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Response of sloping ground with liquefiable materials during an earthquake: a class A prediction

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Keywords: liquefaction, dynamic analysis, centrifuge test, UBCSAND model

ABSTRACT: Currently, liquefaction induced deformations are assessed without consideration of stiffness loss of soil although this will affect the dynamic characteristics of the structure. Stiffness and strength loss can be accounted for in total or effective stress dynamic analyses. An effective stress approach is a fundamental procedure for predicting the pore pressure and deformations. Validation of such procedures can be made using physical model data. This paper presents the results of a Class-A prediction of response of a slope shaken with two subsequent earthquakes events using numerical and centrifuge modeling. It accounts for re-liquefaction of material during the second event.

1 Introduction

A number of structures and life line facilities have failed or suffered large displacements during past earthquakes. In most cases the damage has occurred as a result of a large drop in the stiffness and strength of soil referred to as liquefaction.

Our understanding of the seismic behavior of earth structures has increased over the years due to:

- Observations from field case histories,
- Extensive laboratory testing of soil elements under cyclic loading,
- Model testing of earth structures under simulated earthquake loading, and
- Development of numerical modeling procedures.

Sophisticated numerical methods can enhance our insight into the behavior of earth structures during earthquake. Validation of such methods against measured response is vital for their acceptance as design tools. Ideally, an actual soil structure should be selected. However, even for the best field case histories, such as the San Fernando dams during the 1971 San Fernando earthquake, neither the input motion nor the soil conditions are adequately known. Model tests can be conducted in the laboratory under controlled conditions and their response observed. Because soil behavior is highly stress dependent, however, small models under a 1g acceleration field are not representative of field conditions. On the other hand, centrifuge tests that utilize a high acceleration field, preserve the stress–strain response of the prototype soil and can give a more realistic representation of field behavior. A major validation initiative (Arulanandan and Scott 1993) using centrifuge tests was carried out in the 1990s. Some related issues to that investigation were the boundary conditions in the centrifuge box, the use of water as a pore fluid, the resulting high rate of drainage, and the lack of verification of saturation. The validation process also showed the necessity of models to rationally consider the generation and dissipation of excess pore pressure during shaking. In 2001, a joint research program "Earthquake Induced Damage Mitigation from Soil Liquefaction" using dynamic centrifuge testing and numerical modeling was initiated by University of

British Columbia (UBC) and C-CORE, Canada. UBC is leading the project and carrying out numerical analyses and C-CORE performs the centrifuge tests and also some analyses (see www.civil.ubc.ca/liquefaction for more details).

In this paper a numerical procedure is used in which both generation and dissipation of pore fluid pressure are considered. The procedure is applied to predict the results of centrifuge tests that investigate liquefaction of sloping ground conditions (Class A prediction). The characteristic liquefaction behavior of Fraser River sand used in the models was obtained from undrained cyclic simple shear tests and is the basis for the numerical predictions of the centrifuge tests. The model was shaken by two simulated earthquake events causing liquefaction, with full drainage between events. The analyses consider re-liquefaction of soil deposits to predict centrifuge test results. A comparison of predicted and measured centrifuge model response is presented in this paper. Prior to the prediction, a brief description of the numerical model is presented.

2 Soil liquefaction

Seismic liquefaction refers to a sudden loss in stiffness and strength of soil due to cyclic loading effects of an earthquake. The loss arises from a tendency for soil to contract under cyclic loading, and if such contraction is prevented or curtailed by the presence of water in the pores that cannot escape, it leads to a rise in pore water pressure and a resulting drop in effective stress. If the effective stress drops to zero (100% pore water pressure rise), the strength and stiffness also drop to zero and the soil behaves as a heavy liquid. However, unless the soil is very loose it will dilate and regain some stiffness and strength, as it strains. If this strength is sufficient, it will prevent a flow slide from occurring, but may still result in excessive displacements commonly referred to as lateral spreading. In addition, even for level ground condition where there is no possibility of a flow slide and lateral movements may be tolerable, significant settlements may occur due to dissipation of excess pore water pressures during and after the period of strong ground shaking.

Liquefaction assessment involves addressing the following concerns:

- Will liquefaction be triggered in significant zones of the structure for the design earthquake, and
- If so, could a flow slide occur, and if not,
- Are the displacements tolerable?

These effects can be assessed from state-of-practice total stress analyses procedures or from state-of-art effective stress analysis procedures.

