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Analysis of Effect of Creep on Response of Granular Pile Reinforced Ground

Madhav M. R. *, Suresh K. and Nirmala Peter E. C.*****

Introduction

Due to growing trends of extensive urbanization and industrialization, the demands for the improvement of marginal sites are increasing continuously. The basic concepts of ground improvement, namely drainage, densification, cementation, reinforcement, drying and heating, have been extensively developed in recent years. Among the various techniques available for improving in situ soft ground conditions, stone column/ granular pile (GP) treatment is considered as one of the most versatile and cost-effective ground improvement technique compared to other methods. They are ideally suited for the improvement of soft clays, silts, organic soil and loose granular deposits. Granular piles improve the performance of foundation on soft ground both by reducing the settlement to an acceptable level and by increasing the load carrying capacity. In addition, they increase the time rate of consolidation; improve the stability and resistance to liquefaction.

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_α increases linearly with C_c. Mesri (1973) and Mesri et al. (1975) showed linear relationship between C_α and C_c for

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* Prof. Emeritus, JNTUH College of Engg., Kukatpally, Hyderabad –500 085, A.P., India. e-mail: madhavmr@gmail.com

** Research Scholar, JNTUH College of Engg., Kukatpally, Hyderabad –500 085, A.P., India. e-mail: peterkandru@gmail.com

*** Assoc. Prof. in Civil Engg., JNTUH, Kukatpally, Hyderabad –500 085, A.P., India. e-mail: ecnps@yahoo.co.uk

Chicago blue clay and Mexico city clay, respectively. The concept of constant C_α/C_c ratio was proposed to characterize the compression behaviour of a wide range of natural soils (Mesri and Godlewski, 1977). The C_α/C_c concept is based on the assumption that, for any natural soil, the ratio of C_α 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_α/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_α/C_c for different soils are given in Table 1 (Mesri and Vardhanabhuti, 2005).

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

Formulation and Solution

The layout and section of soft ground reinforced with granular piles at a spacing, S, are shown in Figure 1.

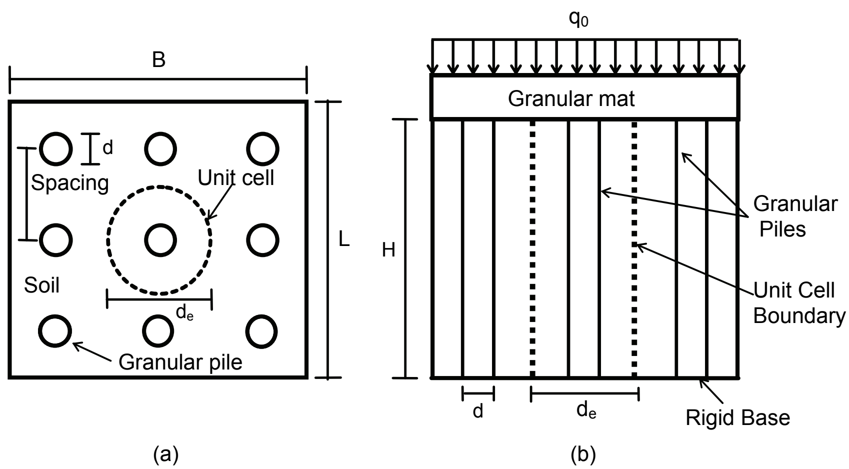

Fig. 1 a) Layout and b) Section of Soft Ground Reinforced with Granular Piles

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 Figure 2.

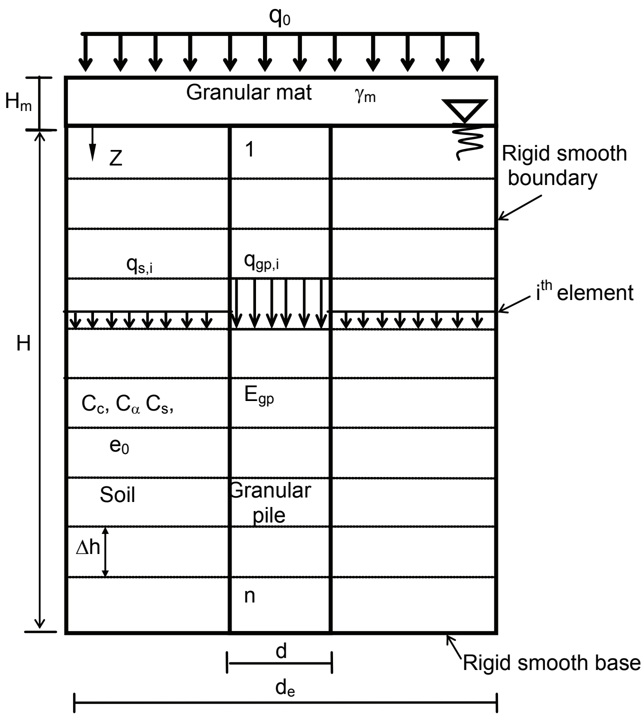

Fig. 2 Unit Cell

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 Figure 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 Figure 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.

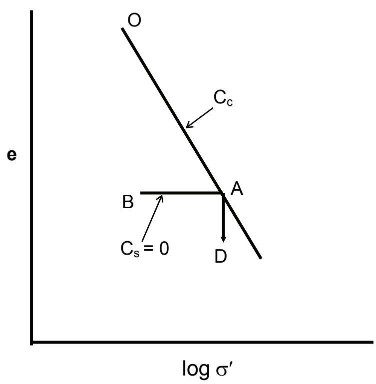

Fig. 3 Void Ratio vs Effective Stress OA - Loading; AD – Creep and AB (C_s=0) - Unloading

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

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

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 (Figure 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, $A_r=(d/d_e)^2$

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

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

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 $\sigma'_{av} = \gamma_{sub} H/2$.

$$q_0^* = \frac{q_0}{\sigma'_{av}} \tag{7}$$

$$q_{s,eop,i}^* = \frac{q_{s,eop,i}}{\sigma'_{av}} \tag{8}$$

$$q_{gp,eop,i}^* = \frac{q_{gp,eop,i}}{\sigma'_{av}} \tag{9}$$

$$\sigma_{oi}^* = \frac{\sigma'_{oi}}{\sigma'_{av}} \tag{10}$$

$$E_{gp}^* = \frac{E_{gp}}{\sigma'_{av}} \tag{11}$$

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

$$q_0^* = q_{gp,eop,i}^* A_r + q_{s,eop,i}^*(1-A_r) \tag{12}$$

and

$$q_{gp,eop,i}^* = \mu\left[\log\left(1+\frac{q_{s,eop,i}^*}{\sigma_{0i}^*}\right)\right] \tag{13}$$

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

Eqs. (12) and (13) 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

where

$$q'_{s,eop,i} = \frac{q^*_{s,eop,i}}{q^*_0} = \frac{q_{s,eop,i}}{q_0} \tag{14}$$

$$q'_{gp,eop,i} = \frac{q^*_{gp,eop,i}}{q^*_0} = \frac{q_{gp,eop,i}}{q_0} \tag{15}$$

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

Shear Stress (τ) during EOP

The shear stress mobilized at the soil-granular pile interface at any element of the soil layer is determined considering the equilibrium forces acting on the i^{th} element (Figure 4 a) of the GP-soil interface.

