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NUMERICAL ANALYSES OF CENTRIFUGE MODELS OF THE BART TRANSBAY TUBE

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ABSTRACT

Centrifuge model tests were performed to study the stability and uplift mechanisms of the BART Transbay Tube. The tube is a cut-and-cover subway tunnel located in a highly seismic area. The low relative density of the backfill material around the tunnel and the low unit weight of the tunnel might make tunnel suffer uplift movement due to buoyancy forces caused by liquefaction of the backfill material during an earthquake. Three uplift mechanisms were observed in the centrifuge model tests: (1) a cyclic ratcheting mechanism of sand moving under the tunnel associated with cyclic lateral deformations of the tunnel;(2) flow of water under the tunnel; and, (3) heave of the soft trench clay. The FLAC program was used to simulate the centrifuge model tests. A sensitivity study was performed to decide on the final mesh and treatment of interfaces in the numerical model. Results of the sensitivity study, numerical simulations and centrifuge model test results are presented and discussed in this paper.

INTRODUCTION

The Bay Area Rapid Transit (BART) Transbay Tube (TBT), constructed in the 1960's, is a cut-and-cover tunnel that connects Oakland to San Francisco; it is a heavily used commuter rail system. Backfill material around the tunnel was placed loosely under water at a relative density less than 50% during the construction. Because of the low density, the backfill material is expected to liquefy during design level earthquakes. BART engaged Fugro West Inc., Oakland, CA to assess the need for ground improvement to mitigate seismically-induced deformations of the tunnel, in particular, the deformations due to uplift of the tunnel in the liquefied backfill. Because of the importance of the project, Fugro recommended centrifuge model tests to explore the deformation mechanisms and calibrate their numerical analyses (Fugro, 2008). This paper describes similar numerical analyses to those performed by Fugro (2008) for the centrifuge experiments and ultimately for the design of the project. However, the analyses in this paper are of a more generic nature and are not directly applicable to the BART project. Using the computer program FLAC, we performed a sensitivity study to decide the parameters (e.g., mesh geometry and interface modeling) used in the numerical simulation. Using the selected parameters, numerical simulations were compared with the centrifuge test data.

RELATED LITERATURE

Koseki (1997) performed several shaking table model tests on partially-buried box structures and completely-buried box structures, manholes and underground pipes and categorized the behavior of the underground structures and the surrounding soils in three components: (1) Lateral soil deformation (2) Movement of pore fluid (3) Reconsolidation. The uplift of underground structures is first caused by the lateral deformation of the surrounding soil and subsequently by movement of pore fluid. The dilation of the soil during shearing reduced the excess pore pressure and may contribute to additional resistance against the uplift of the completely-buried structures.

Yang et al. (2004) performed numerical analyses using the computer program FLAC (www.itascacg.com) and the constitutive model UBCSAND (Puebla et al. 1997; Beatty and

Byrne 1998) and compared to the results of the centrifuge tests conducted by Adalier et al. (2003). The centrifuge test results confirmed the ability of the numerical models to predict the behavior of George Massey Tunnel in Vancouver, B.C., Canada. The numerical analyses and centrifuge tests led to a recommendation that retrofit should be performed and they demonstrated that the proposed retrofit schemes could successfully limit the deformations to acceptable levels.

Fugro (2008) performed numerical finite difference analyses for the BART Offshore Transbay Tube. They concluded that the uplift mechanism is a ratcheting displacement-limited mechanism associated with small tunnel deformations. This conclusion was confirmed by the centrifuge results presented in this paper and guided the recommendation of no-densification retrofit of the loose trench soils to mitigate against tunnel uplift.

CENTRIFUGE EXPERIMENTS

Description of the centrifuge models

As the BART tunnel is approximately 5.5 km in length, there is a variety of geological conditions along the alignment of the tunnel. The alignment was divided by the design engineers (Fugro, 2008) in several zones reflecting differences in the stiffness of the clay surrounding the trench, and the thickness of overburden. Two zones were selected as the prototypes for the two centrifuge tests. Figure 1 shows the idealized cross section applicable to these two zones for the purpose of centrifuge testing. Trench material and its strength are the differences of these two tests. For the first centrifuge test, the material is a stiff low plasticity silty clay in the Merritt-Posey San Antonio (MPSA) formation (Stiff Clay). For the second centrifuge test, the trench material is a lightly overconsolidated high plasticity clay known as Young Bay Mud (YBM). Considering goals of modeling and factors affecting modeling results, a scale factor of 1:40 and a rigid container were chosen for these two centrifuge tests.

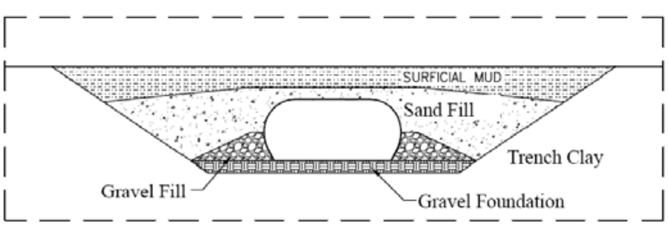


Fig. 1. Idealized cross-section for the centrifuge model

In the first centrifuge test, JCC01, the stiff MPSA silty clay was modeled by compacted Yolo Loam, a locally available low plasticity silty clay. In the second centrifuge test, JCC02, Young Bay Mud obtained from the Hamilton Air Force Base site was used to model the trench material. The backfill materials, "Gravel Foundation", "Gravel Fill" and "Sand Fill" were modeled using Monterey 0/30 sand and Nevada sand. Key properties and element cyclic behavior of both sands have been characterized in laboratory experiments in the past study, Arulmoli (1992), Kammerer et al (2000, 2004). Table 1 lists the sand properties in JCC01 and JCC02. The "Surficial Mud" layer acts as a barrier that restricts some water pressure dissipation from the top of the backfill materials. This layer was modeled by Yolo Loam mixed in a slurry/paste with tap water at the water content close to 1.2 times the liquid limit.

