5ICEGE
5th International Conference on Earthquake Geotechnical Engineering
January 2011, 10-13 | Santiago, Chile
Paper No. ARABO

NUMERICAL SIMULATION OF DRAIN PERFORMANCE IN LIQUEFIABLE SOILS

George BOUCKOVALAS [1], Vasiliki DIMITRIADI [2] Yannis TSIAPAS[3], Alexandra TSIOULOU[4]

ABSTRACT

Modern approaches for the design of drains for liquefaction mitigation are based on the work of Seed and Booker (1977) for infinitely permeable drains and uniform soil with purely horizontal drainage. More recently, Bouckovalas et al. (2009) revisited this seminal work, and showed that it is systematically conservative by overlooking the "shake-down" effects in plastic strain or excess pore pressure accumulation exhibited during cyclic loading of cohesionless soils. After reviewing the basic assumptions and results of the revised methodology of Bouckovalas et al. (2009), this paper presents results from 3-D numerical analyses of the excess pore pressure response of a thin liquefiable sand layer improved with gravel drains. The numerical predictions verify the revisions introduced to the original Seed & Booker (1977) formulation, but also reveal a second source of considerable conservatism hidden into the conventional methods of drain design: the coefficient of volume compressibility $m_{v,3}$, as proposed in the original methodology and adopted in relevant seismic codes world wide, is grossly overestimated leading to a rather delayed reaction of the drains to the earthquake-induced excess pore pressure build up.

Keywords: Earthquakes, liquefaction, drains, numerical analyses

INTRODUCTION

Installation of gravel or prefabricated drains is among the most commonly used methods for earthquake-induced liquefaction mitigation in non cohesive soil deposits. This is mainly due to the dual function of drains, namely: (a) the increase in the liquefaction resistance of the surrounding sand by increasing its initial density, an effect most pronounced when using gravel drains, and (b) the triggering of radial drainage, allowing the partial dissipation of the earthquake generated excess pore pressures from the sand layer towards the more permeable gravel pile. Seed & Booker (1977) were the first to propose a simple, still realistic mathematical formulation of the problem, leading to user friendly design charts which are still part of a number of seismic codes worldwide (e.g. JGS, 1998, USACE, 1999, INA, 2001). Their methodology is based on the following assumptions: (a) the flow from the uniform ground towards the gravel pile is considered purely axisymmetric, (b) the overall stiffness of the improved ground is not affected by the drains, and (c) the gravel pile is considered infinitely permeable compared to the surrounding ground. In practical terms, the Authors suggest that this condition is satisfied when the drains are at least 200 times more permeable than the surrounding natural soil.

Since Seed & Booker (1977), many researchers have managed to improve some of the aforementioned simplifying assumptions and increased the accuracy of the analytical predictions. One of the most

[1] Professor, Department of Civil Engineering, NTUA, e-mail: gbouck@central.ntua.gr
[2] PhD Candidate, National Technical University of Athens e-mail: valiaciv@gmail.com
[3] M.Sc. Student, University of Texas at Austin, U.S.A.
[4] M.Sc. Student, University of California at Berkeley, U.S.A.

noticeable studies was performed by Onoue (1988), who inserted a new dimensionless design parameter, also denoted as drain resistance L. Drain resistance is defined as a function of the ratio of the permeability coefficients of the natural soil (k_s) over that of the drain material (k_d) and practically decreases the dissipating capacity of the gravel drain. The new design charts proposed by Onoue (1988), not only incorporate drain resistance as a separate design parameter, but also consider the case of combined horizontal and vertical (towards the surface) dissipation of excess pore pressures, in the case where the improved soil layer is not covered by an impermeable cap (e.g. a clay layer). At the same time, Matsubara et al. (1988) and Iai & Koizumi (1986) also studied the drain resistance problem and proposed similar design solutions, with minor differentiations of the core methodology. Recent studies by Madhav & Adapa (2007) focus on the effects of the radial ground densification variability and also examine possible dilation effects of the drain material in the event of a seismic shaking. Apart from the above analytical or semi-analytical solutions, Pestana et al. (1997, 1998) devised a general Finite Element solution algorithm (FEQDrain) which, in addition to the cases treated by Seed and Booker (1977) and Onoue (1988), provides an improved simulation of prefabricated drain performance. In particular, Manning's equation may be used to account for the relatively high discharge capacity of such drains, while allowance is made for water storage within the drain.

Papadimitriou et al. (2007) have pointed out the conservatism of the Seed & Booker (1977) design charts, through well established numerical analyses and set the background for the revision of their basic assumptions. Bouckovalas et al. (2009) suggested that one of the basic assumptions in the Seed & Booker (1977) theory, i.e. the relationship describing the rate of excess pore pressure build up in the improved ground does not account for the "shake-down" response characterizing the accumulation of plastic strains during cyclic loading of cohesionless soils and may thus lead to overrated predictions of excess pore pressures. Thus, they modify the original formulation by Seed and Booker (1977) and propose new design charts for gravel drain design, under the limiting assumptions of the original formulation.

In the following paragraphs, the basic assumptions and results of the revised formulation of Bouckovalas et al. (2009) is initially reviewed. In the sequel, the performance of gravel drains within a thin liquefiable sand layer is simulated in a series of 3-D parametric numerical analyses, which employ an advanced bounding surface plasticity model for the effective stress response of sands during cyclic loading, implemented to the Finite Difference code FLAC3D (Itasca 2006). The numerical predictions are qualitatively, compared to the revised analytical predictions in order to verify the presence of shake-down effects. Furthermore, the coefficient of volume compressibility $m_{v,3}$ for the liquefiable soil is back-calculated from a quantitative comparison between numerically and analytically predicted excess pore pressures between the drains, developing during the phases of seismic shaking and subsequent dissipation, and then compared to the respective values recommended by Seed & Booker (1977) and also adopted in contemporary seismic codes.

