EFFECTS OF PARTIAL SATURATION ON LIQUEFACTION

Ahmad Jafari Mehrabadi, Ph.D. candidate, Memorial University of Newfoundland, St. John's, NL, Canada
Radu Popescu, Associate Professor, Memorial University of Newfoundland, St. John's, NL, Canada

ABSTRACT
Deltaic sands usually contain free gas. This significantly affects the compressibility of soils, and as a result it has remarkable effects on liquefaction susceptibility of soils. This paper presents a comparison of the numerical results for an earth slope made of Fraser River sand subjected to seismic motion both for saturated and partly saturated cases. Fully coupled, effective stress nonlinear dynamic analyses are performed using a multi-yield plasticity soil constitutive model implemented in the finite element program DYNAFLOW. The purpose of this paper is to emphasize the effects of partial saturation on liquefaction susceptibility of soils.

RÉSUMÉ
Les sables deltaïques contiennent habituellement du gaz. Ceci affecte la compressibilité de manière significative, et il a des effets remarquables sur la susceptibilité de la liquéfaction de sol. Cet article présente la comparaison des résultats numériques pour une pente faite de sable de fleuve de Fraser soumis au mouvement séismique, pour des cas saturés et partiellement saturés. Des analyses dynamiques non-linéaires sont effectué avec le logiciel DYNAFLOW. Le but de cet article est d'étudier les effets de la saturation sur la susceptibilité de la liquéfaction du sol.

1. INTRODUCTION

Almost all deltaic sands such as Fraser River sand often contain free or dissolved gas (Grozic 2002). Presence of gas in soils can significantly modify the strength and liquefaction susceptibility of the soil. In fact, free gas in soil influences the pore fluid compressibility, and as a result increases the total compressibility of the soil element under undrained loading conditions. Recent cyclic triaxial tests conducted on Ottawa sand samples confirmed that the presence of gas in partly saturated soils enhances the resistance of the soil samples against liquefaction by 200% to 300% (Gorzic 2002, and Grozic et al. 2000b). The lower the initial degree of saturation, the higher the cyclic resistance of the soil sample. Moreover, due to compressibility of gas, the relative density of soil increases leading to higher resistance.

Constitutive relationships for partly saturated soils, based on introducing an equivalent Bulk modulus for the mixture of gas and water, have been suggested in papers by Pietruszczak and Pande (1991) and Pietruszczak and Pande (1996). In a more recent study by Grozic et al. (2000a), a constitutive soil model has been enhanced by considering the compressibility and solubility of the pore gas and liquids into constitutive relations based on the Hilf's equation (Fredlund and Rahardjo 1993). Hilf equation was derived based on the results of a one-dimensional oedometer test on a compacted soil, Boyle's law for ideal gas and Henry's law for solubility of gas into water (Fredlund and Rahardjo 1993) to calculate the change in pore pressure due to the application of total stress.

The purpose of this paper is to compare the numerical results obtained for seismic behaviour of an earth slope made of Fraser River sand with different degrees of saturation, and to emphasize the effects of partial saturation on the liquefaction susceptibility of soils. The numerical model used in this study is the multi-yield plasticity soil constitutive model implemented in the finite element program, Dynaflow (Prevost 2002). In order to simulate the effects of partial saturation using the available software, the value of the Bulk modulus of fluid phase is calculated based on the initial degree of saturation and assumed to remain constant.

2. NUMERICAL MODEL

The multi-yield plasticity soil constitutive model has been validated several times in the past for analysis of liquefaction phenomenon (e.g. Popescu and Prevost 1993). The model is a kinematic hardening model based on a relatively simple plasticity theory (Prevost 1985), and is applicable to both cohesive and cohesionless soils. Fundamental theory behind the model has originated from the concept of a "field of work-hardening moduli" (Mroz 1967) by approximating the nonlinear elastic plastic stress-strain curve into a number of linear segments with constant shear moduli. This is done by means of defining a series of nested yield surfaces in the stress space. Each yield surface corresponds to a region of a constant shear modulus. The outermost surface is related to zero shear modulus, and is called failure surface. Both Drucker-Prager and Mohr-Coulomb type surfaces can be employed in the model for frictional materials (sands) (Popescu 1995).