Table 1. Sand Properties in JCC01 and JCC02

Property	Sand Fill (JCC01) [JCC02]	Gravel Fill (JCC01) [JCC02]	Gravel Foundation (JCC01) [JCC02]
Name	Nevada Sand	Monterey 0/30	Monterey 0/30
USCS Soil Classification	SP	SP	SP
Conventional Permeability (cm/s)	0.014	0.22	0.22
Model Viscosity Scale Factor	(11) [12]	(11) [12]	(11) [12]
Prototype Permeability (cm/s)[1]	0.057	0.88	0.88
Median Grain Size (mm)[2]	0.15	0.35	0.35
% finer than 0.075 (mm)[2]	1 to 2	< 1	< 1
Cone Tip Resistance, qc1 (MPa)	2.9	6.9	4.5
Target Relative Density (%)	(40) [40]	(35) [18]	(35) [18]
Target Void Ratio, e	(0.68) [0.68]	(0.76) [0.87]	(0.76) [0.87]
Target Dry Unit Weight (kN/m3)	(15.5) [15.5]	(14.8) [14.4]	(14.8) [14.4]
Relative Density (As-Built) (%)[3]	(32) [40]	(50) [20]	(40) [20]

[1] Prototype permeability is obtained from conventional permeability times model scale factor (40) divided by viscosity scale factor.

[2] Balakrishnan (2000) and Wu et al. (2003)

[3] Cone tip resistance was converted to relative density using the relationships proposed by Kulhawy and Mayne and Jamiolkowski et al. published in Kulhawy and Mayne (1990).

Instrumentation

Figure 2 shows the sensor layout in JCC02 and names of sensors used in this paper. The instrumentation plan for JCC01 and JCC02 are almost the same except JCC02 added few more sensors. Instrumentation plans and purposes in JCC01 and JCC02 are described in Chou et al. (2008a and 2008b).

Shaking events

Table 2 lists all shaking events during both centrifuge tests including the event number, name and PGA. The target motion for the BART tunnel is the "TCU" event which was obtained by processing the recordings from the TCU station during the 1999 Chi Chi Earthquake in Taiwan (Bechtel, 2005). In numerical simulations, the input motion was TCU motion recorded in the centrifuge tests.

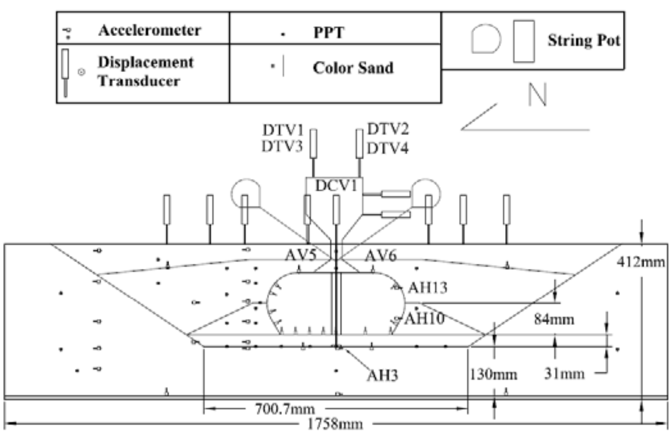


Fig. 2. Layout of Sensors in JCC02
(Units are in model scale)

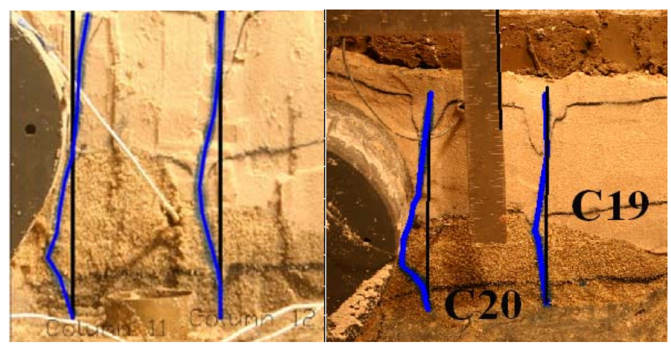
Table 2. Input Ground Motion Information

JCC01			JCC02		
Event No.	Name	PGA (g)	Event No.	Name	PGA (g)
E1	Step Wave	0.015	E1	Step Wave	0.013
E2	Small LP	0.013	E2	Small LP	0.0066
E3	LP	0.144	E3	LP	0.066
E4	Small TCU1	0.053	E4	Small TCU	0.07
E5	Small TCU2	0.141	E5	TCU	0.66
E6	TCU	0.649	E6	Small Joshua Tree	0.05
			E7	Joshua Tree	0.36

Note: LP is used to represent 1989 Loma Prieta – YBI. TCU is used to represent 1999 Chi-Chi Taiwan, TCU078. Joshua Tree is used to represent the modified Joshua Tree recording from the 1992 Landers earthquake.

