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A 2-DIMENTIONAL APPROACH FOR NUMERICAL MODELING OF SEISMIC GRAVEL DRAINS IN LIQUEFIABLE GROUNDS

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ABSTRACT

Liquefaction of water saturated granular soils is one of the major risks that affect the safety and earthquake performance of infrastructure such as bridges, dams, ports, and lifelines in various parts of the world. The seismically-induced ground deformations are often the main concern when liquefaction occurs in significant zones of an earth structure or soil foundation. Recent studies including field data, centrifuge model testing and numerical investigations suggest that one of the promising measures to alleviate large earthquake-induced deformations and ground failures is by installing stone columns and/or gravel drains.

Design of such treatment scheme needs to account for a number of factors involved in a project through a parametric study. Such analysis should be carried out by using numerical modeling in a cost and time-effective manner. To do that, commonly a two-dimensional (2-D) numerical approach is used in practice; however the materials properties (i.e. mechanical and hydraulic properties) should be modified to reflect the three-dimensional (3-D) conditions. The equivalent 2-D analyses should provide comparable results especially in terms of displacements which control the design.

This paper describes the results of a coupled mechanical-hydraulic dynamic analysis carried out for a port structure founded on liquefiable ground treated with stone columns. An effective stress-based procedure was employed to analyze the excess pore water pressure generation, dissipation, and redistribution in the soil layers. Two sets of 2-D analyses using two approaches for equivalent soils parameters were carried out and the results are presented and compared.

INTRODUCTION

Earthquakes have caused severe damage to on-shore and off-shore infrastructures such as buildings, bridges, ports and terminals, dams, and lifelines, particularly where soil liquefaction was involved. Liquefaction of water saturated sandy soils is a major concern in geotechnical engineering in seismic regions. It can occur in saturated granular soils when seismic excitations result in the generation of high excess pore water pressures causing large reductions in soil shear stiffness and strength that lead to large ground deformations or failures. Although notable advancements have been made in understanding the mechanism of soil liquefaction and the remedial measures for dealing with the consequences over the past 2 to 3 decades, most of the significant progress has been confined to assessing the likelihood of liquefaction triggering under undrained conditions. However, the resulting earthquake-induced deformations are the main concern to the engineers, and evidence from past earthquakes indicate that liquefaction-induced large (in the order of meters) lateral spreads and flow-slides have taken place in relatively gentle (no more than a few percent) coastal or river slopes in many regions of the world (Hamada, 1992 and Kokusho, 2003). Seismically triggered submarine slides and marine structure failures were also reported/summarized by Scott and Zukerman (1972); Hamada (1992) and Sumer et al. (2007). More interestingly, flow-slides have occurred not only during but also after earthquake shaking.

Two key factors controlling the response of liquefiable soils to earthquake excitations are:

- Mechanical conditions
- Hydraulic/Flow conditions

Mechanical conditions encompass soil density, compressibility, stiffness, strength, initial static stress state, and earthquake characteristics (amplitude, predominant periods, etc.) that are mostly responsible for the generation of excess pore water pressure during seismic loading. The hydraulic/flow conditions i.e. drainage path, soil hydraulic conductivity/permeability and its spatial variation (permeability contrast) within the earth structure control the redistribution of excess pore water pressure during and after the earthquake. Sharp et al. (2003) and Seid-Karbasi and Byrne (2006a) using centrifuge model tests and numerical analyses, respectively, demonstrated that liquefiable soil deposits with lower permeability suffer greater deformations in an earthquake. Seid-Karbasi and Byrne (2006a) and Seid-Karbasi (2009) also showed that pore water migration is likely responsible for liquefaction onset commonly observed first at shallower depths of uniform soil layers in past earthquakes and physical model tests.

The majority of the previous liquefaction studies was based on the assumption that no flow occurs during and immediately after earthquake loading and were centered on mechanical conditions. However, this condition may not represent the actual conditions, because both during and after shaking, water migrates from zones with higher hydraulic head (e.g. greater excess pore water pressure) towards zones with lower hydraulic head. Recent studies including field investigation by Kokusho and Kojima (2002), physical model testing by Kukosho (1999) and Kulasingam et al. (2004), and numerical analysis by Seid-Karbasi and Byrne (2004a), Seid-Karbasi, and Byrne (2007) show that the presence of low permeability sub-layers acting as hydraulic barriers is likely the cause of flow failures of slopes underlain by loose sandy soils. The presence of such a hydraulic barrier layer impedes the upward flow of water resulting in a very loose zone immediately below the barrier leading to significant strength loss and possible post-shaking failure. This mechanism is also referred to as "void ratio redistribution" since it tends to develop a contracting zone in the lower parts of the liquefied sand layer and an expanding zone in the upper parts of it. The mechanism has been recently studied by researchers at Chuo University, Japan (Kokusho, 1999 and Kokusho, 2003) and the University of California, Davis, U.S (Kulasingam, 2003 and Malvick, 2005) using physical model testing and the University of British Columbia, Canada (Seid-Karbasi, 2009) employing numerical modeling. The severe strength loss due to expansion from void redistribution can lead to flow-slides even in very gentle slopes and after shaking has ceased as demonstrated by Seid-Karbasi and Byrne (2007a).

The risk of liquefaction and associated ground deformations can be reduced by various ground-improvement techniques, including: densification, solidification (e.g., cementation), gravel seismic drains and stone columns. Experience from past earthquakes and data from physical model tests suggest that liquefiable ground treated with seismic drains have better performance compared to unimproved sites (e.g., Hausler & Sitar, 2001; and Martin, et al., 2004). Some centrifuge test data, indicate that the densification method is not an effective treatment technique for liquefiable soils comprising a hydraulic barrier layer (e.g., Balakrishnan, 2000). Use of gravel drains is a rather recent development when compared to the more traditional soil densification techniques. Seismic gravel drains (stone columns), as a liquefaction mitigation measure, were initially studied by Seed and Booker (1977). As noted by Adalier and Elgamal (2004), since then, the gravel drain technique has received increased attention from a number of leading researchers (e.g., Ishihara and Yamazaki, 1980; Tokimatsu and Yoshimi, 1980; Baez and Martin, 1995; Boulanger, et al., 1998; Pestana, et al., 1999; Rollins, et al., 2004; Adalier and Elgamal, 2004; Seid-Karbasi and Byrne, 2004a and 2007; Chang, et al., 2004; Brennan & Madabhushi, 2005; and Shenthan, 2005).

