DILATION OF GRANULAR PILES IN MITIGATING LIQUEFACTION OF SAND DEPOSITS

Madhira R MADHAV[1] And Jaswant N ARLEKAR[2]

SUMMARY

Liquefaction of loose saturated sands still remains a challenging task for geotechnical engineers. Amongst the various remedial measures available, the use of granular piles or stone columns is one of the most popular choices. The dilation of dense granular material achieves three important phenomena, viz. reinforcement, drainage and densification. The effect of dilation on reinforcing action of granular piles has been quantified earlier. In this paper, the dilation effect on drainage function of granular piles/gravel drains is studied by extending Seed and Booker model. It is shown that the effect for the ranges of parameters considered is marginal.

INTRODUCTION

Liquefaction is defined as the state existing in saturated sandy soils as they loose their shearing resistance as a result of reduction in effective stresses due to increased pore water pressure. Shearing stresses transmitted to saturated sands during seismic conditions are one of most frequent and severe cause of liquefaction of ground. This phenomenon is known to amplify earthquake damage in alluvial deposits and reclaimed ground in lowlands. Measures to mitigate damage from liquefaction can be categorised into 1. Treat, improve or modify the soil; 2. Strengthen structures or make them flexible enough to withstand the effects of liquefaction and 3. Prepare auxiliary or supplementary facilities.

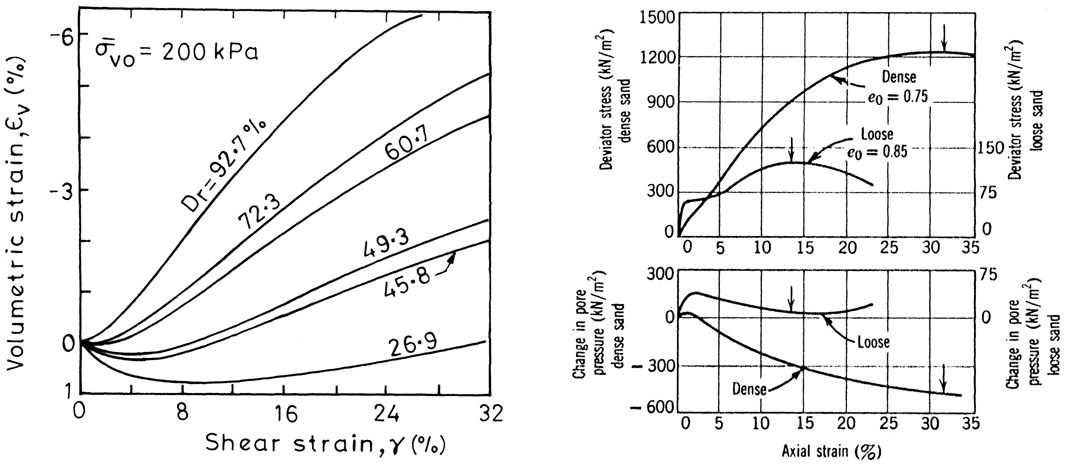
Prevention of possible damage from liquefaction by improving in situ ground or soil conditions is considered the best strategy and is the most preferred choice. Resistance to liquefaction can be improved by 1. Increasing the density, 2. Modifying the grain size distribution, 3. Stabilising the soil structure and 4. Reducing the degree of saturation of the soils, 5. Dissipate the excess pore pressures generated, 6. Intercept the propagation of excess pore pressures, etc. Provision of gravel drains/granular piles/stone columns is one most commonly adopted ground treatment methodology has proved its effectiveness in many instances (Mitchell and Wentz, 1991). In this paper, the effect of dilation of granular material in enhancing the effectiveness of the above method is modelled and quantified.

DILATION OF GRANULAR MATERIALS

Figure 1(a) (from Vaid et al., 1981) is a typical example of stress-strain behaviour of granular material under drained conditions in a simple shear test at different vertical stress conditions. While initially loose samples undergo volume decrease, dense samples experience volume increase (dilation) during shearing. The rate of dilation increases with relative density. The response of saturated sand under undrained triaxial conditions (after Leonards, 1962) can be seen in Figure 1(b). While positive pore pressures are generated in loose sands, generation of very high negative pore pressures can be observed due to suppression of the tendency for dilation. A moderately dense sand (eo=0.75) has developed a negative pore pressure of 300 kPa under a confining stress of 69 kPa.

