Mitigative measures against soil liquefaction: Numerical modeling and centrifuge experiments

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Abstract:
A combined experimental and numerical research program was sponsored by NSERC to study and optimize the liquefaction remediation measures in the Fraser River Delta. The program included laboratory soil tests, eight centrifuge experiments and numerical modeling. This paper presents the numerical predictions performed at Memorial University of Newfoundland (MUN). Nonlinear, dynamic effective stress analyses have been performed using fully coupled solid-fluid equations for porous media, and a multi-yield plasticity soil constitutive model implemented in the finite element code DYNAFLOW. The numerical model has been calibrated based on the results of laboratory soil tests, information from the literature and back analysis of the first three centrifuge experiments. Lessons learned during the process of model calibration including some interesting phenomena such as effects of possible incomplete sand saturation in the centrifuge, effects of a low-permeability (barrier) layer and performance of the improved slope in centrifuge are discussed based on numerical simulations and observations in the centrifuge experiments. Limitations of the numerical model and centrifuge experiments are also discussed.

Key words: Soil liquefaction, Seismic resistant design, Finite elements, Centrifuge experiments.
1 Introduction

Liquefaction induced failures during past earthquakes have caused severe damage to many structures resulting in loss of human life and important economic losses (e.g., Seed et al. 2003; Ferritto 1997; Bardet et al. 1997). Various soil improvement techniques are employed to reduce the risk of soil failure caused by liquefaction. Optimization of such remediation methods may lead to great savings in investments without loss of effectiveness.

The Fraser River Delta in British Colombia is highly prone to liquefaction hazards. In this regard, Natural Sciences and Engineering Research Council of Canada (NSERC) sponsored a Liquefaction Remediation Initiative (LRI) to study and optimize a series of mitigation measures against liquefaction for the Fraser River Delta. LRI included laboratory soil tests for estimating the geomechanical properties of Fraser River sand, eight centrifuge experiments (tests CT1 to CT8) with different configurations to assess the performance of various soil liquefaction countermeasures, and numerical modeling for calibrating numerical procedures used for liquefaction analysis. The laboratory soil tests were performed at the University of British Colombia (UBC). The centrifuge experiments were conducted at C-CORE using a rigid box and a centrifugal acceleration of 70g. Two different methods were used for numerical modeling; one at UBC based on the finite difference code FLAC (Itasca Consulting Group Inc. 2000), and one at Memorial University of Newfoundland (MUN) based on the finite element code DYNAFLOW (Prevost 2002). The numerical predictions were class A predictions, i.e., they were performed and submitted prior to conducting the relevant centrifuge experiments. Industry partners also joined this research program to comment on the soil conditions and suggest mitigation solutions.
All the experimental results and numerical class A predictions are available on the LRI website (EIDMSL 2003). In addition, all the numerical results obtained at MUN, including class A predictions of tests CT1 and CT3 to CT8 and class C predictions (back analyses) of all tests are posted on the GEOSIM website (GEOSIM 2001) as time histories of accelerations, displacements and excess pore water pressures computed at all transducer locations. They are directly compared with the corresponding time histories recorded in the LRI centrifuge experiments.

Nonlinear, effective stress, fully coupled, dynamic finite element analyses of saturated soil slopes subjected to seismic input were performed using a multi-yield plasticity soil constitutive model (Prevost 1985) implemented in the finite element code DYNAFLOW (Prevost 2002). This model has been validated several times in the past for liquefaction analysis (e.g., Popescu and Prevost 1993, 1995). The layout of centrifuge experiments CT1 to CT8 along with the transducer locations is shown in Fig.1. In this figure, P, L, and A indicate pore water pressure transducer, linear variable differential transformer and accelerometer, respectively. The target base input acceleration time history in tests CT2 to CT4 was selected based on a level of seismic risk with 2% probability of exceedance in 50 years proposed for Vancouver area and labeled as event A2475 (Seid-Karbasi 2003). This event is shown in Fig. 2. In tests CT5 to CT8, the target base input acceleration was increased by a factor of two (2×A2475). The base input acceleration for test CT1 consisted of event A475 (shown in Fig. 2 and corresponding to a level of seismic risk with 10% probability of exceedance in 50 years) followed by event A2475. Specifications of all the LRI centrifuge tests are summarized in Table 1.
This paper presents the numerical model calibration procedure and class A predictions performed at MUN. The numerical model was initially calibrated based on the results of laboratory soil test performed at UBC within the framework of LRI. Recalibration of the model was subsequently done based on information from the literature and class C predictions of the first three LRI centrifuge experiments. Due to a large volume of data, only numerical results at a few selected transducers, believed to be representative for the overall slope behavior, are discussed here. However, comparisons of the numerical predictions and centrifuge experimental results at all transducer locations are available online on the GEOSIM website (GEOSIM 2001). A brief discussion on transducer locations selected to be discussed here is included at the beginning of Section 4.

