratio, $C_{□}/C_c$, of 0.04 at t/t_0=1 (EOP) and times, t/t_0=10, 30, 60 and100.

The variations of settlement reduction factor, □, with time, t/t_0, for different area ratios, A_r and for different normalized applied stresses, q^*_0, (Fig. 3.1) are similar to the variation of □ with relative stiffness factor, □ (Fig. 5), the rate of increase of settlement reduction factor□□□□ with t/t_0 increasing with area ratio, A_r. However, the rate of increase of settlement reduction factor,□□□ with t/t_0 decreases with t/t_0 at higher values of time for larger area ratio, e.g. A_r=0.3, as the soft in situ soil gets completely unloaded after a certain time and does not creep further. The time corresponding to attainment of constant settlement reduction factor corresponds to the time for unloading, t_{ul}/t_0. The settlement reduction factor, □, increases with normalized applied stress, q^*_0, at EOP and the same trend manifests with creep. Hence the settlement reduction factor, □, increases with increasing values of q^*_0, the increase being of

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the order of 0.714 and 0.794 for EOP, 0.766 and 0.823 for at a time of t/t_0=100 for normalized applied stress of q^*0=2 and 3.5 respectively for an area ratio A_r=0.1.

The void ratio, e_0, versus log effective stress, log q_s, plots for untreated ground and for the first, i=1 and the tenth, i=10, layers of ground treated with granular piles for the three area ratios, A_r, are presented in Fig. 3.2. The response of the unreinforced ground is explained in Fig.10.

The response of the reinforced ground in terms of void ratio –log effective stress plot with stiffness factor, □ =30, creep ratio $C_{□}/C_c$=0.04 normalized applied stress, q^*_0=2.5 is explained in Fig.15. The final void ratios for reinforced ground at the end-of-primary (EOP) consolidation are 1.21, 1.30 and 1.35 for corresponding effective stresses of 67.87, 47.3 and 38.7 kPa at normalized depth of z/H=0.05 (i=1) and 1.30, 1.34 and 1.37 with corresponding effective stresses of 124.8, 105.3 and 93.71 for mid depth (i=10) respectively for area ratios of A_r=0.1, 0.2 and 0.3. The stresses on the soil due to creep decrease from 47.3 at EOP to 34.25, 28, 24.1 and 22 for i=1, and from 105.3 at EOP to 92.5, 86.4, 82.5 and 79.71 for i=10 at times, t/t_0 = 10, 30, 60 and 100. The corresponding void ratios are 1.30, 1.277, 1.265, 1.258 and 1.253 for i=1, and 1.34, 1.32, 1.31, 1.3 and 1.29 for i=10 respectively for an area ratio, A_r=0.2. Thus these plots depict that the in situ soft soil gets unloaded during creep.

IV. CONCLUSIONS

A simple approach is presented for the analysis of creep of soft ground reinforced by granular piles loaded uniformly through a granular mat. The swell index of the soil is assumed as zero and the granular mat as rigid. Results in terms of variations of stresses transferred to the soft in situ soil and the granular pile, the stress concentration ratio, SCR, void ratio-log effective stress, time for unloading, all with depth, are presented for different values of relative stiffness factors, □, area ratios, A_r, time, t/t_0, and creep ratios, $C_{□}/C_c$.

The in situ soft soil gets unloaded during creep with respect to stresses transferred on to it at the EOP, due to compatibility of displacements with respect to those of granular pile. The settlement reduction factor and the stress concentration ratio, q_{gp}/q_s, increase with time for all area ratios and relative stiffness factors, the rate of increase increasing with increasing values of both these factors.. The soil develops pseudo-preconsolidation stress as a consequence of transfer of stress to the granular pile. This phenomenon is akin but contrastingly different from the development of pseudo-preconsolidation effect due to long term creep or secondary compression identified by Bjerrum (1972).

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