Indian Geotechnical Journal, 38(3), 2008, 295-316

Densification and Dilation Effects of Granular Piles in Liquefaction Mitigation

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Introduction

Earthquakes are constantly posing risk to life and infrastructure facilities. One of the most important causes of damage to structures during earthquakes has been the development of liquefaction in saturated sand deposits. The liquefaction is manifested either by the formation of sand boils and mud-spouts at the ground surface, by seepage of water through ground cracks or in some cases by the development of quick sand conditions over substantial areas (Seed and Idriss 1982). Soil liquefaction is the state at which the soil deposit looses its strength and flows as a fluid. A qualitative understanding of the mechanism underlying the liquefaction of saturated sands, subjected to cyclic loading, such as that induced by earthquakes, had been recognized widely since first being examined by Casagrande in 1936 (Martin et al. 1975). Sawicki and Mierczynski (2006) presented historical developments of mechanics of saturated granular soils in relation to the liquefaction phenomenon, and the development of theoretical approaches to liquefaction-related problems, such as cyclic loading induced compaction and pore pressure accumulation, or cyclic degradation of shearing resistance.

Saturated granular material, when subjected to cyclic loading involving the reversal of shear stresses, tends to get compacted or densify. Under undrained conditions, in cases where the soil consists of loose granular materials under high water table, the tendency to get densified may result in the development of excess hydrostatic porewater pressure during each cycle of loading. If the magnitude of porewater pressure generated equals the confining pressure, the effective stress becomes zero and the soil is said to have liquefied. Pore pressure buildup leading to liquefaction may be due to static or cyclic stress applications and the possibility of its occurrence depends on the initial void ratio or relative density of sand and the confining pressure (Seed 1979). Resistance to liquefaction can be improved by increasing the density (densification), modifying the grain size distribution (grouting/mixing), stabilizing the soil fabric (reinforcing), reducing the degree of saturation, dissipation of the excess pore pressures generated and interception of the propagation of excess pore pressures (drainage), etc. (Madhav and Arlekar 2000).

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A possible method of stabilizing a soil deposit susceptible to liquefaction is to install column-like structures in the ground, like system of gravel or rock drains (Seed and Booker 1977). Provision of sand drains/granular piles/stone columns/aggregate piers is the most commonly adopted ground treatment methodology for liquefaction mitigation. It has proved its effectiveness in many instances (Mitchell and Wentz 1991). Adalier and Elgamal (2004) reviewed the current state of stone column technologies as a liquefaction countermeasure. Due to installation of gravel drains, the generated pore water pressures due to repeated loading get dissipated almost as fast as they are generated. Thus granular piles are effective in mitigating liquefaction damage due to the drainage facility. Granular piles are of the displacement type and hence densify in situ ground during the process of installation (Madhav 2001). The effect of densification is manifested through an increase in the coefficient of earth pressure at rest and in the values of modulus of deformation of the soil (Ohbayashi et al. 1999).

Densification by rammed granular piles (RGP) causes increase in deformation moduli and decreases in the coefficients of permeability and volume change. The densification effect, however, decreases with distance from the center of the compaction point and may become negligible at the periphery of the unit cell. Thus, the coefficients of horizontal permeability, $k_h(r)$, and volume change, $m_v(r)$, of the soil around the granular pile can be considered to vary with distance, r, from the center of the granular pile. Dense granular soils experience volume increase during shearing due to dilatancy. Very high negative pore pressures are generated due to suppression of the tendency for dilation, which further enhances the liquefaction mitigation (Madhav and Arlekar 2000). Hence, in addition to the drainage effect, densification and reinforcement of the ambient soil around the rammed granular pile (RGP) should also be considered along with the dilation effect for total and better evaluation of the improvement.

In this paper, the pore pressure generation and dissipation of the treated ground under earthquake conditions is analyzed to quantify the densification effect of ground treatment with RGP and due to dilation of RGP. Theory of pore-water pressure generation and dissipation developed by Seed and Booker (1977) is applied, with some modifications for evaluating the densification and dilation effects of RGPs together, to the analysis of columnar gravel drains under earthquake conditions.

Rammed Granular Piles

Ground improvement by means of granular piles/stone columns/geo-piers, which is associated with partial substitution of the in–situ soil, originated in sixties. Stone columns generally use gravel or crushed stone as backfill. Numerous publications (e.g. Barksdale and Bachus, 1983; Munfakh et al., 1987; Baez and Martin, 1992; and Brennan and Madabhushi, 2002) describe the use of stone columns for ground reinforcement and their potential to mitigate the liquefaction. Liquefied and non-liquefied subsoil conditions of two reclaimed islands in Kobe City after the 1995 Hyogoken-Nambu earthquake were investigated by Yasuda et al. (1996) and it was found that the sub soils treated with sand compaction piles or rod (vibro) compaction did not liquefy nor subside even though the earthquake shaking was very strong. Ground treated by granular piles provide increased bearing capacity, significant reduction in settlement, free drainage, increase of liquefaction resistance, etc. Granular piles are installed by vibro-compaction, vibro-replacement, cased bore hole (rammed stone columns/RGP) or by simple auger boring methods (Datye and Nagaraju 1981, Balaam and Booker 1981). RGP are installed into the ground by partial or full displacement methods and by ramming in stages, using a heavy falling weight, within a ‘pre-bored casing’ or ‘driven closed end casing’, retracting the casing pipe stepwise. In the latter case, driving of closed end tube itself densifies the surrounding soil.

Densification Effect

Ramming action of the falling weight tends to ram the stone into the sides of the hole, which densifies more and reinforces the ground in addition to compacting the stone column substantially. Tsukamoto et al. (2000) examined the changes in the state of stress due to static sand compaction pile penetration in densifying loose to medium dense soils and presented a chart for evaluating the improved or modified SPT N values (after treatment) in terms of replacement ratio, and initial SPT N (before treatment) values of the soil.

Densification and reinforcement effects cause modifications in the properties of the in situ soil. The densification and modification effects on the soil parameters of the ground are not uniform over the entire zone of surrounding ground but are functions of the distance from the point of densification. During the process of installation of RGP, the soil adjacent to and in the vicinity of the point of treatment gets densified most. This densification effect decreases with the distance from the point of densification. The densification effect can easily be but indirectly quantified by in-situ tests. Ohbayashi et al. (1999) summarized measured values of Swedish Weight Sounding, (N_{sw}), SPT N and CPT (q_c) at different sites wherein the increase in the measured parameters are presented as a function of the distance from the center of the compaction point. The densification effect becomes negligible at a distance of about 2.0 m from the center of the sand compaction piles (SCPs) but the increase depends on the fines content (Fig. 1). Thus, the densification effect of the ground, due to the installation of granular pile, is maximum near the periphery of the granular pile and decreases with the distance from the granular pile. Densification by RGP causes increase in deformation moduli and decrease in the coefficients of permeability and of volume change. Figure 1 shows the variation of increase in SPT N_1 value with distance from pile centre, depicting the diminishing trend of increase in N_1 value with distance for all the fines content. This decrease is hypothesized to be linear or exponential.