[1] *Professor, Department of Civil Engineering, Indian Institute of Technology Kanpur Email:madhav@iitk.ac.in*

[2] *Ph.D. Candidate, Department of Civil Engineering, Indian Institute of Technology Kanpur*



(a) (b)

Figure 1: (a) Stress-strain behaviour for granular material under drained conditions (Vaid et al., 1981); (b) Response of saturated sand under undrained triaxial conditions (after Leonards, 1962).

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. A liquefaction resistance plot similar to that proposed by Seed (1976) but based on dilation angle rather than corrected blow count was presented by Vaid et al., 1981. According to the above results, liquefaction is unlikely to occur regardless of the stress level provided the relative density is greater than 75%, the corrected dilation angle is more than 160 or the corrected SPT N is larger than 20.

Granular drains facilitate rapid dissipation of pore pressures built up during seismic loading and thus prevent liquefaction of loose granular deposits. Seed and Booker (1976), Iai and Koizumi (1986), and Onoue (1987) analyse ground treated by granular drains/ stone columns for no and constant drain resistance cases and provide design charts.

Baez and Martin (1991) present an evaluation of the relative effectiveness of stone columns for the mitigation of liquefaction of soil. They identify three mechanisms through which stone columns/gravel drains/granular piles can mitigate the potential for liquefaction, viz., densification of soil surrounding them, reinforcing the soil by stiff elements and carry higher shear stresses and through drainage for rapid dissipation of pore water pressure built up during seismic events. They also describe tests on footings on soil reinforced with stone columns, which have been used to calibrate a finite element program. The most interesting results obtained are 1. The stone columns experience an increase in effective stress simultaneously with the development of negative pore pressures (Sasaki and Taniguchi 1982, also report similar observation) and 2. The influence of stone column extended up to elements two stone column diameters away. The finite element analysis indicates a redistribution of the load towards the stone columns (reinforcement effect).

Pastana et al. (1998) extend these analyses to include the following cases: (i) perfect drain analysis with ground water level (GWL) at or below ground level (GL); (ii) drain with finite horizontal and vertical permeabilities and with storage capacity; and (iii) new prefabricated drain consisting of a geopipe with an option of being wrapped in a geofabric to prevent clogging. Resistances to flow in both horizontal and vertical directions are incorporated. For GWL at GL, solutions obtained by FEM for cases (i) and (ii) agree with Seed and Booker (1976), Iai and Koizumi (1986), and Onoue (1988) respectively. If the initial water level is at depth, it rises within the drain (storage effect) due to flow from the surrounding ground, leading to uniform 'back pressure' in the drain, which retards subsequent entry of water.

For initial water level 1.0 m below the GL and for drain spacing, d_e/d_w, equal to 5.0 (d_e and d_w are the diameters of unit cell and the gravel drain respectively), complete liquefaction is predicted at cyclic ratio of 1.85, 3.0, 5.6 and beyond 6.0 for k_w/k_e (the ratios of permeabilities of the drain and the soil) values of 50, 100, 250 and 500 and greater respectively. This result is fairly close to Seed and Booker (1976) who predict that liquefaction is unlikely if $k_w/k_e \geq 200$.

For drain spacing, d_e/d_w, closer than 5.0, total liquefaction is unlikely even if the initial GWL is 1.0 m below the GL. For a perfect drain, i.e. no losses in the drain, while the maximum average pore pressure ratio of only 0.35 is predicted for d_e/d_w ratio of 6.0 for GWL at GL, liquefaction is predicted at a cyclic ratio of 3.3 for GWL at 1.0 m below GL.

The efficacy of gravel drains in reducing liquefaction can further be improved by enlarging their diameter at the top to provide a 'pseudo-reservoir' for temporary storage of water accumulated during an earthquake. Even a 40% increase in the 1.0 m diameter of the stone column is adequate to prevent rapid increase of maximum average pore pressure ratio with cyclic ratios up to 6.0 for d_e/d_w = 5.0.

Baez and Martin (1993) assume compatibility of shear strains between the stiff gravel drains/stone columns and the soft liquefiable soil. The shear stresses in the two components would then be proportional to their respective shear moduli. The flexural response is more likely according to Goughnour and Pestana (1998) due to the large slender ratio of stone columns. They conclude that the present design approach, which does not include the flexural response, could result in unconservative assessment of the reinforcement effect of stone columns.

MECHANICAL EFFECT OF DILATION OF GRANULAR PILES

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 dilatant nature of 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 or partially drained state that exists within a gravel drain/granular pile during a seismic event is modelled and analysed in the following section.