REVISED RATE OF EXCESS PORE PRESSURE GENERATION

The relationship describing the rate of excess pore pressure generation in the original Seed & Booker (1977) formulation relates excess pore pressure u_g to the number of cycles with uniform stress amplitude N, as follows:

$$\frac{u_g}{\sigma'_o} = \frac{2}{\pi}\sin^{-1}\left(\frac{N}{N_l}\right)^{1/2A} \quad (1)$$

where σ'_o is the initial mean effective stress for triaxial test conditions, or the initial vertical effective stress for simple shear conditions, N_l is the number of cycles required to cause liquefaction and A is an empirical constant that has a typical value of 0.70. Differentiation of Eq. 1 with respect to the number of loading cycles N leads to the following expression for the rate of excess pore pressure build up:

$$\frac{\partial u_g}{\partial N} = \frac{\sigma_o'}{A\pi N_l} F_1 F_2 \tag{2}$$

where

$$F_1 = \frac{1}{(N/N_l)^{1-1/2A}} \tag{3a}$$

and

$$F_2 = \frac{1}{\sqrt{1-(N/N_l)^{1/A}}} \tag{3b}$$

In the original formulation, the normalized number of cycles N/N_l in Eqs. 3a and 3b has been expressed in terms of the excess pore pressure ratio r_u, implying that the rate of excess pore pressure build up is a sole function of the ever current value of r_u.

Nevertheless, in the revised formulation, it is argued that this assumption is only partially justified. Namely, F_2 is an increasing function of N/N_l, as it expresses the effect of gradually increasing cyclic strain amplitude in stress controlled cyclic liquefaction tests, and it should indeed be related to the ever current value of the excess pore pressure ratio. However, F_1 is a decreasing function of N/N_l, expressing the shake down response which characterizes plastic strain or excess pore pressure accumulation during cyclic loading. This phenomenon is more visible at the initial stages of cyclic loading, and depends primarily on the number of loading reversals (or N) as it is attributed to the evolution of the fabric of soil particles towards a more stable state.

Hence, in the revised formulation, N/N_l is replaced in terms of r_u in F_2 and in terms of time t ($N=tT$ where T is the period of cyclic loading) in F_1, leading to the following final expression for the rate of excess pore pressure build up:

$$\frac{\partial u_g}{\partial N} = \frac{\sigma'_o}{\pi A N_l} \frac{1}{\left(tT/N_l\right)^{1-1/2A} \cos\left(\frac{\pi}{2} r_u\right)} \tag{4}$$

The effect of the proposed modifications on the gravel drain performance may be appreciated from Fig. 1 which compares excess pore pressure time histories obtained with the original as well as with the revised formulation. The comparison is shown for a representative spacing ratio a/b = 0.10, four levels of shaking intensity (N_{eq}/N_l = 1 to 4) and a wide range of dimensionless time factors T_{ad} = 5 to 50, where

$$T_{ad} = \frac{k_s t_d}{\gamma_w m_{v,3} a^2} \tag{5}$$

k_s is the horizontal permeability coefficient of the soil, t_d is the duration of shaking, a is the drain radius, γ_w is the unit weight of water and $m_{v,3}$ is the coefficient of volume compressibility.

Note that the difference between the two methods increases with increasing shaking intensity, N_{eq}/N_l, and drainage capacity T_{ad}, with the revised predictions of r_u, being systematically lower compared to the original ones. It is also interesting that, in the case of successful liquefaction mitigation (i.e. $r_{u,max} < 1$), the revised solution predictions attain a peak value at the early stages of shaking and then gradually decrease. On the contrary, the original predictions consistently increase towards their peak value.

