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OPTIMUM DEPTH OF SEISMIC DRAINS FOR MITIGATING LARGE DEFORMATIONS IN LIQUEFIED GROUND WITH HYDRAULIC BARRIER

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ABSTRACT

Liquefaction of water saturated granular soils is one of the major risks that affect the safety and post-earthquake performance of infrastructure such as bridges, dams, buildings, 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 indicate that large lateral spreads and flow-slides in gentle sandy slopes have taken place when a low permeability silt/clay layer (hydraulic barrier) is present. One of the promising measures to alleviate this barrier effect and ground failures is seismic drains.

Currently the effects of seismic drain configuration in plan are well understood and established in the engineering profession. However, most drain improvement schemes comprise of seismic drains that fully penetrate the liquefied soil layer. This paper describes the results of a coupled stress-flow dynamic analysis to investigate the enhancement effect of drain depth on deformations of liquefied slopes with barrier sub-layer. This study showed that drains that fully penetrate the liquefiable depth do not provide the lowest deformations and as a result may not provide the optimum solution.

INTRUDUCTION

Earthquakes have caused severe damage to onshore and offshore infrastructures such as buildings, bridges, ports or 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 areas. 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 issue 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 concerns to 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, stiffness and 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 were 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) showed 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 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 a few 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), and gravel seismic drains or stone columns. Experience from past earthquakes and physical model tests data 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 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 drains configuration in plan are well understood and established in the engineering profession since the pioneering work by Seed and Booker (1977). However, the effects of penetration depth of drains are not well understood. This paper presents the results of a dynamic, coupled stress-flow analysis carried out to investigate the depth effects of the seismic drains on the behavior of gentle liquefied slopes with hydraulic sub-layer barrier. The results of the study demonstrate the impact of water migration (inflow/outflow) within liquefied grounds that is controlled by drainage capacities during an earthquake excitation.

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) 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, axial strain vs. time and strain path (with Dr_c= 29%) indicate a potential for triggering liquefaction at constant

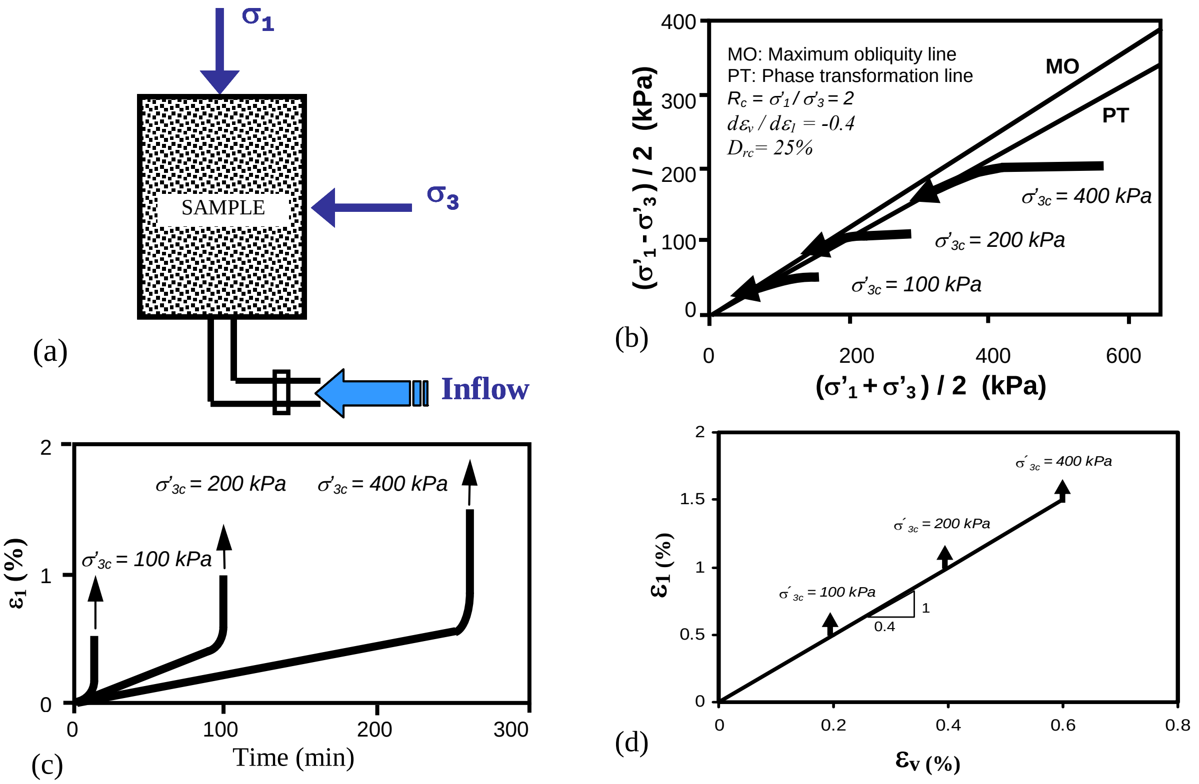


Fig. 1. Partially-drained instability of loose Fraser River sand (data from Vaid and Eliadorani 1998): (a) inflow into triaxial sample (b) stress paths; (c) strain paths and (d) axial strain vs. volumetric strain.

shear stress (σ'_1 - σ'_3 = constant). A small amount of expansive volumetric strains imposed by water inflow resulted in an effective stress reduction and flow failure of 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 σ'_{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 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.

Figure 2 shows a typical volumetric strain profile along the normalized depth of the liquefiable soil layer beneath the hydraulic barrier. More detailed discussion of void redistribution effects may be found elsewhere (i.e. Seid-Karbasi and Byrne, 2007a&b and Seid-Karbasi 2009).