Currently, the effects of seismic drain configuration in plan are well understood and established in the engineering profession since the pioneering work by Seed and Booker (1977). Seid-Karbasi and Byrne (2008) showed that the gravel drains with maximum penetration depth into the liquefiable layer are not the most effective option in all cases.

Design of such treatment scheme needs to account for a number of factors involved in a project through a parametric study. To do that, a two-dimensional (2-D) numerical approach is commonly used in practice; however the materials properties (i.e. mechanical and hydraulic properties) should be modified in respect to the real three-dimensional (3-D) conditions. The equivalent 2-D analyses should provide practically comparable results especially in terms of displacements which govern the design scheme.

This paper describes the results of a coupled mechanical-hydraulic dynamic analysis for a port berth structure founded on liquefiable ground treated with stone columns. The effective stress approach was employed to analyze the excess pore water pressure generation and redistribution in the ground soil layers. Two sets of 2-D analyses using two approaches for equivalent soils parameters were conducted and the results are compared.

SOIL LIQUEFACTION AND HYDRAULIC CONDITIONS

Earthquake-induced soil liquefaction refers to a sudden loss in shear strength and stiffness due to seismic shaking. The loss arises from a tendency for granular soils to undergo volume change when subjected to cyclic loading. When the volume change tendency is in contraction and the actual volume change is prevented or curtailed by the presence of pore water that cannot escape in time, the pore water pressure will increase and the effective stress will decrease. If the effective stress drops to zero (100% pore water pressure rise), the shear strength and stiffness will also drop to zero and the soil will behave like a heavy liquid.

Although a large number of laboratory investigations on liquefaction resistance of sands have been carried out, most of them dealt with the undrained (constant volume) behavior. Recent laboratory studies, (e.g. Vaid and Eliadorani, 1998; Eliadorani, 2000) have demonstrated that a small net flow of water into an element (injection) causing it to expand can result in additional pore pressure generation and further reduction in strength. Chu and Leong (2001) reported the same behavior occurs in loose and dense sands, and called it "pre-failure instability".

Vaid and Eliadorani (1998) examined this phenomenon by injecting or removing small volumes of water from the sample during monotonic triaxial testing as it was being sheared and referred to this as a "partially drained condition" (this test method is also called "strain path" in the literature e.g. Chu and Leong 2001). The results of inflow tests on Fraser River sand shown in Fig. 1 in terms of stress path, samples of sand consolidated to an initial stress state corresponding to $R_c = \sigma'_{1c}/\sigma'_{3c} = 2$, as shown in Fig. 1b, where R_c is the effective stress ratio, and σ'_{1c} and σ'_{3c} are the major and minor principle effective stresses, respectively.. As shown in Fig. 1d, the sample with $\sigma'_{3c} = 100$ kPa failed once the volumetric strain (ε_v) reached about 0.2%. In these tests, expansive ε_v was imposed by injection of water into the samples (see Fig. 1a) at a constant rate of $d\varepsilon_v/d\varepsilon_1 = -0.4$, where ε_1 is the axial strain. The samples were stable under the initial stress state. The stress paths during injection indicate a reduction in effective stresses at a constant shear stress. For each sample with each different initial confining stress as shown in Fig. 1d, the large reduction of shear strength/stiffness (i.e. instability) occurred with little change in shear stress and void ratio and at very small ε_1 of the order of 0.5%. Positive pore pressures continued to develop even beyond the phase transformation line. This occurs because the rate of imposed expansive volumetric strain is greater than the dilation potential of the soil skeleton in drained conditions.