Sujatha (1998) analyzed several field test data and postulated linear and exponential variations of the stiffness of soil with distance as limits for the densification effect. Similar linear and exponential variations are considered for the reductions of flow and deformation parameters of the in situ soil (coefficients of permeability and volume change). This reduction is maximum at the point of densification and reduces with distance towards the periphery of the unit cell reaching the original in-situ values at the farthest end of the unit cell. Murali Krishna and Madhav (2007) analyzed reinforcement and densification effects due to installation of granular piles on the deformation properties of the treated ground and derived the equivalent deformation properties. Murali Krishna et al. (2007) considered the densification effect, by adopting the linear and exponential variations of deformation properties, in the analysis of settlement response of the ground treated with granular piles. Murali Krishna et al. (2006) analyzed the pore pressure generation and dissipation of the treated ground under earthquake conditions to quantify the densification effect by adopting linear variations of the coefficients of permeability and volume change with distance.

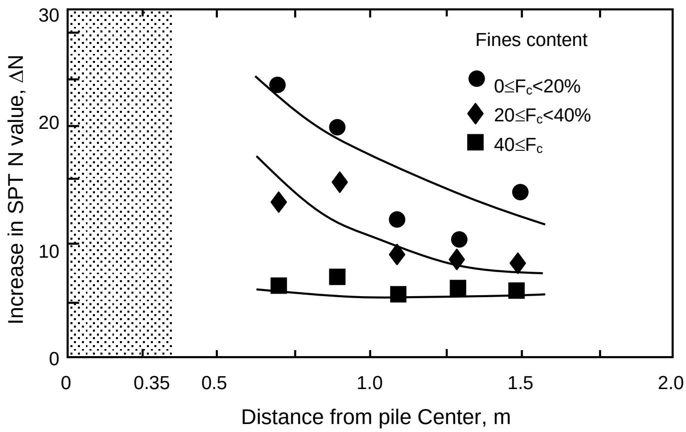

Fig. 1 Variation of Increase in SPT N_1 Value with Distance from Pile Centre (after Ohbayashi et al. 1999)

Variations of the Flow Parameters

Variations of the coefficient of horizontal permeability, k_h, with distance from the center of RGP is considered with k_h value becoming minimum at the edge of RGP (r = a) and increasing to maximum (original) value at the periphery of the unit cell (r = b) (Figure 2).

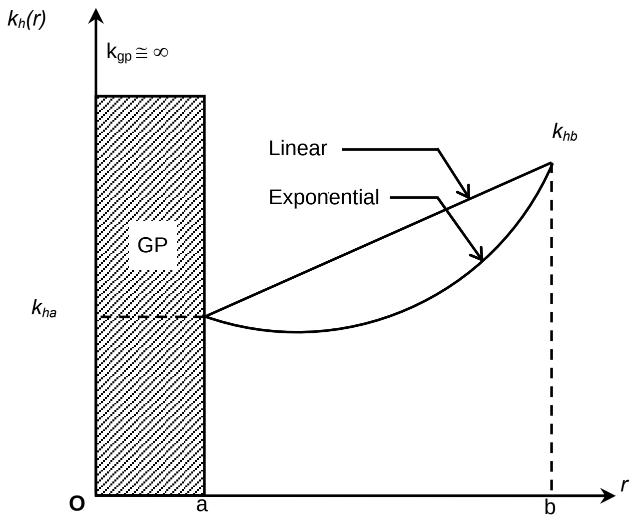

Fig. 2 Variations for Coefficient of Permeability of Soil with Distance

The general expressions for the variations of the coefficient of permeability at a distance r from the centre of the granular pile can be expressed (Murali Krishna et al. 2006) as

$$k_h(r)=\left(\frac{k_{hb}-k_{ha}}{b-a}\right).(r-a)+k_{ha} \tag{1}$$

for linear variation and

$$k_h\left(r\right)=k_{ha}.\exp\left(\frac{\left(r-a\right)}{\left(b-a\right)}\ln\left(\frac{k_{ha}}{k_{hb}}\right)\right) \tag{2}$$

for exponential variation, where $k_h(r)$, k_{ha} and k_{hb} are the coefficients of permeability at distances of r, a and b from the centre of the granular pile respectively.

Similarly, the expressions for the variations of coefficient of volume change, m_v, are

$$m_v(r)=\left(\frac{m_{vb}-m_{va}}{b-a}\right).(r-a)+m_{va} \tag{3}$$

for linear variation and

$$m_v\left(r\right)=m_{va}.\exp\left(\frac{\left(r-a\right)}{\left(b-a\right)}\ln\left(\frac{m_{va}}{m_{vb}}\right)\right) \tag{4}$$

for exponential variation, where $m_v(r)$, m_{va} and m_{vb} are respectively the coefficients at distances r, a and b from the centre.

Dilation Effect

It has been generally recognized that the susceptibility of a given soil to liquefaction is determined to a high degree by its void ratio or relative density. In any given earthquake, loose sands may liquefy but the same materials in a denser condition do not. In the city of Niigata, Japan in 1964, for example, liquefaction was extensive where the relative density of the sand was about 50%, but areas where the relative density exceeded about 70%, the sand did not liquify (Seed and Idriss, 1971).

Shearing of dense dilative soils produces small excess pore pressures at small strains. However, at larger strains, the pore pressures decrease and become negative as the soil grains move up over one another, tending to cause an increase in soil volume (dilation). For dense, saturated sands sheared without porewater drainage, the tendency for dilation or volume increase results in the generation of negative porewater pressure and an increase in the effective stress and shear strength of the granular material. The response of saturated sand under undrained triaxial conditions (Leonards 1962) can be seen in Figure 3. While positive pore pressures are generated in loose sands, generation of very high negative pore pressures can be observed due to the suppression of the tendency for dilation in medium and dense sands. Figure 4

(Vaid et al. 1981) is a typical example of volume change behaviour of granular material under drained conditions in a simple shear test at different vertical stress conditions. While initially loose samples undergo volume decrease initially, dense samples experience volume increase (dilation) during shearing. The rate of dilation increases with relative density.