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.

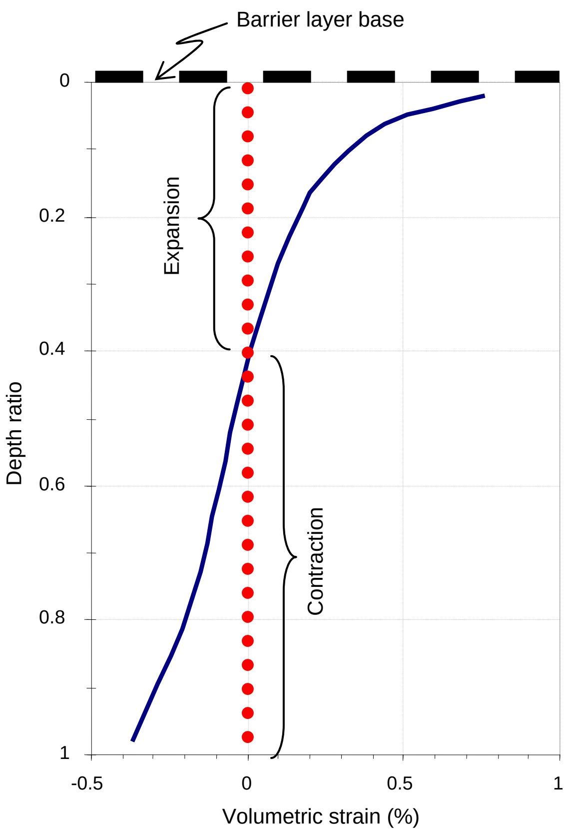


Fig. 2. Typical volumetric strain isochrone beneath the barrier layer with normalized depth for infinite slopes (Seid-Karbasi & Byrne, 2007b).

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. 3.

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

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

$$B^e = \alpha \, . \, G^e \qquad (2)$$

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:

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

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

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:

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

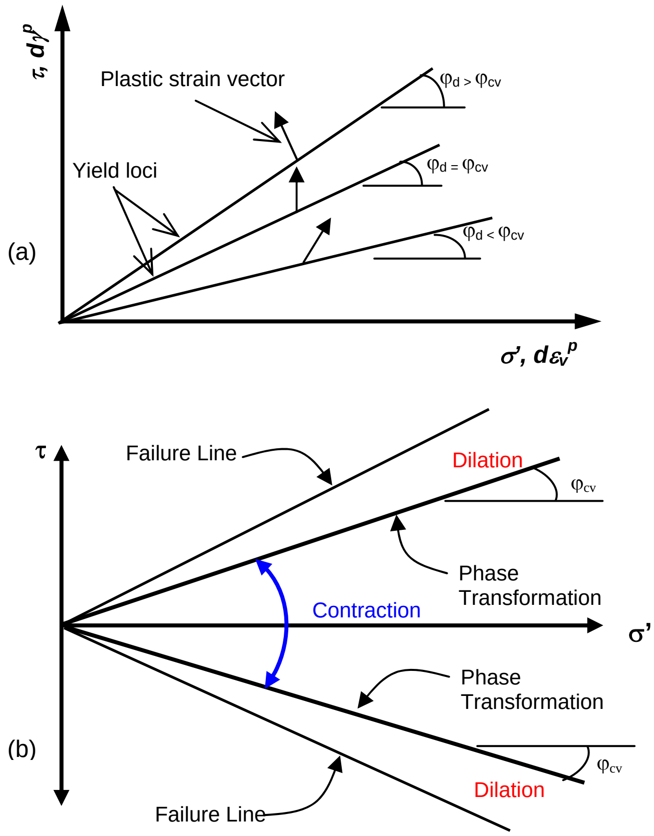


Fig. 3. (a) moving yield loci and plastic strain increment vectors, (b) dilation and contraction regions.

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, $(N_1)_{60}$. The model properties to obtain such agreement are therefore expressed in terms of $(N_1)_{60}$ values.

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.

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.