The plastic potential is assumed to be associative for its deviatoric component and non-associative for its dilatational (volumetric) component. The volumetric component accounts for the dependence of soil dilatational behavior on the mobilized stress ratio. The soil hysteretic behavior and shear stress-induced anisotropic effects are simulated by a purely devaitoric kinematic hardening rule (Prevost 1989). The procedure to calibrate the multi-yield plasticity soil model for the Fraser River sand used in the NSERC liquefaction Remediation Initiative (LRI) can be found in a paper by Jafari Mehrabadi and Popescu (2004). The same constitutive parameters have been used in this study and are shown in Table 1.

Table 1 Multi- yield plasticity soil constitutive parameters for Fraser River sand.

Constitutive parameters of Fraser River sand	Symbol	Values Loose	Values Dense
Mass density (kg /m³)	ρ_s	2710	2710
Porosity	n^w	0.448	0.406
Hydraulic conductivity (cm / s)	k	0.042	0.031
Low-strain shear modulus (MPa)	G_0	30	52.31
Reference effective normal stress (Kpa)	p0	100	100
Power exponent	n	0.5	0.5
Poisson ratio	ν	0.3	0.3
Friction angle at failure	ϕ	36°	42°
Coefficient of lateral earth pressure at rest	k_0	0.43	0.43
Maximum deviatoric strain (C=Compression and T=Tension)	ε_{dev}^{max}	0.08 (C) 0.08 (E)	0.01 (C), 0.008 (E)
Phase transformation angle	ψ	34°	34°
Dilation parameter	X_{PP}	0.48	0.01

3. MODEL ING PARTLY SATURATED SOILS

A simplified model to account for the effects of partial saturation is working with a value of Bulk modulus of the fluid phase as an equivalent Bulk modulus of the mixture of gas and water.

For this purpose, consider a soil element in which the volume of voids, V_v , equals to the sum of the volume of air, V_a , and the volume of water, V_w , i.e., Eq. 1.

$$V_v = V_a + V_w \quad [1]$$

Differentiation of Eq. 1 with respect to absolute pore pressure, u, leads to:

$$\frac{dV_v}{du} = \frac{dV_a}{du} + \frac{dV_w}{du} \quad [2]$$

Using the definition of the Bulk modulus for each phase, x, as $B_x = \frac{-du}{\left(\frac{dV_x}{V_x}\right)}$, the above relationship can have the following form:

$$\frac{V_v}{B_v} = \frac{V_a}{B_a} + \frac{V_w}{B_w} \quad [3]$$

Both V_w and V_a can be written in terms of V_v , and the degree of saturation, S.

$$V_a = (1\text{-}S)V_v \quad [4]$$
$$V_W = SV_v$$

Substituting Eqs. 4 into Eq. 3 results in:

$$\frac{1}{B_v} = \frac{1-S}{B_a} + \frac{S}{B_w} \quad [5]$$

This equation is the same as the equation given by Pietruszczak and Pande (1991) when the effects of fluid surface tension are neglected. In fact, B_v can be considered as an equivalent Bulk modulus of the mixture of gas and water. Note that for S=1(100% saturation) Eq. 5 leads to $B_v = B_w$, which is true for completely saturated soil.

The dimensionless form of Eq. 5 is:

$$\frac{B_v}{B_w} = \frac{1}{\frac{B_w}{B_a}(1-S) + S} \quad [6]$$

From Boyle's law, it can be proved that B_a is equal to the absolute pore pressure. For pore pressure equal to atmospheric pressure, the value of $\frac{B_w}{B_a}$ is about 20000.

Figure 1 shows the variation of $\frac{B_v}{B_w}$ (Bulk modulus ratio) versus degree of saturation for two different absolute pressures based on Eq. 6. As it can be seen, the presence of air (even 1%, i.e., S=0.99) tremendously decreases the value of the equivalent Bulk modulus of the mixture, and consequently increases the compressibility of the mixture of gas and water.

4. BOUNDARY VALUE PROBLEM

Due to man-made activities and the nature of the soil in the Fraser River Delta in British Colombia, this region is highly vulnerable to liquefaction hazards. In this regard, NSERC sponsors research to optimize the required mitigation measures against liquefaction for the Fraser River Delta by means of centrifuge experiments and numerical modeling. The liquefaction remediation initiative (LRI) includes soil laboratory testing, numerical modeling and centrifuge experiments to evaluate the performance of various soil liquefaction countermeasures. The primary goal of LRI is to optimize soil improvement methods for seismic hazards. The recommended base model to be studied is an earth slope made of Fraser River sand, and eight centrifuge tests with different mitigation configurations are being conducted within the framework of LRI (For details see Earthquake Induced Damage Mitigation from Soil Liquefaction website. 2003. Posted at: http://www.civil.ubc.ca/liquefaction/.).