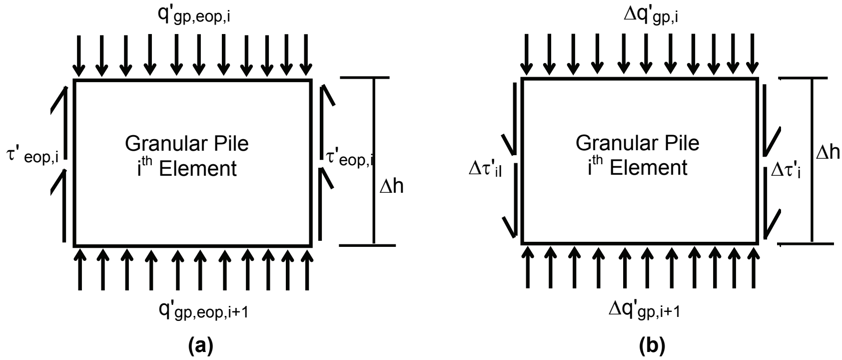

Fig. 4 Stresses on a Typical Granular Pile Element a) at end of Primary Consolidation and b) During Creep

The shear stress at the end of primary consolidation, $\tau'_{eop,}$ is

$$\tau'_{eop} = \frac{d}{4(\Delta h)}\left\{q'_{gp,eop,i} - q'_{gp,eop,i+1}\right\} \tag{16}$$

Eq. (16) in normalized form is

$$\tau'_{eop} = \frac{(d/H)}{4\left(\Delta h/H\right)}\left\{q'_{gp,\,eop,\,i} - q'_{gp,\,eop,\,i+1}\right\} \tag{17}$$

Eq. (17) reduces to

$$\tau'_{eop} = \frac{n}{4\,(D_r)}\left\{q'_{gp,\,eop,i} - q'_{gp,eop,i+1}\right\} \qquad 18)$$

where D_r = (H/d) = the depth ratio. Shear stress for n^{th} element is evaluated based on the assumption that the difference in shear stress between element (n-2) and element (n-1) and that between element (n-1) and n remain the same.

$$\tau'_n = \tau'_{n-1} - \left(\tau'_{n-2} - \tau'_{n-1}\right) \tag{19}$$

The error involved due to the above assumption becomes insignificant for large ‘n’.

Settlements (S) during EOP

Knowing the stresses $q'_{s,eop,i}$ and $q'_{gp,eop,i}$, the settlement of i^{th} element, $\Delta S_{s,eop,i}$ may be obtained from Eq.(4) as

$$\frac{\Delta S_{s,eop,i}}{H} = \frac{C_c}{(1+e_0)}\frac{\Delta h}{H}\log\left(1+\frac{q_{s,eop,i}}{\sigma'_{oi}}\right) \tag{20}$$

The normalized total settlement at the end-of-primary (EOP) consolidation, $S_{s,eop}/H$ of the reinforced ground is obtained by summing the settlements of all the elements as

$$\frac{S_{s,eop}}{H} = \sum_{i=1}^{n}\frac{\Delta S_{s,eop,i}}{H} \tag{21}$$

Stress Concentration Factor (SCF) during EOP

The stress concentration factor (SCF) for any element ‘i’ at the end-of-primary consolidation is evaluated as

$$(SCF)_{eop,i} = \frac{q_{gp,eop,i}}{q_{s,eop,i}} \tag{22}$$

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 Figure 3) are estimated from

$$S_{cr} = \frac{C_\alpha}{\left(1+e_p\right)}H\log\left(\frac{t}{t_0}\right) \tag{23}$$

where C_α 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_α in terms of the ratio C_α/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.

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 $\Delta q_{gp,cr,i}$ while $\Delta q_{s,cr,i}$ 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

$$\Delta q_{gp,cr,i} A_r - \Delta q_{s,cr,i}(1 - A_r) = 0 \tag{24}$$

Stress on Soil and Granular Pile (GP) during Creep

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

$$\Delta q_{s,i} = \sum_{j=1}^{i} \Delta q_{s,cr,j} \tag{25}$$

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

$$\Delta S_{s,cr,i} = \left[\frac{C_\alpha}{(1+e_p)} \Delta h \ \log\left(\frac{t}{t_0}\right) - \frac{C_s}{(1+e_p)} \Delta h \ \log\left(\frac{q^*_{s,eop,i}}{q^*_{s,eop,i} - \Delta q_{s,i}}\right) \right] \tag{26}$$

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

$$\Delta S_{s,cr,i} = \frac{C_c}{(1+e_p)} \Delta h \ \left[\frac{C_\alpha}{C_c} \log\left(\frac{t}{t_0}\right) \right] \tag{27}$$

where $\Delta S_{s,cr,i}$ = compression of the i^{th} 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

$$\Delta q_{gp,i} = \sum_{j=1}^{i} \Delta q_{q,cr,j} \tag{28}$$

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

$$\Delta S_{gp,cr,i} = \frac{\Delta q_{gp,i}}{E_{gp}} \Delta h \tag{29}$$

Compatibility displacements for the any element during creep, is

$$\Delta S_{gp,cr,i} = \Delta S_{s,cr,i} \tag{30}$$

Substituting Eqs. (27) and (29) in Eq. (30), $\Delta q_{gp,cr,i}$ is obtained as

$$\Delta q_{gp,j} = \frac{C_c}{(1+e_p)} E_{gp} \left[\frac{C_\alpha}{C_c} \log\left(\frac{t}{t_0} \right) \right] \tag{31}$$