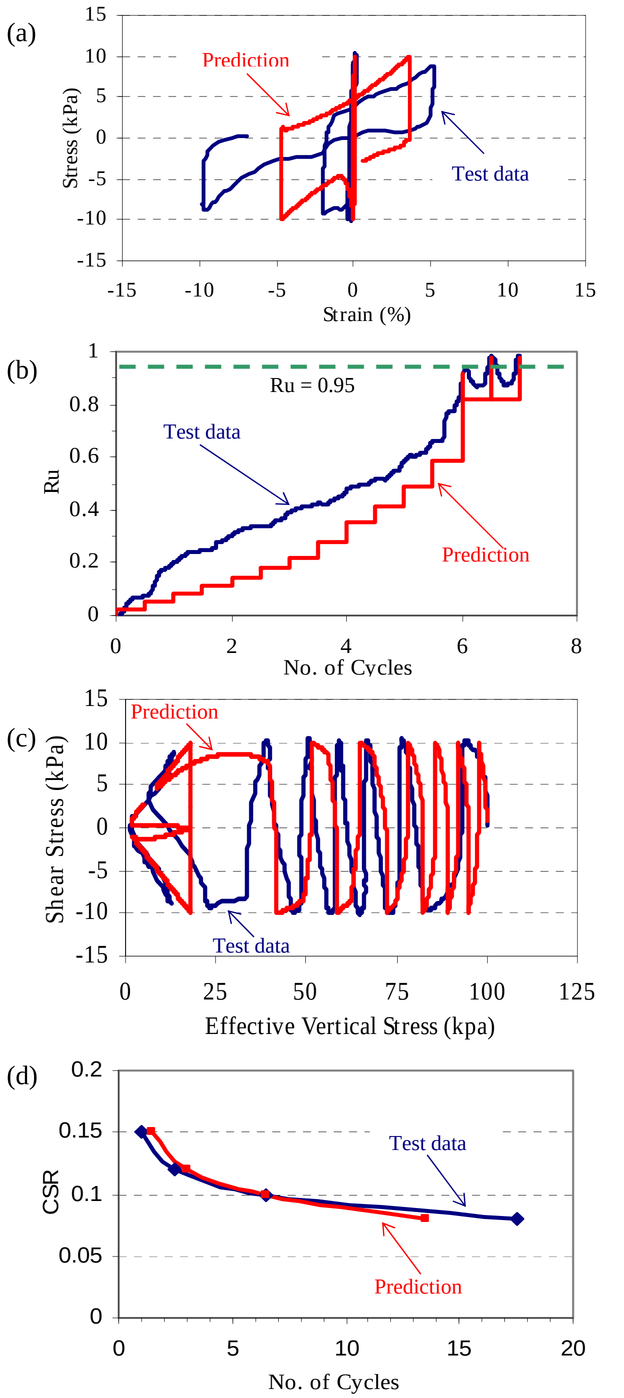


Fig. 4. Comparison of predicted and measured response for Fraser River Sand, D_r = 40% & σ'_v = 100 kPa (a) stress-strain, CSR = 0.1, (b) R_u vs. No. of cycles (liquefaction: $R_u \geq 0.95$), (c) CSR vs. No. of cycles for liquefaction (tests data from Sriskandakumar, 2004).

ANALYZED SOIL PROFILE

The soil profile used in this study is a 10 m sand layer representing a sloping ground of 1° inclination with water table at ground surface as shown in Fig.6. The soil profile

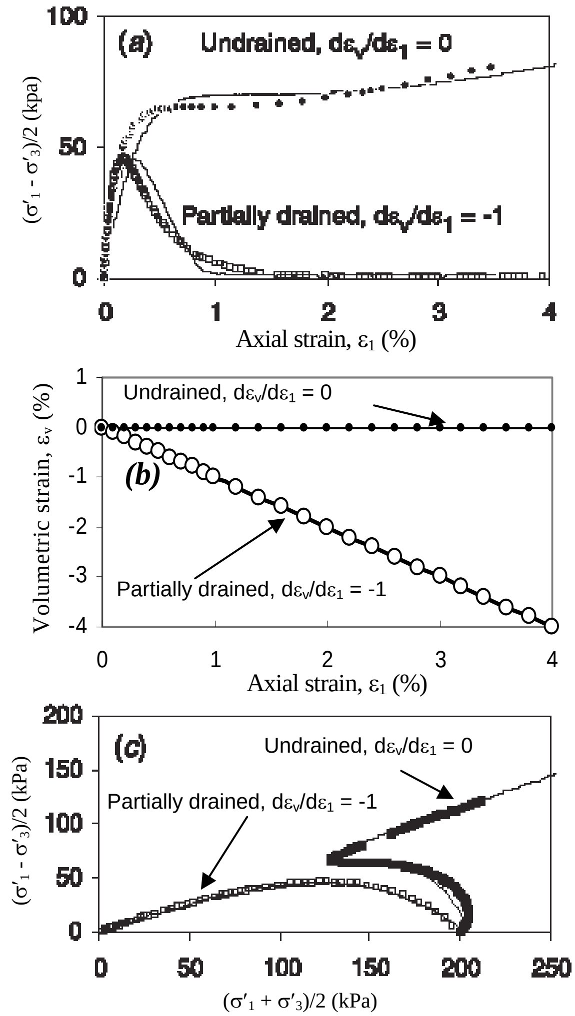


Fig. 5. Soil element response in undrained and partially drained (inflow) triaxial tests for FR River sand, (a) stress-strain, (b) volumetric strain, and (c) stress paths (modified from Atigh and Byrne 2004).

comprises of a loose sand deposit resting on an impermeable rigid foundation. A ground motion in terms of an acceleration time history (*PGA* = 2.5 m/s^2).is applied at the rigid base. Fraser River sand, with relative density *Dr* = 40 % is considered to represent the loose sand. Material properties are listed in Table 1, in which ρ_d, n and k are material dry density, porosity and permeability, respectively. *UBCSAND* model was applied to the loose sand layer with corresponding equivalent *UBCSAND* $(N_1)_{60}$ value. The low permeability silt layer barrier at 4m depth is simulated with a *Mohr-Coulomb* model having friction angle, φ = 30° and permeability, k one thousand times lower than that of the loose sand layer. Its stiffness in terms of bulk modulus and shear modulus was modeled as 1e4 kPa and 0.5e4 kPa, respectively.