The calibrated numerical model presented here has been used by Jafari-Mehrabadi and Popescu (2006) for understanding the mechanisms of seismic induced slope failure, investigating possible limitations of the LRI centrifuge experiments, and extending the scope of the centrifuge experimental program to study various configurations of the proposed mitigation measures as well as new mitigation solutions. Lessons learned during the process of model calibration including the effects of possible incomplete sand saturation in centrifuge, effects of low-permeability soil layers, boundary effects in centrifuge modeling, performance of mitigated waterfront slopes in centrifuge, as well as limitations of the numerical model are discussed in this paper. Limitations of the numerical model are also investigated and discussed.
2 Mathematical Model

The multi-yield plasticity soil constitutive model (Prevost 1985) implemented in the finite element code DYNAFLOW (Prevost 2002) has been used in this study. DYNAFLOW is a finite element code for solving coupled field equations for multiphase media based on an extension of the Biot formulation (e.g., Biot 1962) into the nonlinear regime (Prevost 1989). The multi-yield plasticity soil constitutive model is a kinematic hardening model based on a relatively simple plasticity theory (Prevost 1985). The nonlinear stress-strain curve is approximated by a number of linear segments with constant shear modulus corresponding to a series of nested yield surfaces in the stress space. The outermost surface is the failure surface. Both Drucker-Prager and Mohr-Coulomb type surfaces can be used for frictional materials (Prevost 2002). The plastic flow rule is associative in its deviatoric component. A non-associative flow rule is used for the dilatational (volumetric) component to account for the dependence of soil dilatational behavior on the mobilized effective stress ratio. The soil hysteretic behavior and shear stress-induced anisotropic effects are simulated by a kinematic hardening rule (Prevost 1989).

The soil constitutive parameters are listed in Table 2. They can be divided into state parameters (obtained from general laboratory soil tests), low-strain behavior parameters (describing elastic deformability), yield and failure parameters (used for generating the nested yield surfaces), and dilation parameters (used to calculate the plastic volumetric strain). It should be noted that parameter $k_0$ is only used by the software for generating the deviatoric stress-strain backbone curves (e.g., Griffiths and Prevost, 1990) and the initial locations of yield surfaces in the stress space (Prevost 1989). Its value depends on the type of consolidation employed in the laboratory soil tests used for calibrating the model parameters (anisotropic or isotropic consolidation). All
the necessary parameters of the multi-yield plasticity model except for the dilation parameter can be estimated from results of conventional field or laboratory soil tests. The dilation parameter, $X_{pp}$, is obtained by means of liquefaction strength analysis (e.g., Popescu and Prevost 1993) based on curve-fitting the experimental liquefaction strength curve using element tests (numerical simulations of undrained cyclic triaxial or simple shear tests).

The finite element meshes used in this study are shown in Fig. 3. The finite element model used for numerical predictions of tests CT1 to CT4 (Fig. 3a) consisted of 958 nodes and 890 finite elements and the finite element model used for numerical predictions of tests CT5 to CT8 (Fig. 3b) consisted of 588 nodes and 542 finite elements. The finite elements used in this research are 4-node linear elements with 4 degrees of freedom per node, two for solid displacements and two for fluid velocities. The seismic motion was applied in horizontal direction at the base and lateral boundaries of the finite element models to simulate the rigid box used in the centrifuge experiments. The base and the lateral boundaries of the analysis domain were assumed impervious.

3 Calibration of the Numerical Model

3.1 Set 1 based on cyclic simple shear tests on air-pluviated samples

The procedure for calibrating the multi-yield plasticity soil constitutive model for 40% relative density Fraser River sand based on the results of cyclic simple shear tests performed on air-pluviated Fraser River sand at UBC is presented by Jafari-Mehrabadi and Popescu (2004a). The corresponding constitutive parameters (for both loose and dense sands) are labeled hereafter as set 1 and listed in Table 2.
The state parameters have been selected from the results of laboratory soil tests performed at UBC (EIDMSL 2003). The value of Poisson’s ratio for sand was selected as $\nu = 0.3$ (e.g., Popescu and Prevost 1993). The power exponent, $n$, is used to simulate the dependence of the low-strain shear and bulk moduli, $G$ and $B$, on the effective mean normal stress, $p$, using the following expressions (e.g., Prevost 1989).

\[
G = G_0 \left( \frac{P}{P_0} \right)^n ; \quad B = B_0 \left( \frac{P}{P_0} \right)^n
\]

where $P_0$ is a reference effective confining stress and $G_0$ and $B_0$ are the values corresponding to $P_0$. The commonly accepted value of $n$ for sands is $n = 0.5$ (e.g., Richart et al. 1970).

The elastic range considered in this study for soil deformability (stress point inside the innermost yield surface) corresponds to a range of shear strains between 0 and 0.05%. Therefore, $G_0$ corresponds to the secant shear modulus at a shear strain equal to 0.05%. For a reference effective mean confining stress $P_0 = 100\text{kPa}$ and a shear strain of 0.05%, a value of about 30MPa can be inferred for $G_0$ of Fraser River sand based the results of isotropically consolidated drained triaxial test provided by Vaid and Eliadorani (2000). The corresponding value resulting from the cyclic simple shear tests performed at UBC (EIDMSL 2003) is about 14 MPa. The correlation in Eq. (2) proposed by Belloti et al. (1986) for very low shear strain levels ($10^{-4}$) results in a value of the initial sand shear modulus, $G_{\text{max}}$, of about 70 MPa at $D_r = 40\%$.

\[
G_{\text{max}} = 400p_a \exp\{1.39D_r\} \left( \frac{P_0}{p_a} \right)^{0.43}
\]

In Eq. (2), $p_a$ is the atmospheric pressure. Estimation of the sand shear modulus at a shear strain level of 0.05% based on the modulus degradation curves proposed by Hardin and Drnevich...
(1972) and Ishibashi and Zhang (1993) results in values equal to 66% and 57% of $G_{\text{max}}$, i.e., 46.2 MPa and 39.9 MPa, respectively. Based on the above information, a range for $G_0$ between 30 and 45 MPa seems reasonable. A value of 30 MPa was initially considered for $G_0$ of loose sand in set 1 as it was closer to the value inferred from the cyclic simple shear tests performed at UBC.