Yoshimine et al. (2006), Sento et al. (2004) and Bobei and

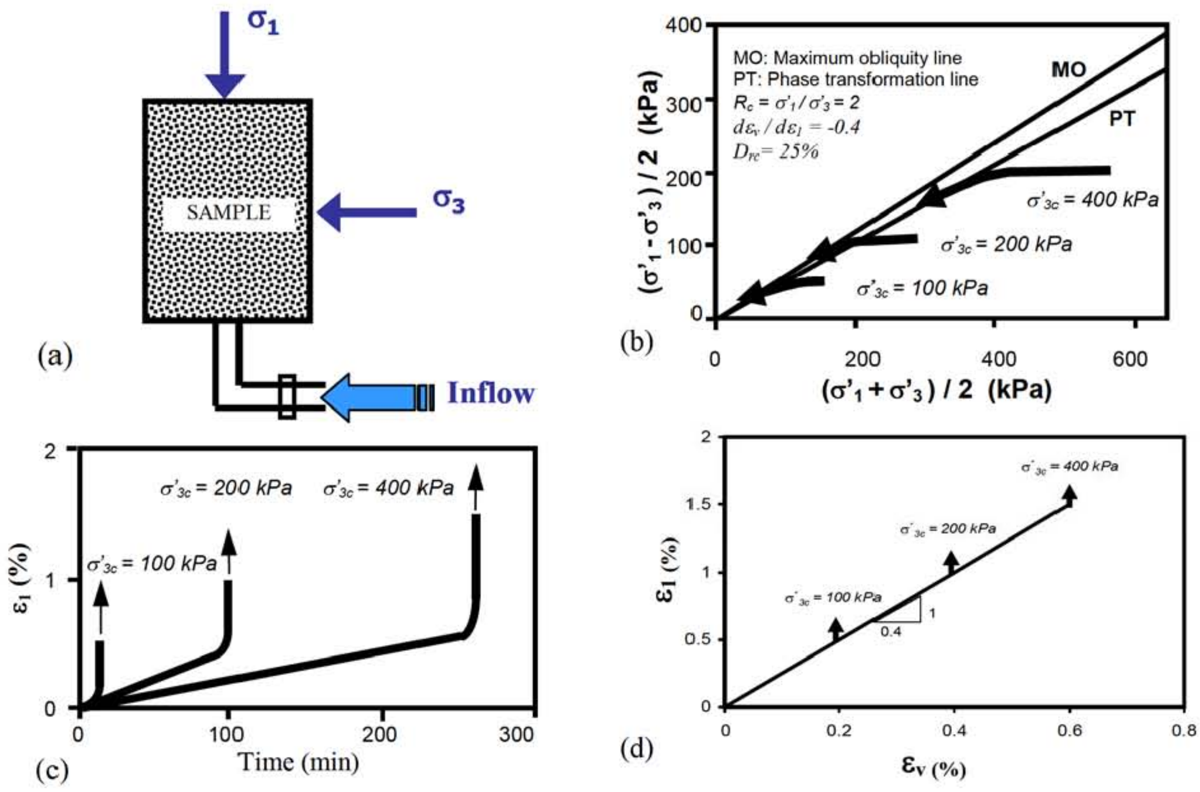

Fig. 1. Partially-drained instability of loose Fraser River sand (data from Vaid and Eliadorani 1998): (a) inflow into triaxial sample (b) stress paths; (c) strain paths and (d) axial strain vs. volumetric strain.

axial strain vs. time and strain path (with $Dr_c = 29\%$) indicate a potential for triggering liquefaction at constant shear stress ($\sigma'_1 - \sigma'_3$ = constant). A small amount of expansive volumetric strains imposed by water inflow resulted in an effective stress reduction and flow failure of Lo (2003) reported similar responses for Toyoura sand and silty sand. As a result, soil elements may liquefy due to expansive volumetric strains that cannot be predicted from analyses based on the results of undrained tests.

The stability conditions of a saturated slope under seismic loads depends largely on whether soil liquefaction will be triggered and what level of soil shear strength and stiffness loss would occur, which in turn depends on the relative rate of pore pressure generation due to seismic shaking and pore pressure dissipation due to drainage. The potential for large lateral displacements or flow slides will be greatly increased if a low permeability layer (e.g. a silt or clay layer) within a soil deposit forms a hydraulic barrier and impedes drainage. The excess pore water generated by seismic loading generally drains upwards and may accumulate underneath the hydraulic barrier layer to form a water film if the water inflow to the soil elements immediately below the barrier exceeds the elements' ability to expand (net inflow). This may result in the formation of a thin layer of soil with near-zero shear strength and eventually flow failure (Seid-Karbasi and Byrne, 2007a). Based on the results of a numerical analysis completed on an idealized infinite slope underlain by a low-permeability layer, which overlies a liquefiable sand layer, Seid-Karbasi and Byrne (2007b) demonstrated that expansion occurs at the upper parts of the liquefiable soil layer while the lower parts contract regardless of the thickness of the liquefiable layer.

ANALYSIS PROCEDURE

In order to evaluate the impact of a low permeability layer on the earthquake-induced ground deformations, it is necessary to simulate the generation, redistribution, and dissipation of excess pore pressures during and after earthquake shaking. This approach requires a coupled dynamic stress-flow analysis. In such an analysis, the volumetric strains of the soil skeleton are controlled by the compressibility of the pore fluid and flow of water through the soil elements. To predict the instability and liquefaction flow, an effective stress-based elastic–plastic constitutive model (*UBCSAND*) was used. The model was calibrated using laboratory and centrifuge test data and is described below.

Constitutive Model for Sands

The *UBCSAND* constitutive model is based on the elasto-plastic stress–strain model proposed by Byrne et al. (1995), and has been further developed by Beaty and Byrne (1998) and Puebla (1999). The model has been successfully used in analyzing the CANLEX liquefaction embankments (Puebla et al., 1997) and predicting the failure of Mochikoshi tailings dam (Seid-Karbasi and Byrne 2004b). It has also been used to examine partial saturation conditions on liquefiable soil's response (Seid-Karbasi and Byrne, 2006) and dynamic centrifuge test data (e.g. Byrne et al., 2004 and Seid-Karbasi et al., 2005). It is an incremental elasto-plastic model in which the yield loci are lines of constant stress ratio ($\eta = \tau / \sigma'$). Plastic strain increments occur whenever the stress ratio increases. The flow rule relating the plastic shear strain increment direction to the volumetric strain increment direction is non-associated, and leads to a plastic potential defined in terms of the dilation angle. Plastic contraction occurs when stress ratios are below the constant volume friction angle and dilation occurs otherwise, as shown in Fig. 2.

The elastic component of the response is assumed to be isotropic and defined by a shear modulus, G^e, and a bulk modulus, B^e, as shown in Eq. 1 and Eq. 2

$$G^e = K_G^e . P_a \left(\frac{\sigma'}{P_a} \right)^{n_e} \tag{1}$$

$$B^e = \alpha . G^e \tag{2}$$

where $K^e{}_G$ is the shear modulus coefficient, P_a represents the atmospheric pressure, $\sigma' = (\sigma'_x + \sigma'_y) / 2$, n_e is an empirical parameter depending on the soils (commonly 0.5), α depends on soil's elastic *Poisson's ratio* (varies from 0 to 0.2 as suggested by Hardin and Drnevich, 1972) and Tatsuoka and Shibuya 1992) and ranges from 2/3 to 4/3. The plastic shear strain increment $d\gamma^P$ and plastic shear modulus are related to stress ratio, $d\eta$ ($\eta = \tau / \sigma'$) as expressed by Eq. 3:

$$d\gamma^p = \left(\frac{d\eta}{\left(\dfrac{G^p}{\sigma'} \right)} \right) \tag{3a}$$

$$G^p = G_i^p \left(1 - \frac{\eta}{\eta_f} R_f \right)^2 \tag{3b}$$

where G^P is the plastic shear modulus defined by a hyperbolic function as Eq. 3b, $G^P{}_i$ is the plastic shear modulus at very low stress ratio level (η near 0), $\eta_f = sin\varphi_f$ is the stress ratio at failure, where φ_f is the peak friction angle, and R_f is the failure ratio. The associated increment of plastic volumetric strain, $d\varepsilon_v{}^P$, is related to the increment of plastic shear strain, $d\gamma^P$, through the flow rule as shown in Eq. 4:

$$d\varepsilon_v{}^P = d\gamma^P . (sin\varphi_{cv} - \eta) \tag{4}$$

where φ_{cv} is the friction angle at constant volume (phase transformation). It may be seen from Eq. 4 that at low stress ratios ($\eta = \tau / \sigma' = sin\varphi_d$) significant shear-induced plastic compaction is predicted to occur, while no compaction would occur at stress ratios corresponding to φ_{cv}. For stress ratios greater than φ_{cv}, shear-induced plastic expansion or dilation is predicted. More detailed discussions about the *UBCSAND* constitutive model were presented previously in Byrne et al. (2004) and Puebla et al. (1997).

The constitutive behavior of sand is controlled by the skeleton. The pore fluid (e.g. water) within the soil mass acts as a volumetric constraint on the skeleton if drainage is fully or partially curtailed. This model has been incorporated into the commercially available computer code *FLAC* (*Itasca*, 2005).

The key elastic and plastic parameters can be expressed in terms of relative density, *Dr,* or normalized Standard Penetration Test values, $(N_1)_{60}$. Initial estimates of these parameters were developed from published data and model calibrations. The responses of sand elements under monotonic and cyclic loading were then predicted and the results compared with the laboratory data. The predictions from the model were matched with the observed responses for sandy soils with a range of relative density or N values. The model was calibrated to reproduce the NCEER 97 chart Youd et al., 2001), is based on field data during past earthquakes and is expressed in terms of normalized Standard Penetration Test, $(N1)_{60}$. The model properties to obtain such agreement are therefore expressed in terms of $(N_1)_{60}$ values.

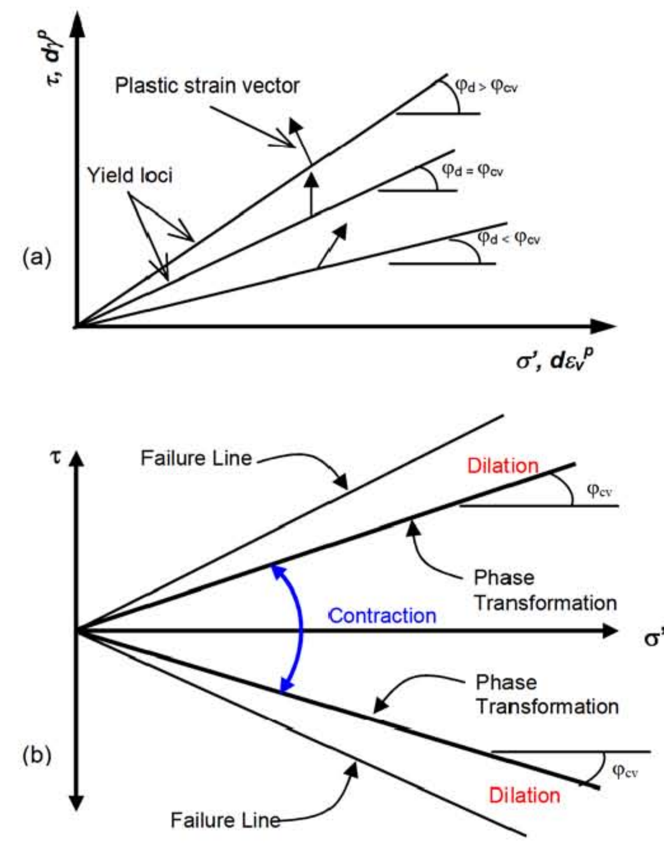

Fig. 2. (a) moving yield loci and plastic strain increment vectors, (b) dilation and contraction regions.

The model has also been modified to reproduce the chart suggested for liquefaction triggering by Idriss and Boulanger (2008). The effect of overburden pressure on liquefaction (i.e. K_σ effect) has taken into account and a good match obtained between the model prediction and that suggested by Idriss and Boulanger (2008). Fig. 3 shows a comparison of the model simulation with that of suggested by those authors for two selected $(N_1)_{60}$ values. This version of the model has been used in this study.

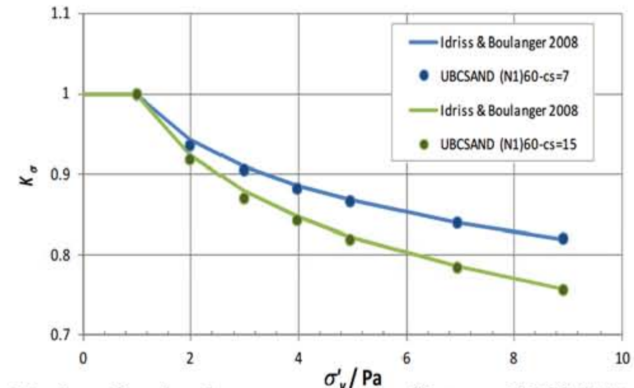

Fig.3. Overburden pressure effects; UBCSAND prediction vs. Idriss –Boulanger (2008) curve

Model Simulation of Laboratory Element Tests

The *UBCSAND* model was applied to simulate cyclic simple shear tests under undrained condition. Figure 4 shows model predictions along with test results on Fraser River sand. The sand tested had an initial vertical consolidation stress σ'_v = 100 kPa and relative density *Dr* = 40%.