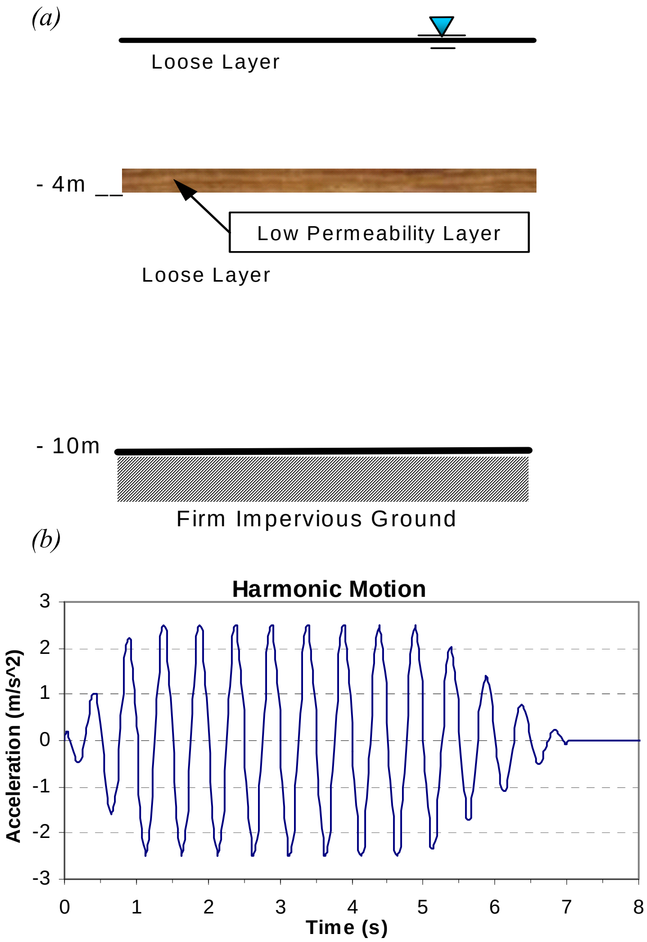


Fig. 6. (a) analyzed soil profile with barrier, (b) acceleration time history for input base motion.

This paper deals with the effects of drain depth in mitigating the observed ground deformations. To examine the influence of penetration depth of drains as remediation measures, the analyses results of three cases with drains are discussed in the following i.e.:

1. Drain with complete penetration (*Case I*)
2. Drain with partial (half) penetration (*Case II*)
3. Drain with minimum penetration (*Case III*)

Inclusion of a drain curtain in a 1-D model converts it to a 2-D model, as the flow properties vary in the horizontal direction. This is also the case for an infinite slope (comprising drains).

Table 1. Materials properties used in the analyses.

Material	ρ_d ($1000\ kg/m^3$)	n	UBCSAND $N_{1(60)}$	k (m/s)
Loose 40%	1.50	0.448	6.2	8.81e-4
Silt barrier	1.50	0.448	----	8.81e-7

The effect of drains spacing in soil layer performance is well recognized; however, little information is available about penetration depth effects and in particular, where a barrier sub-layer is present. Figure 7 shows the meshes used in the analyses for the three cases along with the benchmark model for discussion purposes. The drain properties are identical to those of the surrounding sand layer except that the drain permeability is greater. Thus, other enhancement effects of the stone (drain) columns such as some densification, as noted by Alalier & Elgamal (2004), were not considered in this study.

The study was conducted in plane-strain condition, and the drains were represented by a column (curtain) of permeability 100-times greater than that of the native liquefiable soils in the analyses. For design purposes, the 3-D (or axisymetric) conditions of the drain columns installation-pattern can be treated in a plane-strain analysis by using an appropriate equivalent drain curtain approach suggested by a few investigators (e.g. Indraratna & Redana, 1997 & 2000 among others).

Results of the Analyses

In general, implementing the drains resulted in lower ground deformation and lower induced excess pore water pressures. Figure 8a shows the contours of maximum excess pore water pressure ratio $(R_u)_{max}$ along with flow vectors predicted for the *Case I* (drain with full penetration) at 3.5 s of shaking. It clearly demonstrates that significant drainage occurs though the seismic drain during the shaking. This lowering effect on developed excess pore water pressures was also observed in field model tests of liquefied soils during shaking as reported by Cheng et al. (2004). Figure 8b shows the model (*Case I*) with displacement vectors (at 12 sec.) indicating no deformation concentration within the slope compared to the benchmark case (see Fig. 7a). The analyses for the other cases also showed that the inclusion of a drain column results in less displacement with no deformation concentration. Figure 9 shows the time histories of the ground surface lateral displacement for these three cases comparing to that of the benchmark case reported by Seid-Karbasi and Byrne (2007a).