The values of friction angle are very scattered. For instance, the results of monotonic laboratory tests performed at UBC (EIDMSL, 2003) show an unexpectedly low value of about 27$^\circ$ at 40% relative density. The values of friction angle for Fraser River sand inferred from the results of monotonic undrained triaxial tests performed on very loose Fraser River sand at a relative density of about 20% (Vaid et al., 2001) are about 35.5$^\circ$ and 39$^\circ$ in compression and extension, respectively. A range for friction angle between 36$^\circ$ and 39$^\circ$ is consistent with the recommendations by the US Army Corps of Engineers (1992). A value equal to 36$^\circ$ was considered for the friction angle of Fraser River sand at $D_r = 40\%$ in set 1.

The coefficient of lateral earth pressure at rest, $k_0$, was taken as $k_0 = \frac{\nu}{1-\nu}$ to simulate anisotropically consolidated sand samples used in the cyclic simple shear tests conducted at UBC.

A value of 8% was initially considered for the maximum deviatoric strain at failure based on previous experience with other types of sands (e.g., Popescu and Prevost 1993).

The value of dilation angle (phase transformation angle) is independent of loading mode, type of deformation, and relative density (Vaid et al. 2001). A unique value of about 34$^\circ$ resulted for Fraser River sand from laboratory tests performed under different conditions (Vaid et al. 2001).
The dilation parameter, $X_{pp}$, was estimated by performing liquefaction strength analysis as described by Popescu and Prevost (1993). This analysis is based on back-fitting the experimental liquefaction strength curve using finite element simulations of cyclic undrained triaxial or simple shear tests (element tests). Cyclic undrained simple shear tests with and without initial static shear stress were performed at UBC on air-pluviated Fraser River sand samples (Wijewickreme et al. 2005). As shown by Byrne and Park (2003), liquefaction was considered to occur when excess pore water pressure ratio reached 95%. The tests without initial static shear were used to estimate the value of the dilation parameter in set 1. Fig. 4a shows the liquefaction strength curve obtained from the laboratory soil tests and the back-fitting by numerical simulations. As shown in Figure 4a, the numerical model (set 1) reasonably reproduced the experimental results in terms of liquefaction strength for $X_{pp} = 0.48$. Figs. 4b and 4c show the evolution of effective vertical stress and the stress path for a cyclic stress ratio, $CSR = 0.1$. The dilation parameter has been obtained based on the final number of cycles required for liquefaction.

The constitutive parameters initially estimated for loose ($D_r = 40\%$) and dense ($D_r = 80\%$) states of Fraser River sand are shown in Table 2 and labeled as set 1. In case of dense sand, the state parameters were derived from results of laboratory tests conducted at UBC. The given range for the values of elastic shear modulus of loose sand (i.e., 30-45 MPa) was modified for a relative density of 80% based on Eq. (2) and the modulus degradation curves discussed earlier. Based on the information from literature (e.g., US Army Corps of Engineers 1992) a reasonable range for the friction angle of dense sand would be $42^\circ$-$45^\circ$. The maximum deviatoric strain was estimated to be about 1% both in compression and extension based on the results of the monotonic undrained triaxial tests provided by Vaid et al. (2001). As discussed earlier, the dilation angle
(phase transformation angle) is independent of relative density and is equal to 34°. The dilation parameter of dense sand obtained based on the results of cyclic simple shear tests conducted at UBC (EIDMSL 2003) was $X_{pp} = 0.01$.

Numerical predictions of the first three centrifuge experiments, using set 1 of parameters, resulted in significantly softer predicted behavior (higher displacements and larger pore water pressure build up) than measured in the centrifuge experiments (e.g., Fig. 9 for test CT2). Moreover, the numerical model predicted a deep rotational slope failure (e.g., Fig. 8) while no slope failure was observed in the centrifuge experiments. At this juncture, it is mentioned that the numerical predictions performed at UBC using the UBCSAND constitutive model (e.g., Byrne et al.2004) implemented in FLAC (Itasca Consulting Group Inc. 2000) resulted in a similar predicted behavior with that obtained at MUN using set 1 of soil constitutive parameters (EIDMSL 2003).

**3.2 Discussion of constitutive parameters – Set 1**

Two possible causes of the mismatch between the numerical predictions and the experimental results are discussed here: (1) possible incomplete saturation of the soil in the centrifuge as was initially assumed by the LRI participants, and (2) discrepancy between the sand behavior in laboratory tests and centrifuge model.

The possible effects of incomplete saturation of the centrifuge models were investigated by Jafari-Mehrabadi and Popescu (2004b) by means of numerical analyses for centrifuge test CT2 at different initial degrees of saturation. A range of initial soil degrees of saturation (100%, 99%,
98%, and 96%) were considered. The constitutive parameters used in that study correspond to set 1.

The predicted excess pore water pressure ratio contours at a certain instant during strong shaking \( t = 12 \text{ s} \) shown in Fig. 5 indicate reduction in pore water pressure buildup with reduction in initial degree of saturation. Fig. 6 shows the predicted contours of maximum shear strain at the end of earthquake along with the deformed shapes of the slope for different initial degrees of saturation. In the case of 100% saturation, the predicted failure mechanism extends over the entire analysis domain, while it affects smaller areas for lower degrees of saturation. The predicted strain magnitudes also go down with the initial degree of saturation.

Fig. 7 shows the predicted vertical displacement time histories at location L2. It can be observed that the assumed initial degree of saturation plays an important role in the predicted value of crest settlement. Incomplete saturation significantly reduces the liquefaction-induced settlements. For instance, the predicted settlement decreases from 0.9m at \( S_0 = 100\% \) to 0.63m at \( S_0 = 96\% \).