State-of-Practice procedures address the above three concerns with 3 separate analyses; a triggering analysis, a flow slide analysis, and a displacement analysis. Such analyses are not fundamental and are not likely to improve our basic understanding of the liquefaction process because they do not directly consider pore pressures in the prediction of liquefaction. Pore pressures are indirectly allowed for in the reduced strength and stiffness values used after liquefaction is triggered.

In State-of-Art effective stress dynamic analyses, pore water pressures are generated in response to the applied earthquake motion and the stiffness and strength of the soil modified accordingly as shaking takes place. More rigorous analyses are based on an elastic plastic stress strain law for the sand skeleton that includes shear induced plastic volumetric strains, and it is these strains under the constraint of the pore fluid stiffness that generate pore water pressure changes. Such an approach allows coupled dynamic stress-flow analyses to be carried out in which both generation and dissipation of pore water pressures and their effects are considered for a specific base motion. The calibration and verification of such models is important and generally involve a 2-step process:

- Simulate and capture the element behavior as observed in laboratory cyclic tests; simple shear, triaxial, and hollow cylinder;
- Simulate and compare predicted and observed dynamic response for a soil structure.

Centrifuge tests that utilize a high acceleration field preserve the stress–strain response of the prototype soil and can give a realistic representation of field behavior. Such models, when subjected to a controlled base motion, can provide a database for the validation of numerical approaches. For this reason, verification is currently based on dynamic Centrifuge tests. Fully coupled effective stress approaches have been developed by many researchers. In this paper, the UBCSAND model as described by Beaty & Byrne (1998) is applied to predict response of a model earth structure model during dynamic centrifuge testing. The UBCSAND model is briefly described below.

3 UBCSAND constitutive model

The UBCSAND constitutive model is based on the elastic–plastic stress–strain model proposed by Byrne et al. (1995), and has been further developed by Beaty & 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 & Byrne 2004a). It has also been used to examine dynamic centrifuge test data (Byrne et al. 2004). It is an incremental elastic–plastic model in which the yield loci are lines of constant stress ratio ($\eta = \tau / \sigma'$). The flow rule relating the plastic strain increment directions is non-associated and leads to a plastic potential defined in terms of dilation angle. More details on the model can be found in Byrne et al. (2004).

The response of sand is controlled by the skeleton behavior outlined above. The presence of a fluid (air water mix) in the pores of the sand acts as a volumetric constraint on the skeleton if drainage is fully or partially curtailed. It is this constraint that causes the pore pressure rise that can lead to liquefaction. Provided the skeleton or drained behavior is appropriately modeled under monotonic and cyclic loading conditions, and the stiffness of the pore fluid and drainage are accounted for, the liquefaction response can be predicted.

This model was incorporated in the commercially available computer code FLAC (Fast Lagrangian Analysis of Continua) Version 4.0 (Itasca 2000). This program models the soil mass as a collection of grid zones and solves the coupled stress flow problem using an explicit time stepping approach.

The key elastic and plastic parameters can be expressed in terms of relative density, Dr, or normalized standard penetration test values, $(N1)_{60}$. Initial estimates of these parameters have been approximated from published data and model calibrations. The response of sand elements under monotonic and cyclic loading can then be predicted and the results compared with laboratory data. In this way, the model can be made to match the observed response over the range of relative density or $(N1)_{60}$ values. The model has also been calibrated to reproduce the NCEER 97 (Youd et al. 2001) triggering chart, which in turn is based on field experience during past earthquakes and is expressed in terms of Standard Penetration Test resistance value, $N1_{(60)}$.

The model was applied to simulate cyclic simple shear tests under undrained condition. Figure 1 shows model predictions along with test results on Fraser River sand with vertical consolidation stress σ'_v =100 kPa and Dr=40% (Wijewickreme et al. 2004). The results in terms of stress-strain, and excess pore pressure ratio, Ru compare reasonably well with the laboratory data. 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 Figure 1c and shows good agreement. The above simulations illustrate that the model can generate the appropriate pore pressures and stress strain response for undrained loading. The model can also account for the effect of volumetric expansion caused by inflow of water into an element as demonstrated by Seid-Karbasi & Byrne (2004b).

