NUMERICAL EVALUATION OF LIQUEFACTION-INDUCED UPLIFT FOR AN IMMERSED TUNNEL

Dongdong. Chang[1], Thaleia Travasarou[2], and Jacob Chacko[3]

[1] *Ph.D., Staff Engineer, Fugro West Inc , Oakland, CA*
[2] *Ph.D., P.E., Senior Engineer, Fugro West Inc , Oakland, CA*
[3] *P.E.,G.E., Principle Engineer, Fugro West Inc , Oakland, CA*
Email: dchang@fugro.com

ABSTRACT :

Nonlinear dynamic finite element (FE) analyses results are compared to the experimentally recorded dynamic response of an immersed tunnel in liquefiable soil. Two experiments were performed at the large scale centrifuge facility at UC Davis to model idealized prototype cross-sections along an immersed tunnel in California. The centrifuge models consisted of a trench excavated in either soft or stiff clay. A rigid model tube was placed in the trench, which was then filled with loose Nevada and Monterey sand, to represent the loose sand and gravel of the prototype. The trench was covered by a soft, surficial clay layer, providing an impermeable barrier. The models were shaken with a series of motions progressively from smaller amplitude to the design peak ground acceleration (PGA) of about 0.6g. During the large motions the model tube experienced permanent prototype uplift on the order of 20 cm. The tests were analyzed using the FE software OpenSees with a fully coupled constitutive model for the liquefiable soils in the trench. FE models consisted of a two-dimensional (2-D) mesh of the soil profile with the tube modeled as a rigid block. The selection of parameters for the constitutive model was based on calibrations against laboratory cyclic simple shear tests performed for the project. The experimental records and the numerical simulations show good agreement on overall model responses (e.g., accelerations and pore pressures developed in the liquefiable sand) and the tube uplift. With appropriate calibrations, the FE models were able to reasonably approximate the essential features of soil and tunnel responses.

KEYWORDS: Liquefaction, Immersed Tunnel, Uplift, Centrifuge, OpenSees, Geotechnical Engineering

1. INTRODUCTION

The Offshore Transbay Tube (TBT) of the Bay Area Rapid Transit (BART) system is a 5.5km-long immersed railway tunnel located in a region of high seismicity in San Francisco, California. Fugro was contracted to assess the vulnerability of the TBT to uplift due to liquefaction of the surrounding soils during the design earthquakes and to design retrofits if needed.

To assist with the above goal, two centrifuge experiments funded by the Bay Area Rapid Transit (BART) were performed in September and December of 2007 at the large scale centrifuge facility of the University of California at Davis (Kutter et al 2008, Fugro 2008a). In particular, the goals of the centrifuge tests were to: 1) Study experimentally the uplift mechanism for representative geometry and soil conditions along the TBT alignment; and 2) Provide a basis for calibration of the numerical tools subsequently used in design.

Two-dimensional (2-D) nonlinear dynamic analyses were performed with two different numerical tools. The primary analyses (Fugro 2008b) were conducted using the finite difference computer program FLAC2D (Itasca, 2006), in combination with the UBCSAND (Beaty et al. 1998) constitutive model to represent the liquefiable soils. Additional analyses were performed on a finite element (FE) platform called “Open System for Earthquake Engineering Simulation” (OpenSees) developed by Pacific Earthquake Engineering Research Center (PEER). The purpose of the numerical analyses was to: 1) Confirm their ability to estimate physical response and 2) Calibrate numerical tools to be used in sensitivity studies for design and engineering recommendations.

The problem of potential uplift of relatively light, buried structures such as pipelines, immersed tunnels, manholes, has been addressed in research by means of physical model testing. Sasaki and Tamura (2004) performed a series of centrifuge tests to explore the effect of factors such as the relative density of the liquefiable layer, the intensity and waveform of the ground motion, the width of the structure and the thickness of the liquefiable layer beneath the structure on the liquefaction induced uplift movement of underground structures. Centrifuge model testing was also conducted to evaluate the uplift potential and the effectiveness of two alternative retrofit options for the George Massey Tunnel in Vancouver, Canada (Yang et al. 2004). In the case of the BART TBT project, the primary differences with respect to the previous studies relate to a) the extent of the liquefiable material in the trench, 2) the large ratio of the width of the structure to the thickness of the liquefiable soil and 3) the large permeability of the liquefiable soils. Following a brief discussion of the experiment design, this paper focuses on comparisons between observed response and estimates from the numerical analyses of the two centrifuge experiments conducted for BART TBT.