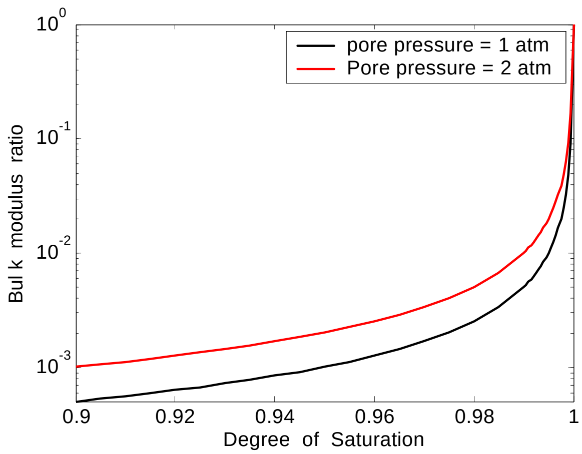

Figure 1. Variation of Bulk modulus ratio due to changes in degree of saturation for two different absolute pore pressures.

The problem being studied in this paper is related to the prototype scale slope used for the second LRI centrifuge test, with no soil improvement. The geometry and the acceleration time history used in this study are shown in Figure 2 (prototype scale). In this figure, EPP, LVDT, ACC are pore water pressure transducer, linear variable differential transducer, and accelerometer, respectively. The earthquake event A2475, shown in Figure 2, corresponds to 10% probability of occurrence in a 50-year period in B.C area (see the corresponding website mentioned above). The finite element model consists of 890 four-node elements with 4 degrees of freedom at each node, i.e. two for solid displacements and two for fluid velocities. The seismic motion is applied in horizontal direction at the base and lateral boundaries of the analysis domain similar to the rigid box used in centrifuge experiments. The base and lateral boundaries are assumed impervious.

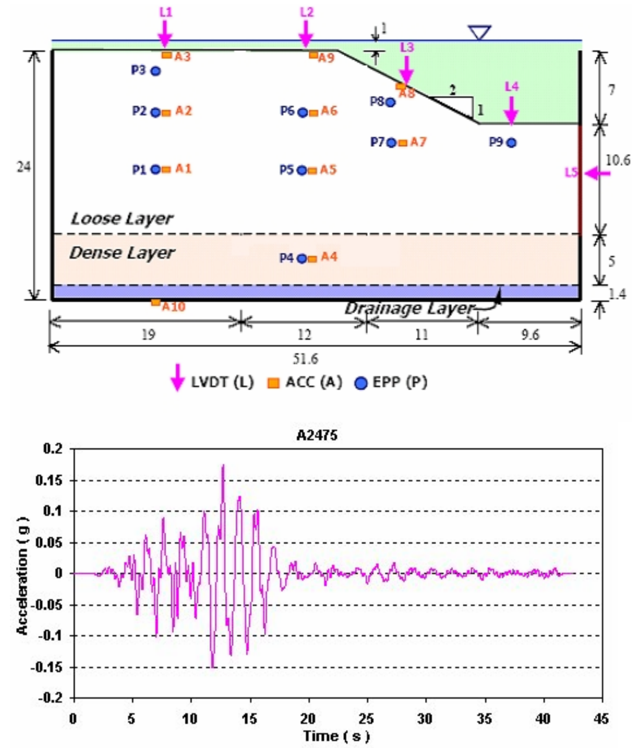

Figure 2. Slope geometry and horizontal acceleration time history applied to the base of the slope (modified from Earthquake Induced Damage Mitigation from Soil Liquefaction website, 2003, posted at: http://www.civil.ubc.ca/liquefaction/).

5. RESULTS AND DISCUSSION

In this section the results of numerical predictions of the aforementioned boundary value problem assuming different degrees of saturation for soil are briefly presented and discussed. In each case the equivalent Bulk modulus of the mixture of gas and water, B_v, is calculated based on Eq.6 assuming atmospheric pressure for B_a. Also, B_v is assumed to remain constant during the analyses. The degrees of saturation considered in this study are 100%, 99%, 98%, 96%, and 90%.