4 Centrifuge model configuration

Figure 2 shows the configuration of the centrifuge model that comprises of a submerged slope of Fraser River sand with Rd = 40% underlain by a dense layer, Dr = 80% of the same sand. At the bottom of the model there is a coarse layer for model saturation. The centrifuge test was carried out in a 70g acceleration field in a rigid container converted from laminar box (flexible boundaries result in undesired displacements due to out of phase motions of the container and unsymmetrical soil model when large deformations occur). All dimensions and parameters are reported in prototype scale. A viscous fluid with viscosity 35 times that of water was used to partially compensate for the discrepancy between diffusion time and dynamic time in the centrifuge test. Figure 2 also shows position of instruments designed to monitor the slope response during shaking. Two input motions called A475 and A2475 shown in Figure 3 were applied at the base of the model.

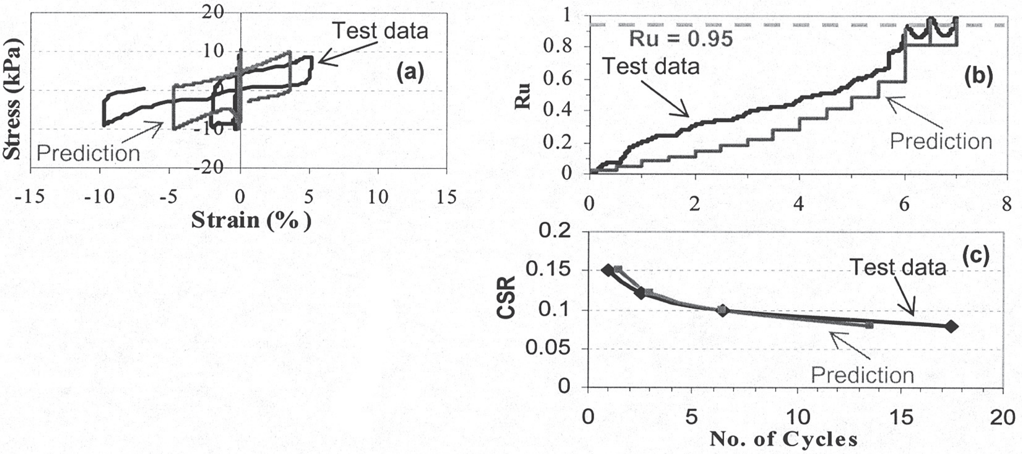

Figure 1. Comparison of predicted and measured response for Fraser River Sand, a) stress-strain, b) & c) Ru & CSR vs. No. of cycles.

The records were filtered to exclude frequencies less than 0.57Hz and greater than 2.86Hz and baseline-corrected. The model was shaken first with the A475 event, and as it did not fail, it was allowed to consolidate and then subjected to the A2475 event.

5 Analyses results

The mesh used in numerical modeling is shown in Figure 4. The materials properties are listed in Table1. The model was first analyzed for A475 event and no significant failure deformation was predicted, in agreement with observation from the test. After dissipation of all excess pore pressure the second event, A2475 was applied. The results for the second event are presented here.

The laboratory test results with two stages of cyclic shearing indicate that materials characteristic properties of the sand at the second stage are different from those of the first stage (Wijewickreme et al, 2004). Table 2 summarizes the laboratory cyclic simple shear tests results on Fraser River sand for the two loading stages. It may be seen that first stage loading results in sand densification and increased number of cycles to cause liquefaction on the second stage provided the sample did not liquefy on the first stage. However, if the sand liquefied during the first stage the number of cycles to cause liquefaction decreased on the second stage. Figure 5 shows variation of No. of cycles vs. Ru observed in the laboratory tests (dashed line) as well as that considered in the analysis (solid lines). The cyclic resistance gain observed after the first stage loading was lost if Ru =100% or dilation occurred. Finn et al (1970) observed the same phenomenon in their pioneering laboratory investigations. Oda et al. (2001) also explained this behavior based on a micro-structural interpretation. During the A475 event the maximum pore pressure ratio and whether or not dilation occurred were monitored for each element. The elements were then allowed to consolidate and their resistance increased in accordance with Figure 5. Predicted values of $Ru_{(max)}$ due to A475 event are shown in Figure 6. It may be seen that high values, in excess of unity, are predicted adjacent to the lateral fixed boundaries. These elements carry low vertical effective stresses due to end side friction induced during consolidation. Shaking causes liquefaction and loss of boundary shear that results in increased total vertical stress and $Ru_{(max)}$ values in excess of unity. It should also be noted that maximum values of Ru do not occur at the same time in all elements. Analyses results in terms of time history of acceleration, excess pore pressure and displacements with corresponding measurements are shown in Figures 7 and 8. The records corresponding to the measurement devices that were not installed or malfunctioned are not presented. Predicted accelerations are in good agreement with measured values. Negative acceleration spike occurs during upward movement due to dilation and is reflected with low values in pore pressure records.