2.CENTRIFUGE TESTING FOR AN IMMERSED TUNNEL IN LIQUEFIABLE SOILS

Two large scale dynamic centrifuge tests were performed at UC Davis in 2007. Details of the two tests are available at Fugro (2008a) and Kutter et al. (2008), which include explanations of model design, material properties, configuration of instrumentation, sequence of input motions, model construction, data from the sensors, and post-earthquake model dissection measurements.

2.1. Overview of Centrifuge Design

The two centrifuge tests designed for the TBT were intended to model two different representative soil conditions around the trench encountered along the TBT alignment. The two tests modeled an idealized cross-section where the trench is surrounded by either stiff Merritt-Posey-San Antonio formation (MPSA-Clay) in the 1st test, or by soft Young Bay Mud (YBM) in the 2nd test. The two models have similar geometry and target trench material properties, with the primary difference being in the stiffness of the clay outside the trench. The centrifuge acceleration for the tests was 40 g. Figure 2.1 shows the model geometry for the 1st test. Monterey sand 0/30 was used to represent the loose gravel as Foundation Course for the tube and as Special Fill that filled the prototype trench up to the tube spring line. Nevada sand was used to represent the loose sand (Ordinary Fill) that filled the trench and provided a minimum cover for the tube. The target prototype properties of these materials are shown in Table 2.1. In the 1st test, the clay surrounding the trench consists of reconstituted Yolo Loam with undrained shear strength of about 100 kPa. In the 2nd test the trench was surrounded by YBM that was consolidated in a press. The strength of YBM based on pre and post experiment vane shear and T-bar tests was approximately 15~35 kPa.

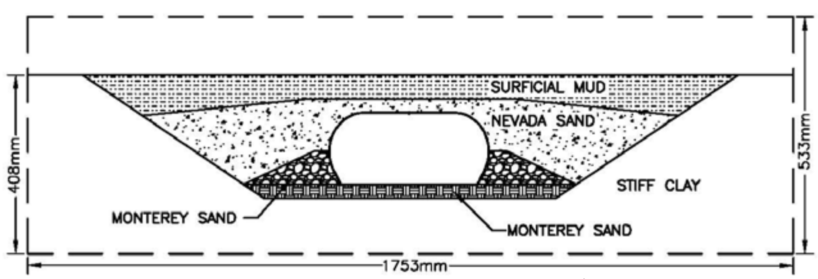


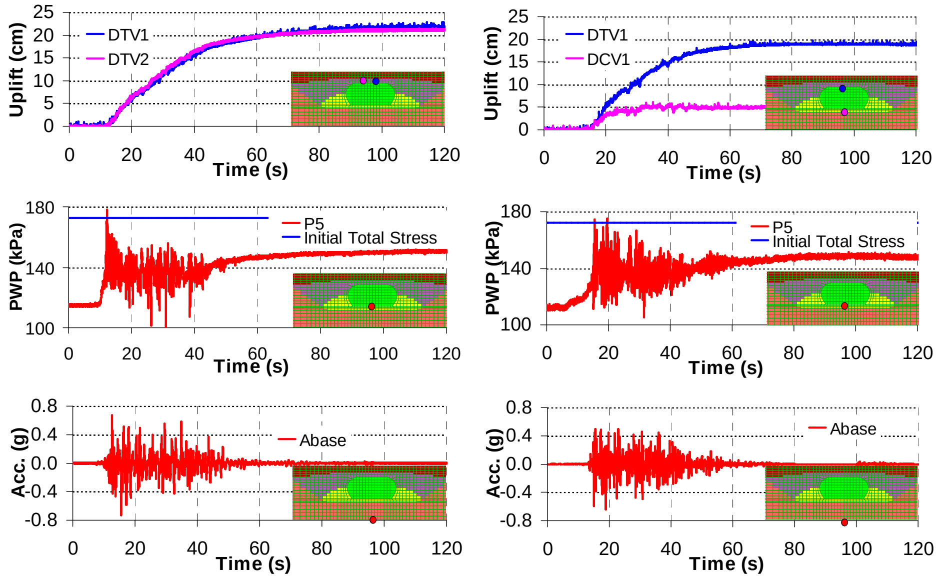
Figure 2.1 Schematic of the cross-section of the 1st centrifuge model.