Partial saturation significantly affects the predicted excess pore water pressure. For instance, Figure 3 shows the predicted pore water pressure ratio time histories at EPP2 for different degrees of saturation. The predicted pore water pressure ratio time history is considerably reduced when degree of saturation decreases from 100% to 90% (RU=1 for S=100% and RU=0.5 for S=90% at t >20 s). Moreover, it is predicted that liquefaction does not occur at this location for S<98%.

The predicted pore water pressure ratio contours at a certain instant during earthquake (t=12 s) are shown in

Figure 4. The lower the degree of saturation, the smaller the pore pressure build-up.

Figure 5 shows contours of maximum shear strains at the end of earthquake along with deformed shapes of the slope for different degrees of saturation. It can be seen from this figure that the lower the degree of saturation, the smaller the predicted strains. In the case of 100% saturation, the predicted failure mechanism extends over the entire analysis domain, while it affects significantly smaller areas for lower degrees of saturation.

Figure 6 illustrates the predicted vertical displacement contours at the end of earthquake. It is predicted that vertical settlement in the free field close to the slope crest are significantly reduced due to decrease in degree of saturation, i.e. from 0.9 m for S=100% to 0.55m for S=90%. Also, lower degree of saturation results in remarkable reduction of the predicted heave at slope toe, i.e. from 1.1 m for S=100% to 0.2 m for S=90%.

Figure 7 shows the predicted vertical displacement time histories at certain transducers, i.e. LVDT1 in the free field upslope, LVDT2 at slope crest, and LVDT4 at slope toe. It is predicted that the assumed degree of saturation plays an important role in the value of vertical displacements predicted at these locations. In other words, partial saturation significantly decreases the liquefaction-induced displacements and settlements as illustrated in Figure 7.

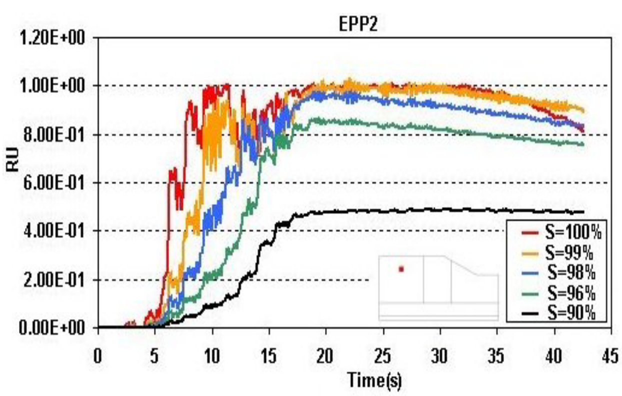

Figure 3. Predicted pore pressure ratio time histories at EPP2 for different degrees of saturation.