Observations

Figure 3 shows pictures taken during the model dissection in JCC01 and JCC02. The deformed blue lines are the deformed shape of blue colored sand columns which were originally installed inside the soils vertically to monitor the deformation of the soil. From the shape of these blue lines, soils near the bottom of the tunnel moved toward the bottom of the tunnel during shaking. Another observation in the model dissection did not show in pictures here is the heave of the trench clay. In JCC01, the clayey soil below the trench was a "Stiff Clay" which suffered insignificant heave during the test. In JCC02, the relatively soft Young Bay Mud experienced noticeable heave during shaking.



(a) JCC01 (b) JCC02

Fig. 3. Colored sand columns indicate lateral flow of sand toward the foundation course beneath the tunnel.

Figure 4 shows excess pore pressure isochrones from sensors on the base of the tunnel (PT1, 2, 3, 4, 5, 6, 7) and selected times during TCU event in JCC01. The excess pore pressure distributions generally had a U-shape during the shaking and these pore pressure data can be converted into hydraulic gradients between sensors (PT1 to 7). The hydraulic gradients indicate a water flow from the edge toward the middle of the model tube. The uplift from flow of pore water can be estimated using Darcy's Law and hydraulic gradients. The detailed calculation procedures and assumptions are explained in Kutter et al. (2008)

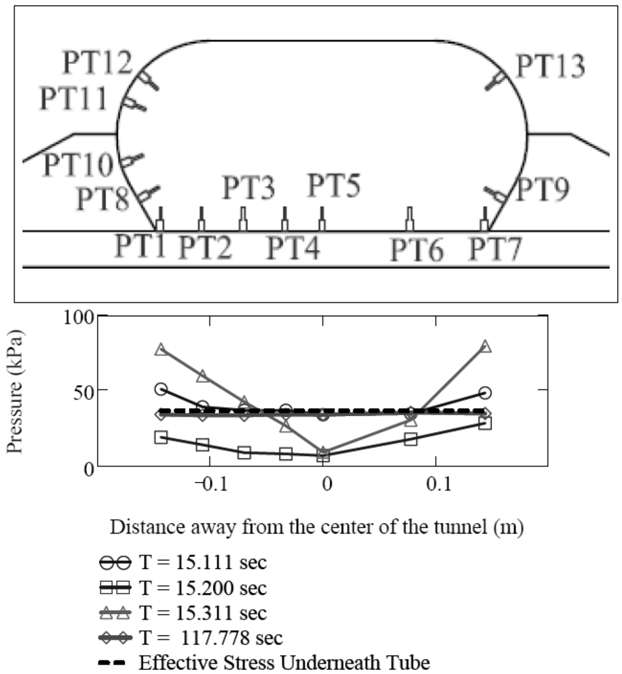
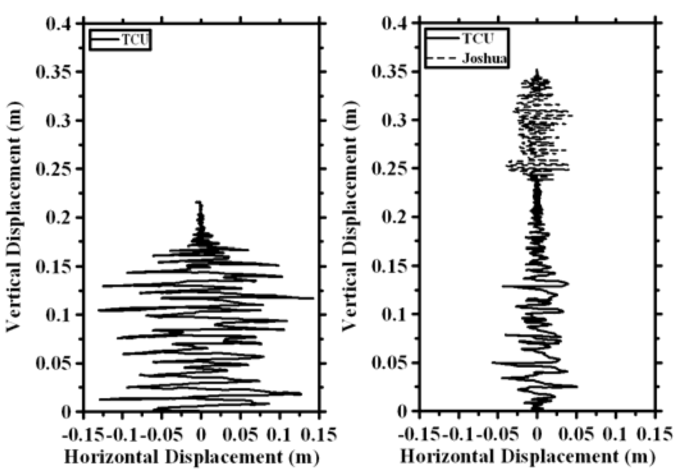


Fig. 4. Plots of excess pore pressure under Tube in TCU motion at different time steps in JCC01

Figure 5 shows experimentally measured displacement trajectories of the center of the base of the tunnel for TCU event in JCC01 and TCU and Joshua Tree events in JCC02. Procedures used to derive the trajectory plots are introduced in Chou et al. (2009).



(a) (b)

Fig. 5. Trajectory movement of the center of the tunnel in different shaking events (a) JCC01 (b) JCC02

After sensor data analyses and observations from the model dissection, uplift mechanisms of the tunnel can be summarized as following three components: (1) ratcheting (sand flow) (2) pore water migration (3) bottom heave of the base of the trench. Detailed explanations and analyses of each uplift mechanism are presented in Kutter et al. (2008).

NUMERICAL SIMULATION

The finite difference program FLAC2D is used in this paper to perform the numerical simulations. Constitutive models used in the simulation and their input parameters are introduced in this section.

Elastic model

The model tunnel in the centrifuge tests was relatively stiff compared to the surrounding soils. Thus, an elastic model with a large shear modulus and bulk modulus is used to simulate the model tunnel. The input parameters are listed in Table 3.

Table 3 Elastic model parameters for tunnel

Parameters	ρ (kg/m^3)	n	υ	G (Pa)
Tunnel	1073	0.05	0.3	8×10^7

Note: n is porosity of the tunnel. n should be zero but for FLAC program simulation a small value, 0.05, is used.