Table 2.1 Prototype target properties for trench material.

Property	Ordinary Fill	Special Fill and Foundation Course
Permeability (cm/s)	0.01 – 0.1 (0.05)	0.8 – 5.0 (1.0)
Cone Tip Resistance	3352 to 5746 (~4788)	1915 to 3830 (~2872)
Relative Density (%)	(40 +/- 5)	(35 +/- 5)
Unit Weight (kN/m^3)	1.99	1.94

2.2. Observations from Centrifuge Testing

Each centrifuge model was shaken with a series of simulated earthquakes that were progressively stronger with maximum base accelerations ranging from 0.1g to 0.6g. The recorded tube and trench uplift, pore pressures under the tube, and the idealized Design Basis Earthquake TCU078 acceleration time history are plotted in Figure 2.2 for both tests. The tube uplift was about 20 cm in both tests, but for the 2nd test 5 cm of the uplift was due to the heave of the soft YBM under the trench (Figure 2.2-b). For both tests, the pore water pressure under the tube remained high for several seconds after shaking stopped, eventually dissipating a few minutes after the end of shaking. More details of the test observations can be found at Fugro (2008a) and Kutter et al. (2008).



(a) The 1st centrifuge test (b) The 2nd centrifuge test

Figure 2.2 Observations from two centrifuge tests during TCU078 motion.

3. FINITE ELEMENT MODELS

The 2-D dynamic FE analyses were performed on OpenSees platform (http://opensees.berkeley.edu). The soil continuum was modeled by soil-fluid fully coupled quadUP elements with the soil constitutive model developed at UC San Diego (Yang et al. 2005). The mixed stress-strain space, pressure-dependent, multiple-yield-surface model was used for sands, and the pressure-independent, multiple-yield-surface model was adopted for clays.

3.1.Constitutive Model for Liquefiable Soils

The constitutive model to represent dynamic response of the liquefiable soil is the PressureDependMultiYield01 (PDMY01) model (Yang et al. 2005), used with solid-fluid fully coupled elements to simulate excess pore water pressure generation and dissipation during earthquake. The PDMY01 is an elastic-plastic material simulating the essential characteristics of pressure sensitive soil under seismic loads, including dilatancy and cyclic mobility. Plasticity is formulated based on the multi-surface (nested surfaces) concept, with a non-associative flow rule to reproduce dilatancy effect. The yield surfaces are of the Drucker-Prager type. This model is fully defined by means of 16 parameters and a set of user-defined backbone curve parameters, (Table 3.1). The maximum shear modulus is determined from shear wave velocity measurement, and the various yield surfaces are determined by the specified modulus degradation (G/G_{max} vs strain) relation and ultimate shear strength. The parameters controlling liquefaction triggering are mainly contrac and the phase transformation angle (PtAng); the parameters controlling dilation behavior are mainly dilat1 and dilat2, and the parameters controlling post-liquefaction shear strain are mainly liquefac1, liquefac2, and liquefac3. Although, all the parameters interact with each other, specific values can be developed to produce a desired cyclic resistance ratio (CRR) and limiting cyclic shear strains. The PDMY01 model has been calibrated against laboratory tests (Elgamal et al. 2002, 2005), and tested by large centrifuge modeling to evaluate soil-pile interaction in liquefied soils (Chang. et al. 2005).

3.2. Model Parameter Calibrations

PDMY01 materials were calibrated to produce similar results in terms of cyclic resistance to liquefaction as a function of number of cycles to those obtained in laboratory tests. For the Monterey sand, cyclic simple shear tests performed by Kammerer et al. (2004) at UC Berkeley were used as a basis for the calibration. The tests were performed at a confining stress of 80 kPa and relative densities of 35%, 45%, and 60%. For the Nevada sand, cyclic simple shear tests performed by Kammerer et al. (2000) and Arulmoli et al. (1992) as summarized in Beaty et al. (1998) were used. The tests were performed at relative densities of 40%, 60%, and 80%.