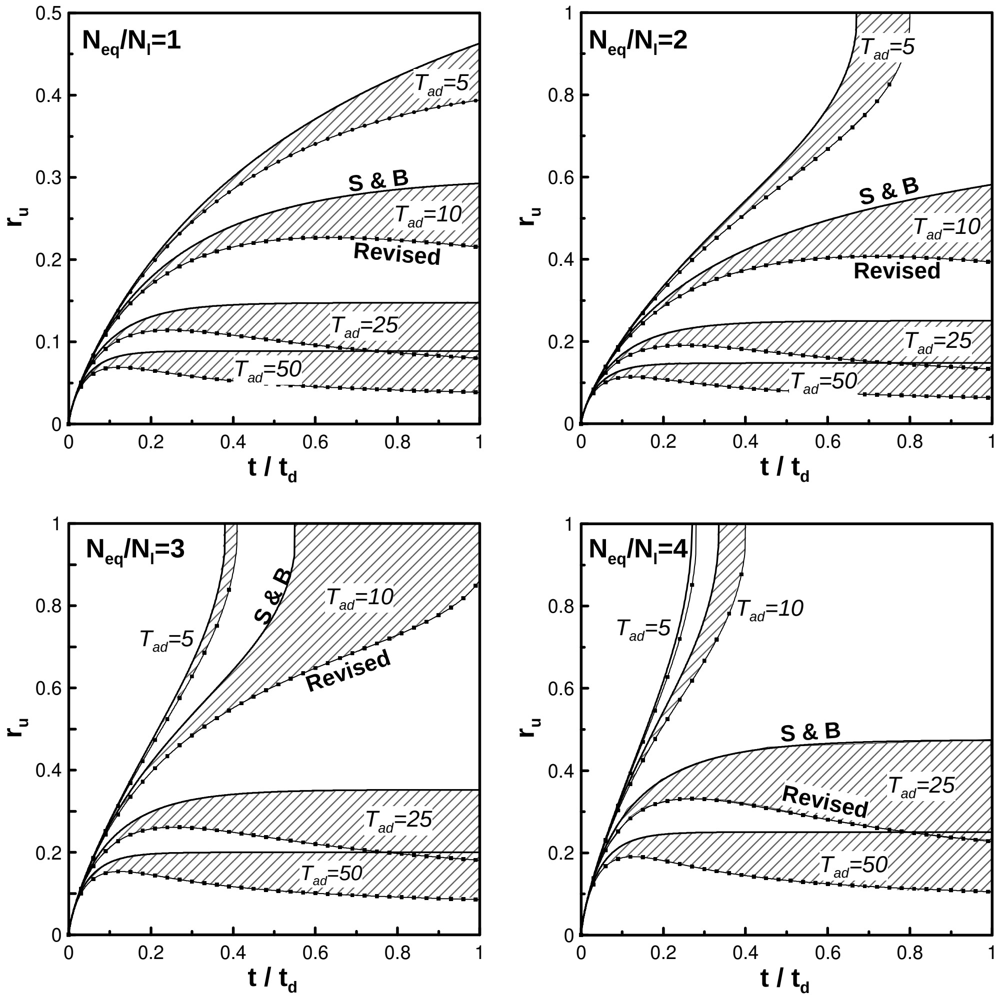


Figure 1: Comparison between predictions of excess pore pressure ratio evolution according to the revised and the original (S&B) design method.

Fig. 2 displays the original (dashed lines) and the revised (solid lines) design charts for the computation of the maximum excess pore pressure ratio, $r_{u,max}$, as a function of the gravel pile spacing ratio (*a/b* = 0 to 0.5, *b* is half of the axis to axis distance of the drains), drainage capacity (T_{ad} = 2 to 200) and shaking intensity (N_{eq}/N_l = 1 to 4). Observe that, for the same allowable maximum excess pore pressure ratio $r_{u,max}$, the revised design charts lead to a reduced *a/b* ratio, implying also a smaller replacement ratio α_s. For instance, for $N_{eq}/N_l = 2$, $T_{ad} = 25$ and $r_{u,max} = 0.40$, the original design methodology renders a spacing ratio *a/b* equal to 0.25, whereas with the revised design charts the equivalent spacing ratio is 0.22. For a triangular gravel drain configuration, where $\alpha_s = 0.91\ (a/b)^2$, the original method renders a_s=0.057 whereas the revised method renders a_s= 0.044, or about 23% less volume of drains.

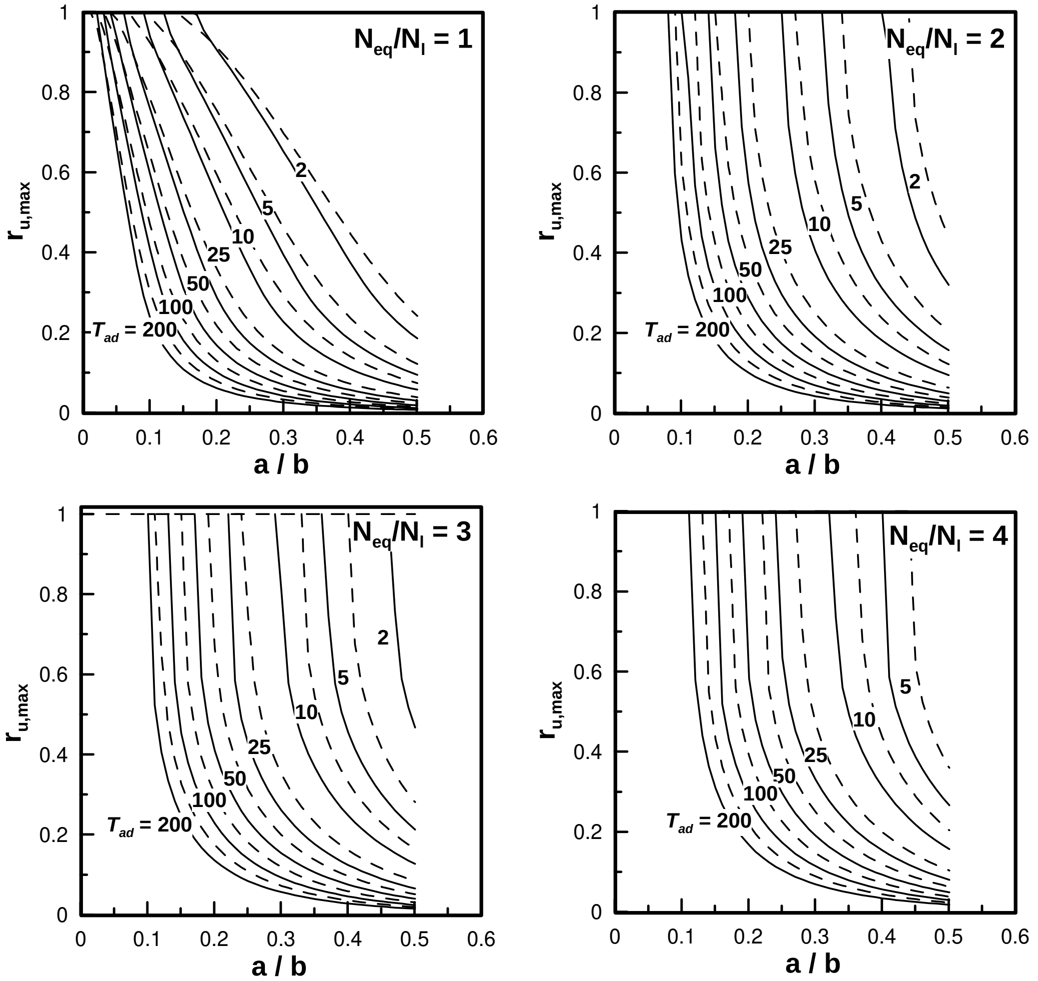


Figure 2: Peak excess pore pressure ratios ($r_{u,max}$) versus gravel drain spacing a/b and drainage potential (T_{ad}) for various intensities of seismic shaking (N_{eq}/N_l), based on the revised (solid lines) and the original Seed & Booker (1977) methodology (dashed lines).