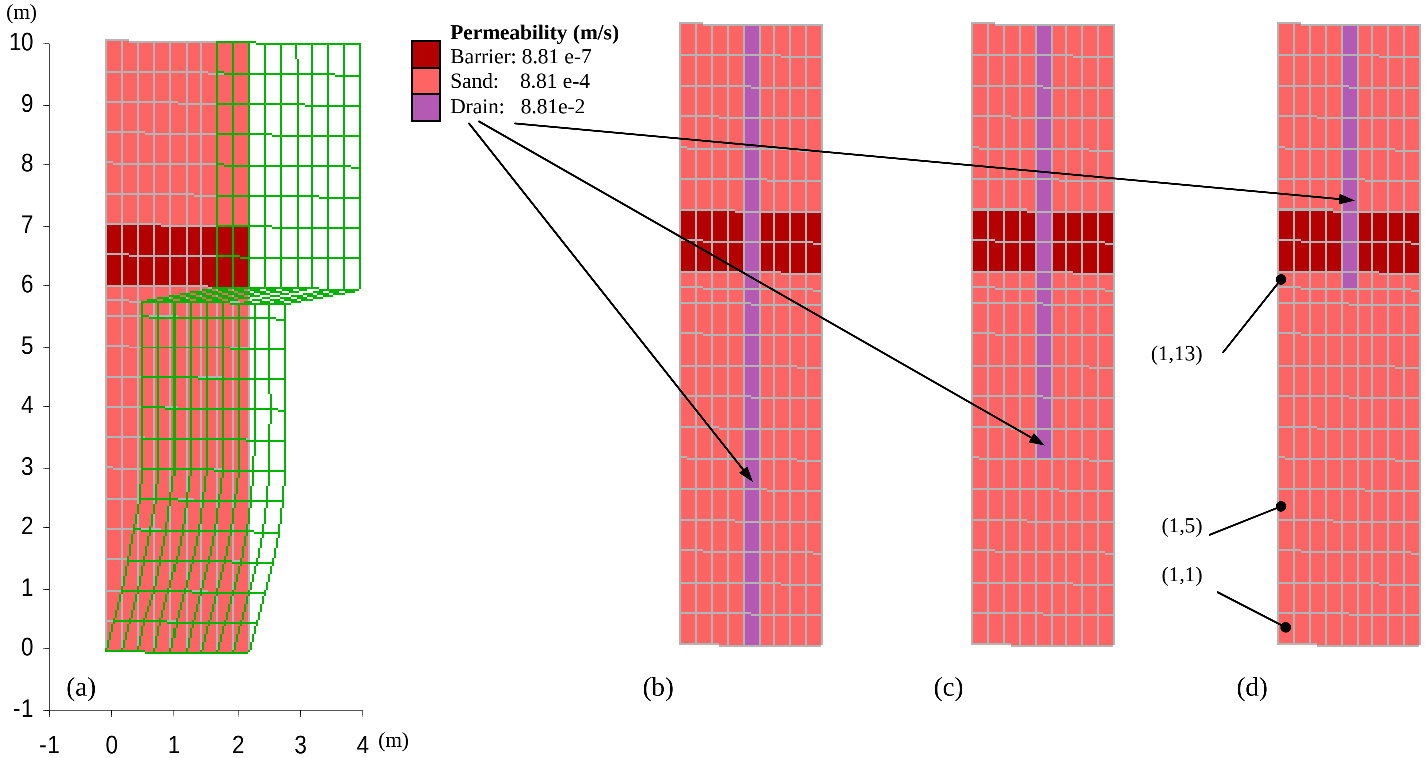


Fig. 7: Analyzed models representing (a) deformed 1°- slope with hydraulic sub-layer barrier, benchmark case, (b) improved with full penetration drain, (c) improved with half penetration drain, and (d) improved with minimum penetration drain.

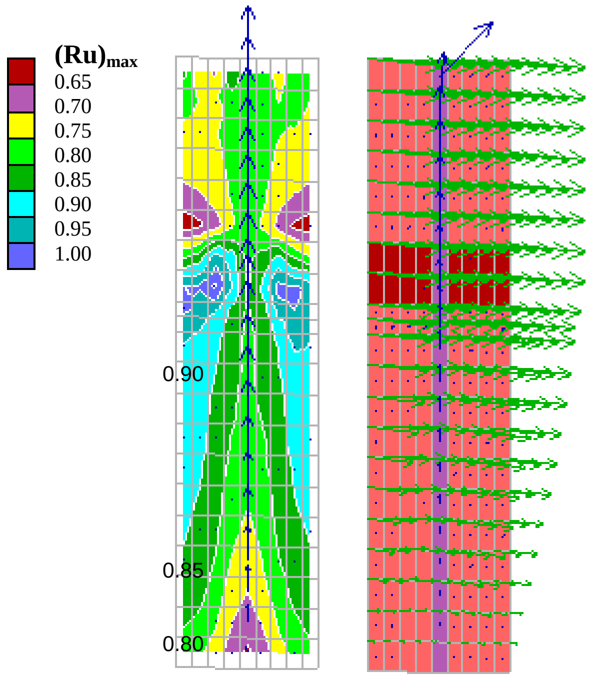
As maybe seen, the application of the seismic drains leads to significantly lower displacements. However, the ground displacement of the model with half penetration drain (*Case II*) gives the lowest displacement. Figure 10 shows the improvement obtained from drains in terms of displacement decrease vs. drain penetration depth below the barrier normalized with respect to the liquefied layer thickness. In the following, this issue will be discussed in more detail.

DISCUSSION

Deformation patterns (given in Fig. 7a and Fig. 8b) indicate that the insertion of drains in the liquefiable slopes (with barrier layer) significantly influences its response to shaking. In all analyzed cases, the ground deformations are considerably lower than that of the unimproved case (see Fig. 9). Figure 11 presents contours of $(R_u)_{max}$ for the three cases at 12 sec. The figure suggests that implementing a fully penetrating drain causes greater excess pore water pressure in most parts of the liquefied layer. Figure 12 shows the effects of drain depth on the time histories of excess pore water pressure, R_u in the mid-depth (i.e. element [1, 5]) of the liquefiable layer along with the specific vertical discharge, *Y-Flow* (flow volume through per unit area) beneath the barrier (element [1, 13]; see Fig. 7 for element positions). It indicates that a drain with half penetration (*case II*) results in the lowest average $R_{u.}$ Nevertheless, the dissipation rate for excess pore water pressure is greater in *case I*, as expected.

The figure also shows that the pore water pressure spikes become greater as the seismic drain extends to a greater depth. It is observed from Fig. 12b that the minimum inflow into the farthest element i.e. [1, 13] (see Fig. 7 for element position) at the barrier base occurs in *case II.* The minimum penetrated drain (*case III*) results in larger inflow into this farthest element (from the drain) as the inflow (resulting in greater expansion) continues for a longer time after shaking ceases (at 7 sec.), due to the low-capacity of the drainage system. In an ideal situation, with optimum drainage system, the rate of inflow and outflow for the farthest element from the drain column are balanced and no expansion occurs in this element at the barrier base.. It should be noted that the flow through the base element of the drain column increases with the drain penetration depth, therefore more drainage capacity is necessary at shallower depths. In practice, this can be fulfilled with a combination of deep and shallow seismic drains. Drain systems of some deeper penetrated seismic drains can also be implemented where the position of the sub-layer barrier varies in depth.