UBC Sand model

The UBC Sand model (Beaty and Byrne, 1998) is used to simulate the behavior of liquefiable soils, Nevada Sand and Monterey 0/30 Sand, in the centrifuge tests. Table 4 shows the input parameters for Nevada Sand and Monterey Sand in the numerical simulation. For Nevada Sand, the relationships between relative density and input parameters are calibrated using undrained cyclic test results. Default relationships between relative density and input parameters recommended by the model developers are used for Monterey Sand.

Mohr Coulomb model

A Mohr Coulomb model available in FLAC2D is used to model soils (Stiff Clay, YBM and Surficial Mud) that do not liquefy during shaking. Characterization of these two layers for numerical analyses is described in a design report prepared by Fugro (2007). Table 5 shows input parameters of Mohr Coulomb model.

Table 4. Parameters of UBC Sand model

Parameter	Nevada Sand	Monterey Sand
KGE Elastic shear modulus multiplier	$(\rho_t \times V_{s1}^2)/Pa \times (2/(1+K_0))^{0.5}$	$434 \times N_{1,60}^{0.333}$
me, Elastic shear exponent	0.5	0.5
KB Elastic bulk modulus multiplier	1×KGE	0.7×KGE
ne, Elastic bulk exponent	0.5	0.5
KGP Plastic bulk modulus multiplier	KGE× $(0.41667+0.0175 \times D_r)$	KGE×0.003 $\times N_{1,60}^2$+100
np, Plastic bulk exponent	0.4	0.4
ϕ_{Cs}, Critical state friction angle	33	33
ϕ_{Peak}, Peak friction angle	ϕ_{Cs}+ $1.5227 \times 10^{-2} \times D_r + 2.8409 \times 10^{-4} \times D_r^2$	ϕ_{Cs}+ $(N_{1,60}/10)$
R_f, Failure ratio	1+ $1.5901 \times 10^{-4} \times D_r - 5.1136 \times 10^{-5} \times Dr^2$	1- $(N_{1,60}/100)$
hf1, Model parameter	1	1
hf2, Model parameter	1	1
hf3, Model parameter	1	1
anisofac, Model parameter	1	1

Note: ρ_t is the total unit weight of the soil. K_0 = 1- sin(ϕ_{Cs}). Pa is the atmosphere pressure. $N_{1,60}$ = SPT blow count for an energy ratio 60% and the vertical effective stress at 1 atmosphere. Dr = Relative Density in percentage.

Table 5. Parameters of Mohr Coulomb model and Hysteresis model

Parameter of Mohr Coulomb	Surfical Mud	Stiff Clay	YBM
ρ, (kg/m^3) Dry density	1400	1934	985
n, Porosity	0.5	0.27	0.63
G, (Pa) Shear Modulus	$(46+1.6\times Z)^2 \times \rho$	$(201+2\times Z)^2 \times \rho$	$(61+3.4\times Z)^2 \times \rho$
ϕ_{Cs}, Friction angle	0	0	0
C, (Pa) Cohesion	(50+19.7×Z) × 47.88	96000	(100+39.4× Z) × 47.88
ϕ_D, Dilation angle	0	0	0
υ, Poison ratio	0.3	0.3	0.3
Parameters of Hysteresis	Surfical Mud	Stiff Clay	YBM
a	1	1	1
b	-0.6	-0.48	-0.5
x_0	-0.85	-0.75	-0.85

Note: Z is the depth of the soil in meters.

Hysteresis model

Because Mohr Coulomb model does not model the nonlinear behavior of soils that occurs in cyclic loading, a hysteresis model needs to be added in the numerical simulation to account for the nonlinear stress-strain curve and the stress-strain loops which represent the energy dissipation of the soil during the cyclic loading. This model is a default model in FLAC2D and the detailed description of the model can be found in FLAC2D manual. In this model, the reduction of the shear modulus is described using:

$$\frac{G}{G_{max}} = \frac{a}{1+\exp(-(L - x_0)/b)} \quad (1)$$

where L is $\log_{10}$(shear strain(%)) and a, x_0 and b are model parameters listed in Table 5. Parameters are decided by fitting the target stress-strain curve of each layer. The target curves are shown in a report, Fugro (2007).

Fluid model

In the fluid model of FLAC program, the mobility coefficient, K, was used instead of the hydraulic conductivity, k. Mobility coefficients of soils are listed in Table 6. The equation used to convert k to K is:

$$K(\mathrm{m}^2/(\mathrm{Pa\text{-}sec})) \equiv k(\mathrm{m/sec}) \times 1.02 \times 10^8 (\mathrm{m/Pa}) \quad (2)$$

Table 6. The mobility coefficients

Soil layer	Mobility Coefficient	
	K_H	K_V
YBM	2×10^{-12}	1×10^{-12}
Stiff Clay	2×10^{-12}	1×10^{-12}
Gravel Foundation	9×10^{-7}	9×10^{-7}
Gravel Fill	9×10^{-7}	9×10^{-7}
Sand Fill	5.8×10^{-8}	5.8×10^{-8}
Surficial Mud	2×10^{-12}	1×10^{-12}

Interface elements

Interface elements are used between the soil and the tunnel in the numerical simulation. Interface elements allow the soil mesh to move relatively to the tunnel mesh. The elastic stiffness of the normal and shear springs along the interface elements are set to be much greater than the stiffness of soils around the tunnel, but the shear resistance will be limited by friction angle. The resulting behavior is nearly rigid-plastic. Table 7 lists input parameters of the interface elements.