Single element cyclic simple shear tests were modeled in OpenSees using the PDMY01 model with a fully coupled quadUP element. A series of cyclic stress ratios (CSR) were applied to the single soil element and the number of cycles (N) to liquefaction estimated. Liquefaction was assumed to occur when the vertical effective stress reaches zero. The constitutive model parameters were adjusted till the desired cyclic resistance was obtained. For the 1st centrifuge the as-built relative densities were estimated based on two cone penetrations tests performed during spinning. Based on these test results, the relative density is about 35% for the ordinary fill (Nevada sand), about 50% for the special fill (Monterey sand), and 40% for the foundation course (Monterey sand). Results of the single element calibration are shown in Figure 3.1 in terms of CSR versus N values.

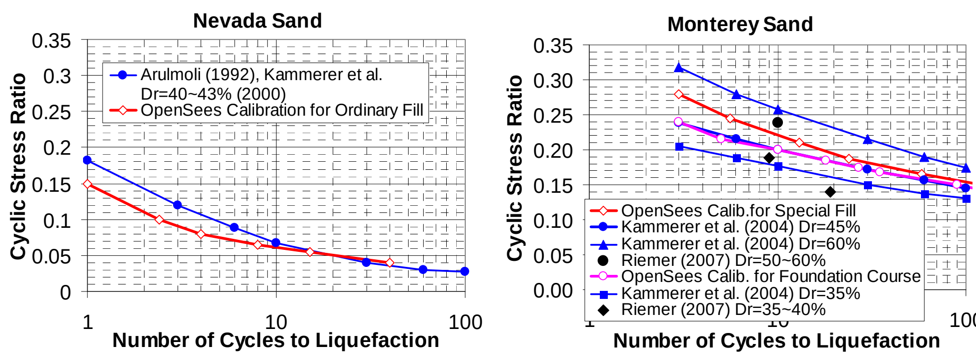


Figure 3.1 Liquefaction resistance calibration of PDMY01 material for the 1st centrifuge test.

Figure 3.2 shows the comparisons between the single soil element calibration of PDMY01 model and DSST lab data at CSR=0.19 for Monterey Sand with D_R=40%. It shows generally good agreement in the element level behaviors. Compared to the experimental data, the simulated response depicts generally less dilative behavior.

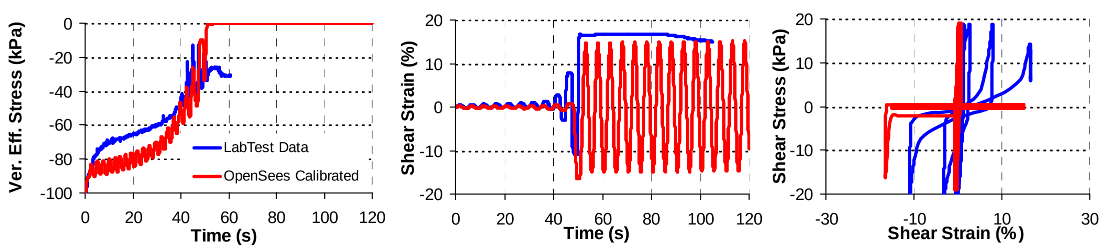


Figure 3.2 Single element dynamic behavior for PDMY01 model calibration (CSR=0.19).

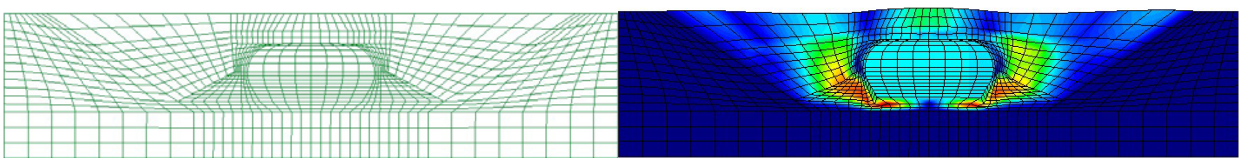
Table 3.1 lists the parameters used in the calibrated PDMY01 model to best match the DSST laboratory data. The shear modulus and bulk modulus of soil were chosen to be consistent with the in-flight shear wave velocity measurements and those used in the UBCSAND model in FLAC-2D analyses (Fugro 2008b). The modulus reduction curves are from Pyke et al. (1995). The friction angles were calculated based on the measured N_{1-60} values and critical state friction angles, and the phase transformation angles were estimated based on recommendations for PDMY01 material (Yang et al. 2005) adjusted by calibrations using the DSST lab data.