The results of the model prediction, expressed in terms of stress-strain and excess pore pressure ratio, R_u, and stress path, compared reasonably well with the laboratory data as shown in Fig.4. It should be noted that as unloading is considered elastic, the excess pore pressure is constant while unloading takes place during cyclic shearing. A comparison of model prediction with tests results in terms of required number of cycles to trigger liquefaction for different cyclic stress ratios, *CSR* is shown in Fig. 3c and reasonable agreement is observed. The predicted apparent step-wise increase in the excess pore pressure with the number of cycles is numerically induced. This is because the cycle count is updated at every half cycle and the pore pressure itself is computed at every step.

The model was also used to study the effects of both the undrained and the partially drained conditions and the model predictions were compared with the observations during triaxial monotonic tests. The partial drainage tests involved injecting water into the sample to expand its volume as it was sheared. The injection causes a drastic reduction in soil strength. The same amount of volumetric expansion was applied in the numerical model and the results shown in Fig. 5 (solid line for model prediction) are in good agreement with the measured data.

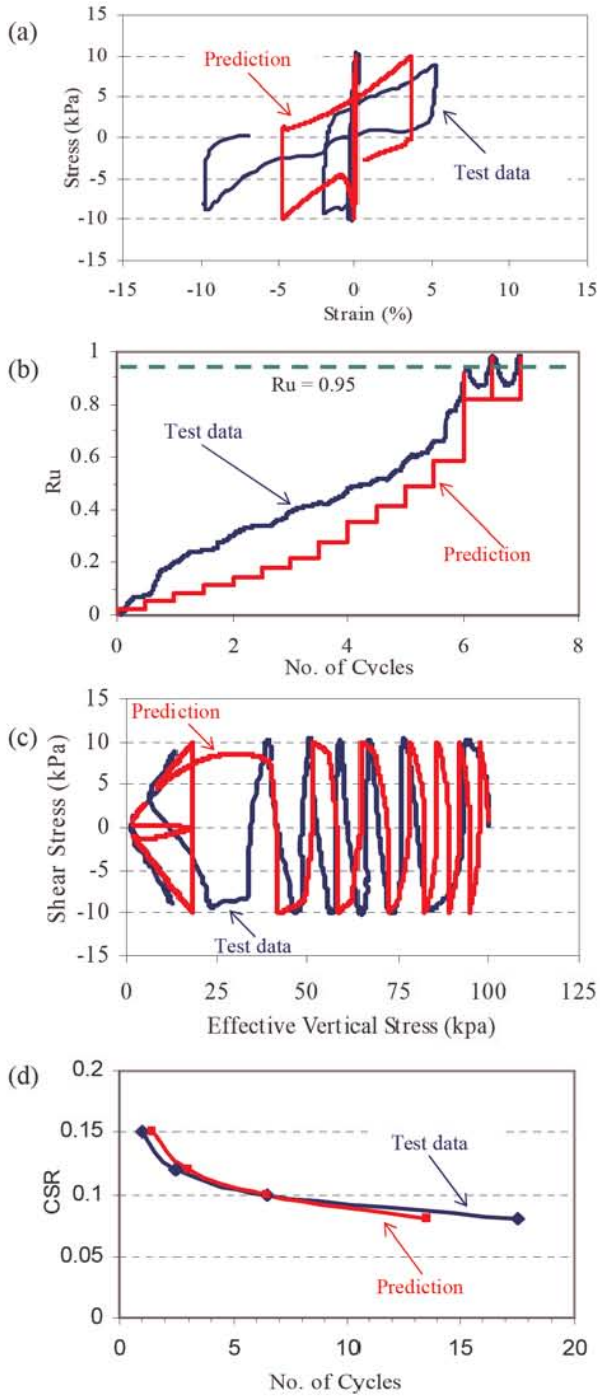

Fig. 4. Comparison of predicted and measured response for Fraser River Sand, D_r = 40% & σ'_v = 100 kPa (a) stress-strain, CSR = 0.1, (b) R_u vs. No. of cycles (liquefaction: $R_u \geq 0.95$), (c) CSR vs. No. of cycles for liquefaction (tests data from Sriskandakumar, 2004).

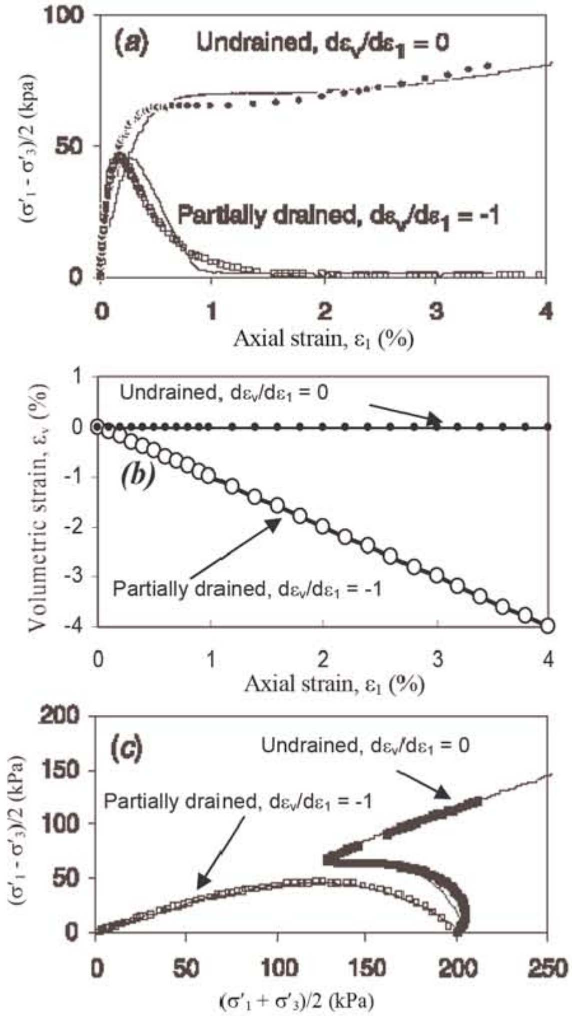

Fig. 5. Soil element response in undrained and partially drained (inflow) triaxial tests for FR River sand, (a) stress-strain, (b) volumetric strain, and (c) stress paths (modified from Atigh and Byrne 2004).