The predicted results summarized in Figs. 5 to 7 for test CT2 (using set 1 of the soil constitutive parameters) indicate that the magnitude of predicted displacements are strongly influenced by the degree of saturation. However, for a degree of saturation as low as 96%, the numerical model continued to predict rotational slope failure and relatively large displacement (Figs. 6 and 7), which were not observed in the centrifuge experiment.
The second possibility for the mismatch between numerical class A predictions and centrifuge experimental results was hypothesized to be due to some differences in the soil behavior between the 1g soil laboratory tests and the centrifuge experiments. Air-pluviation method was used to prepare the centrifuge models in the LRI project at a relative density of 32%, which is lower than the target value (40%) to account for stress densification (Park and Byrne 2004b) due to mechanical handling and centrifuge model swing up (C-CORE 2004, and Park and Byrne 2004a). The soil samples in the laboratory cyclic simple shear tests were air-pluviated at 40% relative density, as no further densification was expected before starting the experiments.

During transportation of the centrifuge container with a lift truck the model was subjected to ambient vibrations for about 5-10 minutes (Tu 2005). Those vibrations at 1 g may have affected the very loose saturated sand in a similar manner as tapping or vibrating affects water pluviated samples. It is noted that specimens prepared by water-pluviation generally exhibit higher cyclic strength as compared to those prepared by air-pluviation (e.g., Amini and Chakravry 2004, Vaid et al. 1999, Vaid and Negussey 1988, and Tatsuoka et al. 1986a and 1986b). This is also valid for Fraser River sand (Wijewickreme et al. 2005).

Based on the above considerations and following a series of sensitivity analyses for various parameters of the multi-yield plasticity model used in set 1, the main reason for the mismatch between the predicted and recorded behavior was found to be related to differences between liquefaction resistance of the sand in the cyclic undrained laboratory experiments performed at UBC with air pluviated samples and that of the sand in the centrifuge experiments. Subsequent numerical calibration of the model based on laboratory soil tests conducted by Vaid et al. (2001)
on water-pluviated samples of Fraser River sand resulted in better predictions (e.g., Figs. 8 and 9). In the case of dense sand the environmental factors mentioned above are believed not to have influenced significantly the sand properties. In order to have a consistent calibration procedure, the dense sand parameters were also estimated in this study based on the results of the available tests on water pluviated samples (Vaid et al. 2001). Based on a numerical study, it was concluded that this change in constitutive parameter values for dense sand did not affect the overall results.

3.3 Set 2 based on cyclic triaxial tests on water-pluviated samples

A new set of soil constitutive parameters was developed based on the liquefaction strength derived from the cyclic undrained triaxial tests conducted on 40% relative density Fraser River sand by Vaid et al. (2001). Also, a series of other soil parameters (i.e., friction angle, low-strain shear modulus, maximum deviatoric strain, and permeability) were modified based the results of back-analyses of the first three centrifuge experiments within the range discussed earlier and information from Vaid et al. (2001). In this respect, friction angle and low-strain shear modulus were selected to be 39° and 45MPa corresponding to the highest values in the ranges discussed in Section 3.1. The value of maximum deviatoric strain was modified according to the results of monotonic triaxial tests provided by Vaid et al. (2001). The sand hydraulic conductivity was also modified based on the pore water pressure dissipation rates recorded in the first three centrifuge experiments, to match the experimentally observed behavior. All those new values are listed in Table 2 under set 2.
The modified soil parameters (set 2 in Table 2) were used in a new liquefaction strength analysis for estimating $X_{pp}$. The new liquefaction strength curve for loose sand (set 2) is shown in Figure 4a along with the results obtained for set 1. The correlation proposed by Castro (1975), Eq. (3), was used to obtain a unique cyclic stress ratio in both cyclic triaxial and simple shear tests:

$$\left[\frac{\tau_b}{\sigma_v}\right]_{\text{simple shear}} = \frac{2}{3} \frac{(1+2k_0)}{\sqrt{3}} \left[\frac{\sigma_d}{2\sigma_c}\right]_{\text{triaxial}}$$

In Eq. (3), $\tau_b$ is the amplitude of cyclic shear stress, $\sigma_v$ is the initial effective vertical stress, $\tau_b/\sigma_v$ is the CSR in cyclic simple shear tests, $\sigma_d$ is the amplitude of cyclic deviatoric stress, $\sigma_c$ is the initial mean effective stress, and $\sigma_d/2\sigma_c$ is the CSR in cyclic triaxial tests. For isotropically consolidated cyclic triaxial tests ($k_0 = 1$), Eq. (3) can be written as:

$$[\text{CSR}]_{\text{simple shear}} = \frac{2}{\sqrt{3}} [\text{CSR}]_{\text{triaxial}}$$

Comparisons of the numerical results using constitutive parameters corresponding to sets 1 and 2 in Table 2 with the experimental results of test CT2 are provided in Figs. 8 to 10. All results are shown at prototype scale.

Figs. 8a and 8b show the predicted contours of maximum shear strains along with the deformed shape of the model at the end of analysis. As discussed before, set 1 of soil parameters resulted in large displacements and a rotational failure mechanism (Fig. 8a), while when using set 2 of parameters, the numerical model predicted significantly lower displacements and no slope failure, similar to the experimental records. The predicted slope settlements (at L2) shown in Fig. 9a are much closer to the experimental records for set 2 than for set 1.
Figs. 9b and 9c show the predicted and measured excess pore water pressure ratios at P2 and P7, respectively. At P2, in the free field, both sets of parameters resulted in very good agreement with the results of the centrifuge experiment. At the location below the slope (P7 in Fig. 9c), the numerical model with set 2 captured the residual excess pore water pressure much better than with set 1.

Fig. 10 shows the predicted (sets 1 and 2) and measured acceleration time histories at location A7 under the slope. The predicted acceleration time history corresponding to set 2 is in very good agreement with the experimental results both in trends and in magnitudes.