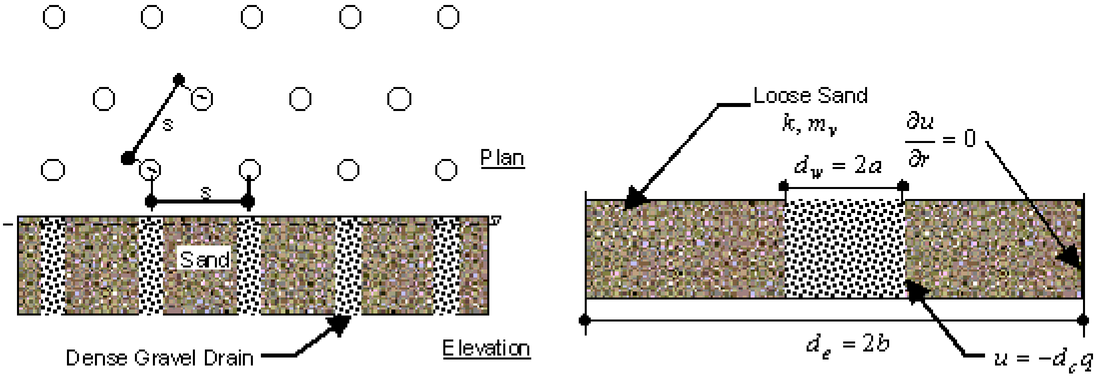
MODELLING OF LIQUEFACTION RESISTANCE OF DILATANT GRANULAR PILE

The work follows from Seed et al. (1975) who observe that if the pore water pressures generated in a soil mass by cyclic loading can to some extent be dissipated as they are created, the potential for liquefaction may get reduced. One of the frequently resorted methods for improving loose sand deposits susceptible to liquefaction, is to install granular piles or gravel drains in a regular array (triangular or square). See Figure 2(a). The granular piles not only reinforce the ground but also function as drains. The response of the treated ground can be assessed by considering a unit cell as shown in Figure 2(b), consisting of a granular pile of diameter, 2a, with its zone of influence, 2b (=cS), where c is a constant and S is the spacing between the drains. c=1.05 and 1.13 for triangular and square patterns of arrangement respectively.

For flow into gravel drain, assuming purely radial flow, and constant coefficient of permeability (k_h) and coefficient of volume compressibility (m_{v3}), the governing equation for the phenomenon can be written as (Seed and Booker, 1977):

$$\frac{k_h}{\gamma_w m_{v3}}\left(\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\frac{\partial u}{\partial r}\right)=\frac{\partial u}{\partial t}-q \qquad (1)$$

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, $q = (\partial u_g / \partial N)(\partial N / \partial t)$, u_g is the pore pressure generated and N the number of cycles. Seed et al. (1975b) propose a relation between u_g and N as



(a) (b)

Figure 2: (a) Arrangement of granular piles; (b) Unit cell.

$$\frac{u_g}{\sigma'_o} = \frac{2}{\pi} \arcsin\left(\frac{N}{N_l}\right)^{1/2\alpha} \tag{2}$$

where σ'_g is the initial mean bulk (vertical) effective stress, N_l the number of cycles required for liquefaction and α - an empirical parameter (typical value of 0.7). Differentiating Eq. 2 , one gets

$$\frac{\partial u_g}{\partial N} = \frac{\sigma'_o}{\alpha \pi N_l} \frac{1}{\sin^{2\alpha-1}\left(\frac{\pi}{2} r_u\right) \cos\left(\frac{\pi}{2} r_u\right)} \tag{3}$$

where $r_u = u/\sigma'_o$ = the pore pressure ratio. The irregular cyclic loading induced by an earthquake is converted (Seed et al. 1975a) to an equivalent number, N_{eq} ,of uniform cycles at a stress ratio, τ_h / σ'_o, occurring over a duration of time t_d . Hence,

$$\frac{\partial N}{\partial t} = \frac{N_{eq}}{t_d} \tag{4}$$

Seed and Booker (1977) suggest that in using these results, it must be note that the rate of pore pressure generation $\partial u_g / \partial N$ depends on the previous cyclic history of the soil which may be represented approximately by the accumulated pore pressure, u, at time, t. This approach facilitates consideration of the past history of strain cycles with some degree of approximation. Eq. 1 is solved for the following boundary conditions by Seed and Booker (1975) who do not consider the effect of dilatancy of gravel drains. At $r = a$, $u = 0$ (gravel drain is infinitely permeable) and at $r = b$, $\partial u / \partial r = 0$ (the outer boundary of the unit cell).