NUMERICAL METHODOLOGY

The numerical analyses were carried out with the non-linear dynamic Finite Difference method combined with a new constitutive model which belongs to the family of the bounding surface plasticity models with a vanished elastic region, incorporating the framework of Critical State Theory. It is based on a recently proposed model (Papadimitriou et al., 2001; Papadimitriou & Bouckovalas, 2002), which has been developed to simulate the cyclic behavior of non-cohesive soils (sands and silts), under small-medium-large cyclic shear strain amplitude using a single (sand-specific) set of constants, irrespective of the initial stress and density conditions. The adoption of a vanished yield surface differentiates the new model which is used herein from the original of Papadimitriou & Bouckovalas (2002). This has led to a number of other modifications, such as (Andrianopoulos, 2006; Andrianopoulos, 2007; Andrianopoulos et al., 2010): (i) the introduction of a new mapping rule, and (ii) the modification of the existing interpolation rule. The new model was implemented in the codes FLAC and FLAC3D, for 2D and 3D dynamic analyses respectively, using their UDM capability (Andrianopoulos, 2006; Andrianopoulos et al., 2010; Karamitros, 2010). Integration of the constitutive equations is performed with a modified (two-step) Euler scheme. In order to control the global integration error in the computation of stresses, the sub-stepping technique with automatic error control by Sloan et al. (2001) was adopted.

The capabilities of the new model have been verified via extensive comparison with laboratory element test results on Nevada sand at relative densities of D_r = 40 & 60% and initial effective stresses between 40 and 160 *kPa* (Arulmoli et al, 1992), including resonant column tests as well as direct simple shear and triaxial liquefaction tests. The use of the new numerical methodology for liquefaction related boundary value problems has been validated on the basis of centrifuge experiments from the well known VELACS experimental project, simulating the one-dimensional response of a liquefiable soil layer under level and gently sloping ground conditions, as well as the response of a shallow foundation resting upon liquefiable soil (Andrianopoulos, 2006; Andrianopoulos et al., 2010; Karamitros, 2010). Furthermore, Papadimitriou et al. (2007) verified the accuracy of the numerical methodology against, relevant to this study centrifuge experiments, for the seismic response of a liquefiable soil layer improved with a cluster of gravel drains.

The present numerical study was performed with the help of the 3D grid configuration presented in Fig. 3a, having dimensions $5.60m\times1.40m\times1.40m$. It consists of a 0.5 *m* thick sand layer, encased between two clay layers, each one being 0.45 *m* thick. The two 1m diameter gravel drains are placed at a center – to – center distance of 2.80 *m*, rendering a replacement ratio *a/b* equal to 0.36. The soil between the gravel drains is discretized into a mesh with increased density around the gravel drains, aiming at depicting in detail the behavior of the soil elements in contact with the gravel piles. Initial stresses were calculated assuming geostatic conditions, and a unique buoyant saturated mass density of 1 Mgr/m^3. The liquefiable sand layer consisted of Nevada sand at relative density *Dr*=60%, initial void ratio e_o=0.661 and porosity *n*=0.398. The drains and the clay layers are assumed to be elastic, with the material properties of Table 1.

Table 1: Material properties for the drains and the clay layers

Dry density ρ_d	1.61 Mgr/m^3
Saturated density ρ_{sat}	2.00 Mgr/m^3
Bulk modulus – K	130 MPa
Shear modulus- G	60 MPa
Poisson ratio	0.3
Damping	10%

Both clay layers have a permeability coefficient of at least 500 times smaller than the sand, prohibiting the excess pore pressure dissipation in the vertical direction, thus ensuring the basic assumption for purely horizontal flow within the sand layer. In particular, the permeability coefficient of the clay layer was selected as 10^{-8} *m/sec*, whereas the permeability coefficients of the sand layer and the gravel drain material varied in each parametric analysis as shown in Table 2. The permeability coefficient of the gravel drain material was chosen as 100 times greater than that of the liquefiable sand, instead of the 200 value grossly proposed in Seed and Booker (1977) for ensuring infinitely permeable drain performance. Taking into account that the time required to complete the numerical analyses increases with decreasing soil permeability, this difference between gravel drain and sand layer permeability coefficients, was chosen after a number of trial-and-error (fine tuning) numerical analyses aimed to find the minimum drain permeability which does not affect excess pore pressure build up in the sand.