For the analyzed slope, the inflow effect is also reflected in the predicted stress-strain response of element [1,13] beneath the barrier (see Fig. 7) as shown in Fig. 13 for the three cases. It indicates that (high) inflow in full penetration case results in large strains in the farthest element from the drain at the barrier interface. It shows that liquefaction occurs in earlier stage of shaking in this case as a result of water inflow through the drain with full penetration depth.



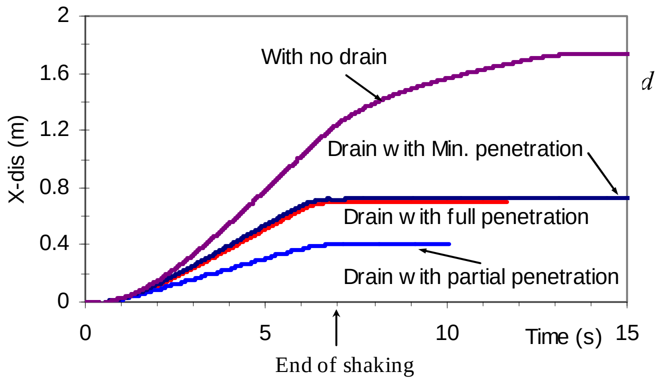


Fig. 9: Comparison of surface lateral displacement time

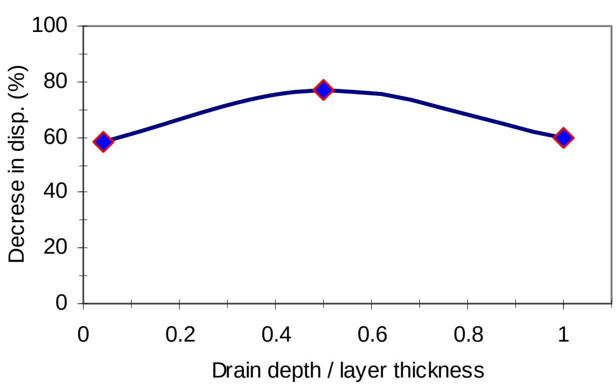


Fig. 10: Decrease in maximum surface lateral displacement vs. normalized drain penetration depth below the barrier.

Liquefaction in *case II* occurs at late stages of shaking and in *case III* the onset is between these two. This finding suggests that liquefaction and soil weakening can occur due to inappropriate drainage system because of easier water circulation within the model in earthquakes.

The drain depth effect is also well pronounced in the acceleration records. Fig. 14 shows the acceleration time histories at the base of the barrier layer (i.e. node [1, 14]) for the three cases. From the figure, some of the relatively smaller displacements in *case III* can be attributed to lower transmitted motion (base isolation effect) compared to those in *case I*. Thus, despite the greater motion in *case II*, the displacements are smaller due to the lower average R_u, driven by the (practically) optimum capacity of the drainage system. Deformations, in this case, show a relatively greater influence over the excitation inertia effect, as reflected in the surface lateral displacement record (see Fig. 9).

A similar observation regarding transmitted motion was addressed for the densification improvement method, based on centrifuge model tests (Mitchell, et al., 1998). In general, the ground deformations take place because of the interplay of applied loads (transmitted motion), available average strength within the liquefiable medium (Fig. 11), and drainage capacity, as observed in these cases.

Some of the above-mentioned effects from seismic drain application were already noticed in centrifuge test models of liquefiable soils and foundations by a few researchers (e.g., Liu & Dobry, 1997; Cooke, 2000; Hausler, 2002; Ghosh & Madabhushi, 2003; Brennan & Madabhushi, 2005 and 2006). Likely, an inappropriate drain system only facilitates more net flow and exacerbates the situation, as the outcome of drain installation is controlled by various factors. Therefore, this study show that, the engineering design of seismic drains improvement systems should be carried out with an account for penetration depth effects along with the drains spacing.

CONCLUSIONS

The liquefaction induced ground deformations in earthquakes are controlled by two major factors i.e.

- Mechanical conditions
- Hydraulic/Flow conditions

Recent studies including field data, centrifuge/shake table model tests, and numerical investigations indicate that large lateral spreads and flow-slides are taken place in gentle sandy slopes when a low permeability silt/clay layer (hydraulic barrier) is present. To cope with hydraulic impact from a sub-layer barrier, one of the promising measures to alleviate this barrier effect and ground failures is seismic drains.