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

$$\Delta q^*_{s,i} = \frac{\Delta q_{s,i}}{\sigma'_{av}} \tag{32}$$

$$\Delta q^*_{gp,i} = \frac{\Delta q_{gp,i}}{\sigma'_{av}} \tag{33}$$

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

$$\Delta q^*_{gp,i} A_r - \Delta q^*_{s,i}(1 - A_r) = 0 \tag{34}$$

$$\Delta q^*_{gp,i} = \mu \left[\frac{C_\alpha}{C_c} \log\left(\frac{t}{t_0} \right) \right] \tag{35}$$

$$\Delta q^*_{s,i} = \frac{\Delta q^*_{gp,i} \quad A_r}{(1 - A_r)} \tag{36}$$

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

$$\Delta q'_{s,i} = \frac{\Delta q^*_{s,i}}{q^*_0} = \frac{\Delta q_{s,i}}{q_0} \tag{37}$$

$$\Delta q'_{gp,i} = \frac{\Delta q^*_{gp,i}}{q^*_0} = \frac{\Delta q_{gp,i}}{q_0} \tag{38}$$

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

$$q'_{s,cr,i} = q'_{s,eop,i} - \Delta q'_{s,i} = q_{s,eop,i} - \Delta q_{s,i} \tag{39}$$

$$q'_{gp.cr.i} = q'_{gp,eop,i} + \Delta q'_{gp,i} = q_{gp,eop,i} + \Delta q_{gp,i} \tag{40}$$

Shear Stress (τ_{cr}) during Creep

The normalized change in shear stress, $\Delta\tau'$, mobilized at the soil-granular pile interface shown in Figure (4b) for any element of the soil layer during creep is

$$\Delta\tau' = \frac{d}{4(\Delta h)}\left\{\Delta q'_{gp,i+1} - \Delta q'_{gp,i}\right\} \tag{41}$$

on simplification Eq. (41) reduces to

$$\Delta\tau' = \frac{n}{4(D_r)}\left\{\Delta q'_{gp,i+1} - \Delta q'_{gp,i}\right\} \tag{42}$$

The normalized total shear stress, τ'_{cr}, mobilized at the soil-granular pile interface during creep is

$$\tau'_{cr} = \left(\tau'_{eop,i} - \Delta\tau'_{,i}\right) \tag{43}$$

Shear stress for n^{th} element is evaluated as before (Eq.19).

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

$$\frac{\Delta S_{t,i}}{H} = \frac{\Delta h}{H}\left[\frac{C_c}{(1+e_0)}\log\left(1+\frac{q_{s,eop,i}}{\sigma'_{oi}}\right)+\frac{c_\alpha}{(1+e_p)}\log\left(\frac{t}{t_0}\right)-\frac{C_s}{(1+e_p)}\log\left(\frac{q_{s,eop,i}}{q_{s,eop,i}-\Delta q_{s,i}}\right)\right] \tag{43}$$

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. (43).

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

$$\frac{S_t}{H} = \sum_{i=1}^{n} \frac{\Delta S_{t,i}}{H} \tag{45}$$

Unreinforced Ground (S_{unt})

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

$$\frac{\Delta S_{unt,i}}{H} = \frac{C_c}{(1+e_0)} \frac{\Delta h}{H} \log\left(1+\frac{q_0}{\sigma'_{oi}}\right) + \frac{C_\alpha}{1+e_p} \frac{\Delta h}{H} \log\left(\frac{t}{t_0}\right) \tag{46}$$

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

$$\frac{S_{unt}}{H} = \sum_{i=1}^{n} \frac{\Delta S_{unt,i}}{H} \tag{47}$$

Settlement Reduction Factor (β) during Creep

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

$$\beta = \frac{S_t}{S_{unt}} \tag{48}$$

Void Ratio (e) during Creep

For normally consolidated soils, void ratio (e) decreases linearly with log effective stress during primary consolidation (Figure 3). The void ratio decreases with time under constant effective stress during secondary compression.

The final void ratio, $e_{f,i}$, of the i^{th} layer of the ground reinforced with GP is

$$e_{f,i} = e_0 - C_c\left[\log\left(1+\frac{q_{s,eop,i}}{\sigma'_{0i}}\right) - \frac{C_s}{C_c}\log\left(\frac{q_{s,eop,i}}{q_{s,eop,i} - \Delta q_{s,i}}\right) + \frac{C_\alpha}{C_c}\log\left(\frac{t}{t_0}\right)\right] \tag{49}$$

with $t = t_i$ for $t > t_i$ as explained above in Eq. (43). Hence void ratios for $t > t_i$, correspond to those for $t = t_i$.

Stress Concentration Factor (SCR) during Creep

The stress concentration factor, $SCF_{cr,i}$ for any given element 'i' during creep is

$$(SCF)_{cr,i} = \frac{q_{gp,\,cr,\,i}}{q_{s,\,cr,\,i}} \tag{50}$$

Overconsolidation Ratio (OCR) during Creep

The overconsolidation ratio, $OCR_{cr,i}$ for any given element 'i' during creep is the ratio of effective stresses during creep with respect to those at the end of primary consolidation, i.e.,

$$\left(OCR\right)_{cr,i} = \frac{q_{s,\,eop,\,i}}{q_{s,\,cr,\,i}} \tag{51}$$

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, β, and the e - log σ' 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, μ: 10, 30 and 60
Creep ratio, C_α/C_c: 0, 0.02, 0.04 and 0.06 and
Normalized applied stress, q^*_0: 2, 2.5, 3.0 and 3.5.

Effect of Stiffness Factor (μ)

Figures 5 to 11 show the effect of stiffness factor, μ=10, 30 and 60, for normalized applied stress, q^*_0, of 2.5, area ratio, A_r, of 0.1, creep ratios, C_α/C_c, of 0.02, 0.04 and 0.06 at times, t/t_0=1 (EOP) and times, t/t_0=10, 30, 60 and100.

The variations of the normalized total settlement, S_t/H with normalized depth, z/H and EOP and at time t/t_0=100 are depicted in Figure 5 for μ=10, 30 and 60. The normalized settlements, St/H, decrease with depth and stiffness factor, μ but increase with time, t/t_0. The increase of settlements with depth is uniform for all relative stiffness factors as it is assumed that creep settlements to be independent of stress on soft ground. The normalized settlements, S_t/H, are 0.104, 0.082, and 0.06 at EOP and 0.123, 0.101 and 0.08 at time, t/t_0=100 for μ equal to 10, 30 and 60 respectively.