The above simulations illustrate that the model can appropriately simulate the pore pressure and stress-strain response under undrained loading, and can also account for the effect of volumetric expansion caused by inflow of water into an element.

TWO-DIMENSIONAL EQUIVALENT APPROACH FOR GROUND IMPROVEMENT SCHEMES

Deformation analysis of a soil foundation system (as-is condition) is commonly conducted using a two-dimensional (2-D) plane strain approach assuming that the loading and material properties are constant in out-of-plane direction. However inclusion of improvement measures (e.g. stone column, deep soil mixing DSM, seismic drains etc.) violates the adopted 2-D conditions.

The use of three-dimensional (3-D) dynamic analysis is a time-consuming task especially for an effective stress approach in a time domain analysis. The computer codes that can handle advanced constitutive models that have been bench-marked for 3-D analyses are not readily available. Therefore, using an equivalent/transformed 2-D analysis is a cost-effective and prudent approach.

Many researchers have attempted to deal with this boundary-value problem; however, they only focused on one aspect of the issue (i.e. equivalent 2-D mechanical properties, and/or equivalent 2-D hydraulic properties). Seid-Karbasi and Byrne (2006) showed that the permeability of material has a significant impact on liquefiable earth structures behavior in earthquakes. In a hydro-mechanical analysis two kinds of equivalent properties should be defined that may not follow the same rule for transformation necessarily.

Stress-deformation analysis of a mechanical problem, the equivalent 2-D properties (e.g. stiffness) for the ground condition with inclusion are commonly defined based on the ratio of improved/replaced area to total area (Martin et al. 1999). Bouckovalas et al. (2006) using strain and stress equivalence approximations showed a good match between results of a 2-D dynamic analysis (in terms of ground surface spectral accelerations) with that of widely used SHAKE-type 1-D equivalent linear ground response analysis. Papadimitriou et al. (2006) examined three different approaches to approximate the effects of DSM inclusion on results of the 2-D ground response analyses. They compared the results with that of a 3-D analysis and concluded that the 2-D analysis with equivalent section moduli (W = I/Y) provides a better match. Papadimitriou et al. (2007) using the same approach investigated the effects of seismic gravel drains on earthquake-induced deformations.

For hydraulic properties which are required in a coupled stress-flow analysis, the majority of works are focused on predicting deformations of soft grounds improved by drains to accelerate the consolidation deformations (Schweiger and Pande 1988, Indraratna and Redana 1997 among others). For consolidation analysis, it is necessary to convert the spatial flow into the laminar one in the 2-D plane-strain model, so some authors introduced equivalent hydraulic conductivity, k (e.g. Shinsha et al 1982). Bergado and Long (1994) using this concept (i.e. equivalent permeability) introduced an approach based on inclusion area ratio, α with respect to the drain pattern to model them in 2-D plane-strain as drain walls with equivalent thickness.

In this approach, the permeability of the soil between drain walls (i.e. native soil), k_m is modified to have discharge capacity of the 2-D model same as that of the actual case (Eq. 5). In this approach the drain wall thickness is defined based on the area ratio as shown in Eq. 6 (Bergado et al. 1996).

$$k_m = \frac{\pi(1-a_s)D}{2S \cdot Log_e(\alpha \cdot n)} \tag{5}$$

$$a = \frac{A_s}{A_s + A_c} \tag{6a}$$

$$a_s = C_1\left(\frac{D}{S}\right)^2 \tag{6b}$$

$$t_s = a_s \cdot S \tag{6c}$$

Where, D, As, Ac, d are drain (columns/walls) spacing (center to center), drain column area, native soil area, drain column diameter, respectively, and n=D/d, S=0.866D, α= 1.05 (for a triangular drain column pattern).

Two analyses using area ratio concept for mechanical property were conducted but in the first analysis the equivalent permeability, k was determined based on Bergado et al. (1996) suggestions *(Case I)* whereas for the second analysis the same approximation rule as mechanical properties was employed for hydraulic conductivity *(Case II)* and the results are compared.

ANALYZED PORT BERTH STRUCTURE

A simplified configuration for a port berth structure consisting of a caisson founded on liquefiable foundation soils is shown in Fig. 6. The soil foundation mainly comprises liquefiable soils that are improved with dense fill and stone columns in the vicinity of the 20 m-wide caisson structure. The model is 90 m and 60 m thick in land-side and water-side, respectively and its length is 600 m. The caisson-foundation system is represented by 466 × 64 elements with a nominal height of 1.5 m in horizontal and vertical directions, respectively. Water table El. is at 3 m which is representative of the mean tidal water level. The free field boundary conditions and horizontal quiet boundary condition were applied at the sides and the base of the model, respectively. The model was subjected to an earthquake motion with a PGA of 0.45g depicted in Fig. 7 which was applied as shear stress at the bottom boundary with a compliant base.

Table 1 lists the parameters for the different materials used in the analyses. The granular soils are modeled as *UBCSAND* model and presented by different values for $(N_1)_{60}$ whereas the caisson was treated as elastic material. The hydraulic conductivity; k for the treated zone in *Case II* was changed to an equivalent value based on the area ratio concept. Figure 8 shows the mechanical properties for the materials in the vicinity of the caisson.