Table 7. Parameters of interface elements

Friction Angle	Dilation Angle	Permeability (cm/sec)	Cohesion (Pa)	kn (Pa)	ks (Pa)
23°	0°	0	0	3.3×10^9	3.3×10^9

Note: kn is the spring stiffness for the normal spring and ks is for the shear spring.

SENSITIVITY STUDY

Before starting the numerical simulation of the centrifuge tests, the mesh geometry for the numerical model needs to be decided. A sensitivity study on mesh size and interface element types is conducted to confirm that acceptable results (from accuracy and efficiency points of view) will be obtained from the numerical model.

Mesh size

In numerical simulation, the spacing of the nodes affects the accuracy of results and the computer simulation time greatly. Mesh sizes of the whole numerical model and the vertical mesh in Gravel Foundation are studied.

Mesh Size of whole model. Ideally, the simulation results will approach an ultimate value when the mesh size becomes finer and finer but the computer simulation time will become longer and longer. Analysis using Coarse, Medium and Fine Meshes were performed to study the sensitivity to the mesh size. Table 8 lists the number of nodes and the computer simulation time for a 60 second dynamic event. Figure 6 shows tunnel movements for coarse, medium, and fine meshes. Computer simulation times of the three runs are all in the practical time range. Simulation times longer than 34 hours were considered impractical. In addition as the mesh becomes finer, element geometry problems become more prevalent in the large strain mode of FLAC program. The Fine Mesh was chosen as the basis for the numerical models presented in this paper.

Table 8. Mesh sizes, simulation time, and resulting displacement

Name	Smallest element Size	No. of Nodes	Computer Simulation Time	Tunnel Movement
Coarse	0.6 x 0.5	1024	8 hours	0.23 m
Medium	0.55 x 0.5	1505	14 hours	0.23 m
Fine	0.35 x 0.5	2035	34 hours	0.35 m

Node spacing in Gravel Foundation. As the space between the tunnel and trench is limited, it was thought that the discretization of Gravel Foundation would be especially important. Three runs with the fine mesh, but with 3, 5 and 10 layers of elements in Gravel Foundation were conducted. 5 layers (illustrated in Figs. 11 and 13) was the number used in the Fine Mesh model described in the previous section. Computer simulation times and mesh sizes in Gravel Foundation were listed in Table 9.

Table 9. Computer simulation time for different numbers layers of elements in Gravel Foundation

Elements in Gravel Foundation (GF)	Mesh Size in GF (m)	Tunnel Movement (m)	Computer Simulation Time
3 layers	1 x 0.43	0.52	30 hours
5 layers	1 x 0.26	0.35	36 hours
10 layers	1 x 0.13	0.27	172 hours

Figure 6 shows tunnel movements of different mesh size simulations. Comparing the tunnel movements and the simulation times, the simulation time for 5 layers in Gravel Foundation is 1.2 times longer than for 3 layers. The simulation time for 10 layers is 4.8 times longer than for 5 layers. It is interesting that increasing the number of layers tends to decrease the computed movement. It is believed that smaller elements tended to result in earlier bad geometry during large deformation, which might have some how constrained the calculated movement. Considering the accuracy and the acceptable simulation time, 5 layers of elements in Gravel Foundation was chosen for this paper.

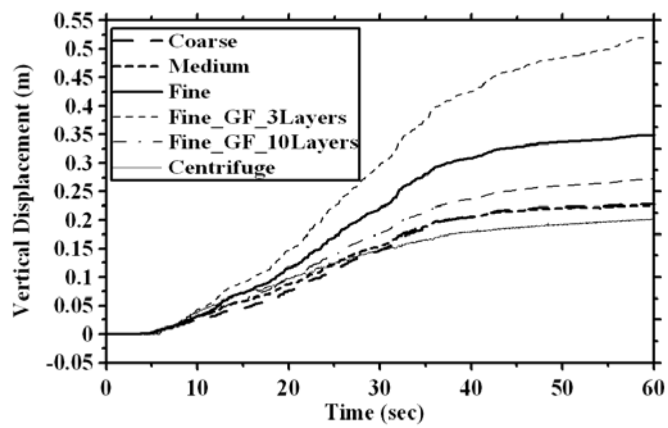


Fig. 6. Tunnel uplift results of different mesh sizes

Interface elements

Interface elements were placed around the tunnel to allow relative movement between the tunnel and the soils. In the numerical simulations, the interface elements are placed around the tunnel via two methods: Method(1) an interface element layer (X marks shown in Fig. 7) is placed around the tunnel. Method(2) In addition to the interface element layer around the tunnel, an additional interface element layer is placed between the bottom of the tunnel and Foundation Course. X marks were placed extending outward and down from the corners to enable the interface elements move relatively to soil elements at the corners of the tunnel.