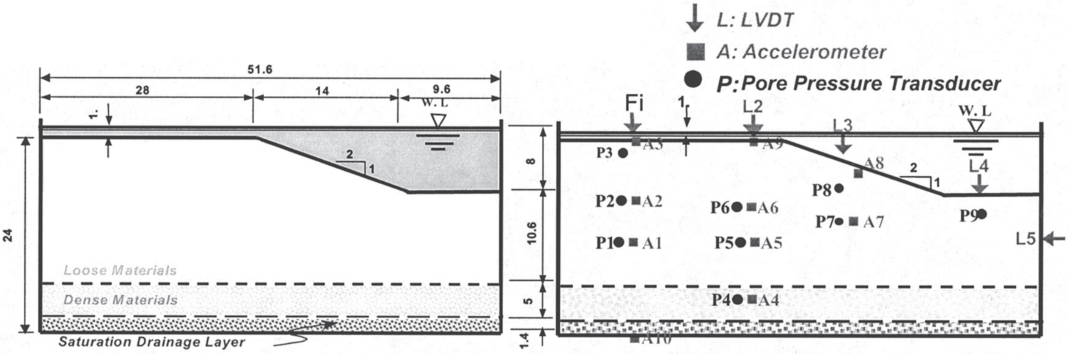

Figure 2. Model configuration for centrifuge testing (prototype scale) and instrumentation.

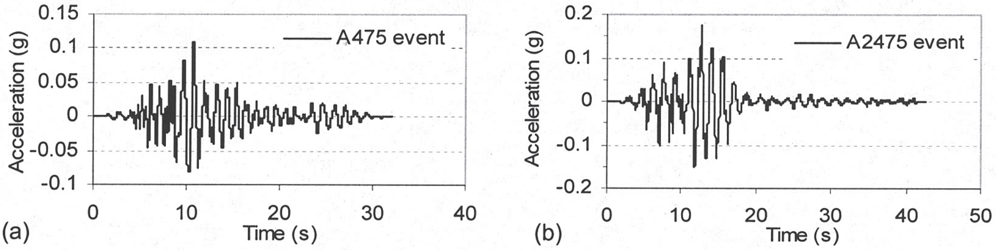

Figure 3. Acceleration time histories for (a) A475 event, and (b) A2475 event.

Table 1. Materials properties used in analyses.

Material	ρ_d [1000kg/m^3]	n	UBCSand $N1_{(60)}$	k [m/s]
Loose	1.50	0.448	6.2	8.8e-4
Dense	1.61	0.406	21.1	6.4e-4
Saturation layer	1.61	0.406	21.1	6.4e-2

Metolose viscosity = 35 times of that of water
Ng =70g, ρ_{water} =1000 kg/m^3, g =10 m/s^2
Drain permeability is considered 100 times of that of dense sand.
ρ_d, n and k are dry density, porosity and permeability respectively.

Table 2. Summary results of re-liquefaction tests on Fraser River sand.

σ'c (kpa)	CSR	Stage I			Stage II		
		Dr (%)	Ru (%)	No. of Cycles	Dr (%)	Ru (%)	No. of Cycles
100	0.1	41	100	8.5	52.0	100	6.0
100	0.1	40	100	6	51.5	100	7.0
100	0.1	41	85	8	46.0	100	27.0
100	0.1	40	55	5	41.0	100	25
100	0.1	41	45	5	42.0	100	27.0

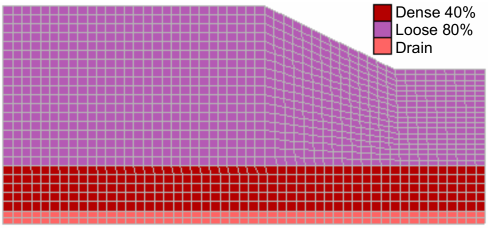

Figure 4. Grid model with different materials types.

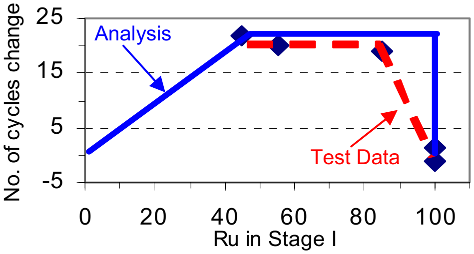

Figure 5. Increase of required No. of cycles vs. Stage I Ru.