The numerical model with set 2 calibrated based on cyclic triaxial test results (Vaid et al. 2001) and back analysis of CT2 was checked for CT1 and CT3. Fig. 11 shows the recorded vs. predicted (sets 1 and 2) settlement time histories at location L2. Again, in terms of predicted displacements and deformation modes, predictions using set 1 of the soil parameters resulted in significantly larger crest settlements than recorded in the centrifuge. When using set 2 of parameters, the numerical model predicted lower displacements and the results were close to the recorded responses in the centrifuge models (all the other numerical and experimental results for tests CT1 and CT3 are available online on the GEOSIM website, GEOSIM 2001).

3.4 Calibration of Other Soil Materials

As indicated in Fig. 1 and Table 1, centrifuge experiments CT4, CT5 and CT8 included drainage dykes as a mitigation strategy. In tests CT7 and CT8 an inclined silt layer with a slope of 1:5.7 was also included.
Relatively little information was available for the material properties of drainage dykes at the
time of class A predictions. This included grain size distribution, values of maximum and
minimum void ratios, hydraulic conductivity (about 100 times higher than that of the loose sand),
and mass density of the solid grains (C-CORE 2004). Therefore, all the other soil parameters
related to low-strain behavior, yield and dilation have been assumed equal to those calibrated for
loose sand except for the friction angle, which was assumed to be 41° (e.g., US Army Corps of
Engineers 1992).

No information was available for the silt properties. A trial set of parameters for this material
(based solely on engineering judgment) was used in class A prediction of test CT5. The silt
parameters were subsequently corrected following comparisons between predictions and
experimental records for test CT5 and also using the results of a centrifuge test (sand slope with
a silt layer) performed for another project (namely COSTA-Canada, see C-CORE 2005 for
details). The corrected values of the silt parameters used in class A predictions of tests CT7 and
CT8 are listed in Table 2.

The sand slopes in centrifuge tests were shaped using a vacuum cleaner to provide the designed
slope face. The model surface in all tests and also the sand below the silt layer in tests CT5, CT7,
and CT8 have been processed in this manner. It is believed that this procedure induced a thin
layer of very loose sand at the slope surface. While presence of such a layer at the surface of the
model may not influence the overall slope behavior, existence of a very loose sand layer
immediately below the silt may have important effects on pore water pressure development.
Therefore, it was decided to include a narrow layer of sand (about 1.5 cm thick at the model
scale) below the silt layer to simulate the effects of model preparation. The relative density of the sand in this layer was assumed to be 20%. A set of constitutive parameters was estimated for this narrow sand layer based on its assumed relative density and the dilation parameter was estimated based on the results of isotropically consolidated cyclic triaxial tests performed on water-pluviated Fraser River sand samples (Vaid et al. 2001). The constitutive parameters for the drainage dykes, silt layer and the narrow loose sand below the silt layer are also listed in Table 2.

4 Class A Predictions for tests CT4 and CT6 to CT8

Test layout and geometry of test CT5 is similar to those of test CT8, and this test is not discussed in this paper. Its class A prediction is available on the LRI project website (EIDMSL 2003). All the numerical results discussed hereafter correspond to set 2 of the soil parameters. Due to a large volume of data, only the results at selected transducers are discussed here. The selected locations and responses are: (1) slope crest, which is a critical point for characterizing slope settlements (transducers L2 for tests CT1 to CT4 and L3 for CT5 to CT8), (2) free field upslope where most waterfront structures are built (pore pressure transducers P2 for tests CT1 to CT4 and P5 for CT5 to CT8), and (3) the region under the slope to study the effects of initial static shear stresses (pore pressure transducer P7 and horizontal accelerometer A7). These locations are also consistent with the selected transducer locations discussed in Section 3.3 for test CT2 (Figs. 9 and 10). It is mentioned that comparisons of numerical and centrifuge experimental results at all transducer locations are available online on the GEOSIM website (GEOSIM 2001). Similar to test CT2, the predicted contours of maximum shear strain along with the deformed shape of the model at the end of analysis are also shown for the tests discussed in this section.
In tests CT6 to CT8, the predictions labeled here as “class C” were performed with the same soil parameters as used in the class A predictions. The only difference from class A predictions is using the base input accelerations recorded in the centrifuge tests instead of the target input accelerations.

4.1 Test CT4: Slope with a drainage dyke

The predicted and recorded displacement time histories at the slope crest (L2) are shown in Figure 12a. One possible reason for the difference between the numerical and experimental results may be associated with the soil parameters assumed for the drainage dyke. However, the recorded crest settlements in tests CT2 and CT4 (Figs. 9a and 12a) show an unexpected trend; a larger crest settlement was recorded in the centrifuge for the improved sand slope (about 26 cm in test CT4) than for the unimproved one (about 13 cm in test CT2), while the numerical model predicted 15 cm in test CT2 and 13 cm in test CT4 after soil improvement. It is possible that the dyke material in centrifuge was softer than assumed in the numerical model. At location P2 in the free field, the recorded and predicted excess pore water pressure ratio time histories are in good agreement (Fig. 12b). The numerical model predicted less pore water pressure than recorded at P7, located under the slope area with static shear stresses (Fig. 12c). Fig. 12d shows the predicted maximum shear strain contours at the end of analysis along with the deformed shape of the model. As it can be seen from this figure, the slope was predicted to be stable as observed in the centrifuge model. The predicted acceleration time history at A7 (Fig. 13) is in reasonable agreement with the recorded response for the class C prediction, where the actual recorded input motion is used in numerical predictions.
It is also mentioned that test CT1 to CT4 were not conclusive for the purpose of the LRI project as the unmitigated slopes did not fail. Consequently, a different slope geometry (Fig. 1b) and a larger target base input acceleration (2x A2475) were used in the remaining four LRI centrifuge experiments (CT5 to CT8) to investigate the performance and effectiveness of mitigation measures.