In this paper, since the granular piles tend to dilate under undrained conditions, they develop negative pore pressure, which is estimated in a manner very similar to the estimation of positive pore pressure in loose sand deposits. The pore pressure at $r = a$ is then

$$u_g / \sigma'_o \big|_{r=a} = -d_c (2/\pi) \arcsin(N/N_l)^{1/2\alpha} \tag{5}$$

where d_c is a constant that depends on the degree of dilatancy of the granular material and depends on the densification achieved during the installation of granular piles. Eq. 1 is non-dimensionalised with respect to pore pressure and solved using the implicit finite difference method. The non-dimensionalised form of Eq. 1 is:

$$T_{bd}\left(\frac{\partial^2 w}{\partial R^2}+\frac{1}{R}\frac{\partial w}{\partial R}\right)=\frac{\partial w}{\partial T}-\frac{\partial w_g}{\partial N}N_{eq} \tag{6}$$

where, $w = u/\sigma_o'$ is the normalised pore pressure, which is same as the porepressure ratio r_u (Seed and Booker, 1977), $T_{bd} = \left(\frac{k_h}{\gamma_w}\frac{t_d}{m_{v3}}\right)\frac{1}{b^2}$, $R = r/b$, $T = t/t_d$, t_d is the total duration of earthquake and $w_g = u_g/\sigma_o'$ as defined by Eq. (2).

RESULTS

Numerical results are obtained by discretising the zone of soil draining in to the gravel drain and solving Eq. 6.

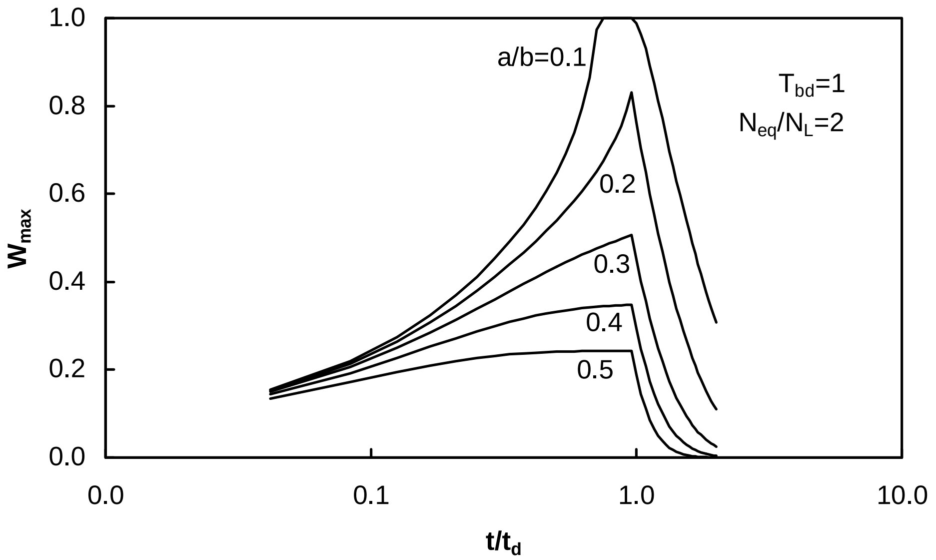


Figure 3: Maximum pore pressure ratio for no dilation of gravel drain material.