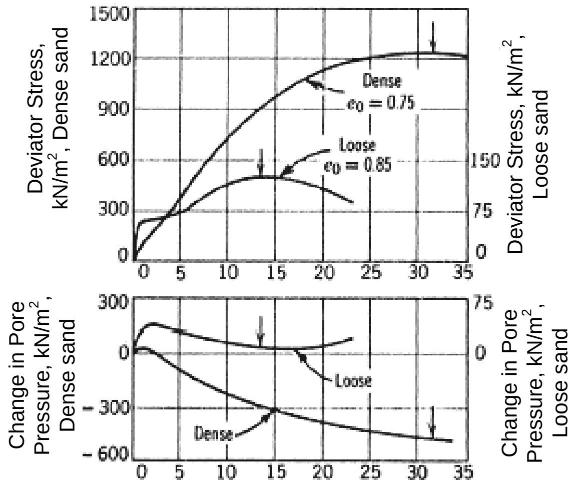

Fig. 3 Response of Saturated Sand under Undrained Triaxial Test Conditions (after Leonards 1962)

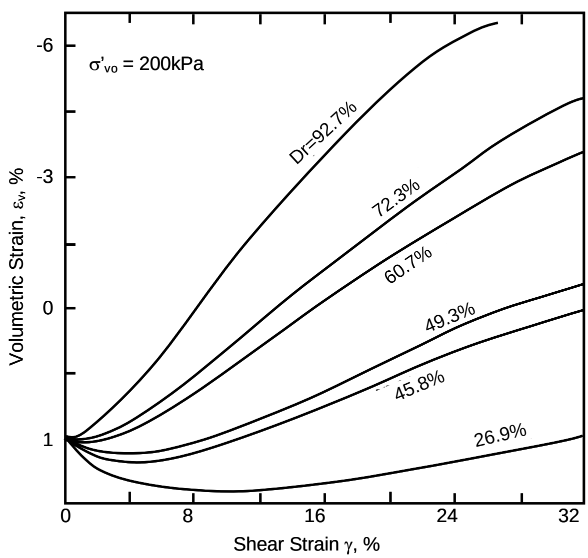

Fig. 4 Volume Change Behavior for Granular Material under Drained Conditions (after Vaid et al. 1981)

The dilation angle is one single parameter which can be readily obtained from both laboratory (drained triaxial or simple shear) and in situ (self-boring pressuremeter) tests, which can give a measure of the liquefaction resistance. Baez and Martin (1992) presented an evaluation of the relative effectiveness of stone columns for the mitigation of liquefaction of soil. They also described tests on footings on soil reinforced with stone columns, which were used to calibrate a finite element program. The most interesting result obtained is that the stone columns experience an increase in effective stress simultaneously with the development of negative pore pressures.

The effect of dilation of granular pile material on settlement response of granular pile treated ground has been investigated by Poorooshasb and Madhav (1985) and Van Impe and Madhav (1992). In the former, the response of a granular pile reinforced soil subjected to uniform loading through a relatively rigid raft is studied considering the granular pile material to follow the rigid plastic dilatant strain hardening postulates of Poorooshasb et al. (1966). The tendency for dilation is resisted by the soil, which offers larger interaction (confining) stresses. As a result, both the pile and the soil become stiffer with increasing applied stress. As a consequence of the dilatant nature of the granular material, the settlement versus the intensity of loading curve exhibits a non-linear relation. The mechanical effect of dilatancy of granular material in increasing the stiffness of soft or loose soil deposits has been quantified in the above two works. The effect of dilatancy during undrained state that exists within a gravel drain/granular pile during a seismic event is also considered in the analysis.

Seed & Booker (1977) Model

Seed and Booker (1977) were the first to propose an analytical model for the generation and dissipation of pore pressure in a soil deposit with vertical drains. Under the assumptions of purely radial drainage, constant coefficient of compressibility and infinite permeability of drains, design charts were developed to evaluate the effects of drain diameter and spacing for the expected earthquake loading on excess pore pressure ratio. In most practical cases, the horizontal permeability of sand/gravel deposit will be several times greater than its vertical permeability and the spacing between vertical drains is closer than the distance required for porewater to drain vertically towards the free surface. Furthermore, many natural deposits of sand are interspersed with narrow horizontal layers of relatively impermeable silt, which may severely inhibit vertical drainage. For these reasons, the dominant mechanism in the operation of a gravel drain system is one of pure horizontal drainage (Seed and Booker 1977).

Thus, for flow into a gravel drain, assuming pure radial flow, and constant coefficients of permeability (k_h) and volume compressibility (m_v), the governing equation for the phenomenon can be written as (Seed and Booker 1977)

$$\frac{k_h}{\gamma_w . m_v}\left(\frac{1}{r}\frac{\partial u}{\partial r}+\frac{\partial^2 u}{\partial r^2}\right)=\frac{\partial u}{\partial t}-\frac{\partial u_g}{\partial N}.\frac{\partial N}{\partial t} \tag{5}$$

where u is the excess pore pressure at a radial distance, r, from the centre, t is time, γ_w the unit weight of water, and u_g = peak excess hydrostatic pore water pressure generated by the earthquake. The rate of generation of pore pressure during an earthquake event is defined by

$$\frac{\partial u_g}{\partial N} = \frac{\sigma'_o}{\alpha \pi N_l} \frac{1}{\sin^{2\alpha-1}\left(\frac{\pi}{2} r_u\right)\cos\left(\frac{\pi}{2} r_u\right)} \tag{6}$$

where $r_u = u / \sigma'_o$ = the pore pressure ratio, σ'_o = the initial mean bulk effective stress for axi-symmetric conditions or the initial vertical effective stress for simple shear conditions; N_l is the number of cycles required to cause liquefaction and α = an empirical constant which is a function of the soil properties with a typical average value of 0.7. The irregular cyclic loading induced by an earthquake is converted (Seed et al. 1975) to an equivalent number, N_{eq}, of uniform cycles at an amplitude of 65% of the peak cyclic shear stress, i.e. $\tau_{cyc} = 0.65 \tau_{max}$, occurring over a duration of time, t_d, and

$$\frac{\partial N}{\partial t} = \frac{N_{eq}}{t_d} \tag{7}$$

Assumptions and Limitations

It is assumed that the flow of the pore water is governed by Darcy's Law; the coefficients of permeability and compressibility remain constant; drainage or flow is horizontal; granular filler material is far more permeable than the surrounding sand layer.

Limitations: Infinite gravel drain permeability is assumed so that no excess pore water pressures are developed in the gravel columns. The smear resistance is not accounted for as also the possible clogging of the drain due to migration of fine sediments towards the drain due to pore pressure dissipation. The model applies to low pore pressure ratio values where a linear process of consolidation is valid.

Recent Studies on Seed & Booker's Model

Design diagrams by Onoue (1988) and Iai and Koizumi (1986) incorporated the effects of drain resistance in the analyses of Seed and Booker (1977). Baez and Martin (1992) presented an evaluation of the relative effectiveness of stone columns for the mitigation of liquefaction of soil. Pestana et al. (1997) developed a finite element code for analyzing three-dimensional pore pressure generation and dissipation with vertical drains in place. Pestana et al. (1998) analysed the development of excess pore pressure in a layered soil profile, accounting for vertical and horizontal drainage with a non-constant 'equivalent hydraulic conductivity' and head losses due to horizontal flow into the drain and also considered presence of a reservoir directly connected to the drain. Boulanger et al. (1998) evaluated the drainage capacity of stone columns or gravel drains for mitigating liquefaction hazards. Murali Krishna et al. (2006) incorporated the densification effect of granular pile in the form of linear variation of flow parameters for the ambient soil.