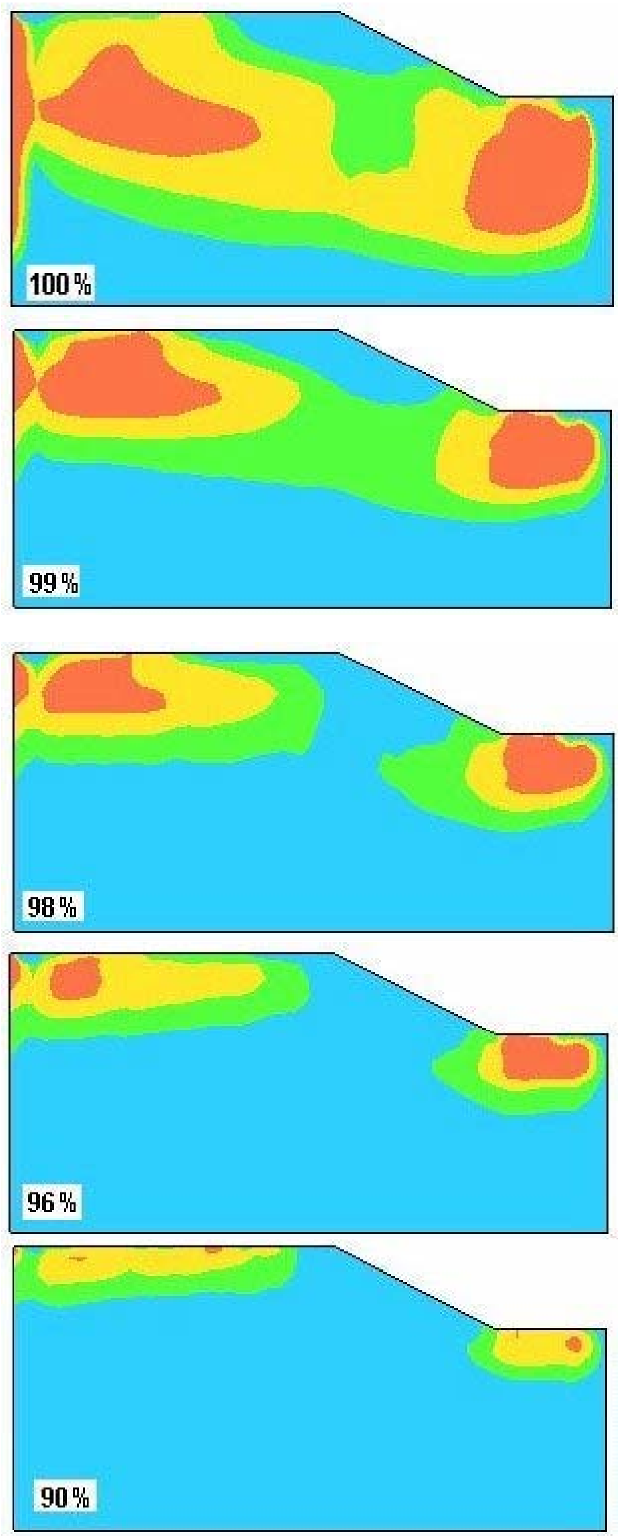

Figure 4. Predicted pore water pressure ratio contours at t=12 s after the beginning of the earthquake for different degrees of saturation.

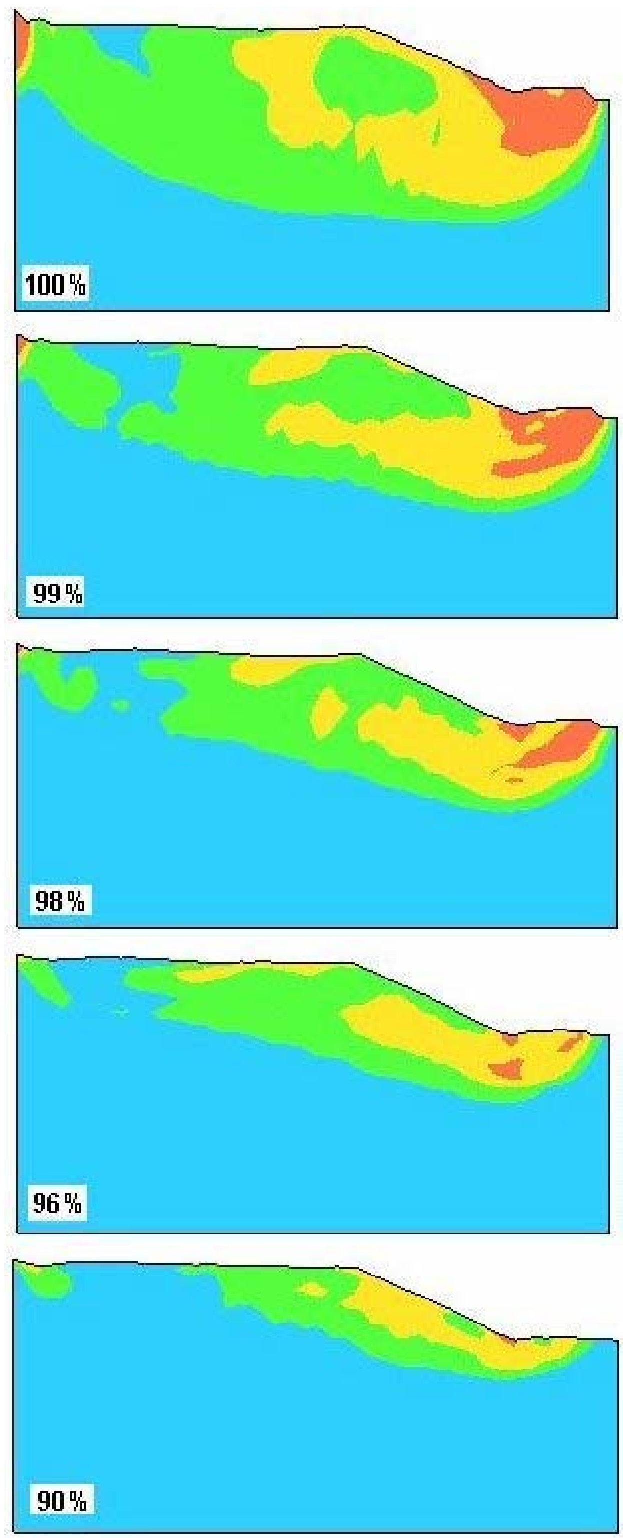

Figure 5. Predicted Maximum shear strain contours and deformed shape at the end of shaking for different degrees of saturation. Deformation magnification factor = 1.