The settlement reduction factor, β, increases with creep (Figure 6). The increase is marginal for relative stiffness factor, μ=10 increasing from 0.94 at EOP to 0.95 and 0.96 at t/t_0 value of 100 for creep ratios of C_α/C_c=0.02 and 0.06. The rate of increase in settlement reduction factor, β with t/t_0 increases with increasing values of relative stiffness factor, μ, the settlement reduction factor, β increasing from 0.548 at EOP to 0.587 and 0.646 at t/t_0=100 for μ=60 for creep ratios of C_α/C_c=0.02 and 0.06 respectively.

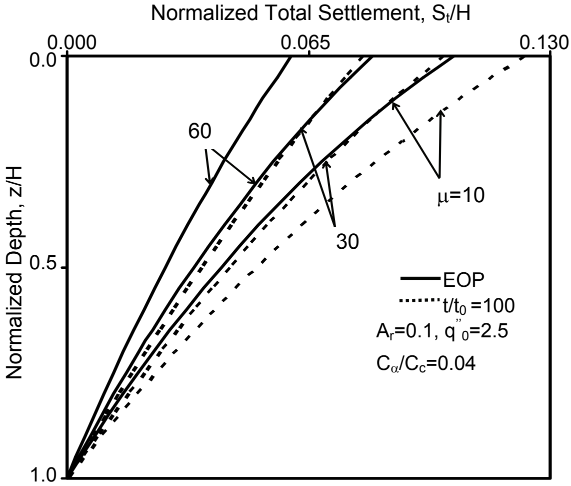

Fig. 5 Normalized Total Settlement of the Soil, S_t/H vs Normalized Depth, z/H Effect of Relative Stiffness Factor, μ

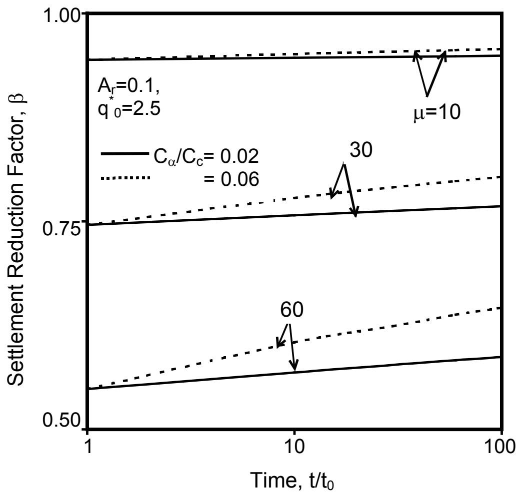

Fig. 6 Settlement Reduction Factor, β, vs Time, t/t_0-Effect of Relative Stiffness Factor, μ & Creep Ratio, C_α/C_c

The settlement at EOP is relatively small for soft ground reinforced by less stiff (μ=10) granular piles. The creep settlements independent of the relative stiffness factor, contribute significantly to the total settlement of the reinforced ground. Hence values of settlement reduction factor, β, increase marginally with time, t/t_0, for less stiff granular pile reinforced ground and significantly for very stiff granular pile reinforced ground.

The variation of normalized stress on soil, q'_s, with normalized depth, z/H, of the ground reinforced with granular piles is shown in Figure 7 for A_r=0.1, q^*_0=2.5 and C_α/C_c=0.4. The normalized stress on the soil at the end of primary consolidation for μ=10 increases from about 0.81 at the top of the soft layer to nearly 0.98 at the base, due to increase in the stiffness, i.e., of the constrained modulus, D_s, with depth for constant compression index, C_c. The normalized stresses on soil (q'_s), reduce from 0.81, 0.46 and 0.24 at EOP to 0.77, 0.34 and 0.01 at t/t_0=100 respectively for μ=10, 30 and 60 at a normalized depth, z/H= 0.025. The reductions at depth, z/H =0.975 are of the order of 0.98, 0.77 and 0.58 at EOP to 0.94, 0.66 and 0.35 at t/t_0=100 respectively. The reduction in the stresses on the soil increases with increasing relative stiffness, μ, of the GP material. The stresses from the soft soil get transferred to the granular pile as the former undergoes creep while the latter does not, to satisfy the compatibility of displacements, the reduction being larger for stiffer granular pile material.

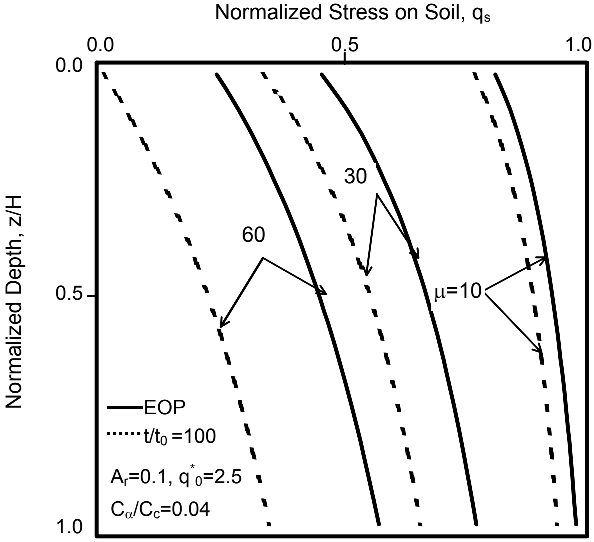

Fig. 7 Normalized Stress on Soil, q'_s vs Normalized Depth, z/H - Effect of Relative Stiffness Factor, μ

The stress concentrations factor, SCF (= q'_{gp}/q'_s), at all depths increases with time for A_r=0.1, q^*_0=2.5 and $\tilde{C}_\alpha C_c$=0.4 (Figure 8). The ratio, SCF, increases with t/t_0, since the stresses acting on the soft soil decrease while those on the granular layers increase due to the stress transfer from the former to the latter due to creep. The stress concentration factor (SCF= q'_{gp}/q'_s) decreases from 3.3, 12.8 and 31.8 at the top of the soft layer to 1.2, 3.9 and 8.4 nearly at the base of the soft layer at the end of primary consolidation for μ=10, 30 and 60. The stress concentration factor, SCF at a normalized depth of 0.025, increases marginally from 3.3 to 4 for μ=10, from 12.8 at EOP to 20.6 for μ=30 and significantly from 35 to 736 for μ=60 for t/t_0=100. This result follows from the observation made above, that is, the magnitude of stress transferred from soft ground to stiff granular pile increases with increasing relative stiffness, μ, of the ground. The net stress on the soft ground becomes negligibly small with time in case of stiff granular pile reinforced ground.