New Model Considering Densification and Dilation of Ground

The governing equation (Eq. 5) of Seed and Booker (1977) is modified to include the densification effect of RGP in the dissipation of the excess pore-water pressures. In the ground treated with RGP, coefficient of permeability (k_h) and coefficient of volume compressibility (m_v) are considered to depend on the distance, r, instead being constant values. Considering an element of soil, in polar coordinates as shown in Figure 5, laminar flow and using Darcy's law, the expression for the flow in radial direction is obtained as,

$$\frac{k(r)}{r}\frac{\partial h}{\partial r}+k(r)\frac{\partial^2 h}{\partial r^2}+\frac{\partial k(r)}{\partial r}\frac{\partial h}{\partial r}=\frac{S}{1+e}.\frac{\partial e}{\partial t} \tag{8}$$

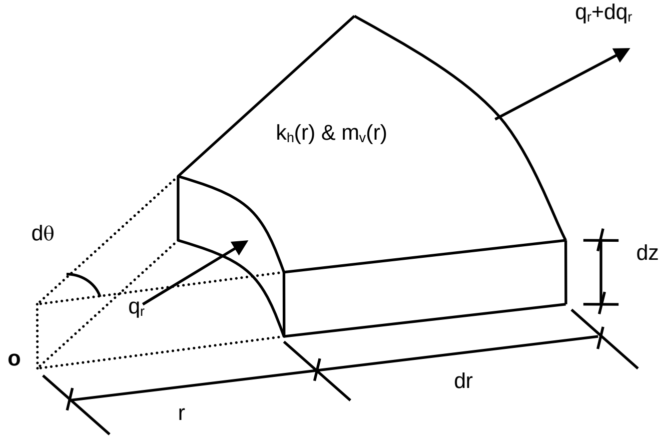

Figure 5 Flow through Soil Element

From the stress-strain relations and definitions of coefficient of compressibility (a_v) and coefficient of volume change (m_v) and for S =1(fully saturated),

$$\frac{1}{1+e}.\frac{\partial e}{\partial t}=m_v\left(\frac{\partial u}{\partial t}-\frac{\partial \sigma}{\partial t}\right) \tag{9}$$

From Eq. 9 and with $h = u/\gamma_w$; (u = excess pore pressure; γ_w = unit weight of water) Eq. 9 is rewritten as:

$$\frac{k_h(r)}{\gamma_w.m_v(r)}\left(\frac{1}{r}\frac{\partial u}{\partial r}+\frac{\partial^2 u}{\partial r^2}\right)+\frac{1}{\gamma_w.m_v(r)}.\frac{\partial(k_h(r))}{\partial r}.\frac{\partial u}{\partial r}=\frac{\partial u}{\partial t}-\frac{\partial \sigma}{\partial t} \tag{10}$$

where $k_h(r)$ and $m_v(r)$ are defined in Eqs. (1) and (3) or (2) and (4) respectively according to the variation (linear or exponential) considered. Considering the rate of change of total stress $(\partial\sigma/\partial t)$ as the rate of pore pressure generation,

$$\frac{\partial u_g}{\partial t} = \frac{\partial u_g}{\partial N} \cdot \frac{\partial N}{\partial t} \tag{11}$$

The final form of Eq. 10 is,

$$\frac{k_h(r)}{\gamma_w . m_v(r)}\left(\frac{1}{r}\frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial r^2}\right) + \frac{1}{\gamma_w . m_v(r)} \cdot \frac{\partial(k_h(r))}{\partial r} \cdot \frac{\partial u}{\partial r} = \frac{\partial u}{\partial t} - \frac{\partial u_g}{\partial N} \cdot \frac{\partial N}{\partial t} \tag{12}$$

Eq. 12 is a more generalized equation considering all forms of non-homogeneity. In non-dimensional form, with normalized pore pressure ratio, $W = u/\sigma'_o$, which is the same as the pore pressure ratio r_u (Seed and Booker 1977) and $W_g = u_g/\sigma'_o$. Eq. 12 becomes

$$\frac{k_h(r)}{\gamma_w . m_v(r)}\left(\frac{1}{r}\frac{\partial W}{\partial r} + \frac{\partial^2 W}{\partial r^2}\right) + \frac{1}{\gamma_w . m_v(r)} \cdot \frac{\partial(k_h(r))}{\partial r} \cdot \frac{\partial W}{\partial r} = \frac{\partial W}{\partial t} - \frac{\partial W_g}{\partial N} \cdot \frac{\partial N}{\partial t} \tag{13}$$

Using normalized terms for r, k_h, m_v and t as R ($=r/b$), $R_k(r)$ ($=k_h(r)/k_{hi}$), $R_{mv}(r)$ ($=m_v(r)/m_{vi}$) and T ($=t/t_d$) respectively (where k_{hi} and m_{vi} are the flow parameters for the virgin soil). Eq. 13 becomes

$$T_{bd} \cdot \frac{R_k(R)}{R_{mv}(R)}\left(\frac{\partial^2 W}{\partial R^2} + \frac{1}{R}\frac{\partial W}{\partial R}\right) + \frac{T_{bd} \cdot}{R_{mv}(R)} \frac{\partial(R_k(R))}{\partial R} \cdot \frac{\partial W}{\partial R} = \frac{\partial W}{\partial T} - \frac{\partial W_g}{\partial N} N_{eq} \tag{14}$$

where,

$$\frac{\partial W_g}{\partial N} = \frac{1}{\alpha \pi N_l} \cdot \frac{1}{\sin^{2\alpha - 1}\left(\frac{\pi}{2} r_u\right) \cos\left(\frac{\pi}{2} r_u\right)} \tag{15}$$

and

$$T_{bd} = \left(\frac{k_{hi}}{\gamma_w} \frac{t_d}{m_{v_i}}\right) \cdot \frac{1}{b^2} \tag{16}$$

Eq. 14 is solved numerically using finite difference approach, discretizing the unit cell radially into a number of elements, for the appropriate boundary and initial conditions (Murali Krishna 2003).

Boundary Conditions

The material in the drains is far more permeable than the surrounding sand layer. If the effect of dilatancy of gravel drain is not considered the excess pore-water pressure in the drain is effectively zero i.e., at $r = a$ or $R = a/b$, $u = 0$ or $W = 0$. Granular piles tend to dilate under undrained conditions, and develop negative pore pressures which are estimated in a manner very similar to the estimation of positive pore pressure in loose sand deposits (Madhav and Arlekar 2000). Thus the pore pressure at $r = a$ is,

$$W_{g\ at\,r=a} = \left(\frac{u_g}{\sigma'_o}\right)_{at\,r=a} = -d_c\left(\frac{2}{\pi}\text{arc sin}\left(\frac{N}{N_l}\right)^{1/2\alpha}\right) \quad (17)$$

where d_c is a constant that depends on the degree of dilatancy of the granular pile material and the densification achieved during the installation of granular piles.