Table 3.1 PDMY01 model parameter for sand in OpenSees analyses

Soil Type	Ordinary Fill / Nevada Sand	Special Fill / Monterey Sand	Foundation Course/ Monterey Sand
Rho (ton/m^3)	1.99	1.94	1.94
Reference Shear Modulus (kPa)	$6.3e^4$	$7.0e^4$	$6.0e^4$
Reference Bulk Modulus (kPa)	$1.5e^5$	$1.1e^5$	$1.0e^5$
Reference Pressure (kPa)	80	100	100
G/Gmax[1]	Pyke 20 ft	Pyke 20-50 ft	Pyke 20-50 ft
Friction Angle	33.3	34.7	34.0
Phase Transformation (PT) Angle	21.5	14.9	15.4
Peak Shear Strain (at pr'=80kPa)	0.1	0.1	0.1
Pressure Dependent Coefficient	0.5	0.5	0.5
contrac	0.3	0.5	0.5
dilat1	0.1	0.15	0.1
dilat2	1.5	1.5	1.5
liquefac1 (kPa)	10	10	10
Liquefac2	2	2	2
Liquefac3	3	3	3
Permeability (cm/s)	0.057	0.88	0.88

4. COMPARISON BETWEEN NUMERICAL AND EXPERIMENTAL OBSERVATIONS

Finite element Pre- and post- processing software GiD was used as an interface for OpenSees analyses. The 2-D FE mesh generated in GiD for the 1st centrifuge test is shown in Figure 4.1, where 4.1(a) is the undeformed mesh, and 4.1(b) is the deformed mesh after the large TCU078 motion with the colors representing the computed displacement magnitudes.



(a) Mesh grids for FE analyses (b) Deformed mesh with displacement contour fill after shake

Figure 4.1 FE meshes for the 1st centrifuge test.

4.1. The First Centrifuge Test

Figures 4.2 through 4.5 present comparisons between experimental and numerical results for Test 1. The numerical results are from the OpenSees analyses using the as-built soil properties and input motions that were recorded input motion at the base of the container during the experiment.

Figure 4.2 compares the recorded and computed time histories of the tube uplift up to 120s. The computed tube uplift is approximately 20 cm as against 22 cm observed in the experiment. Similar trends are observed both during and subsequent to shaking.

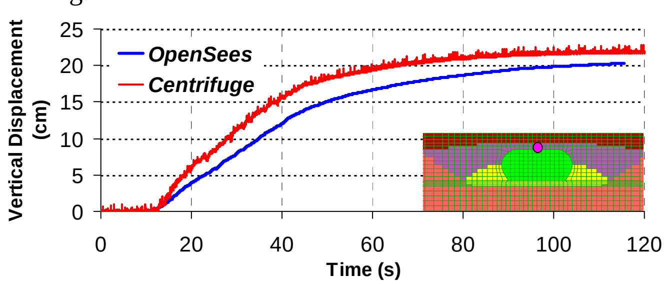


Figure 4.2 Computed and recorded tube uplift during TCU078 motion for the 1st centrifuge test.

Figure 4.3 compares the recorded and computed acceleration time histories along a vertical column of accelerometers located about 1.5 meters away from the edge of the tube (see blue dots on Figure 4.3 for sensor locations). The computed accelerations in general agree with the recorded ones. Although, dilation appears to be underpredicted at some locations, the numerical estimates appear to capture the reductions in ground shaking at shallow depths in Nevada sand due to liquefaction/pore pressure generation in the underlying layers.