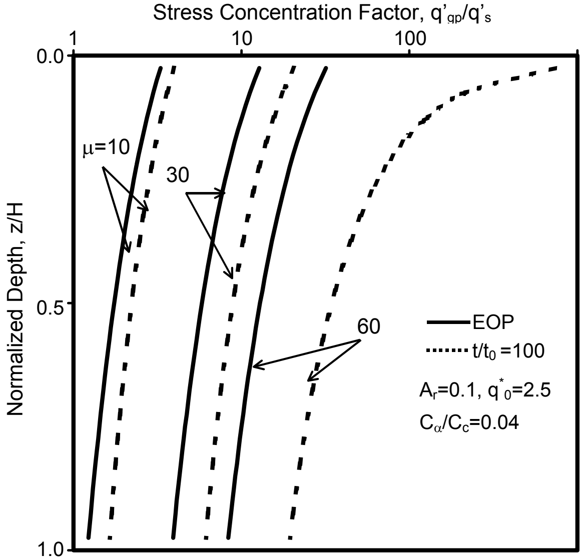

Fig. 8 Stress Concentration Factor, SCF, vs Normalized Depth, z/H - Effect of Relative Stiffness Factor, *μ*

Figure 9 shows the variation of the normalized shear stress, τ'_{cr}, with normalized depth, z/H, for A_r=0.1, q^*_0=2.5 and C_α/C_c=0.4.

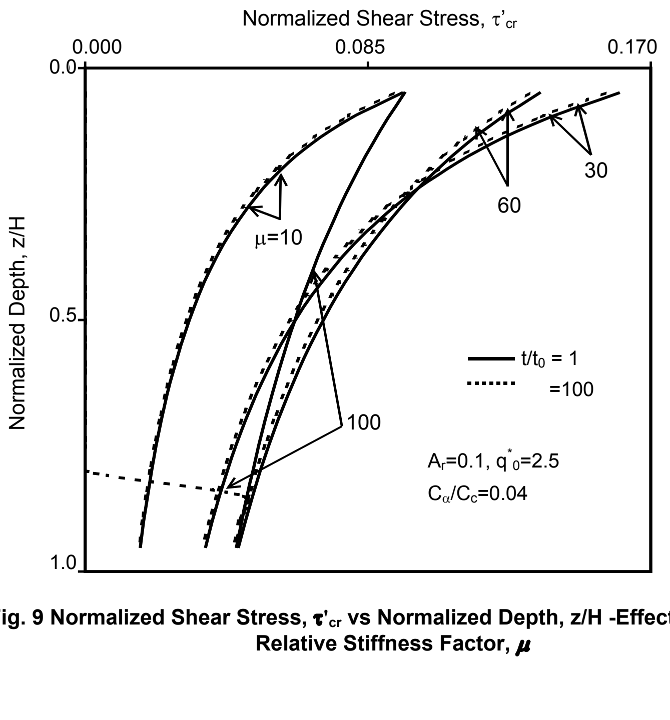

Fig. 9 Normalized Shear Stress, τ'_{cr} vs Normalized Depth, z/H -Effect of Relative Stiffness Factor, *μ*

The normalized shear stress decreases with depth in all the cases. The decrease is from 0.160 at z/H=0.025 to 0.034 at z/H=0.975 for stiffness factor, μ, of 30 at EOP. The normalized shear stresses increase with stiffness factor and with decrease time, t/t_0. The shear stresses in the upper region increase with μ till μ values of the order of 30 but decrease with further increase in μ. This result appears anomalous but can be easily explained. The increase in relative stiffness of granular pile near the top leads to greater stresses to be transferred to it there. However, this ratio is smaller at depth than near the top since the modulus of deformation of the soil increases with depth, leading to more stresses being transferred to the soil in the lower half of the layer. The shear stresses decrease sharply with depth for axially loaded compressible piles (Poulos and Davis 1980). However, with further increase in values of μ beyond 30, the granular pile tends to become a rigid pile and carries larger loads at all depths. The stress transfer to the soil at depth is much less and consequently the shear stresses too. The axial load is constant with depth for an infinitely rigid (granular) pile and the shear stresses become zero. This trend can discerned for μ values increasing beyond 60. The shear stresses for μ equal to 100 are less the corresponding values for μ equal to 60 at all depths. The in situ soil deforms considerably further due to creep. The GP material deforms equally by transfer of stress (Figure 4b) from the soil to the GP. This transfer is more in the upper part of GP than in the lower half. Consequently, the shear stresses tend to decrease uniformly with depth during creep. The normalized shear stresses, τ'_{cr}, at normalized depth, z/H=0.025 are 0.0951, 0.1602, and 0.1365 at EOP and 0.0887, 1504 and 0.1288 at time, t/t_0=100 for μ equal to 10, 30 and 60 respectively.

The variation of void ratio, e_0, with log effective stress, q_s, is plotted for untreated ground and ground treated with granular piles for stiffness factor, μ, equal to 10, 30 and 60 for the first (i=1) and the tenth (i=10) layers in Figure 10.

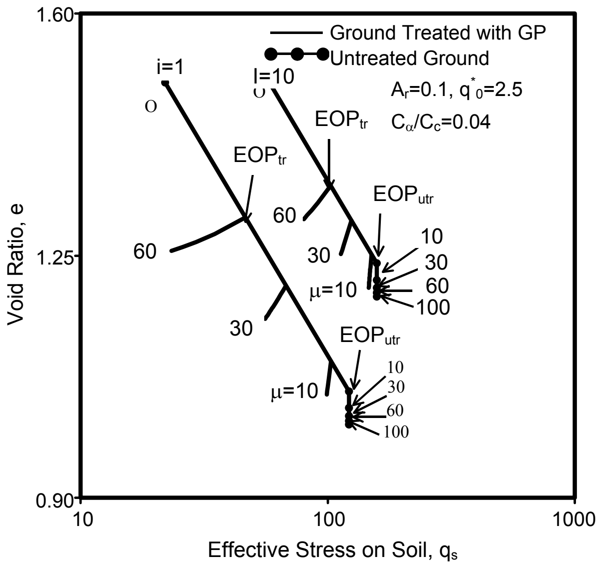

Fig. 10 Void Ratio, e vs Effect Stress on Soil, q'_s -
Effect of Relative Stiffness Factor, μ -Treated and Untreated Ground