At the outer boundary of the unit cell, due to symmetry, rate of change of porewater pressure in the radial direction is zero, i.e., at $r = b$ or $R = 1$,

$$\frac{\partial u}{\partial r} = 0 \text{ or } \frac{\partial W}{\partial R} = 0 \quad (18)$$

Initial Condition

At $t = 0$ or $T = 0$, pore pressures at all the nodes in the soil are equal to the average of pore water pressure generated over the initial time period of dt (or dT), i.e., the average of pore-water pressure generated over an initial cycle interval, dN.

$$W_{g\ at\,T=0} = \left(\frac{u_g}{\sigma'_o}\right)_{at\,t=0} = \frac{1}{2}\left(\frac{2}{\pi}\text{arc sin}\left(\frac{dN}{N_l}\right)^{1/2\alpha}\right) \quad (19)$$

Limitations

The assumptions and limitations associated with the original Seed and Booker (1977) model are carried to the new model as well. The smear effect can be included through densification effect on the flow parameters (reduction in the permeability and coefficient of volume change). However, the limitations due to the assumption of infinite permeability for the drain and neglect of the possible clogging of the drain with sediment still remain.

Results and Discussion

Under the assumption of pure radial flow, the rate of pore pressure dissipation, $W = u/\sigma'_o$, with time throughout the deposit depends on the dimensionless parameters: a/b = a ratio representing the geometric configuration of the RGPs; N_{eq}/N_l = cyclic ratio characterizing the severity of the earthquake shaking in relation to the liquefaction characteristics of the sand; T_{bd}, relating the duration of the earthquake to the consolidation properties of the sand; α, a parameter characterizing the shape of the pore pressure generation curve.

α = 0.7 fits the experimental data well (Seed and Booker, 1976) and hence adopted; $R_k(r)$ & $R_{mv}(r)$, functions of radial distance, r, define the variations of coefficients of horizontal permeability and volume compressibility of the in situ soil respectively and d_c, a constant that depends on the degree of dilatancy of the granular material and the degree of densification achieved during the installation of granular piles.

‘No Densification & No Dilation’ Case for Validation

If the coefficients of permeability and volume change of the surrounding soil are unchanged and there is no dilation effect due to installation of RGP, the conditions represent the ‘No Densification & No Dilation’ case. This case in which the ambient soil has constant flow and compressibility properties (k_{hi} and m_{vi}), i.e., without any variation (i.e., R_{ka}= R_{kb}= R_{ma}= R_{mb}= 1) and d_c = 0, is identical to the problem solved by Seed and Booker (1977). The effect of *a/b* ratio on the variations of pore water pressure with time from the present study are shown in Figure 6 for cyclic ratio (N_{eq}/N_l) of 2 and a range of *a/b* (0.1 to 0.4) with T_{bd} value of 1 along with the results obtained by Seed and Booker (1977) for similar conditions but based on finite element program LARF (Liquefaction Analysis for Radial Flow). Maximum pore pressure ratio, $W_{\max}(T)$, i.e. maximum value of u/σ'_o throughout the layer is plotted against T (t/t_d). The results obtained in the present study based on finite differences agree reasonably closely with the results of Seed and Booker (1977). Small deviations discernible between the solutions may be due to the methods of solution and the time steps involved. The deviations decrease with increase in the area ratio. For cyclic ratio, (N_{eq}/N_l) equal to 2, if no drains are present (*a/b* = 0) the soil liquefies at $T_l = 1/(N_{eq}/N_l)$ i.e., at T = 0.5. For *a/b* = 0.1, initial liquefaction is deferred but eventually occurs at about T = 0.6. The liquefied state, $W_{\max}$ = 1, continues until the end of the period of strong shaking. Thereafter W_{max} decreases as pore-water pressure gets dissipated. Initial liquefaction is prevented at higher *a/b* values as the maximum pore pressure ratio decreases with increase in *a/b* value.

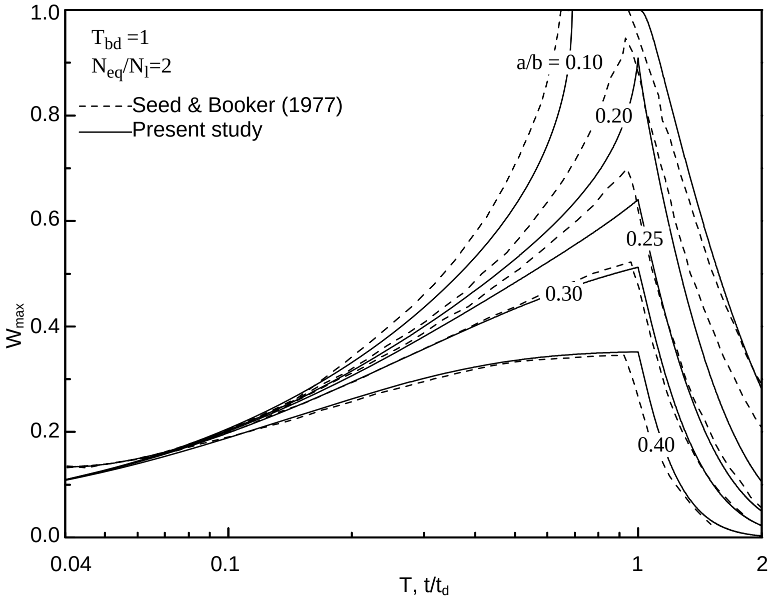

Fig. 6 Effect of Area Ratio on W_{max} for ‘No Densification & No Dilation’ Case

Densification and Dilation Effects

Densification effect, but with no dilation effect (d_c = 0), with respect to coefficient of volume change only (i.e., no change in coefficient of permeability (R_{ka}=R_{kb}= 1)) on maximum pore pressure ratio, W_{max}, is shown in Figure 7 for densification at the near end, R_{ma}, only and no modification in the coefficient of volume change at the farthest end (R_{mb}=1). Both, linear and exponential variations of $m_v(r)$ with distance are considered. A range of R_{ma} (1-0.3) for T_{bd} = 1, a/b = 0.3 and N_{eq}/N_l = 2 are considered. It can be observed that W_{max} decreases with decrease in near end densification. Maximum pore pressure ratio, W_{max}, decreased from 0.512 to 0.408 and 0.377, for linear and exponential variations respectively, for a decrease in R_{ma} from 1 to 0.3. The effect of the type of variation (linear or exponential) considered for the variation of coefficient of volume change with distance, is more for relatively lesser R_{ma} values.