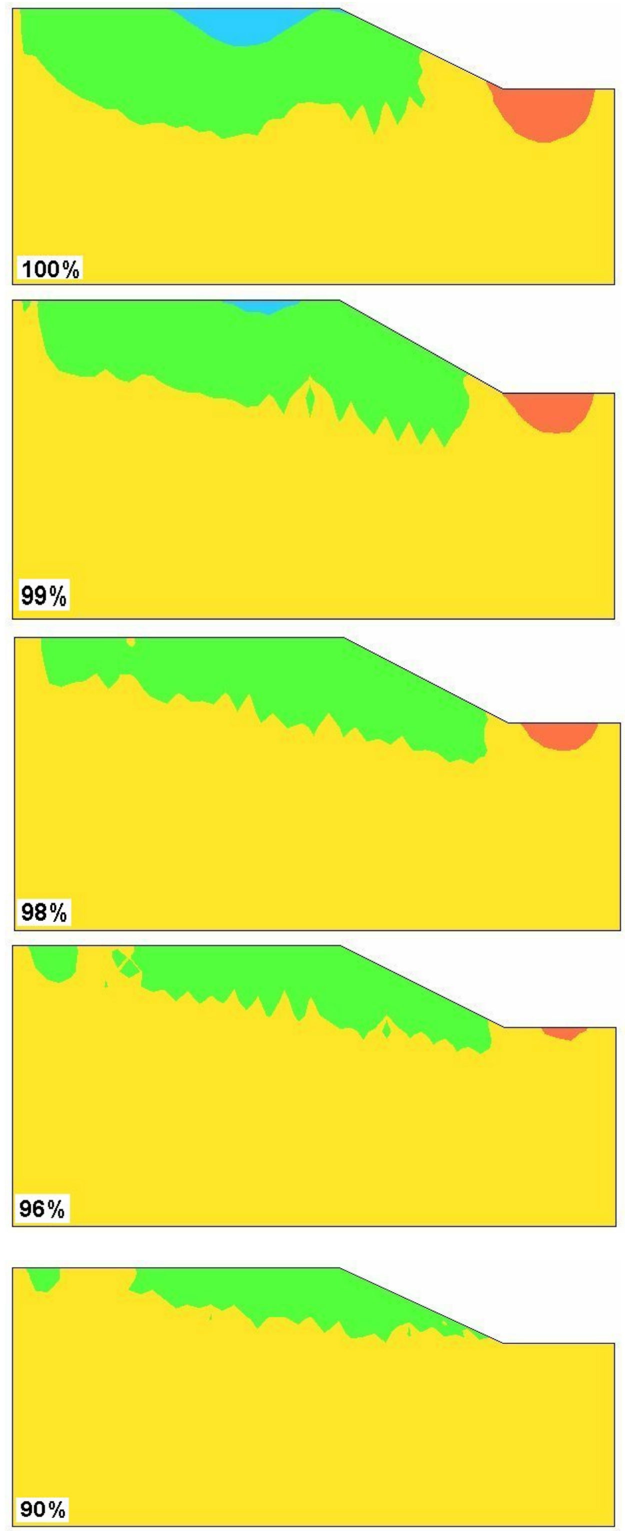

Figure 6. Predicted vertical displacement contours at the end of shaking for different degrees of saturation