The initial void ratios and the effective overburden pressures in the first (Point O) and the tenth (Point O') layers are taken as e_0=1.5 for σ'_{01}= 22 kPa and 1.5 and σ'_{01}= 58 kPa respectively. Stress, q_0= 100 kPa is applied through the granular mat or pad. The final void ratios, $e_{f,i}$, for the untreated ground are 1.054, 1.03, 1.18, 1.01 and 1.0 for the first (i=1) and 1.24, 1.215, 1.20, 1.196 and 1.19 for the tenth (i=10) for time of EOP, 10, 30, 60 and 100 respectively. While the final void ratio at the end-of-primary (EOP) consolidation is 1.054 for untreated ground for an applied stress of 100 kPa, the corresponding effective stress on soil and the void ratio at the end-of-primary (EOP) consolidation are 67.9 kPa and 1.21 for the first and 125 kPa and 1.3 for the tenth layers respectively for stiffness factor, μ=30 for the ground reinforced with granular pile. During creep, the stresses on the soil decrease from 67.9 kPa at EOP to 61.8 kPa, 58.9 kPa, 57.1 kPa and 55.8 kPa for the first and from 125 kPa at EOP to 119 kPa, 116.2 kPa, 114.5 kPa and 113.2 kPa for the tenth layers with time, t/t_0 = 10, 30, 60 and100. The corresponding void ratios are 1.1, 1.182, 1.171, 1.164 and 1.157 for the first and 1.30, 1.276, 1.265, 1.258 and 1.252 for the tenth layers respectively. These results signify the unloading of the soft ground during creep. The unloading is more for stiffer granular piles as they are able to carry larger stresses for the same amount of creep deformation of the soft ground.

The ratio of effective stresses with respect to the stress at the end of primary consolidation was calculated (Eq.51) and termed as overconsolidation ratio, OCR. The variation of overconsolidation ratio (OCR) with time for different relative stiffness factors, μ, and for different creep ratios, C_α/C_c, is depicted in Figure 11.

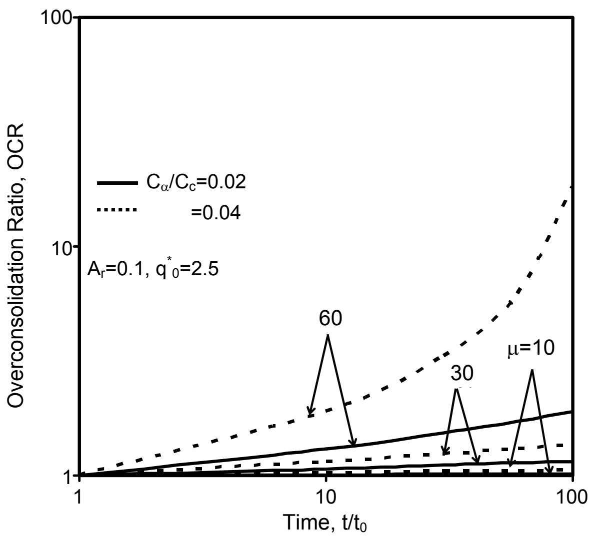

Fig. 11 Overconsolidation Ratio, OCR vs Time t/t_0- Effect of Relative Stiffness Factor, μ & Creep Ratio, C_α/Cc

OCR values increase with increasing time, t/t_0, as the process of unloading continues with increasing creep deformations. The rate of increase of OCR increases with increases of time, the creep ratio, C_α/C_c, and the relative stiffness factor, μ. The overconsolidation ratio increases with the increase in the relative stiffness factor, μ, for any given time. The overconsolidation ratio is 1.027, 1.152 and 1.896 for creep ratio C_α/C_c = 0.02 and 1.055, 1.358 and 18.243 for creep ratio C_α/C_c =0.04 for the relative stiffness factor, μ, equal to 10, 30 and 60 respectively for t/t_0=100.

Effect of the Area Ratio (A_r)

Figures 12 to 17 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, μ=30, creep ratio, C_α/C_c, of 0.04 at t/t_0=1 (EOP) and times, t/t_0=10, 30, 60 and100.

Figure 12 shows effect of area ration on the variation of the normalized total settlement, S_t/H with normalized depth, z/H. The normalized total settlements, S_t/H, decrease with increase of area ratio A_r, and increase with time from EOP. The ground becomes stiffer and reinforced more with increasing area ratio and thus records lower values of settlements. However, the normalized settlements, S_t/H, of0.082, 0.063, and 0.05 at a normalized depth, z/H-0.025 and at EOP increase to 0.101, 0.082 and 0.065 at time, t/t_0=100 respectively. The creep settlements are assumed independent of all these parameters and thus lead to the above effect.

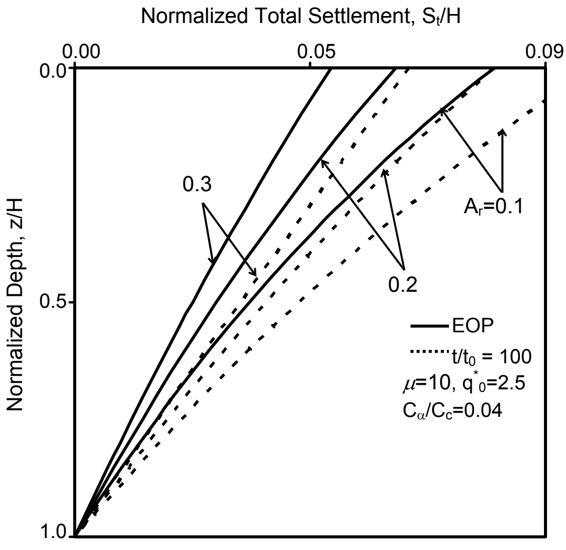

Fig. 12 Normalized Total Settlement, S_t/H vs Normalized Depth, z/H- Effect of Area Ratio, A_r

The variations of settlement reduction factor, β, with time, t/t_0, for different area ratios, A_r and for different normalized applied stresses, q*0, (Figure 13) are similar to the variation of β with relative stiffness factor, μ (Figure 5), the rate of increase of settlement reduction factor, β with t/t_0 increasing with area ratio, A_r. However, the rate of increase of settlement reduction factor, β, 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, β, increases with normalized applied stress, q^*_0, at EOP and the same trend manifests with creep. Hence the settlement reduction factor, β, increases with increasing values of q^*_0, the increase being of 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.