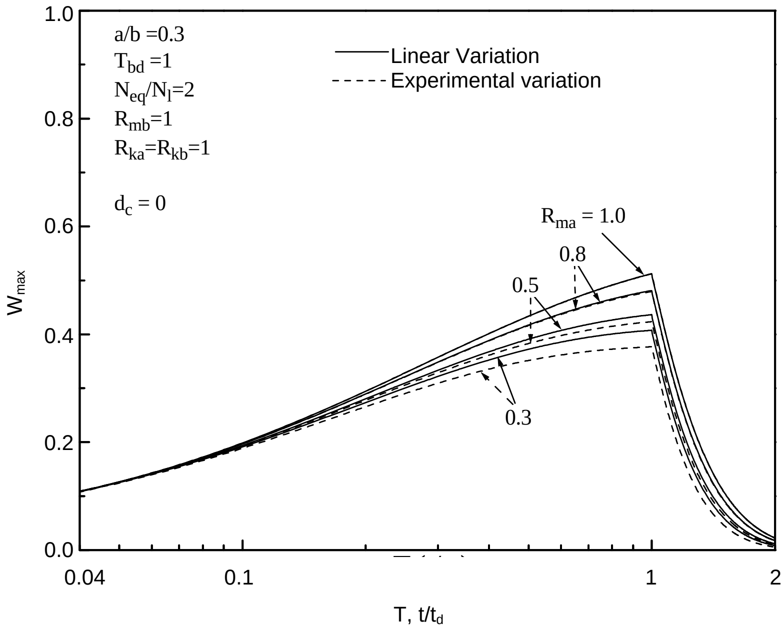

Fig. 7 Effect of R_{ma} on W_{max} for Linear and Exponential Variations

Densification effect in terms of reduction in coefficient of permeability only, i.e., no change in coefficient of volume change (R_{ma}=R_{mb}= 1), but with no dilation effect (d_c = 0), on maximum pore pressure ratio is depicted in Figure 8 with densification effect only at the near end, R_{ka}, and with no modification in the coefficient of permeability at the farthest end (R_{kb}=1). Both, linear and exponential variations are considered. A range of R_{ka} (1-0.3) for T_{bd} = 1, a/b = 0.3 and N_{eq}/N_l = 2 is considered. W_{max} increases from 0.512 to 0.650, for both the variations, for a decrease in R_{ka} from 1 to 0.8. For near end permeability decreasing by 50%, i.e., R_{ka}=0.5, the maximum pore pressure ratio reaches 1, signifying the liquefied state. Further decrease in R_{ka} reduces the time to attain the liquefaction state. Consideration of the densification effect with respect to coefficient of permeability alone is similar to the smear effect. While the maximum pore pressure ratio increases because of it, the type of variation (linear or exponential) of permeability with distance has no significant effect on the generation and dissipation of pore pressures.

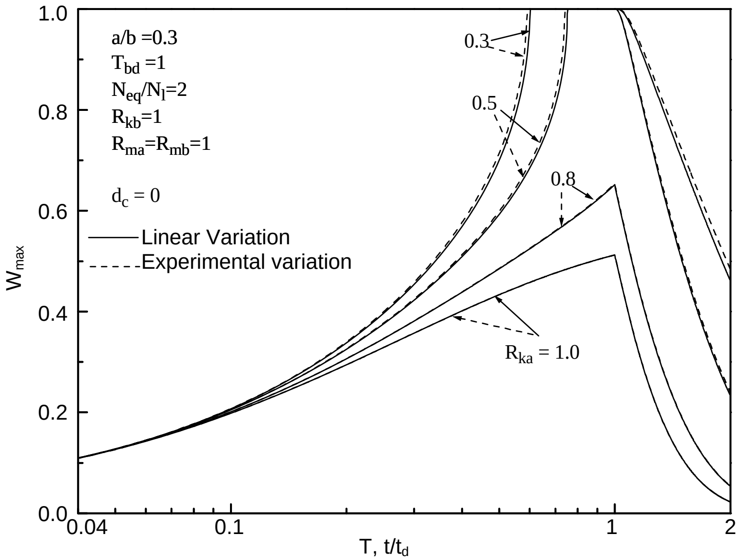

Fig. 8 Effect of R_{ka} on W_{max} for Linear and Exponential Variations

Densification effect with respect to both compressibility and permeability but without dilation effect (d_c = 0), on maximum pore pressure ratio, W_{max}, is shown in Figure 9 considering the densification effect on both the coefficients only at near end with the coefficients of permeability and volume change remaining unaffected at the farthest end (R_{kb} = R_{mb} =1). The same range of densification effect R_{ka} and R_{ma} (1-0.3) for T_{bd} = 1, a/b = 0.3 and (N_{eq}/N_l) = 2 are considered. Maximum pore pressure ratio increases from 0.512 to 0.602 for a decrease in R_{ka} and R_{ma} from 1 to 0.8, for both linear and exponential variations. W_{max} reaches 1, i.e. the ground attains liquefaction state, at T =1 for R_{ka}=R_{ma}=0.5. Further decreases in R_{ka} and R_{ma} reduces the time to liquefaction state, W_{max}=1. Very little difference exists for linear and exponential variations for R_{ka} and R_{ma} of 0.5 and 0.8, with exponential variation values being slightly less.

Effect of dilation alone, with no densification effect (i.e., R_{ka}= R_{kb}= R_{ma}= R_{mb}= 1), on maximum pore pressure ratio is presented in Figure 10. The effect of the dilation coefficient, d_c (0, 2 and 5), on the maximum pore pressure ratio, W_{max}, for T_{bd} = 1, a/b = 0.2 and N_{eq}/N_l = 2, is presented in Figure 10. The negative pore pressures generated in the dilating gravel drain reduce the possible liquefaction by permitting faster rates of dissipation of pore pressures induced. The curves for d_c equal to 2 and 5 indicate reductions in maximum pore pressures of the order of 13 and 19%, with values of W_{max} of 0.909, 0.790 and 0.737 for d_c values of 0, 2 and 5 respectively.

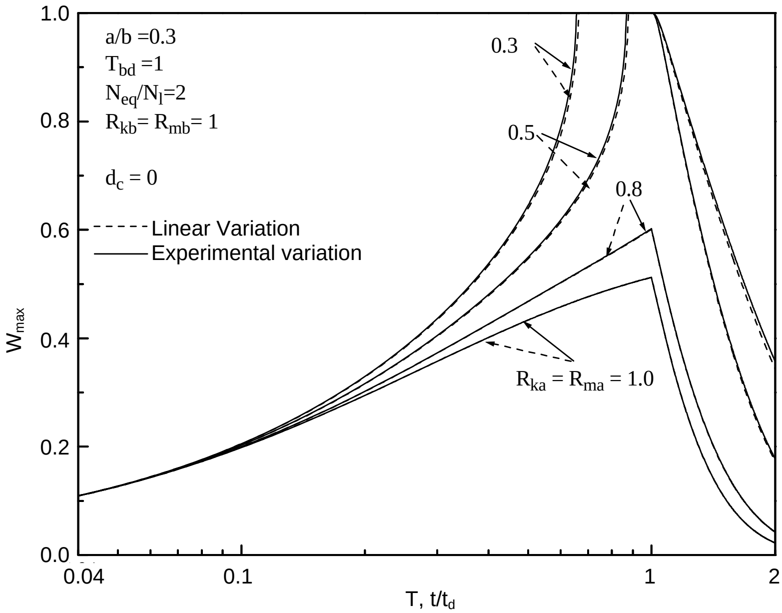

Figure 9 Effect of R_{ka} and R_{ma} on W_{max} for Linear and Exponential Variations