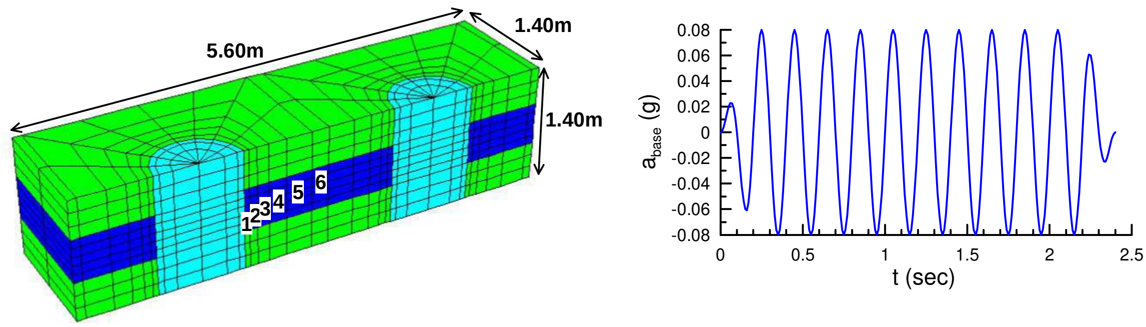
The lateral boundaries of the discretized soil-drain model were tied to one-another to ensure the same horizontal and vertical displacements of the two borders, in an attempt to reproduce the deformation pattern of a laminar box device in a centrifuge test. This type of boundary condition was assessed, in terms of deformed mesh in selected time moments, horizontal acceleration (x_{acc}) and excess pore pressure ratio (r_u) time histories, against the alternative option of free field boundaries offered by FLAC3D and was found to simulate more efficiently the actual soil vibration.

Table 2: Permeability coefficients used in the numerical analyses.

Analysis	Permeability coefficient (*m/sec*)	
	Sand	Gravel Drain
a	0 (undrained)	-
b	$1*10^{-4}$	$1*10^{-2}$
c	$5*10^{-5}$	$5*10^{-3}$
d	$2*10^{-5}$	$2*10^{-3}$
e	$1*10^{-5}$	$1*10^{-3}$
f	$5*10^{-6}$	$5*10^{-4}$

It was also ensured that the effective vertical stress in the middle of the sand layer was equal to σ'_o=100*kPa*. For that purpose, a uniform vertical pressure of 186 *kPa* and a uniform pore pressure of 93*kPa* were applied on the ground surface. This additional surcharge enhanced the numerical stability of this surficial soil deposit, while the application of a uniform pore pressure ensured continuous saturation of the soil deposit throughout the duration of the seismic shaking.

An essentially sinusoidal seismic excitation was applied at the base of the configuration consisting of 12 cycles with period T=0.20*sec* and peak acceleration 0.08*g* (Fig. 3b). A cycle of smaller acceleration amplitude was added at the beginning and at the end of the sinusoidal excitation in order to avoid erroneous numerical predictions due to the abrupt change of the imposed shaking. Note that the use of a harmonic seismic excitation is consistent with a basic assumption of the original as well as the revised analytical solutions, since the empirical relationship for excess pore pressure build up (Eq. 1) is strictly applicable to results from element laboratory tests subjected to successive cycles of constant cyclic shear stress amplitude.



(a) **(b)**

Figure 3: (a) Grid configuration of drain improved liquefiable sand encased between two impermeable clay layers. (b) Seismic excitation applied at the base of the drain-soil system.

PARAMETRIC ANALYSES

As mentioned in previous paragraphs, a set of six parametric numerical analyses was performed, varying the permeability coefficient of the liquefiable sand from 0 (undrained shaking) to $5*10^{-6}$ *m/s,* as listed in

Table 2. In all numerical analyses, the improved sand layer was first subjected to the excitation of Fig. 3b and was consequently left at rest, to dissipate the generated excess pore pressures. The initial numerical analysis for fully undrained conditions provided a practically uniform response throughout the sand layer as observed in Fig. 4, displaying the excess pore pressure time histories from elements (zones in FLAC terminology) 1 through 6 which cover the area from the drain periphery to the middle distance between two neighboring drains (Fig. 3a). To obtain an estimate of the number of cycles to liquefaction N_l, a basic input parameter for the analytical prediction of drain performance, the empirical relationship (Eq. 1) of Seed Martin & Lysmer (1975) was fitted to the numerical predictions of Fig. 4. Note that the first loading cycle of lower amplitude, (i.e. for $t = 0 - 0.2sec$) was not taken into consideration in the application of Eq. 1, so that the analytically computed liquefaction curves correspond to a uniform sinusoidal loading. A fair agreement between the analytical and the numerical predictions is obtained for required time to liquefaction between 1.2 and 1.4*sec*, corresponding to N_l=5÷6.

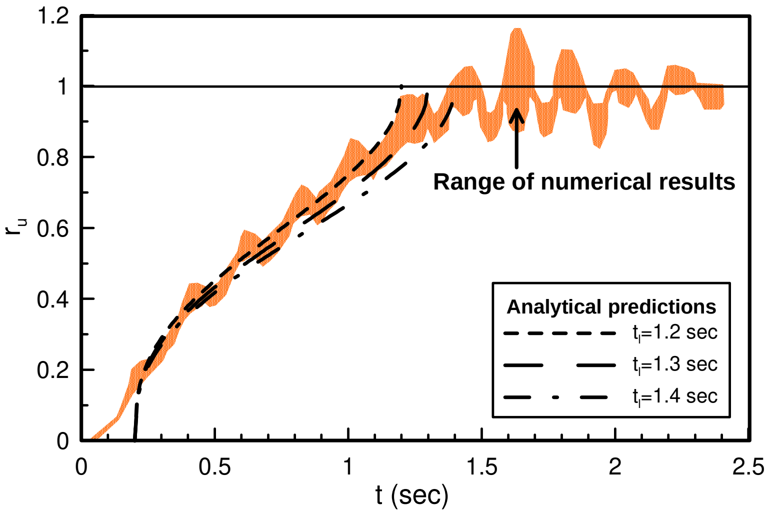


Figure 4: Comparison of the analytically computed liquefaction curves for t_l = 1.2, 1.3 & 1.4 *s* and the numerical results for undrained soil conditions.