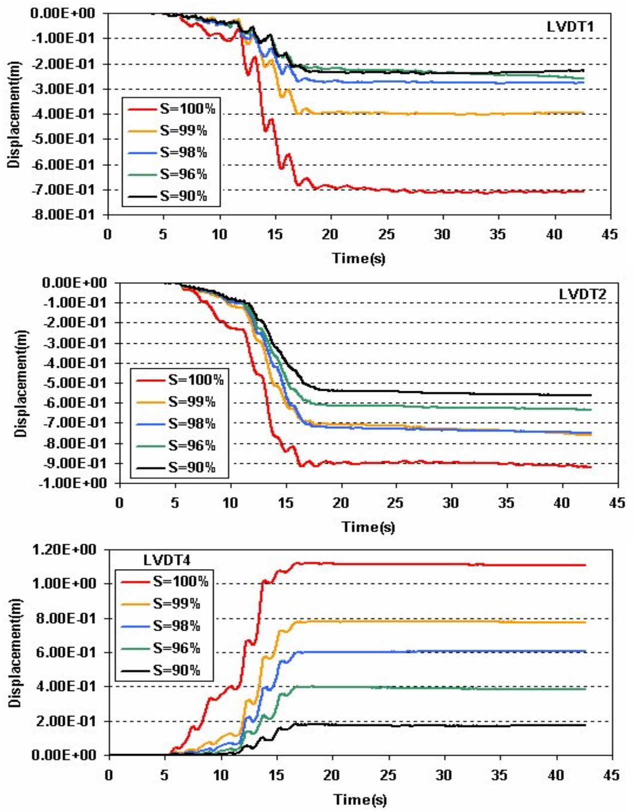

Figure 7. Predicted vertical displacement time histories at LVDT1, LVDT2, and LVDT4 for different initial degrees of saturation (see Figure 2 for location of LVDT's).

6. CONCULDING REMRAKS

The predicted results indicate slope failure along with settlement at slope crest and heave at toe in all cases. However, the magnitude of predicted displacements seems to be strongly influenced by the degree of saturation, and it resulted from this study that partial saturation may significantly affect the amount of seismically induced pore water pressure. Consequently, correct numerical simulation of the effects of partial saturation may result in significant savings in the cost of remediation techniques against liquefaction.

7. ACKNOWLEDGMENT

This research is financially supported by NSERC as part of a liquefaction remediation initiative and Grant No. RG203795-02. This support is gratefully acknowledged. Also, the authors are indebted to Professor J. H. Prevost, for providing the finite element code, Dynaflow, used in this paper.

8. REFERENCES:

Earthquake Induced Damage Mitigation from Soil Liquefaction website. 2003. Posted at: http://www.civil.ubc.ca/liquefaction/.

Fredlund, D.G. and Rahardjo, H. 1993. Soil mechanics for unsaturated soils. John Wiley & Sons, Inc., New York, New York.

Grozic, J.L.H. 2002. Liquefaction potential of gassy marine sands. Proceedings, First International Symposium on Submarine Mass Movements and Their Consequences, eds., Locat, J., and Mienert, J., Nice, Franc.

Grozic, J.L.H., Imam, S.M.R., Robertson, P.K., and Morgenstern, N.R. 2000a. Constitutive modeling of gassy soil behavior. Accepted for publication in the Canadian Geotechnical Journal.

Grozic, J.L.H., Robertson, P.K., and Morgenstern, N.R. 2000b. Cyclic liquefaction of loose gassy sand. Can. Geotech. J., Vol. 37: 843-856.

Jafari Mehrabadi, A. and Popescu, R. 2004. Solutions for mitigating soil liquefaction effects: A numerical study. Submitted to 13th World Conference on Earthquake Engineering. Vancouver, B.C., Canada

Mroz, Z., 1967. On the behavior of anisotrpic workhardening. J. Mech. Phys. Solids, 15: 163-175.

Pietruszczak, S. and Pande, G.N. 1996. Constitutive relations for partially saturated soils containing gas inclusions, Journal of Geotechnical Engineering, 122(1): 50-59.

Pietruszczak, S.,and Pande, G. N. 1991. On the mechanics of partially saturated soils. Computers & Geotechnics, vol. 12, no. 1, 1991.

Popescu, R. 1995. Stochastic variability of soil properties: Data Analysis, Digital Simulation, effects on system behavior. Ph.D. Thesis, Princeton University, Princeton, NJ.

Popescu, R., and Prevost, J.H. 1993. Centrifuge validation of a numerical model for dynamic soil liquefaction. Soil Dynamics and Earthquake Engineering, 12: 73-90.

Prevost, J.H., 2002. Technical manual of DYNAFLOW - Version 02, Release 01 A. Princeton University, Princeton, NJ.

Prevost, J.H. 1989. DYNA1D, a computer program for nonlinear seismic site response analysis. Technical report NCEER-89-0025. National Center for Earthquake Engineering research. State University of New York at Buffalo.

Prevost, J.H., 1985. A simple plasticity theory for frictional cohesinless soils. J. Soil Dynam. and Earthq. Eng., 4(1): 9-17.