4.2 Test CT6: Uniform loose sand slope with increased seismic load

For this test, the numerical model predicted limited slope failure (crest settlement higher than 30cm with shear strains higher than 5% over an extended zone as shown in Figs. 14a and 14d). The recorded behavior was somehow stiffer (smaller displacements). This may be due to the fact that in test CT6 the model was spun twice to 70g; first for checking the model saturation by means of P-wave velocity measurements, and the second time for performing the actual test. This might have caused a higher degree of sand densification (Tu, 2005) and stiffer soil behavior in the centrifuge model. The recorded and predicted excess pore water pressure ratio time histories in the free field (P5) are in reasonable agreement (Fig. 14b). The numerical model predicted significantly less pore pressure build up at P7 (under the slope area) than recorded in the centrifuge experiment (Fig. 14c). Fig. 15 shows the comparison between the predicted and recorded acceleration time histories at A7. The numerical model did not capture the large negative spikes recorded in the centrifuge.

4.3 Test CT7: Similar to test CT6 with an inclined silt layer

The predicted crest settlements shown in Fig. 16a are in very good agreement with the experimental records. The pore water pressure time history at P5 was not recorded; however,
comparison between the numerical and centrifuge experimental results at an adjacent location (transducer P4 shown Fig. 16b) shows a reasonable agreement. In particular, the residual excess pore water pressure is well captured by the numerical model. As discussed in Section 5.2, it is believed that the significant dilative behavior predicted by the numerical model and recorded in the centrifuge model at location P4 is related to the proximity of the rigid box wall. As shown in Fig. 16c, at P7 the recorded response shows significant fluctuations, which were not captured by the numerical model; however, the predicted response is in the range of the mean values of the recorded excess pore water pressure. The predicted maximum shear strain contours of test CT7 at the end of analysis are shown in Fig. 16d along with the deformed shape of the model. The silt layer and the soil above it had a rigid block down-slope movement as also observed in the centrifuge model.

The predicted and recorded acceleration time histories at A7 are compared in Fig. 17. The negative recorded acceleration peaks seem to indicate sudden stops of down-slope displacements followed by small upslope movements as recorded by L3 (Fig. 16a). The numerical model did not capture the negative high values, and the predicted acceleration time histories are more symmetrical.

4.4 Test CT8: Similar to test CT7 mitigated by three drainage dykes

The comparison of the predicted and measured response at the slope crest (L3) is presented in Fig. 18a. The predicted settlements are lower than those measured in the centrifuge. However, as discussed for tests CT2 and CT4, again the recorded final crest settlement in test CT7 (about 45cm - without mitigation) is smaller than that recorded in test CT8 (about 65cm - with
mitigation. At P5 (Fig. 18b), during the last part of the earthquake, the numerical model predicted significant dilative response, not observed in the centrifuge; however, liquefaction was predicted to occur by the end of the earthquake. The residual excess pore water pressure ratio higher than 1 recorded at this location may be related to sinking of the transducer. Also, the numerical model predicted significantly more dilation at P7 compared to the centrifuge results as shown in Fig. 18c. The predicted maximum shear strain contours of test CT8 along with the deformed shape of the model at the end of analysis are shown in Fig. 18d. Due to the presence of drainage dykes, the numerical model predicted lower maximum shear strains below the silt layer, as compared to test CT7 (compare with Fig. 16d). Similar to test CT7, the predicted accelerations at location A7 are in the same range as the recorded ones, but the numerical model did not capture the large negative spikes recorded in the centrifuge experiment (Fig. 19).

5 Discussion of some numerical simulation results

5.1 Evolution of pore water pressure after the end of shaking

Fig. 20 shows the predicted excess pore water pressure ratio contours at different instants in tests CT7 and CT8. The numerical model predicted upward migration of the pore water after the end of the earthquake and its subsequent trapping below the silt layer. The same behavior was also observed in the centrifuge models (e.g., P5 in test CT8 shown in Fig. 18b). In fact, due to existence of a low-permeability soil layer in a liquefiable sand deposit, after pore water pressure buildup during an earthquake water may be trapped below a stratum with a relatively low permeability. This forms a water-rich seam under that layer causing reduction of the shear strength of soil along the seam (e.g., Kulasingam et al. 2004). If drainage is hindered for a long
time after the earthquake, delayed flow failure may take place, as observed in experimental
studies performed by Kokusho (1999).

Correct solution of the coupled field equations for nonlinear saturated porous media allows
simulation of pore water pressure build-up and dissipation as well as transient seepage and
enables the numerical model to capture this water migration after shaking.

5.2 Boundary effects due to rigid box

A rigid container was used in the LRI centrifuge experiments. This influenced the seismic soil
response in the proximity of lateral boundaries. A numerical study was performed to investigate
the effects of rigid boundaries by extending laterally the original FE mesh used in the numerical
predictions. The centrifuge experiment CT7 was selected for this study. The input motion was
the actual horizontal acceleration time history recorded on the rigid box in test CT7. The original
finite element mesh was extended on both sides in three stages by 25m, 50m, and 75m,
respectively. The difference between the results related to mesh extensions #2(50 m) and
#3(75m) was not significant in terms of crest settlement; consequently, an extension of 50m on
both sides was considered adequate to eliminate the boundary effects in the slope area in the
finite element analysis. The recorded excess pore water pressure ratio time histories at P4 and P7
are shown in Fig. 21. Also, the predicted responses at P4, P5 and P7 corresponding to the
original finite element mesh and finite element mesh extensions #1 and #2 are shown in Fig. 22.
As shown in Fig. 22a, the significant dilative behavior recorded at P4 and predicted at this
location using the original finite element mesh disappeared completely by extending the analysis
domain. Also, extending the finite element mesh resulted in decreasing the dilative responses
predicted at location P5 as shown in Fig. 22b. The predicted response at location P7 (Fig. 22c) did not significantly change by extending the analysis domain. This may be related to the fact that this transducer was far enough from the lateral walls of the rigid box to be affected by boundary conditions and soil dilative behavior at this location was due to presence of static shear stress. The proximity of rigid lateral boundaries may have also affected crest settlements at location L3 (Fig. 23). The finite element model with extended analysis domain predicted crest settlements about 20% larger than those predicted using the original finite element mesh.