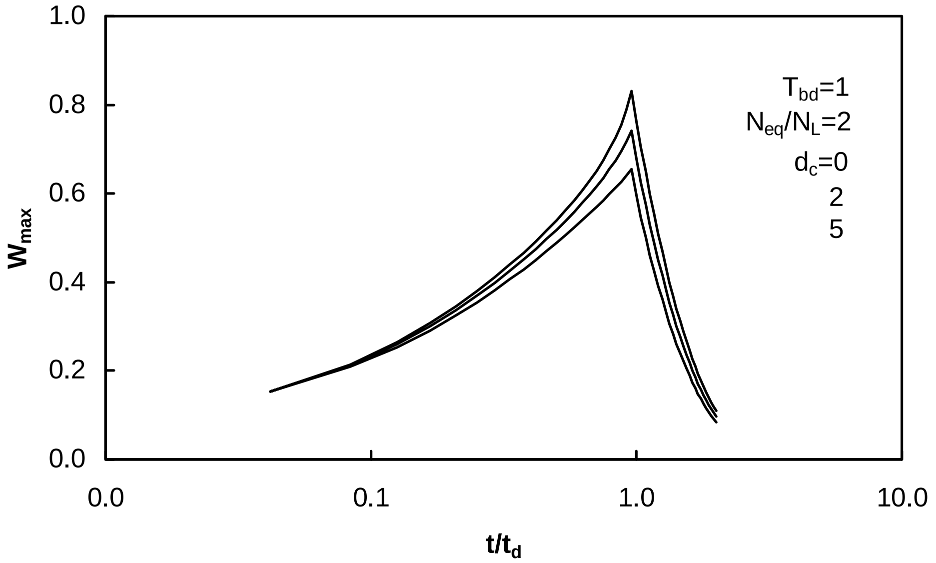


Figure 4: Maximum pore pressure ratio for different values of dilation coefficient.

The results are compared with Seed and Booker (1977) for no dilation of gravel drain material (Figure 3). The results from the present analysis agree closely with Seed and Booker (1977) for a/b values of 0.2 and greater. Small differences are discernible for a/b less than 0.2 possibly due to larger time step considered in the present analysis.

The effect of the dilation coefficient, d_c, on the maximum pore pressure generated for $T_{bd} = 1$, $a/b = 0.2$ and $N_{eq}/N_l = 2$, is presented in Figure 4. The negative pore pressures generated in dilating gravel drain reduce liquefaction induced pore pressures by permitting faster rates of dissipation. The curves for d_c equal to 2 and 5 indicate reductions in maximum pore pressures of the order of 11 and 17%. The maximum effect is obviously felt at t/t_d equal to 1.0.

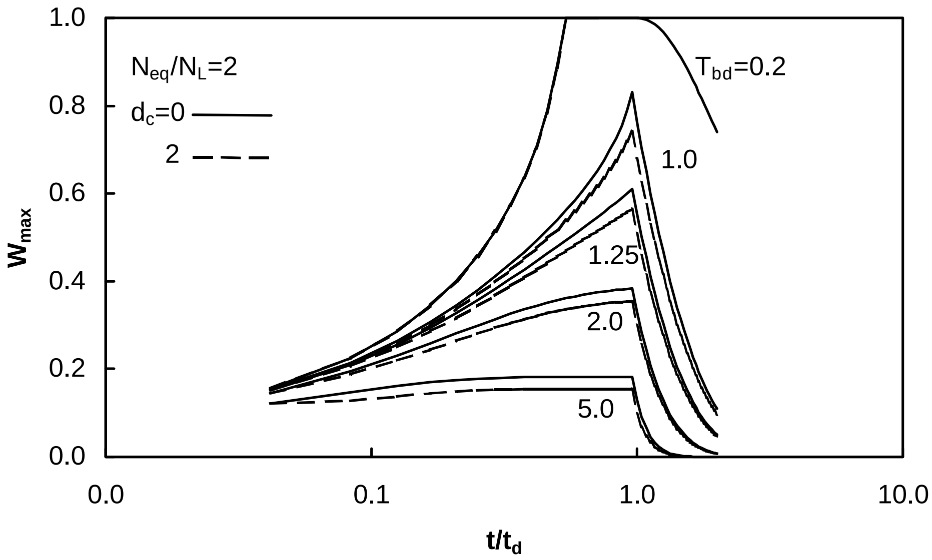


Figure 5: Effect of dilation for different values of T_{bd}.

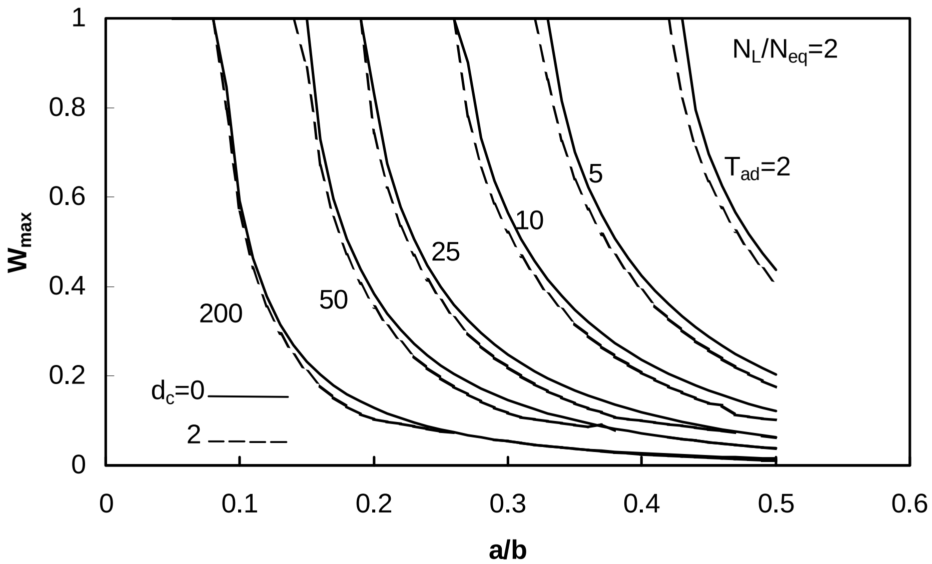


Figure 6: Effect of dilation for different values of T_{ad}.