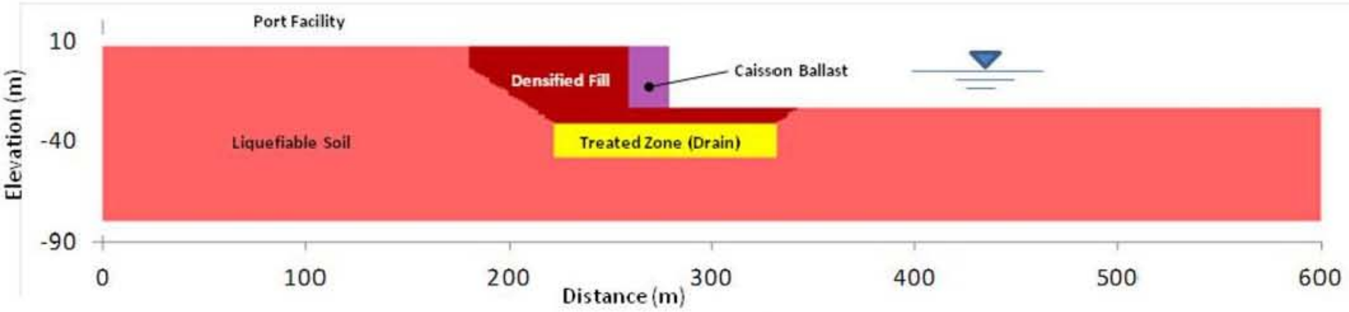

Fig. 6: Simplified model of port structure used in the 2-D plane strain analyses

Table 1: Materials parameters used in FLAC 2-D plane strain analyses

Material	Dry Density (kg/m^3)	$(N_1)_{60}$	k (m/s)	G (kPa)	B (kPa)
Liquefiable Foundation Soil	1458	12	1.00e-5	----	----
Dense Fill	1558	30	7.00e-2	----	----
Treated Zone, Native Soil	1488	25	5.64e-6[1]	----	----
Treated Zone, Gravel Drain	1590	30	1.00e-2[1]	----	----
Caisson Material	1730	----	1.00e-7	6.0e5	1.60e6

1) The permeability for the treated zone in the second analysis was changed to an equivalent value of 5.00e-3 (m/s).

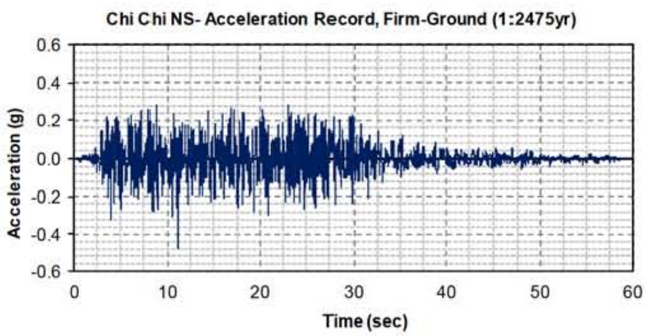

Fig. 7. Scaled acceleration time history used in the analyses

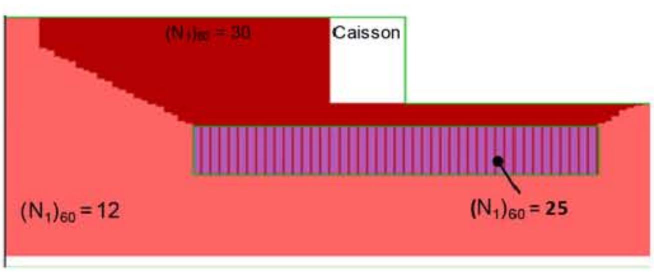

Fig. 8. Mechanical properties in the caisson foundation

RESULTS OF THE ANALYSES

In general, the use of gravel drains results in reduced ground deformations and lower induced excess pore water pressures as demonstrated by Cheng et al. (2004) and Seid-Karbasi and Byrne (2008) using physical and modeling procedures, respectively.

Fig. 9 shows the flow vectors at the toe of the treated zone for *Case I* at 10.0 sec. of shaking. It clearly demonstrates that significant drainage/redistribution of water occurs though the seismic drains during shaking. This lowering effect on developed excess pore water pressures can be seen readily from Fig. 10 which shows the distribution of the excess pore water pressure ratio, R_u at the same location and

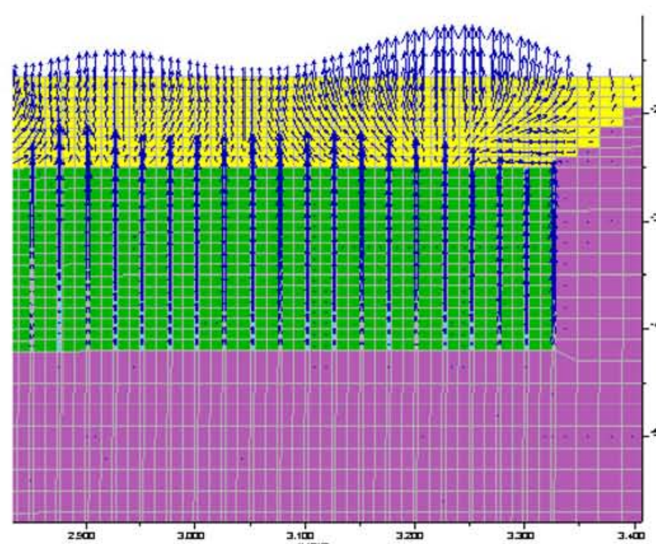

Fig. 9. Flow vectors at the toe of the treated zone (Case I)

shaking time. The main purpose of the improvement scheme is to alleviate and lessen the earthquake-induced deformations in the foundation to a tolerable level; therefore lateral displacement can be accounted for as performance criteria for an improvement option for this complex. Fig. 11 shows the contours of horizontal displacement in the vicinity of the caisson structure at 30sec. of shaking. The results show the caisson foundation experiencing significant movement towards water (in excess of 2.5 m).