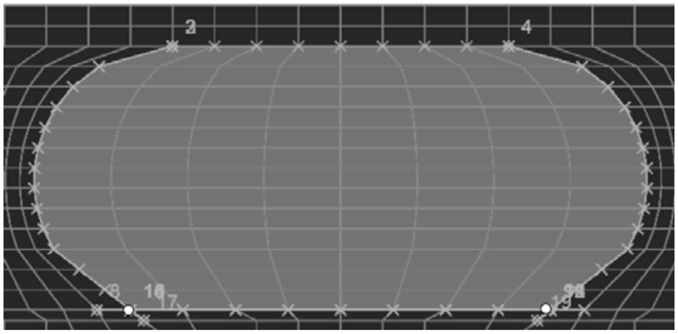


Fig. 7. Interface elements(X marks) placed around the tunnel

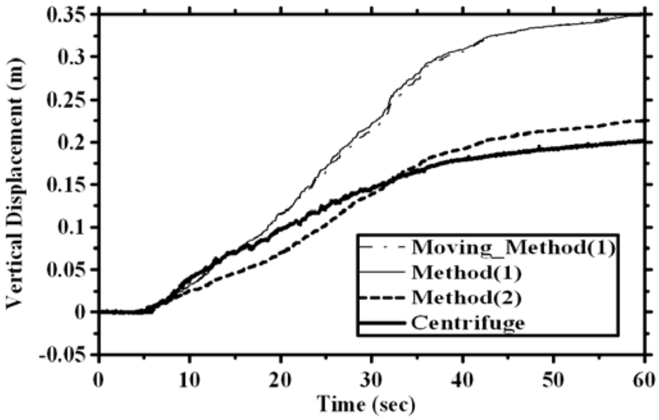


Fig. 8. Tunnel uplift results of Method (1), Method (2) with and without moving nodes and the centrifuge test

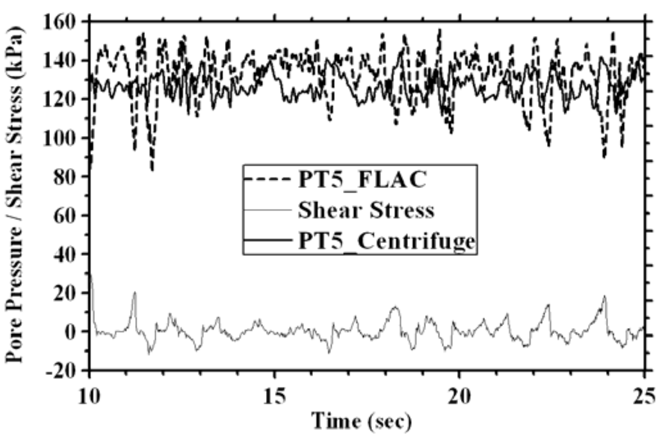


Fig. 9. Comparison of shear stress and pore pressure underneath the tunnel for Method (1)

Figure 8 compares the tunnel movements of the centrifuge test with predictions using the Fine Mesh with Method(1) and Method(2). Simulation results of Method(2) are closer to the centrifuge test results than Method(1). However, neither method captured the rate of the tunnel movements perfectly.

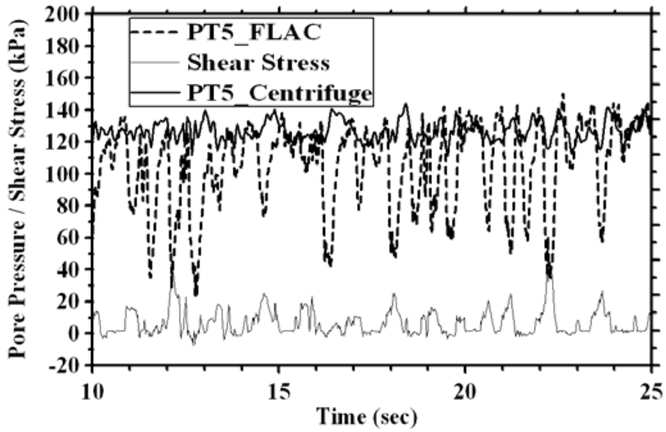


Fig. 10. Comparison of shear stress and pore pressure underneath the tunnel for Method (2)

Figure 9 & 10 show the pore pressure at PT5 between 10 and 25 seconds in numerical simulations, the pore pressure of Method(2) is somewhat lower than the centrifuge test results and Method(1). Figure 9 & 10 also show the shear stress underneath PT5 for Method(1) and Method(2). From figures, the negative pore pressure spikes associated with dilatancy are more exaggerated at PT5 in Method(2) than in Method(1) which leads to the lower average pore pressure at sensor PT5 in Method(2) than in Method(1). The more frequent dilation and the lower pore pressure are reasons that Method(2) results in less movement than Method(1).

<u>Movement around the corners of the tunnel</u>

Because of interface elements are used around the tunnel, the soil nodes can slide relative to the tunnel. But the soil node, next to the bottom corner of the tube (White o marks in Fig. 7) is constrained so that it cannot slide; the interfaces allow movement along the direction of the interface, but constrain movement normal to the interface. At any corner where interfaces intersect at an angle other than 180 degrees, the interface node is constrained so it cannot move perpendicular to either interface; thus, it is not allowed to slide in either direction. There is actually a small relative movement (associated with kn values) that occurs prior to complete locking. To solve this problem, a crude remeshing scheme was attempted. After a small movement occurs at the corner soil node, the corner node is moved back to the nodes at the corner of the tunnel. A function programmed using FISH language in FLAC2D was used to calculate the distance between soil nodes and the tunnel nodes and to move soil nodes. When the distance reaches the target value, this function will move soil nodes back to the tunnel nodes. The distance which makes the interlocking occur is about 0.8mm and the target value was chosen as 0.8mm.