A comparison (Figure 5) of curves for different values of T_{bd} (duration of seismic event) and for $d_c = 0$ and 2, the effect of dilation appears to be of the same order in all the cases considered. However, the effect appears marked for lower values of T_{bd} as the peak values are reduced significantly.

The variation of maximum pore pressure at the end of the seismic event with the ratio a/b is depicted in Figure 6. In this figure $T_{ad} = \left(\frac{k_h}{\gamma_w} \frac{t_d}{m_{v3}} \right) \frac{1}{a^2} = T_{bd} \left(\frac{b}{a} \right)^2$. Increasing values of a/b imply closer spacing of gravel drains. As is to be expected, a dilating gravel drain would cause the maximum pore pressure to be less than otherwise.

REFERENCES

1. Baez, J.I. and Martin, G.R. (1991), "Quantitative evaluation of stone column technique for earthquake liquefaction mitigation," *10th World Conference on Earthquake Engineering*, 1477-1483.

2. Baez, J.I. and Martin, G.R. (1993), "Advances in the design of Vibro-systems for improvement of liquefaction resistance," *Proceedings of the Symposium on Ground Improvement*, Canadian Geotechnical Society, Vancouver.

3. Goughnour, R.R. and Pestana, J.M. (1998), "Mechanical behaviour of stone columns under seismic loading," *2nd International Conference on Ground Improvement Techniques*, Singapore, pp.157-162.

4. Leonards, G.A. (ed.), (1962), *Foundation Engineering*, McGraw-Hill, New York.

5. Mitchell, J.K. and Wentz, F.J. (1991), "Performance of improved ground during the Loma Prieta earthquake," UCB/EERC Report No. 91/12, *EERC and Department of Civil Engineering, Berkeley*, California.

6. Pestana, J.M., Hunt, C.E., Goughnour, R.R. and Kammerer, A.M. (1998), "Effect of storage capacity on vertical drain performance in liquefiable sand deposits," *2nd International Conference on Ground Improvement Techniques*, Singapore, pp.373-380.

7. Poorooshasb, H.B., Holubec, I. and Sherbourne, A.W. (1966), "Yielding and flow of sand in triaxial compression," Canadian Geotechnical Journal, 3:179-190.

8. Poorooshasb, H.B. and Madhav, M.R. (1985), "Application of rigid plastic dilatancy model for prediction of granular pile settlements," *4th International Conference on Numerical Methods in Geomechanics*, Nagoya, Japan, 3:1805-1808.

9. Sasaki, Y. and Taniguchi, E. (1982), "Shaking table tests on gravel drains to prevent liquefaction of sand deposits," *Soils and Foundations*,. 22, No. 3, pp.1-14.

10. Seed, H.B. and Booker, J.R. (1977), "Stabilisation of potentially liquefiable sand deposits using gravel drains," *Journal of Geotechnical Engineering*, ASCE, 103(7), pp.757-769.

11. Seed, H.B., Idriss, I. M., Makdisi, F. and Bannerjee, N. (1975a), "Representation of irregular stress time histories by equivalent uniform stress series in liquefaction analyses," UCB/EERC, Report No. 75-29, *EERC and Department of Civil Engineering, Berkeley*, California.

12. Seed, H.B., Martin, P.P. and Lysmer, J. (1975b), "The generation and dissipation of pore water pressures during soil liquefaction," UCB/EERC, Report No. 75-26, *EERC and Department of Civil Engineering, Berkeley*, California.

13. Vaid, Y., Byrne, P.M. and Hughes, J.M.O. (1981), "Dilation angle and liquefaction potential," *Journal of Geotechnical Engineering*, ASCE, 103(7), pp.1003-1008.

14. Van Impe, W.F. and Madhav, M.R. (1992), "Analysis and settlement of dilating stone column reinforced soil," *Austrian Geotechnical Journal*, 137:114-121.