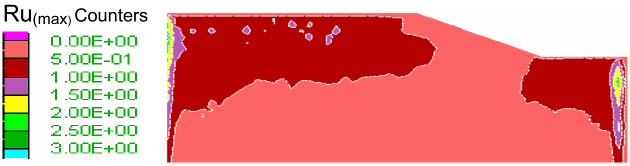

Figure 6. Maximum Ru within the model due to A475 event.

Accelerometers at near the top surface, e.g. A3 recorded spikes with higher values than predicted. Kutter and Wilson (1999) based on nonlinear wave propagation theory argued that these sharp spikes in acceleration time history of liquefied soil are due to different velocities of the wave front as the soil stiffens upon dilation. They used the term "de-liquefaction" to denote strain-stiffening behavior of soil at post liquefaction stage. It should be noted that this strain stiffening was observed in the simple shear tests in Figure 1a and is accounted for in the UBCSAND model, but not to a sufficient extent. As shown on Figure 8, predicted displacements back from the slope crest (LVDT L1) are in good agreement with the measurements, however in the vicinity of the slope the predictions overestimated the displacements. At the toe of the slope (LVDT L4) heave was predicted whereas the measurements showed settlement. The rationale for this difference is not understood.

6 Conclusions

Liquefaction of water saturated sandy soils is a major concern in geotechnical engineering in seismic areas. Significant advancements have occurred over the past decades in both understanding and practice with regard to the triggering of liquefaction, however, the ability to predict liquefaction induced deformations has only recently developed. When liquefaction occurs in significant zones of an earth structure the resulting deformations are the main issue. Numerical methods have been developed that can be used to predict liquefaction induced deformations and enhance our insight into the seismic response of earth structures. Validation of such methods is vital if these methods are to be accepted in engineering practice. Ideally, actual field case histories would be used for validation, however, even for the best field case histories, such as the 1971 near-failure of the San Fernando dams, neither the input motion nor the soil conditions are adequately known. Within a dynamic centrifuge model both the input motion and soil properties are known and this has proven to be a useful tool for validation numerical models. In this paper a Class-A prediction of the response of a slope shaken with two subsequent earthquake events using an effective coupled stress-flow analyses was made and the results are compared with those from centrifuge model tests. The procedure captures sand element behavior in monotonic and cyclic laboratory tests. The sand exhibits different properties in the second stage based on strain history in the first stage. This effect was accounted for in the analyses.