In the sequel, the Bouckovalas et al. (2009) methodology was applied to provide a best fit to the numerically predicted excess pore pressure time histories at zone 6, i.e. at mid-distance between the drains. This was achieved by a trial-and-error procedure, where the coefficient of volume compressibility $m_{v,3}$ was varied until a good agreement was obtained for the peak excess pore pressure ratio ratio, $r_{u,max}$. The comparison between analytical and numerical predictions is shown in Fig. 5, for three different values of the required time to liquefaction: t_l = 1.2*s*, 1.3*s* and 1.4*s*. The numerical predictions are displayed with solid lines, whereas the analytical predictions are displayed with dashed lines. The first thing to observe in this figure is that all numerical predictions confirm the characteristic shape of the excess pore pressure time histories predicted when shake down effects are taken into account. Namely, they rise fast at the beginning of shaking, then they attain a peak value and consequently they decrease while shaking is still in progress. It is reminded that, according to the original Seed and Booker (1977) formulation, excess pore pressures should stabilize at the maximum value, as shown previously in Fig. 1. In addition to the above fundamental observation, which essentially confirms the validity of the revised theory, one may also appreciate the quantitative accuracy of the analytical predictions which becomes optimum for t_l=1.4*s*.

The second issue which could be addressed with the aid of the aforementioned parametric analyses was the value of the coefficient of volume compressibility $m_{v,3}$ which has to be used for the design of drains in practice. It is reminded that, according to Seed and Booker (1977), $m_{v,3}$ should be estimated from Eq. 6, on the basis of cyclic triaxial liquefaction tests, where excess pore pressure is raised under undrained conditions to a desired maximum value Δu_{max} (= 0.40 ÷ 0.60) and is consequently left to dissipate while measuring the resulting volumetric strain $\Delta\varepsilon_{vol}$:

$$m_{v,3} = \frac{\Delta\varepsilon_{vol}}{\Delta u_{max}} \tag{6}$$

Furthermore, Seed and Booker (1977) argue that $m_{v,3}$ remains more or less constant for the excess pore pressure ratios of interest, and could be therefore treated as a constant. To check the validity of the above assumptions, Fig. 6 relates the inverse of $m_{v,3}$, which resembles an equivalent Bulk Modulus of the soil skeleton, to the $r_{u,max}$ values, based on three different estimates: (a) the trial-and-error fitting of the numerical predictions with the revised theory of Bouckovalas et al (2009), (b) numerically simulated triaxial (TX) liquefaction tests, as recommended by Seed and Booker (1977), and (c) similar simple shear tests (DSS) which are admittedly closer to the stress and boundary conditions of soil elements in the free field. It is clarified that the triaxial as well as the simple shear tests were simulated with the same numerical tools and the soil parameters which were used to perform the parametric numerical analyses for the drain improved sand layer.

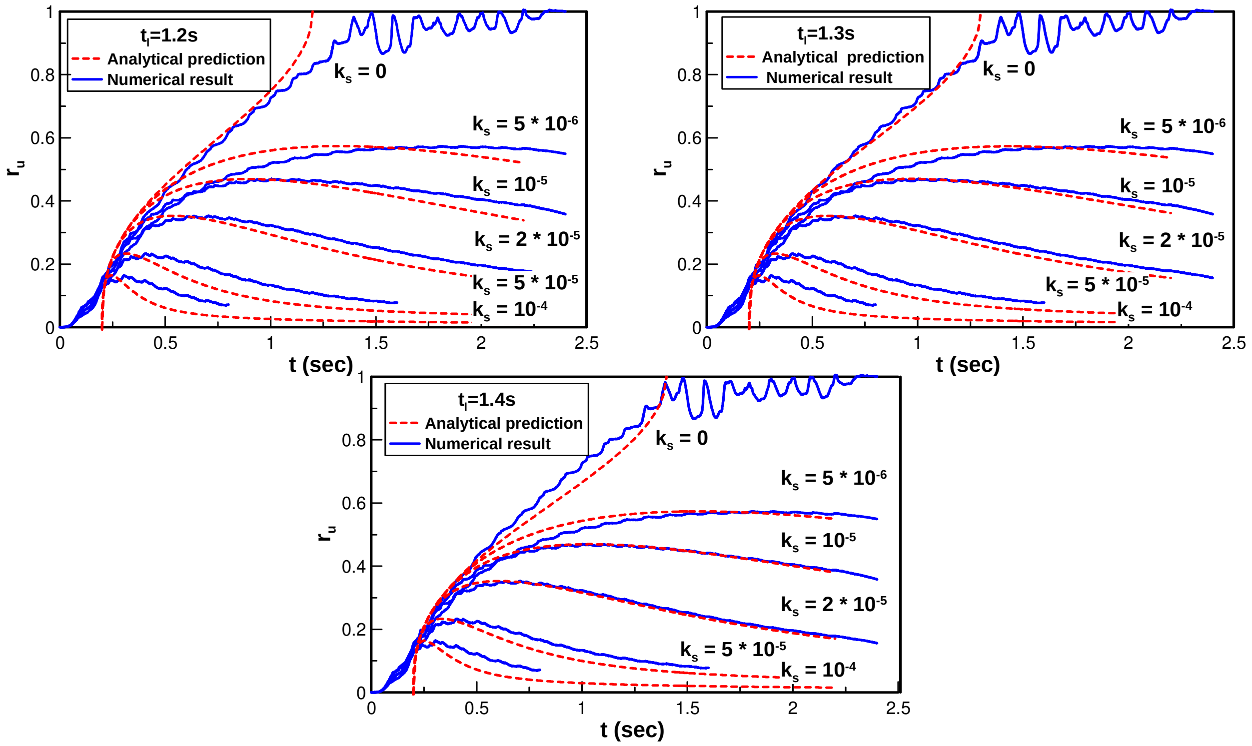


Figure 5: Best-fit comparison between analytical and numerical predictions of excess pore pressure ratio evolution, for time to liquefaction t_l = 1.2s, 1.3s and 1.4 s