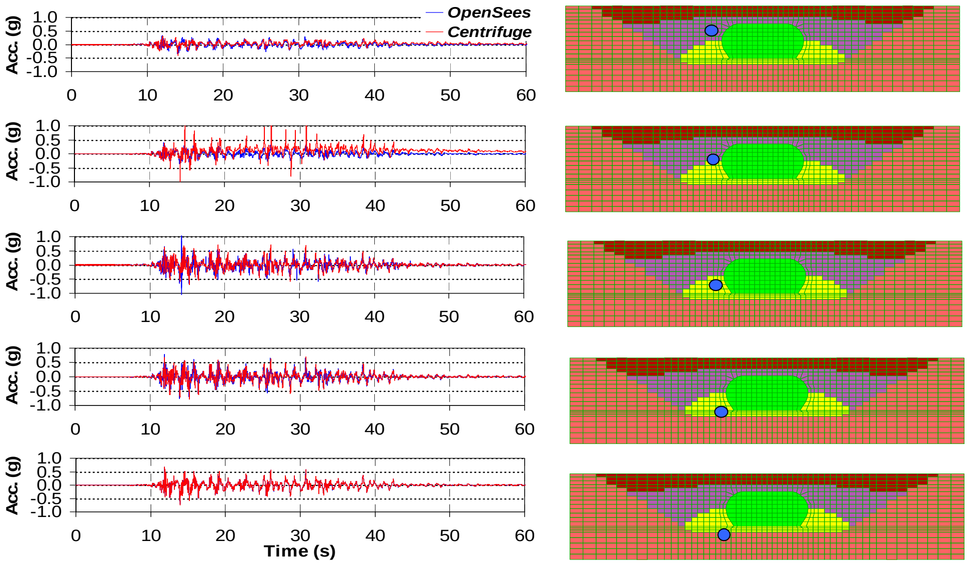


Figure 4.3 Accelerations in a vertical column during TCU078 motion for the 1st centrifuge test.

Figure 4.4 compares the recorded and computed time histories of a vertical array of pore pressure transducers located about 1.5 meters away from the edge of the tube (see blue dots on Figure 4.4 for pore pressure transducer locations). The computed pore pressures generally compare well with the centrifuge measurements at all locations, but appear to slightly underestimate the shear-induced dilation of soil. The numerical analyses also capture the post-shaking pore pressures trends reasonably well. Both computed and recorded data show that pore pressure dissipates relatively quickly from the Monterey sand near the tube base, but stay high in the Nevada sand near the sand-surface clay interface after shaking. After shaking, the computed pore pressures seem to dissipate slightly faster than recorded for the Nevada sand but slower than recorded for the Monterey sand long.

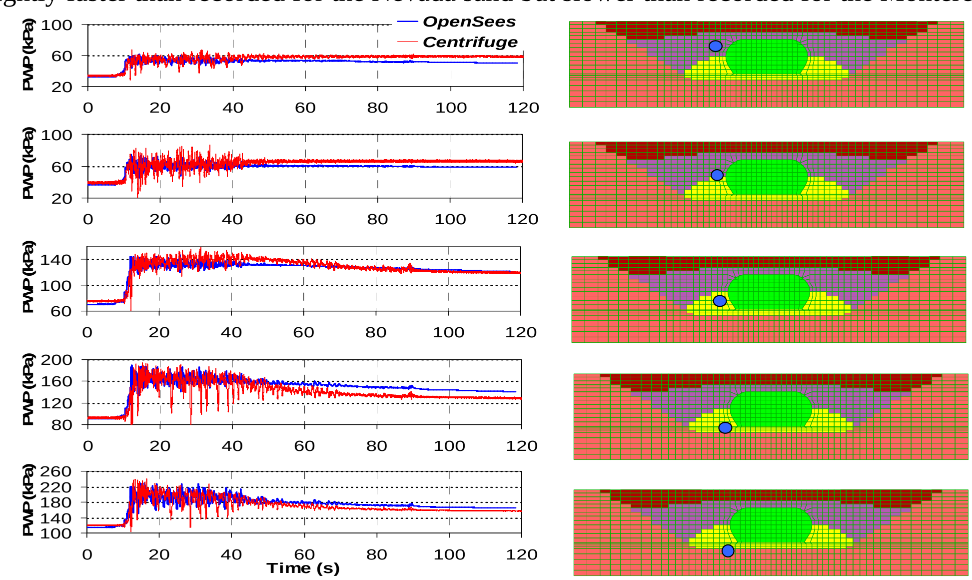


Figure 4.4 Pore water pressures in a vertical column during TCU078 motion for the 1st centrifuge test.