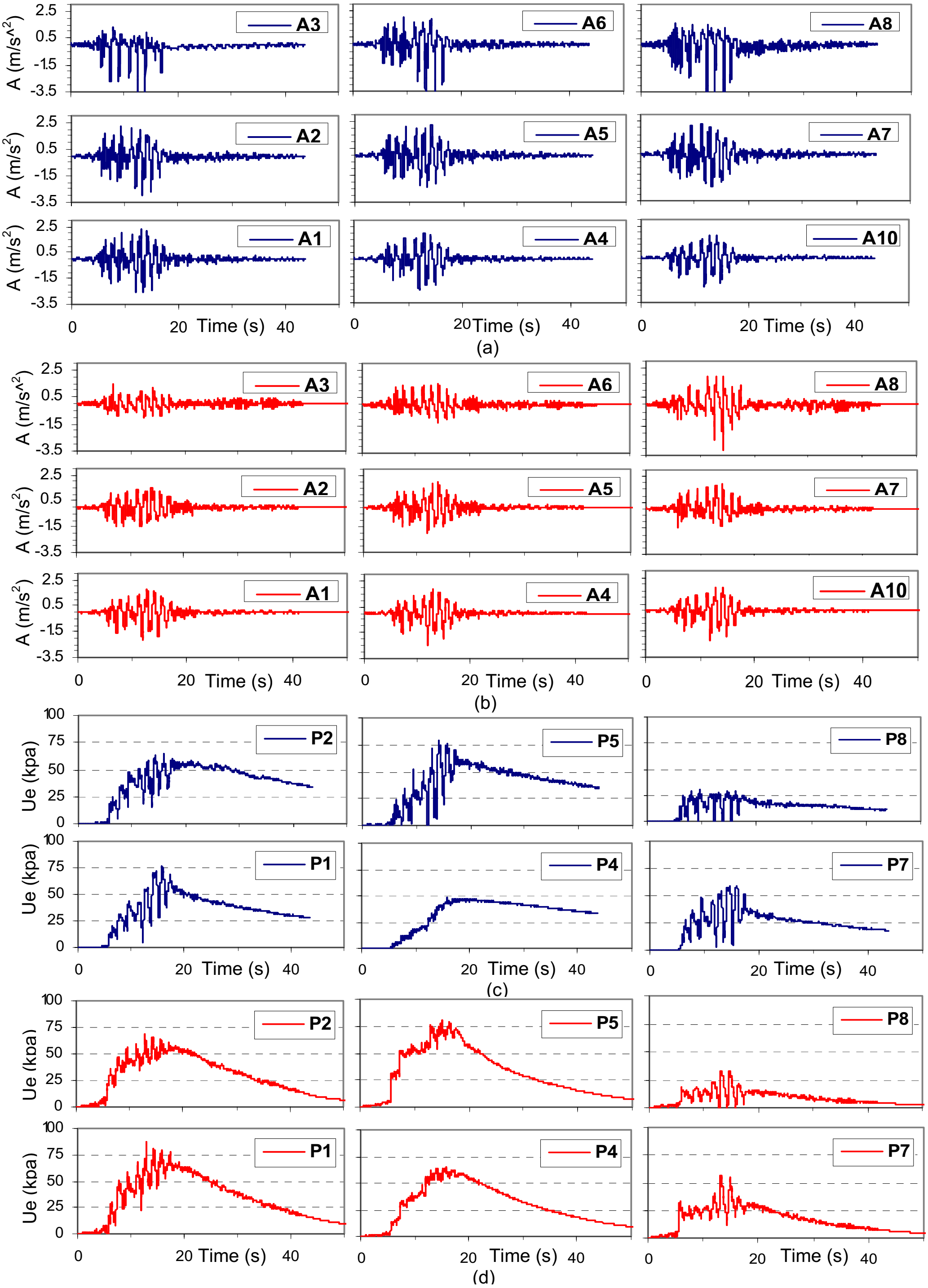

Figure 7. Measured and predicted slope response, a) measured acceleration, b) predicted acceleration, c) measured pore pressure, d) predicted pore pressure.

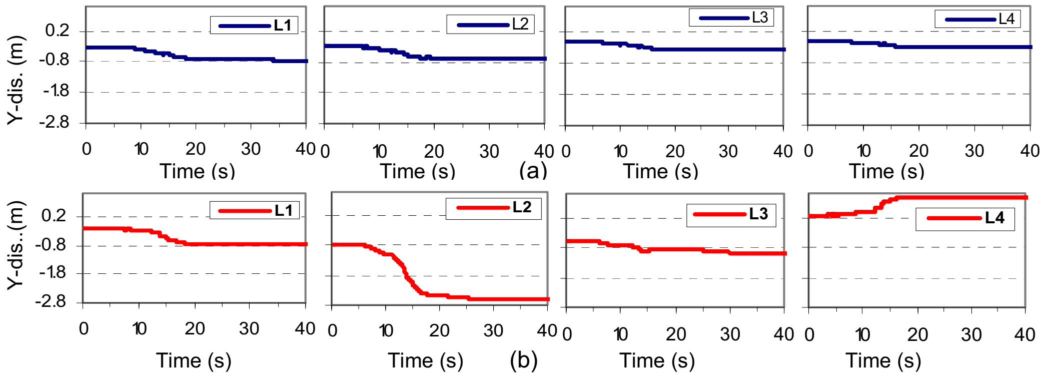

Figure 8. Measured and predicted displacements, a) measured, b) predicted.

The comparison of prediction and measurement demonstrated that the analysis procedure used is capable of predicting the general behavior of the slope during first as well as subsequent re-liquefaction conditions. The prediction in terms of pore pressure and accelerations agreed well with the measurements. The predicted displacements in the vicinity of the slope were generally greater than the measured values. Heave of the toe was predicted whereas settlement was measured.

7 Acknowledgement

The financial support from BC-Hydro through the Professional Partnership program is appreciated. The support of the National Scientific and Engineering Research Council, Canada through Strategic Liquefaction Grant No. NSERC 246394, and NSERC COSTA No. 03608-CG068625 is acknowledged. Thanks are extended to Mr. S. Sriskandakumar for conducting laboratory tests.

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