Figure 8 and 11 show results of Method(1) with and without moving corner nodes. Tunnel movements and the pore pressure of Method(1) with and without the moving nodes procedure are very close. Therefore, this remeshing procedure was abandoned.

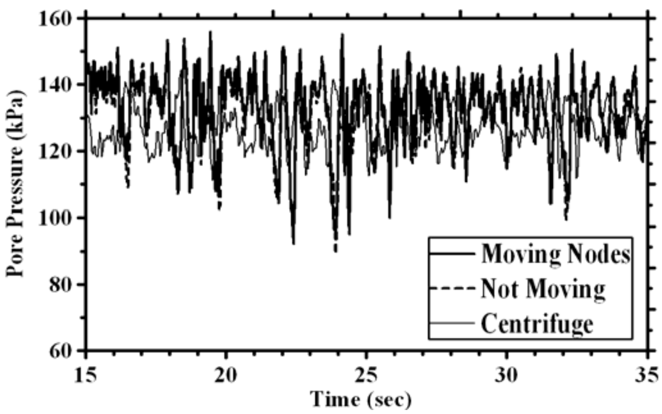


Fig.11. Pore pressure at sensor PT5 of the centrifuge test and Method (1) with and without moving nodes

Numerical model for the centrifuge test simulation

After the studies associated with the design of the mesh and interface elements, the selected mesh to simulate the centrifuge tests is described as: (1) Fine Mesh with five layers in Gravel Foundation; (2) One interface element layer around the tunnel is used in the numerical simulation model. Comparing the pore pressure underneath the tunnel, Method(1) seems superior to Method (2). Therefore, Method(1) was selected.

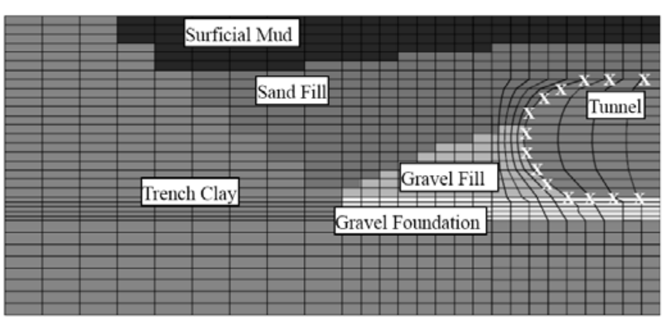


Fig. 12. FLAC program mesh for simulation

Figure 12 shows the left half of the final numerical model mesh used for numerical simulations. The right half is the same as the left half. The input motion used in this paper for simulations is TCU motion recorded in JCC01. Interface elements are indicated by X in Fig 11.

COMPARISONS OF NUMERICAL SIMULATION AND CENTRIFUGE RESULTS

The numerical simulation results and the centrifuge test results are compared to make sure that the numerical model can capture the main mechanisms observed in the centrifuge tests. Only comparisons of JCC01 are presented.

Movement of the tunnel

Figure 13 shows the trajectory of the tunnel in TCU motion of the numerical simulation model. Comparing Fig. 13 and Fig. 5(a), regardless of the magnitude difference of the vertical and horizontal movements, the trajectory plot of the numerical simulation model has the same pattern as the centrifuge test. The vertical movement of the tunnel gradually accumulates in an upward direction while large cyclic horizontal movements occur.

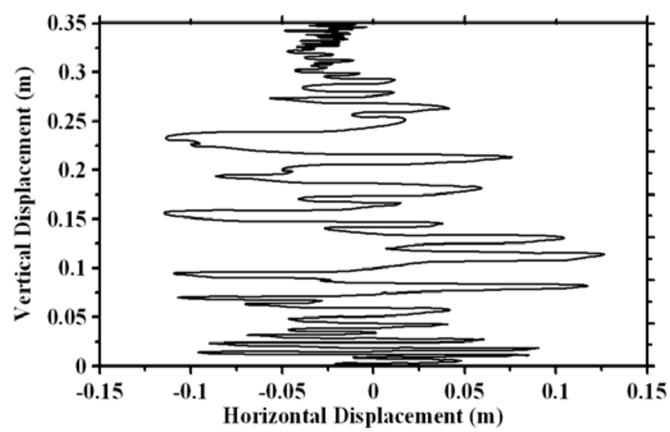
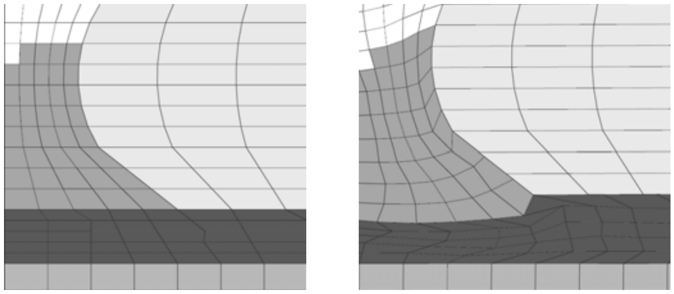


Fig. 13. Trajectory of the tunnel in the numerical simulation model

Deformation pattern

Figure 14 show the mesh before and after the shaking event. Comparing with Fig. 3, the numerical model has a similar deformation pattern of soils surrounding the tunnel with the observed deformation pattern in the centrifuge tests.