Figures 4.5 compares the deformation patterns of soil observed during model excavation with the predictions from the OpenSees analyses. Both observations and calculations show the Monterey sand adjacent to the tube to move inwards and underneath the tube while the Nevada sand move outwards as the tube lifts up.

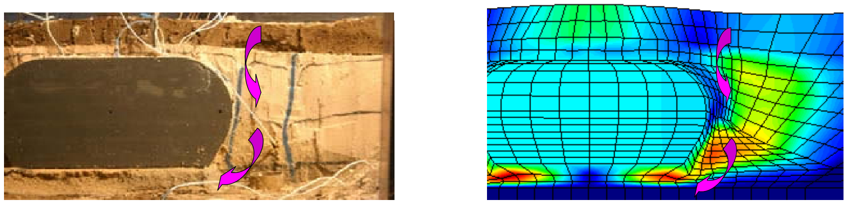


Figure 4.5 Comparison between observed and computed soil deformation pattern.

4.2. The Second Centrifuge Test

Figure 4.6 compares the recorded and computed time histories of the tube and trench uplifts for the second centrifuge test. The computed tube uplift is approximately 19 cm, which agrees well with the recorded value. During the second centrifuge tests the soft YBM around the trench heaved and contributed to uplift of the tube. The heave in the trench was recorded with a displacement transducer to be about 5 cm. However, measurements during excavation, and interpretation of results from pore pressure sensors suggested that the heave may have been larger and on the order of 7 to 8 cm. The computed trench uplift is approximately 10 cm which somewhat overestimates the recorded response.

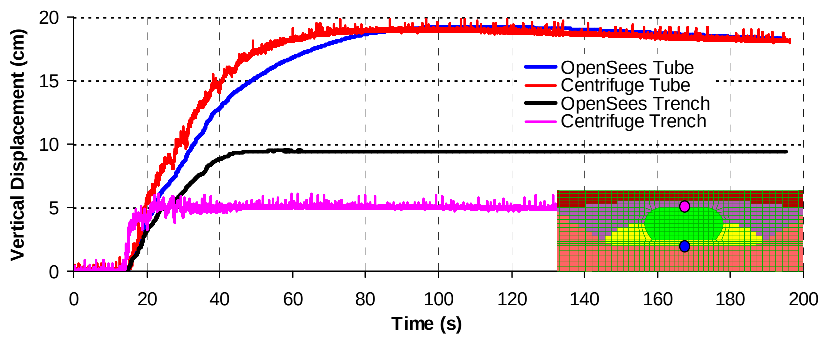


Figure 4.6 Computed and recorded tube and trench uplifts during TCU078 motion for the 2nd centrifuge test.

5. CONCLUSIONS

Nonlinear dynamic FE analyses were performed with the PDMY01 model in OpenSees for two centrifuge tests of an immersed model tube surrounded by liquefiable soils. In general, the analyses are in good agreement both qualitatively and quantitatively with the observed response. Numerical estimates of tube uplift appear to capture both the co-seismic and post-seismic component of the vertical tube displacements. In addition, both the amplitude and rate of pore pressures dissipation in the liquefiable trench soils were captured satisfactorily by the numerical analyses. The deformation patterns obtained by the numerical analyses were similar to those observed during model dissection showing the movement of the liquefied soil inward and underneath the tube. Finally a contribution to the uplift from the soft clay heaving in the 2nd centrifuge experiment was suggested, although slightly underestimated by the numerical methods. Overall, despite their limitations, the numerical methods and constitutive models used appear to capture the primary features of this complex problem involving dynamic soil response, liquefaction, flow, and soil-tube interaction. The overall good comparisons indicated that the calibrated FE model using OpenSees could be used to help evaluate the vulnerability of the prototype.

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