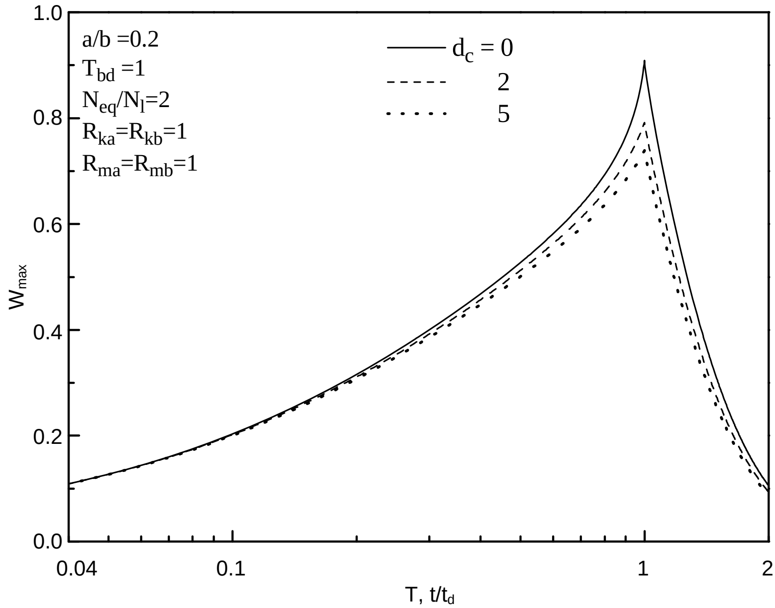

Figure 10 Maximum Pore Pressure Ratios for Different Values of Dilation Coefficient

Figure 11 provides the comparison of 'W_{max} versus T' curves for 'no dilation' (d_c = 0) and with dilation effect for d_c = 2, along with the densification effect with respect to coefficient of permeability at the near end (R_{ka} = 1, 0.8 and 0.6) and its linear variation. The dilation effect reduces the maximum pore pressure ratios by 6 and 5 % for R_{ka} values of 0.8 and 1 respectively while for no densification (R_{ka} = 1), the time to attain W_{max} = 1 state is delayed by 5 %.

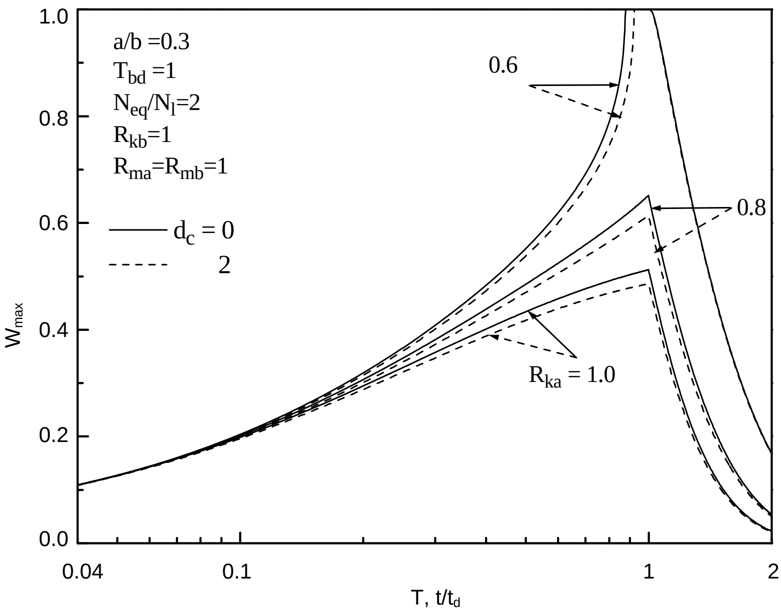

Fig. 11 Effect of Densification with respect to R_{ka} and Dilation on W_{max}

Figure 12 shows both the dilation and densification effects on W_{max}. The densification effect is considered with respect to coefficient of volume change at the near end (R_{ma} = 1, 0.8, 0.5 and 0.3) and its linear variation. The dilation effect reduces the maximum pore pressure ratios further by an amount of about 5.1 % for all R_{ma} values implying that dilation effect is not sensitive to the densification effect with respect to coefficient of volume compressibility only.

The densification effect is considered with respect to both the coefficients of volume change and permeability at the near end (R_{ma} = R_{ka} = 1, 0.8, 0.5and 0.3) and its linear variation along with dilation effect (d_c = 0 and 2) is shown in Figure 13. The dilation effect reduces the maximum pore pressure ratios by 5.0 and 5.5 % for R_{ma} =R_{ka} = 1 and 0.8 values, while for R_{ma} =R_{ka} = 0.5 and 0.3 values, the time to attain W_{max} = 1 state is delayed only slightly.

Figure 14 shows the variation of 'Maximum W_{max}' (maximum of W_{max} over the duration of t_d, the duration of earthquake) with *a/b* considering different combinations of improvement effects. Two extreme values of T_{bd} (0.5 and 2) are considered to obtain a range of variations between 'Maximum W_{max}' and *a/b*. Densification effect with respect to both the coefficients of volume change and permeability at the near end (R_{ma} = R_{ka} = 0.5) and its linear variation and the dilation effect with d_c = 2 are considered. The effect of dilation appears to be of the same order in all the cases considered in reducing the maximum pore pressures. As is to be expected, the effect of densification with respect to both reductions in permeability and volume compressibility causes an increase in the pore pressures than otherwise.

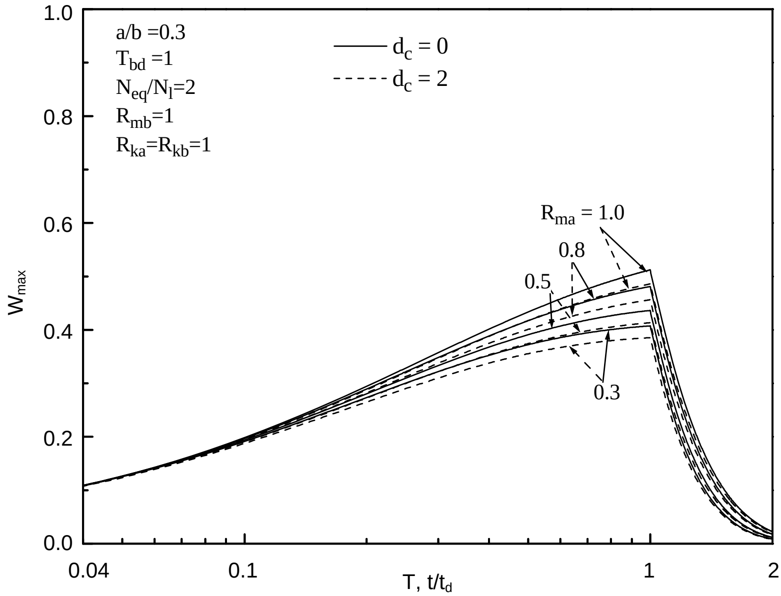

Fig. 12 Effect of Densification with respect to R_{ma} and Dilation on W_{max}

Practical applicability of the proposed model is illustrated by considering a typical practical situation with reasonable input parameters. A soil layer with γ'_w = 9.81 kN/m^3; k_{hi} = 10^{-5} m/s and m_{vi} = 7.13 × 10^{-5} kN/m^2, subjected to an earthquake that is represented by twenty four uniform stress cycles in a period of 70 s, is considered. Under undrained conditions (no granular drains or RGP) the soil would liquefy under this sequence of stress application after 12 cycles so that N_{eq}/N_l = 2. If granular piles of 0.6 m diameter at 2 m c/c (a/b = 0.3 & b = 1.0 m) spacing are installed then:

$$T_{bd} = \left(\frac{k_{hi}}{\gamma_w}\frac{t_d}{m_{vi}}\right).\frac{1}{b^2} = \left(\frac{10^{-5}\times 70}{9.81\times(7.13\times 10^{-5})}\right).\frac{1}{1.0^2} = 1.0$$