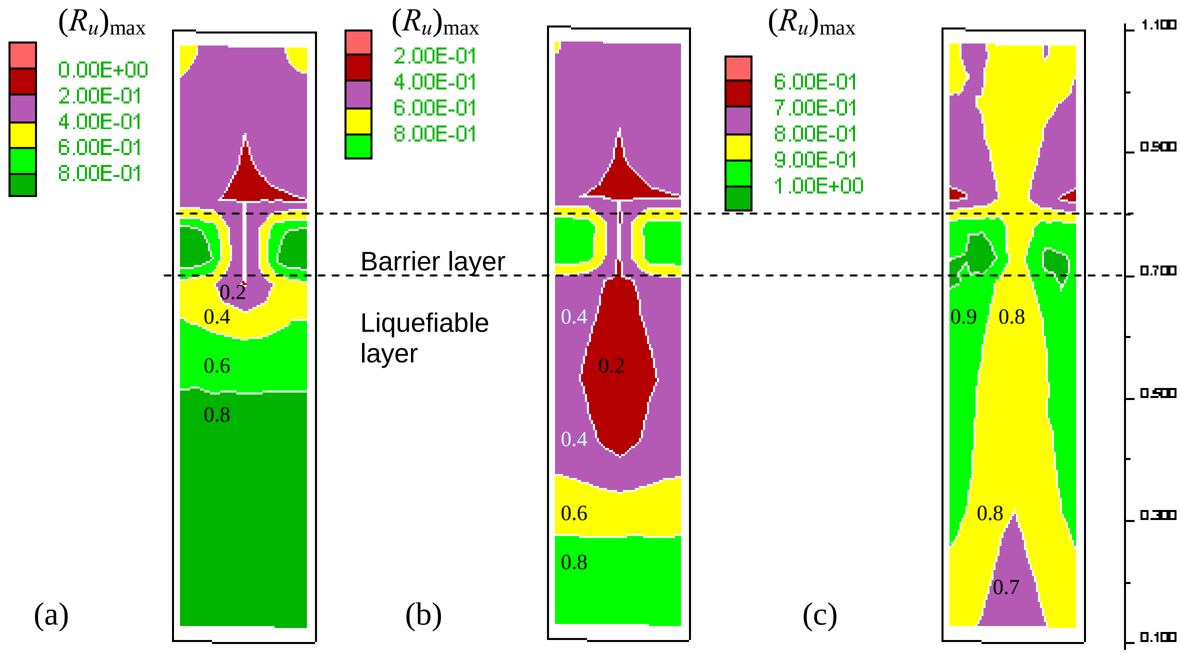


Fig. 11: Contours of $(R_u)_{max}$ within the model treated by drain (at 12 sec.) (a) case III, (b) case II, and (c) case I.

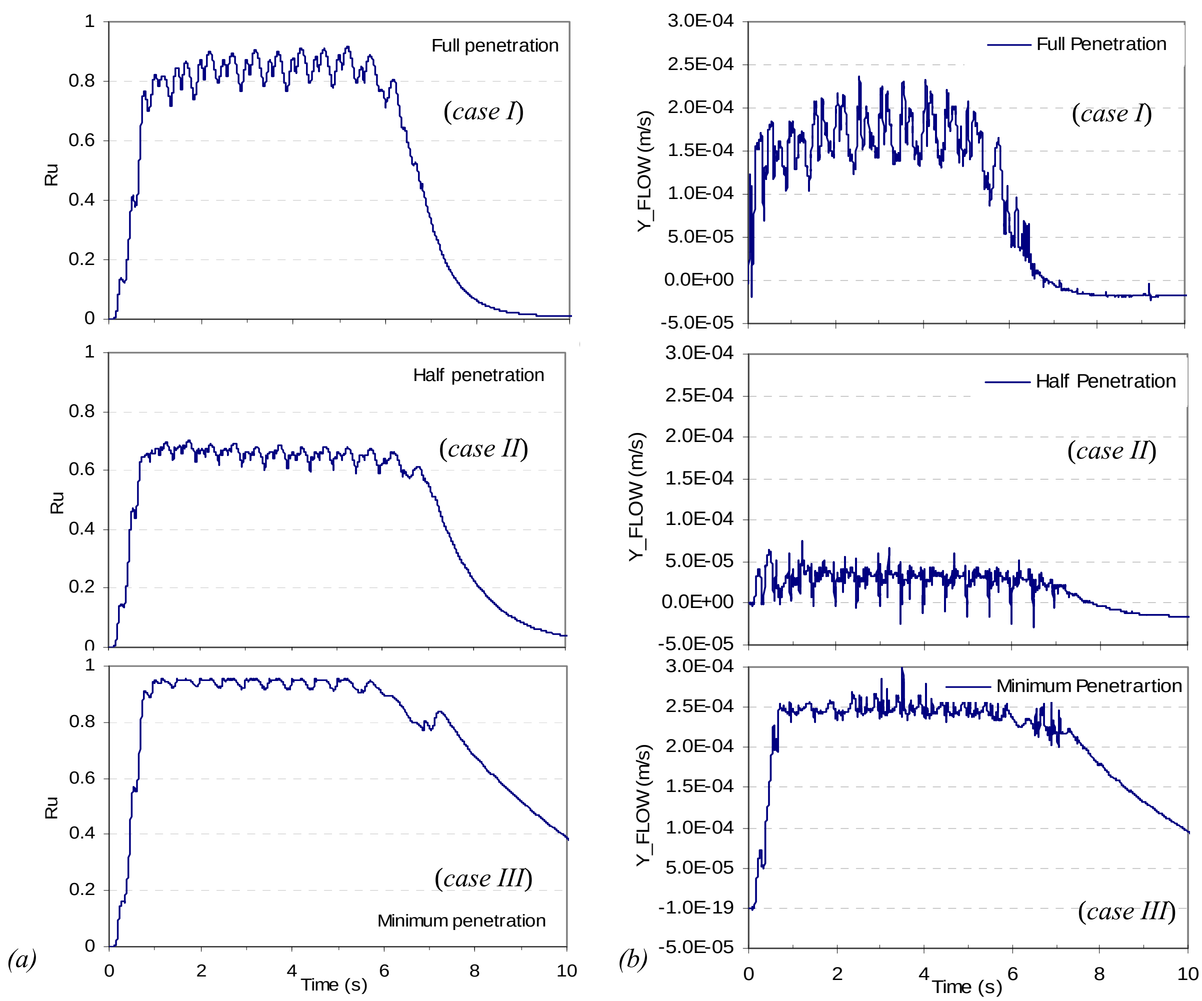


Fig. 12: Effects of penetration depth of the seismic drains in terms of time history of (a) R_u at mid-depth i.e. element [1, 5], and (b) vertical specific discharge at barrier base i.e. element [1, 13].