It is first observed that, regardless of the method of estimation, $m_{v,3}$ varies with $r_{u,max}$. Still, this variation is relatively mild and, in the case of the back-calculated $m_{v,3}$ values, not systematic. Hence, it is indeed reasonable in practice to assume a constant mean value. However, there is a significant difference between the three sets of $m_{v,3}$ values, with the TX estimates being at least twice as large as the estimates back-calculated from fitting the numerical predictions. The DSS estimates appear more reasonable, as they over predict the back-calculated values by only 30-40%.

To appreciate the practical importance of the differences observed in Fig. 6, we recall the example presented earlier, at the end of the second chapter, in order to estimate in quantitative terms the conservatism of the original over the revised design methodology. In that case, the use of the revised methodology reduced the required replacement ratio by about 23%, from 0.057 with the original methodology to 0.044 with the revised one. If we further assume that the actual $m_{v,3}$ value is one half of that recommended by the original methodology, then the dimensional time factor T_{ad} should be increased from 25 to 50 and the respective replacement ratio should be further decreased to 0.18. Hence, the overall economy in the use of drains, considering both the shake down effects and more realistic m_{v3} estimates rises to roughly 48%!

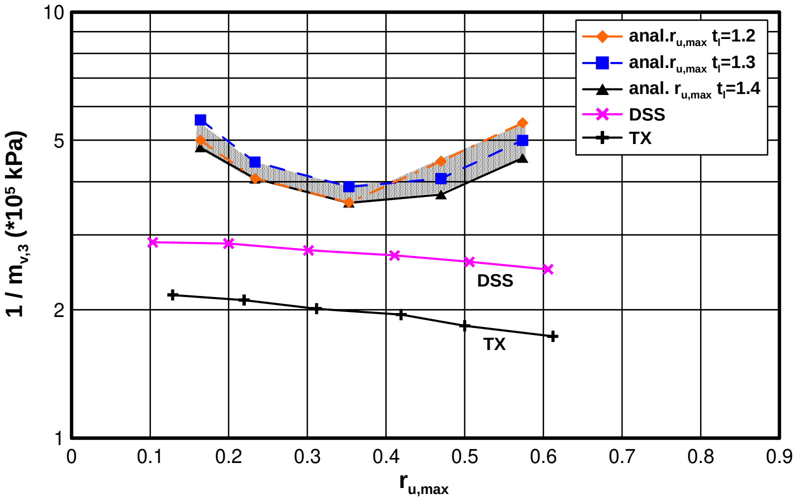


Figure 6: Comparison of $1/m_{v,3}$ values obtained from back-analysis of the numerical predictions, as well as numerically simulated cyclic triaxial (TX) and simple shear (DSS) liquefaction tests.,

CONCLUSIONS

The pioneering work of Seed and Booker (1977) for liquefaction mitigation using infinitely permeable drains has been revised by Bouckovalas et al (2009) in order to incorporate the sand fabric evolution and the associated shake down effects on the free field rate of excess pore pressure generation due to seismic loading. This paper provides a brief review of the revised design methodology, but it is mainly devoted to the presentation of 3D elastoplastic numerical analyses which aim at providing further insight to this complex drain-soil interaction problem. In summary, the findings of practical interest from this study are the following:

(a) Unless the improved ground liquefies, excess pore pressures reach a peak value at the early stages of seismic shaking and then decrease, while shaking continues. Note that the original formulation predicts a steady asymptotic increase of excess pore pressures reaching their peak value at the end of shaking.

(b) The revised predictions of peak excess pore pressure $r_{u,max}$ in the improved ground are systematically lower than the original ones. For moderate seismic shaking with $N_{eq}/N_l = 2$, T_{ad}=25 and target $r_{u,max}$ = 0.40 the revised method renders about 23% less volume of required drains.

(c) The parametric analyses, performed in order to simulate numerically the seismic response of a thin liquefiable sand layer improved with gravel drains for a wide range of sand permeability coefficients, verified with characteristic accuracy the pattern of excess pore pressure build up in the improved soil, as described in (a) above.

(d) A fairly good agreement could be established between numerical and analytical predictions of excess pore pressure build up for a more less unique set of values for the coefficient of sand compressibility $m_{v,3}$, regardless of the target value of the maximum excess pore pressure ratio $r_{u,max}$ and the assumed value of sand permeability coefficient. This finding comes in support of the basic assumption of the original as well as the revised analytical formulations which treat $m_{v,3}$ as a material constant.

(d) The values of $m_{v,3}$, back calculated from matching analytical to numerical predictions of excess pore pressure build up, are at least 50% lower than the values obtained from liquefaction triaxial tests, as proposed by Seed and Booker (1977) and also recommended in many contemporary seismic codes. Adopting this difference in the computation example referred in (b) above, reduces further the required volume of drains to about 50%.

AKNOWLEDGEMENTS

The second author has been financially supported by the State Scholarships Foundation of Greece. Yannis Chaloulos (Civil Engineer M.Sc.) provided valuable guidance for the use of FLAC 3D. These contributions are gratefully acknowledged.