If densification and dilation effects are not considered, the maximum pore pressure ratio generated is observed to be 0.512 (Figure 13). If densification effect alone is considered in terms of reductions in the values of permeability and compressibility to 0.8 times of their initial values at the periphery of the granular pile (near end) and no effect at the farthest end, then maximum pore pressure ratio generated increases to 0.602. If dilation effect alone is considered with d_c = 2, then W_{max} will be of 0.485 while the both dilation and densification effects are considered as mentioned above the W_{max} is 0.569. It may is observed from Figure 13 that if the densification effect reduces the permeability and compressibility values significantly to 0.5 times the initial values or lower, maximum pore pressure ratios will reach 1 indicating the initiation of liquefaction even with dilation effect.

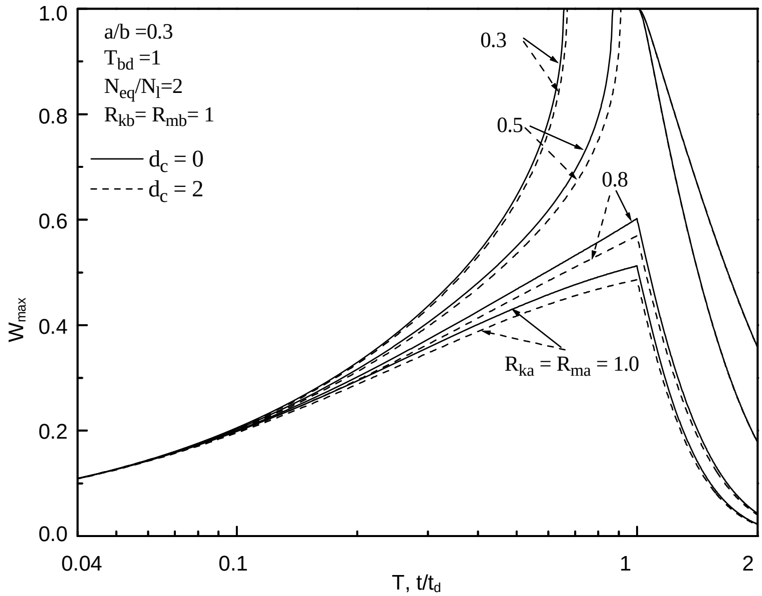

Fig. 13 Effect of Densification with respect to R_{ka} & R_{ma} and Dilation on W_{max}

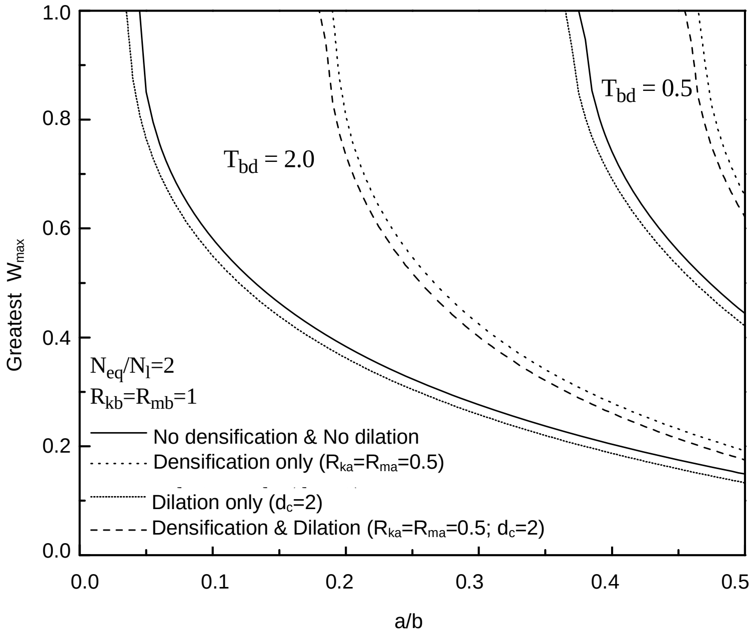

Fig. 14 Variation of 'Greatest W_{max}' with a/b: Different Effects

Hence, it is recommended that modified permeability and compressibility values and the corresponding dilation effect should be considered for the effective analysis of pore pressure generation and dissipation. Similar mechanisms operate in the ground treated with vibrated granular piles that are installed by vibro-replacement/displacement process. In this case the densification caused by vibration may affect the flow and compressibility parameters.

Summary and Conclusions

Liquefaction analysis, using modified pore pressure generation and dissipation model, for the ground treated with gravel drains (stone columns) considering the effect of installation in densifying the ground and its dilation effect is proposed. Both the coefficients of volume change and permeability are considered to be affected due to densification as they decrease because of densification. This densification decreases with distance from the granular pile. Densification effect on the coefficient of volume change is positive in that the maximum induced pore water pressure ratios get reduced and is sensitive to the type of variation considered as pore pressure ratios are lesser for the exponential variation. Densification effect, on the coefficient of permeability alone or in addition to effect on coefficient of volume change, increases the maximum pore water pressure ratios giving a negative effect. The pore pressures ratios are not sensitive to the type of variation of permeability with distance. Densification effect on both coefficients of permeability and volume change result in either a slightly negative or positive effect depending on the degree of densification.

Dilation effect generates negative pore water pressures in the granular piles. The negative pore pressures generated in dilating gravel drain reduce potential liquefaction induced pore pressures by permitting faster rates of dissipation, and hence enhance liquefaction mitigation. The negative effect of the densification (reduction in permeability) is offset by the dilation effect thus proving the effectiveness of granular piles in liquefaction mitigation. It is recommended that the densification and dilation effects should be considered while designing granular pile/stone column treatment for liquefaction mitigation.

Notation

a	radius of the granular pile
b	radius of the unit cell
GP	granular pile
$k_h(r)$	horizontal permeability of treated ground
k_{hi}	horizontal permeability of untreated ground
$m_v(r)$	coefficient of volume compressibility/volume change of treated ground
m_{vi}	coefficient of volume compressibility/volume change of untreated ground
N	equivalent number of uniform stress cycles associated with any period of earthquake shaking

N_{eq} equivalent number of uniform stress cycles induced by earthquake

N_l number of uniform stress cycles required to cause liquefaction

R non-dimensionalized radial distance, r/b

r radial distance measured from the center of granular pile

SPT N standard penetration test number

SPT N_1 SPT N value corrected for the overburden stress of 100 kPa

T normalized time, t/t_d

t time

T_{bd} dimensionless time factor

t_d duration of earthquake

u excess hydrostatic pressure

u_g excess hydrostatic pressure generated by earthquake shaking

W or r_u pore pressure ratio

$W_{\max}$ maximum pore pressure ratio W throughout the layer at a given T

σ'_0 the initial mean bulk effective stress

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