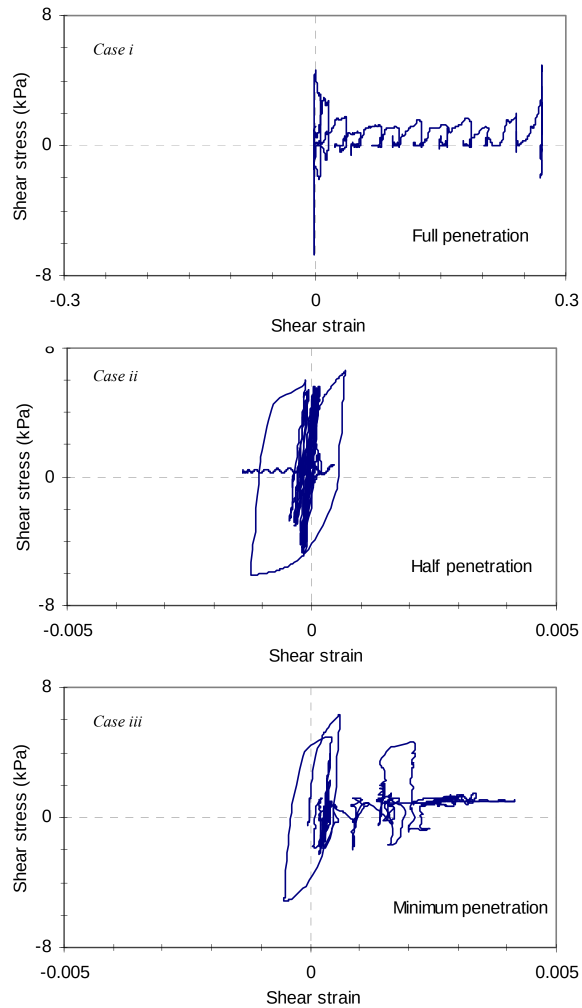


Fig. 13: Stress-strain response for the barrier base element, [1,13].

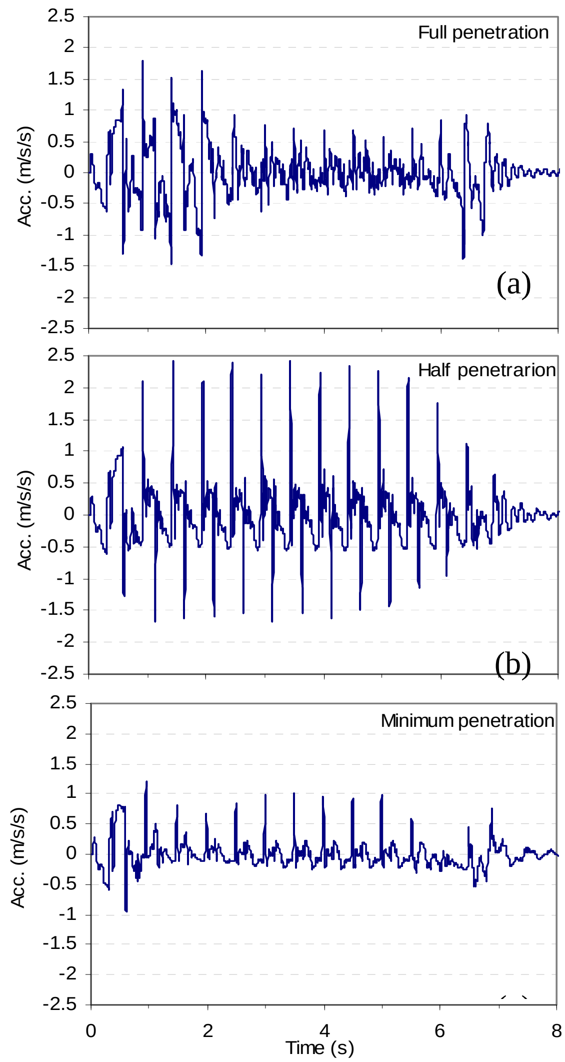


Fig. 14: Acceleration time history at the barrier base (node [1, 14]) for (a) case I, (b) case II, and (c) case III.

Currently the effects of seismic drains configuration in plan are well understood and established in the engineering profession. However, most drain improvement schemes comprise of seismic drains that are fully penetrated in the liquefied soil layer.

This paper described the results of a study using a coupled stress-flow dynamic procedure to investigate the enhancement effects of drain depth on deformations of liquefied slopes with barrier sub-layer. This study showed that

1. Drains can alleviate the barrier layer effects and reduce the lateral deformations (shear failure). This agrees with physical modeling data and experience from past earthquakes, as noted by others.

2. The improvement obtained from drain system depends on the system capacity.

3. For a given configuration of seismic drains, the effective ness of the drain system varies with the depth of drains.

4. Seismic drains have multiple effects on the response of liquefiable soil layers to earthquakes i.e.,
   a. Dissipation effect,
   b. Facilitating flow within the medium.
   c. Alleviation of the base isolation effect of liquefied soil.

5. The extent of improvement from drains installation reflects the interplay of the above effects.

6. Seismic drains with full penetration through the liquefiable layer are not the optimum measure in all cases. Drains with partial penetration are the optimum solutions for providing minimum deformations. They are also more cost-effective.

7. Drains with minimum penetration can be a promising economic measure for providing improvement.

8. The drains get "loaded" from the bottom up and could feed water into the barrier base and exacerbate the situation.

9. As more drain capacity is needed near the surface, a combination of drains penetrated to deeper zones below

the barrier and some to the base of the barrier should be considered.

10. Generally, it is likely that the location of the sub-layers barrier is not known very well in which case some drains to greater depths would seem wise

11. Systematic studies and numerical modeling, using a coupled stress-flow approach, can provide insights into drain behavior during seismic loading and provide guidance on optimizing improvement solution for engineering projects.

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