REFERENCES

Andrianopoulos, K.I. (2006) “Numerical modeling of static and dynamic behavior of elastoplastic soils”, Doctorate Thesis, Department of Geotechnical Engineering, School of Civil Engineering, National Technical University of Athens (in Greek)

Andrianopoulos, K.I., Papadimitriou, A.G., and Bouckovalas, G.D. (2006) “Implementation of a bounding surface model for seismic response of sands”, Proceedings, 4th International FLAC Symposium on Numerical Modeling in Geomechanics, May

Andrianopoulos, K.I., Papadimitriou, A.G., and Bouckovalas, G.D. (2007) “Use of a new bounding surface model for the analysis of earthquake-induced liquefaction phenomena”, Proceedings, 4th International Conference on Earthquake Geotechnical Engineering, June

Andrianopoulos K. I., Papadimitriou A. G. ,Bouckovalas G. D., (2010), "Explicit integration of bounding surface model for the analysis of earthquake soil liquefaction", International Journal for Numerical and Analytical Methods in Geomechanics (in press)

Andrianopoulos K. I., Papadimitriou A. G., Bouckovalas G. D. (2010), “Bounding surface plasticity model for the seismic liquefaction analysis of geostructures”, Soil Dynamics and Earthquake Engineering, 30(10): 895-911

Arulmoli, K., Muraleetharan, K.K., Hossain, M.M. and Fruth, L.S. (1992) “VELACS: verification of liquefaction analyses by centrifuge studies; Laboratory Testing Program – Soil Data Report”, Research Report, The Earth Technology Corporation

Bouckovalas G.D., Papadimitriou A.G., Niarchos D. (2009) “Gravel drains for the remediation of liquefiable sites: the Seed & Booker (1977) approach revisited”, Proceedings, International Conference on Performance-Based Design, June

DeAlba, P., Seed, H.B. and Chan, C.K. (1976) “Sand liquefaction in large-scale simple shear tests”, Journal of the Geotechnical Engineering Division, ASCE, 102(GT9): 909-927

Iai, S. and Koizumi, K. (1986) “Estimation of earthquake induced excess pore pressure for gravel drains”, Proceedings, 7th Japan Earthquake Symposium, 679-684

INA (2001) “Seismic design guidelines for port structures”, Working Group No 34 (eds), Maritime Navigation Commission, International Navigation Association, A.A. Balkema, Roterdam, The Netherlands.

Itasca Consulting Group, Inc. (2006) “FLAC3D – Fast Lagrangian Analysis of Continua in 3 Dimensions”, Version 3.1, User’s Manual, Minneapolis: Itasca

JGS (1998) “Remedial measures against soil liquefaction”, The Japanese Geotechnical Society (ed.), A.A. Balkema, Roterdam, The Netherlands.

Karamitros Dimitrios (2010) “Development of a numerical algorithm for the dynamic elastoplastic analysis of geotechnical structures in two (2) and three (3) dimensions”, Doctorate Thesis, Geotechnical Department, School of Civil Engineering, National Technical University of Athens.

Matsubara, K., Mihara, M. and Tsujita M. (1988) “Analysis of gravel drain against liquefaction and its application to design”, Proceedings, 9th World Conference on Earthquake Engineering, Vol. III 249-254

Madhav M.R., & Adapa M.K. (2007) “Liquefaction mitigation of sand deposits by granular piles: densification and dilation effects”. Proceedings, 4th International Conference on Geotechnical Earthquake Engineering, June.

Papadimitriou, A.G., Bouckovalas, G.D. and Dafalias, Y.F. (2001) “Plasticity model for sand under small and large cyclic strains”, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 127, no. 11, 973-983

Papadimitriou, A.G. and Bouckovalas, G.D. (2002) “Plasticity model for sand under small and large cyclic strains: a multiaxial formulation”, Soil Dynamics and Earthquake Engineering, 22, (3) 191-204

Papadimitriou, A.G., Moutsopoulou, M.-E, Bouckovalas, G.D., and Brennan, A.J. (2007) “Numerical investigation of liquefaction mitigation using gravel drains”, Proceedings, 4th International Conference on Earthquake Geotechnical Engineering, Paper 1548, Thessaloniki, Greece, June 25 – 22

Pestana M. J., Hunt C., Goughnour R. (1998) “Use of prefabricated drains for the reduction of liquefaction potential”, ASCE 12th Engineering Mechanics Conference, San Diego, California, 1025 – 1028

Pestana M. J., Hunt C., Goughnour R. (1997) “FEQDrain: A finite element computer program for the analysis of the earthquake generation and dissipation of pore water pressure in layered sand deposits with vertical drains”, Report No. EERC 97-15.

Seed, H.B., and Booker, J.R. (1977) “Stabilization of potentially liquefiable sand deposits using gravel drains”, Journal of the Geotechnical Engineering Division, ASCE, 103(GT7): 757-768

Seed, H.B., Martin, P.P., and Lysmer, J. (1975) “The generation and dissipation of pore pressures during soil liquefaction”, Report No. EERC 75 – 26, Earthquake Engineering Research Center, University of California, Berkeley, CA.

Sloan, S.W., Abbo, A.J., Sheng, D. (2001) “Refined explicit integration of elastoplastic models with automatic error control”, Engineering Computations, 18 (1/2), 121 - 154

Onoue, A. (1988) “Diagrams considering well resistance for designing spacing ratio of gravel drains”, Soils and Foundations, 28(3): 160-168

USACE (1999) “Engineering and design guidelines on structure improvement for structures and facilities”, Publ. No ETL 1110-1-185, US Army Corps of Engineers, Engineering Division, Directorate of Civil Works, Washington DC, U.S.A..