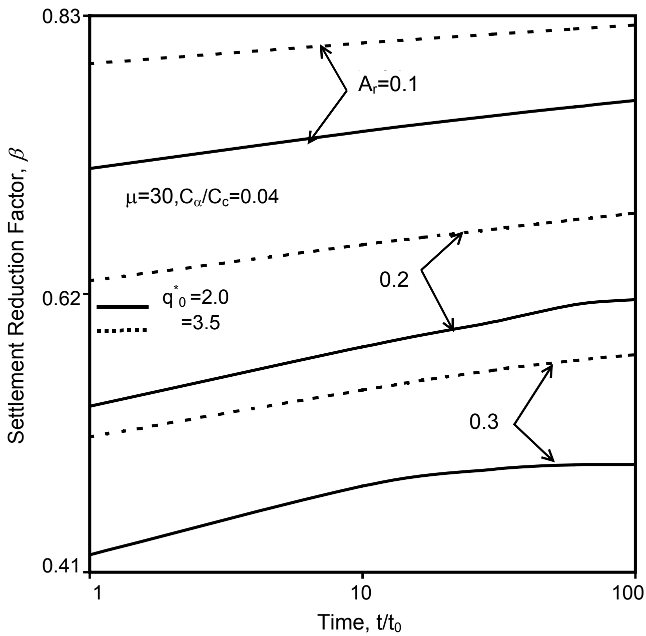

Fig. 13 Settlement Reduction Factor, β vs Time, t/t_0 - Effect of Area Ratio, A_r & Normalized Applied Stress, q^*_0

The variation of normalized stress, q'_s, on soil, with normalized depth, z_i/H, of the ground reinforced with granular pile shown in Figure 14 for μ = 30, q^*_0 = 2.5 and C_α/C_c=0.4. The normalized stresses on soil (q'_s), at normalized depth, z/H= 0.025, reduce from 0.46, 0.25 and 0.17 at EOP due to creep to 0.34, zero and zero at time, t/t_0=100 for area ratios, A_r=0.1, 0.2 and 0.3 respectively. The soil gets unloaded completely for area ratio, A_r= 0.3 over a depth range of 0 to 0.775 respectively.

The stress concentrations factor, SCF (= q'_{gp}/q'_s), at all depths increases with time and area ratio, A_r (Figure 15) as a consequence of increase of stresses on GP and decrease of stresses on the elements of the soil. The stress concentration factors, SCF= q'_{gp}/q'_s, are12.8, 15.77 and 17.62 at a normalized depth of 0.025 and at the EOP and increase respectively to 20.6, infinity and infinity for t/t_0=100 for area ratios A_r=0.1, 0.2 and 0.3 respectively. The SCR tending to infinity implies that the soft ground is completely unloaded from the applied stress.

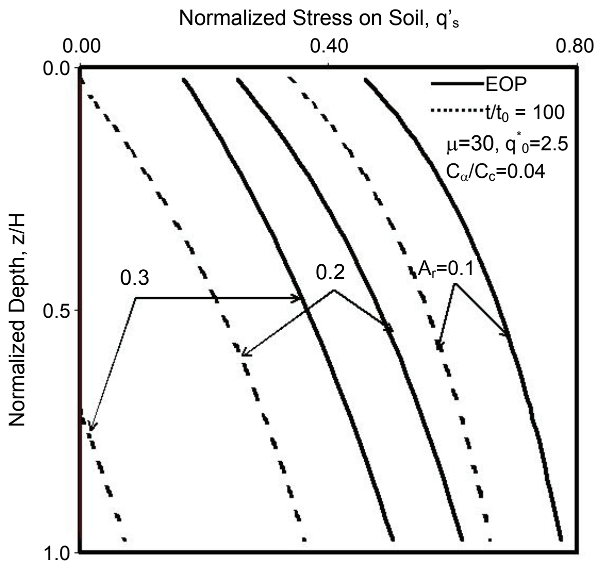

Fig. 14 Normalized Stress on Soil, q'_s vs Normalized Depth, z/H – Effect of Area Ratio, A_r

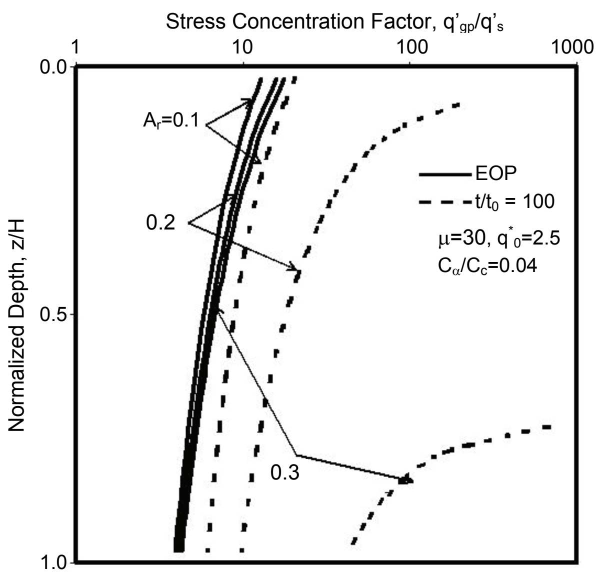

Fig. 15 Stress Concentration Factor, SCR vs Normalized Depth, z/H Effect of Area Ratio, A_r

The variations of the normalized shear stress, τ_{cr}, with normalized depth, z/H, and for different area ratios, are presented in Figure 16. The normalized shear stresses decrease with area ratio and with time. The normalized shear stresses, τ_{cr}, are 0.1602, 0.064, and 0.029 at EOP and at a normalized depth, z/H-0.025and reduce to 0.1504, zero and zero at time, t/t_0=100 for area ratios A_r=0.1, 0.2 and 0.3 respectively. The decrease of shear stresses with time is marginal as was the case depicted in Figure 9 for the effect of relative stiffness factor. The shear stress is zero once the top layers get unloaded.

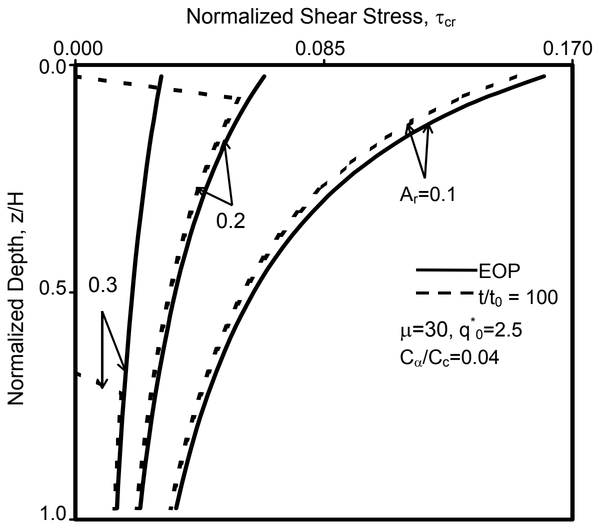

Fig. 16 Normalized Shear Stress, τ'_{cr} vs Normalized Depth, z/H Effect Area Ratio, A_r

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 Figure 17. The response of the unreinforced ground is explained in Figure 10.