The predicted contours of maximum shear strains and the deformed shapes for the original finite element mesh and extensions #1 and #2 are shown in Fig. 24. Relatively large local shear strains close to the upslope boundaries are predicted when using the original mesh and extension #1 (Figs. 24a and 24b). For extension #2 these effects are significantly attenuated. This may lead to the conclusion that high shear strains predicted close to the lateral boundaries are also related to the proximity of these boundaries to the slope. Moreover, it is obvious that proximity of lateral rigid boundaries affects the failure mechanism. As the lateral boundaries are moved away from the slope, a deep failure surface extends both laterally and in depth. In this study the effects of the rigid base of the box were not assessed (it was assumed that this model simulated the presence of a rigid sand layer at a certain depth). It is obvious from the results in Figs. 24b and 24c that the base rigid boundary also affects the failure mechanism. This is also reinforced by the predicted crest settlements shown in Fig. 23, that are slightly lower for extension #2 (with failure mechanism more affected by the base boundary) than for extension #1.
5.3 Performance of the improved slope in test CT8

As shown in Fig. 1b, three vertical drainage dykes were placed in the LRI centrifuge test CT8 (as well as CT5) to mitigate the consequences of soil liquefaction in the slope. While existence of these dykes in test CT8 locally reduced the excess pore water pressure below the silt layer (e.g., compare the predicted results shown in Fig. 20 for tests CT7 and CT8), this measure did not reduce the slope crest settlement as compared to the unmitigated model (test CT7). Results of numerical simulations of test CT8 with extended finite element domain performed by Jafari-Mehrabadi and Popescu (2006) indicated that the drainage dykes used in test CT8 reduce excess pore water pressure build up only in their immediate vicinity, and that they are not strong enough to resist high shear stresses developed along the silt layer. In fact, in both centrifuge experiment CT8 and its numerical simulations (with and without extended finite element analysis domains) drainage dykes are sheared at the level of the silt layer. Based on a numerical study, Jafari-Mehrabadi and Popescu (2006) showed that use of a single sheet pile in front of the slope is more effective than the drainage dykes used in test CT8 for a wide range of earthquake intensities.

5.4 Limitations of the numerical model

More dilative behavior for the soil below the slope (regions with initial static shear stress) was consistently predicted by the numerical model than recorded in the centrifuge experiments (i.e., Figs. 9c, 12c, 14c, and 18c). The same tendency was observed during calibration of set 1, where a lower liquefaction resistance obtained in the laboratory for the undrained cyclic simple shear tests with static shear stress than for the ones without static shear stress could not be reproduced by the numerical model. While larger liquefaction resistance (more dilative behavior) in the presence of static shear is a common characteristic of most sands, for sands in a very loose state,
static shear stress may reduce the liquefaction resistance (e.g., Vaid et al., 2001 and Youd et al., 2001). Apparently, the latter feature is not correctly simulated by the numerical model. It is noted however that another version of the multi-yield plasticity model with a more advanced dilatancy formulation is already implemented in DYNAFLOW. This is a double plastic potential model based on the experimental work of Pradhan and Tatsuoka (1989) that leads to a more accurate simulation of plastic dilation under cyclic loading. This model, however, has two dilation parameters, instead of one, namely $X_{pp}$ in the model used in this research. Calibration of two dilation parameters requires very detailed results of cyclic undrained tests that were not available for the research presented here. Consequently, it was decided to use the model with single plastic potential that could be reasonably calibrated from soil data available for this study.

The fact that the numerical model did not capture large negative acceleration peaks observed in some of the centrifuge experiments and possibly associated with short up-slope displacements (e.g. Seid-Karbasi et al. 2005) may be also related to the limitation discussed before: the slope soil in the numerical model was too stiff to allow such displacements.

6 Conclusions

A multi-yield plasticity soil constitutive model implemented in the finite element program DYNAFLOW was used in this study to perform class A predictions of the LRI centrifuge tests. The numerical model was calibrated using the laboratory soil data on Fraser River sand, information from the literature, and results of the first three centrifuge experiments. The results were presented in terms of time histories of accelerations, excess pore water pressures, and displacements and were directly compared to the centrifuge experimental records.
The mitigation measures studied in the LRI research program consisted of dense soil dykes and drainage dykes. However, the first four experiments, including both types of dykes, were not conclusive due to the fact that the unmitigated slopes did not fail. Therefore, only one type of mitigation, drainage dykes crossing an impervious barrier layer, as included in the second series of centrifuge experiments was discussed here. The drainage dykes tested in these experiments reduced locally the pore water pressures below the barrier layer (and thus they were able to prevent a possible delayed slope failure) but, they were too weak to resist significant shear deformations during the earthquake.

Some phenomena that could have induced significant differences between numerical predictions and experimental results were investigated in the process of numerical model calibration. These included effects of possible incomplete sand saturation in centrifuge models and possible discrepancy between the liquefaction resistance of the soil samples in laboratory cyclic simple shear tests and that of the soil in centrifuge models. Accumulation of excess pore water pressure under a low-permeability soil layer after the shaking and boundary effects due to use of a rigid box in centrifuge were also discussed based on numerical simulation results.