Fig. 12 shows the deformed mesh at the toe of the caisson foundation at the end of shaking (50 sec.). As maybe seen the majority of the deformations occur below the treated zone (unimproved soil) and this area remains essentially undistorted after large lateral movements. Note that the elements were vertically aligned before the earthquake.

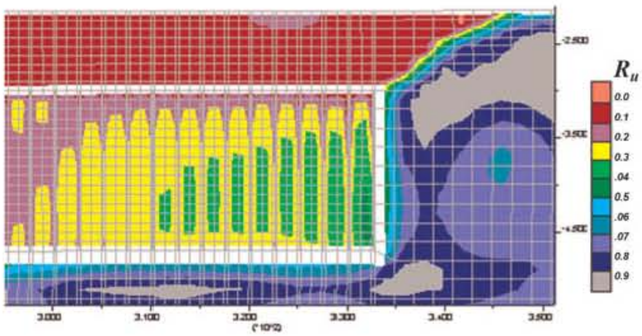

Fig. 10. Distribution of R_u at toe of the treated zone

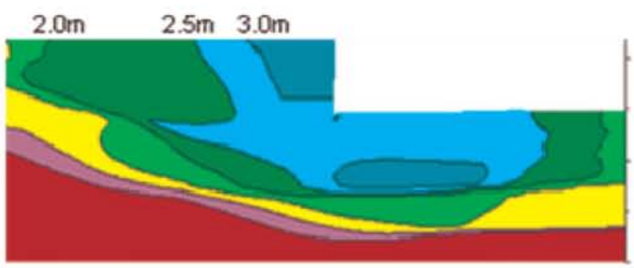

Fig. 11. Contours of lateral displacements (Case I)

The second case was analyzed with the same parameters as presented in Table 1 except that an equivalent permeability based on area ratio concept was assigned for the treated zone. The analysis results were of similar pattern of that of *Case I;* however the larger deformations caused a "bad geometry" at 34 sec at which point the simulations could not continue. Fig. 13 shows the contours of horizontal displacements for *Case II*. Comparing with that of *Case I* shown in Fig. 11 it was concluded that, for this project using equivalent permeability derived based on area ratio concept results in larger deformations and was used in further parametric analyses.

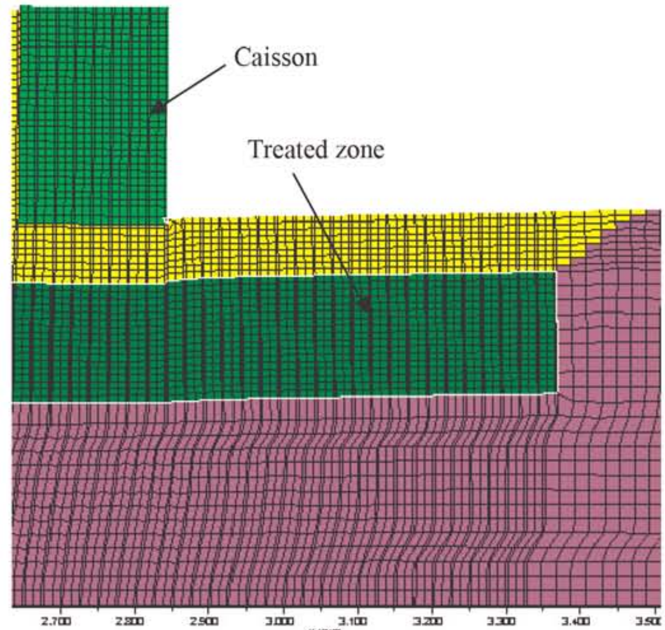

Fig. 12. Distorted mesh at the end of shaking (Case I)

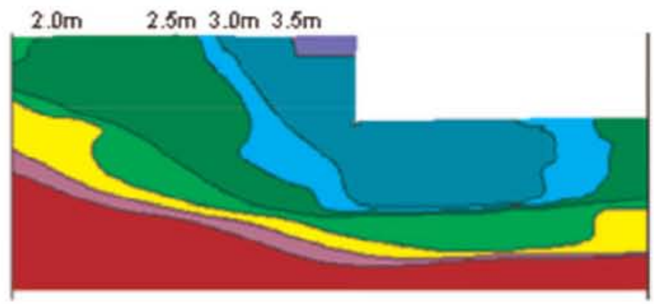

Fig. 13. Contours of lateral displacements (Case II)

Incorporating drain column (wall) in *FLAC* model needs small-size elements (a minimum of 2 elements, but preferably more) representing the drain wall, which is a main factor in controlling computational time-step (mechanical time-step) in a time-domain analysis procedure. Also, the presence of small-sized elements with high permeability (i.e. drain walls) decreases the (hydraulic) time-step significantly. Therefore a mesh with larger elements that can provide results in the safe side is a time- and cost-effective approach in a parametric analysis in large projects requiring many computer analyses.

CONCLUSIONS

Liquefaction of water saturated granular soils is one of the major risks that affect the safety and earthquake performance of infrastructure such as bridges, dams, ports., and lifelines in various parts of the world. Recent studies suggest that one of the promising measures to alleviate large earthquake-induced deformations and ground failures is seismic drains.

Design of such treatment scheme needs to account for a number of factors involved through a parametric study. Such analyses can be carried out using numerical modeling in a cost and time-effective manner.

This paper describes the results of a coupled mechanical-hydraulic dynamic analysis carried out for a marine structure founded on liquefiable ground treated with stone columns. An effective stress-based procedure was employed to analyze the excess pore water pressure generation and redistribution in the ground soil system. Two sets of 2-D analyses using two approaches for accounting for the 3-D effects of drain inclusion in a plane-strain procedure were carried out and the results were compared. The results of the study suggest that the commonly used area ratio concept to determine the equivalent material properties can also be employed in a coupled stress-flow analysis. This approach provided larger and hence conservative ground deformations when compared to the equivalent permeability concept; *k* method proposed by Bergado et al. (1996).

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