The response of the reinforced ground in terms of void ratio –log effective stress plot with stiffness factor, μ =30, creep ratio C_α/C_c=0.04 normalized applied stress, q^*_0=2.5 is explained in Figure 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.

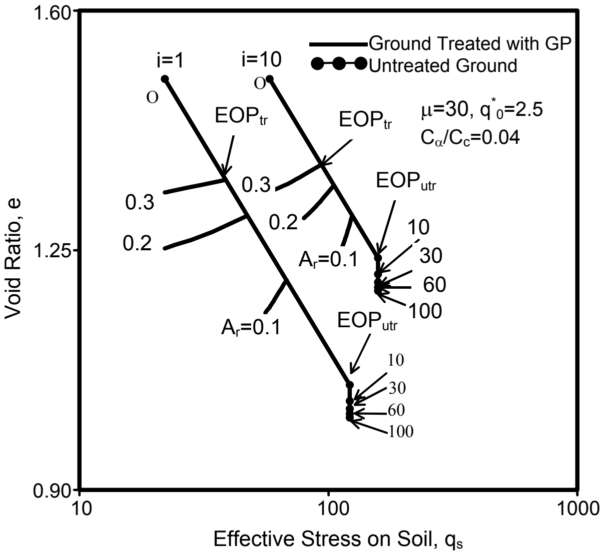

Fig. 17 Void Ratio, e vs Effect Stress on Soil, q'_s – Effect of Area Ratio, A_r- Treated and Untreated Ground

Time for Unloading due to Creep

The interesting phenomenon of unloading due to creep of soft soil in granular pile reinforced ground has been established and quantified in the preceding discussion. Soils at different depth get completely unloaded at different times depending on the relative stiffness, area ratio and creep rates. Figure 18 shows times for unloading (t_{ul} /t_0) vs normalized depth, z/H, for creep ratios, C_α/C_c, of 0.02, 0.04 and 0.06 for stiffness factors, μ, of 30 and 60, for an area ratio, A_r=0.3 and normalized applied stress of q^*_0=2.5. The time for unloading, (t_{ul}/t_0) decreases from about 5.83 to 1.52 for creep ratio of C_α/C_c=0.04, at normalized depth, z/H=0.05 for an increase in the relative stiffness factor, μ, from 30 and 60. The time for unloading decreases from 34, 5.83 and 3.24 with increase the creep ratio, C_α/C_c from 0.02, 0.04 and 0.06 for relative stiffness factor, μ =30 and the normalized depth, z/H=0.05.

Figure 19 shows times for unloading (t_{ul}/t_0) versus normalized depth, z/H with the effect of creep ratio, C_α/C_c, for area ratios of A_r of 0.2 and 0.3, for relative stiffness factor, μ=60 and normalized applied stress of q^*_0=2.5. The time for unloading (t_{ul} /t_0) decreases from 2.99 to 1.52 with increase in the area ratio, A_r, from 0.2 and 0.3 for creep ratio of C_α/C_c=0.04, at normalized depth, z/H=0.05. The time for unloading decreases from 8.9, 2.99 and 2.07 with increase in the creep ratio, C_α/C_c from 0.02, 0.04 and 0.06 for area ratio =0.2 at normalized depth, z/H=0.05.

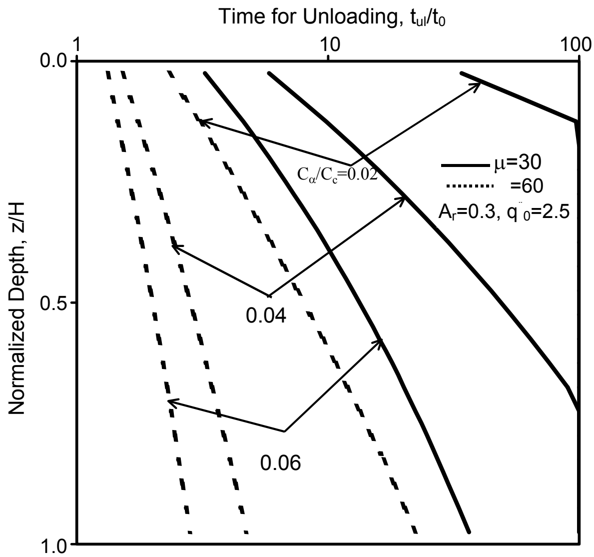

Fig. 18 Time for Unloading, tul/t_0 vs Normalized Depth, z/H – Effect of Creep Ratio, C_α/C_c & Effect of Relative Stiffness Factor, μ

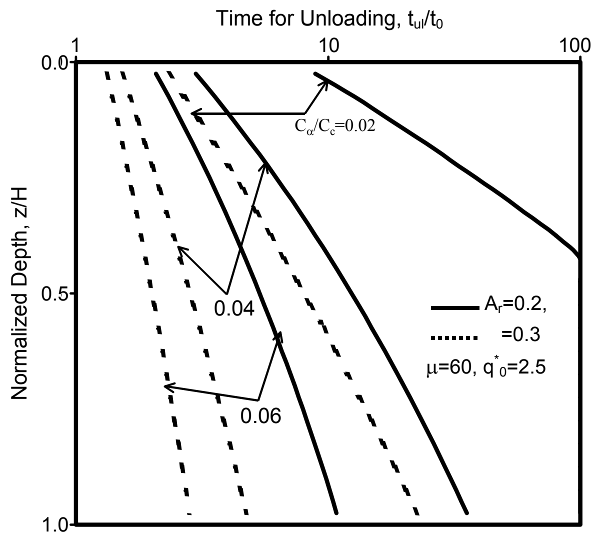

Fig. 19 Time for Unloading, tul/t_0 vs Normalized Depth, z/H – Effect of Creep Ratio, C_α/C_c & Effect of Area Ratio, A_r

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, μ, area ratios, A_r, time, t/t_0, and creep ratios, C_α/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 interface shear stresses between the granular pile and the soft soil decrease marginally with time. 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). The over-consolidation defined with respect to the stress at the EOP increase with increasing time, area ratio and relative stiffness factor. On similar lines, the time for complete or total unloading of stress on to soft soil decreases with increasing time, area ratio and relative stiffness factor but increases with depth of the layer.

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