Some limitations of the numerical model were discussed based on the cases where significant discrepancies between the predicted and recorded responses were observed.
Acknowledgments

This research was supported by NSERC as part of a liquefaction remediation initiative and Grant No RG203795-02. This support is gratefully acknowledged. The authors are also indebted to Professor J. H. Prevost for providing the finite element code DYNAFLOW, to Dr. R. Phillips and the geotechnical team at C-CORE for providing the results of centrifuge experiments, to Professor P. M. Byrne for discussions and result analysis and to his collaborators at UBC for providing the laboratory test results.
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<table>
<thead>
<tr>
<th>Centrifuge experiments</th>
<th>Test configuration and Mitigation strategy</th>
<th>Target Acceleration</th>
<th>Figure</th>
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<tr>
<td>CT1</td>
<td>Uniform loose sand No soil improvement</td>
<td>A475 followed by A2475</td>
<td>1a</td>
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<td>Uniform loose sand No soil improvement</td>
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<td>1a</td>
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<td>CT3</td>
<td>Uniform loose sand Densification at dyke</td>
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<td>CT4</td>
<td>Uniform loose sand Drainage dyke</td>
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<td>Uniform loose sand with silt (barrier) layer; Drainage through 3 vertical dykes</td>
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<td>1b and 1c</td>
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<td>Uniform loose sand without silt (barrier) layer No soil improvement</td>
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<td>1b and 1d</td>
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<td>CT7</td>
<td>Uniform loose sand with silt (barrier) layer No soil improvement</td>
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<td>1b and 1d</td>
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<tr>
<td>CT8</td>
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<td>2 × A2475</td>
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Table 2 Constitutive parameters of different soil materials.

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<th>Set 2</th>
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<th>Set 2</th>
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<th>Silt</th>
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(1) Below the silt layer in tests CT7 and CT8
(2) At a mean effective confining stress $p_0 = 100$ kPa
(C) Compression
(E) Extension
Fig. 1 Layout of the LRI centrifuge experiments: (a) slope geometry and instrumentation layout in tests CT1 to CT4 (modified from EIDMSL 2003), (b) slope geometry in tests CT5 to CT8, (c) instrumentation layout in tests CT5 and CT8, and (d) instrumentation layout in tests CT6 and CT7. In this figure, P, L and A indicate pore water pressure transducer, linear variable differential transformer and accelerometer, respectively. All dimensions are in meters.
Fig. 2 Events A475 and A2475 used as the target base input acceleration time histories in LRI centrifuge experiments (see Table 1).
Fig. 3 Finite element meshes used in numerical class A and class C predictions of the LRI centrifuge tests.
Fig. 4 Liquefaction strength analysis: (a) Experimental and numerical liquefaction strength curves for Fraser River sand, (b) effective vertical stress vs. number of cycles corresponding to the middle point of the liquefaction strength curve from UBC shown in Figure 4a, and its numerical simulation using set 1 of soil parameters, and (c) experimental and numerical stress paths corresponding to Fig. 4b.
Fig. 5 Test CT2: Predicted excess pore water pressure ratio contours at $t = 12s$ after the beginning of the earthquake for different initial degrees of saturation.
Fig. 6 Test CT2: Predicted Maximum shear strain contours and deformed shape of the slope at the end of shaking for different initial degrees of saturation. Deformation magnification factor = 1.
Fig. 7 Test CT2: Recorded and predicted vertical displacement time histories at location L2 for different initial degrees of saturation.
Fig. 8 Predicted contours of maximum shear strain and deformed shapes at the end of analysis in test CT2 using (a) set 1 of soil parameters, and (b) set 2 of soil parameters.
Fig. 9 Test CT2: Recorded vs. predicted (sets 1 and 2) time histories at locations (a) L2, (b) P2, and (c) P7 (IEVS = initial effective vertical stress).
Fig. 10 Test CT2: Recorded vs. predicted acceleration time histories at location A7 (sets 1 and 2 of soil parameters).
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Fig. 16 Test CT7: (a) Recorded vs. predicted (class A and class C) time histories at location L3, (b) predicted time histories at location P5 (no record available), (c) recorded vs. predicted time histories at location P7, and (d) predicted maximum shear strain contours at the end of analysis (IEVS = initial effective vertical stress).
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Fig. 18 Test CT8: (a), (b), and (c) Recorded vs. predicted (class A and class C) time histories at locations L3, P5, P7, respectively, and (d) Predicted maximum shear strain contours at the end of analysis (IEVS = initial effective vertical stress).
Fig. 19 Recorded vs. predicted acceleration time histories in test CT8 at location A7.
Fig. 20 Predicted contours of excess pore water pressure ratios during and after the end of earthquake in tests CT7 and CT8.
Fig. 21 Recorded excess pore water pressure ratio time histories in test CT7 at locations (a) P4, and (b) P7 (IEVS = initial effective vertical stress).
Fig. 22 Test CT7: Comparisons of the predicted excess pore water pressure ratio time histories using the original FE mesh and FE mesh extensions #1 and #2 at locations (a) P4, (b) P5 and (c) P7, respectively. Mesh extensions 1 and 2 correspond to 25 m and 50 m extension of the original FE mesh analysis domain from both sides (IEVS = initial effective vertical stress).
Fig. 23 Comparison of the recorded and predicted crest settlement time histories in test CT7 for the original FE mesh and extensions #1 and #2.
Fig. 24 Predicted contours of the maximum shear strains at the end of analysis in test CT7 using, (a) the original FE mesh, (b) extension #1 and (c) extension #2.