(a) (b)

Fig 14. Meshes of liquefiable soils around the tunnel from FLAC model (a) Before shaking (b) After shaking

<u>Pore pressure isochrones underneath the tunnel</u>

Figure 15 shows the pore pressure isochrones at different time steps from sensor PT1 to PT7 underneath the tunnel during the TCU event in JCC01. Comparing Fig. 15 and Fig. 4, both of them show U-shape curves of the pore pressure underneath the tunnel. Although the variation of the pore pressure across the base of the tunnel is not as large in Fig 15 as it is in Fig 4, it appears that the numerical simulation can capture the mechanisms observed in the centrifuge tests pretty well.

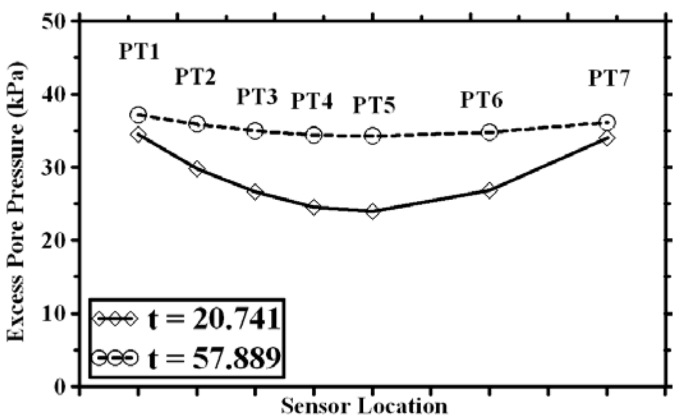


Fig 15. Pore pressure at sensors in the bottom of the tunnel

DISCUSSION

In the sensitivity study, the node in the soil at the corner of the tunnel is constrained to move with the tunnel. A crude remeshing procedure is tried to fix this problem, but the remeshing did not seem to help. This procedure only moves the soil nodes back to the tunnel nodes but the soil elements at the corners of the tunnel still can not flow underneath the tunnel therefore this procedure does not help. A very fine mesh (Finer than 0.8mm) with interface elements placed around the corners of the tunnel might be able to make the soil element flow underneath the tunnel but the geometry problem will be more prevalent in this approach and needs to be solved. Also, the simulation time will increase significantly. Alternatively, a discrete element treatment of the soil near the corner might be able to solve the problem of the discontinuity at the corner of the tunnel.

The tunnel movement from the numerical simulation (fine mesh with Method(1) interface treatment) is about 75% larger than the movement of the centrifuge test. A few reasons could explain this discrepancy. The first possible reason is that the estimated relative densities input in the numerical simulations did not match the real densities of the soils. The relative densities of the liquefiable soils in the centrifuge tests were estimated using a small scale CPT in the centrifuge. Therefore, a discrepancy of the estimated and the real densities could occur because of the different CPT equipment. Also, the way of interpreting CPT data might lead to a discrepancy between the estimated and the actual densities. A second reason for discrepancies is the imperfection of the numerical models. The centrifuge model is complicated and many mechanisms contributed to the tunnel movement. The constitutive models used in FLAC are not perfect and might miss some mechanisms. Constitutive models which can model the soil behavior better could be used to improve the numerical simulation results. The numerical simulation results were sensitive to changes in the mesh geometry and interface treatment, and the results cannot be shown to converge to a stable result.

The pore pressures distribution of the numerical simulation in the bottom of the tunnel had the same shape as the centrifuge test. But the gradient of pore pressure between the edge of the tunnel and the center of the tunnel is less in the simulation than the experiment. Thus, the tunnel movement caused by the pore water migration in the numerical simulation will be less than that in the centrifuge model.

CONCLUSION

Two centrifuge model tests were performed to study the seismic performance of the BART Transbay Tube. Three mechanisms of the tunnel movement were categorized from the centrifuge test data and model dissection: (1) Ratcheting (Sand flow) (2) Pore water migration (3) Bottom heave of the base of the trench. Those mechanisms were suggested from the numerical analyses performed by Fugro (2008) prior and following the two centrifuge experiments and were also confirmed experimentally.

A finite difference program, FLAC, is used to simulate the centrifuge model tests. The parameters of constitutive models used in FLAC program are introduced. A sensitivity study is performed to decide the mesh sizes, the mesh sizes of Gravel Foundation and the use of interface elements in the numerical simulation model. Considering both the practical restrictions on computer simulation time and the accuracy of the numerical results, the Fine Mesh with five layers of elements in Gravel Foundation is used for the numerical simulation model. The pore pressure distribution along the base of the tunnel seemed to be better modeled if only one interface element layer is used around the tunnel.

The tunnel movement of the numerical simulation is about 75% higher than the centrifuge test results but the trends of the tunnel movement are similar. Second, deformation patterns of soils are compared. The deformation patterns of the liquefiable soils of the numerical simulation and the centrifuge test are similar. Liquefiable soils move toward the bottom of the tunnel in a ratcheting mode during the shaking event. Third, pore pressures across the bottom of the tunnel are compared. The numerical simulation data show the same U-shape pore pressure distribution as discovered in the centrifuge test. Similarities of these key features indicate this numerical model could capture the main mechanisms causing the movement of the tunnel, but precise agreement between the experiment and simulation cannot be expected.

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