Seismic Ground Behavior in Tokyo Port at the 2011 off Pacific Coast of Tohoku Earthquake

- An effective stress dynamic analysis focusing on the impact of the aftershock -

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ABSTRACT

Seismic ground motions and behavior of excess pore water pressures on a reclaimed land during the 2011 off Pacific coast of Tohoku earthquake are studied numerically by using a computer program, "FLIP ROSE" (Iai et al., 1992, 2011). Long duration time of 30 minutes which includes both the main shock and the aftershock is taken into account in the dynamic analysis considering build-up and dissipation of excess pore pressures as well. The site studied is located in Tokyo port, Japan, where seismic ground motions are obtained with vertically arrayed seismographs by Tokyo metropolitan government. It is shown that observed peak ground accelerations at each depth of ground are reasonably simulated. The ratio of excess pore water pressure is calculated as maximum of about 0.4 during aftershock, agreeing with the fact of no significant liquefaction at the site. It is noted that significant rise of excess pore water pressure is calculated in the aftershock, suggesting significant effect on liquefaction.

Introduction

During the the 2011 off Pacific coast of Tohoku earthquake in north eastern Japan (M_w=9.0, hereafter, Earthquake 3.11) , soil liquefaction occurred in a wide area of reclaimed land along Tokyo Bay. Despite of very large epicentral distance about 380km to 400km, and the relatively small peak ground accereration (PGA) of about 100gal to 150 gal (Fukutake et al, 2013), significant liquefaction occurred in Urayasu city resulting in sever damage of many houses, lifelines, civil structures. The security camera in a junior high school in Urayasu city captured the onset of sand boiling during the mainshock first, and it expanded in the aftershock (M=7.7) which occurred about 30 minutes after the mainshock (Urayasu city government, 2011, Ueda et al, 2013). In addition, it is reported that some inhabitants in Urayasu city testified sand boiling did not occurre during the mainshock but the aftershock (Yasuda et al, 2013). This suggests some amount of excess pore water pressure buildup by the cyclic loading due to mainshock and reach complete liquefaction by following aftershock. In this regard, some analytical studies have been conducted focusing on the effect of aftershock to liquefaction at Tokyo Bay area in the Earthquake 3.11.

Morikawa et al (2011) undertook a numerical analysis on the ground in Urayasu city by effective stress method to clarify the effect of long duration time. It was shown that the characteristics of ground would have changed by aftereffect of stress by precedent earthquake,

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and turn easily into the state of liquefaction or re-liquefaction even by small intensity of aftershock. Fukutake et al (2013) studied the effect of aftershock on the settlement due to liquefaction by effective stress analysis for both liquefied and unliquefied ground in Tokyo Bay area. The mainshock and aftershock was considered in a row on the reponse analysis. It was mentioned that the excess pore water pressure continuously rise even after the cease of mainshock with a slower pase by the motion of low amplitude of acceleration. The aftershock would cause the large ammount of settlements after earthquake, estimating the 40% of all settlement could occurr in Urayasu city. Ueda et al (2014) conducted the numerical study on the re-liquefaction phenomena targeting on the liquefied ground at Takasu district in Urayasu city. The excess pore water pressure buildup and dissipation during and after earthquake motion are considered in the study. It was shown that re-liquefaction would occur by after-shock in the condition of high excess pore water pressure due to mainshock.

It was thus clarified by past studies that there would be high possibility of re-liquefaction when the ground is in the process of dissipation of pore water pressure, supporting the existence of effect of aftershock on liquefaction. However, the past studies were focusing on mainly the liquefied ground where exhibition of liquefaction was observed. It will be meaningful in engineering point of view to study on non liquefied ground to add further case histories.

The authors have studied numerically the seismic ground behavior of non liquefied ground in Tokyo port at the earthquake 3.11. The reclaimed ground at Shin-Ariake in Tokyo Port is targeted where the seismic acceleration record is observed by vertically arrayed seismograph. First, ground behavior is assessed by reproducing stress strain hysteresis in the ground by using the recorded acceleration time histories. Next, the ground model is constituted at the site for dynamic effective stress response analysis. Finally, the effect of aftershock is studied using the developped model by considering reproduced mainshock and aftershock. The same constitutive model for soil as used in the past study by Ueda et al (2013) is used which is utilised in a computer program "**F**inite element analysis program of **Li**quefaction **P**rocess/ Response of Soil-structure systems during Earthquakes (FLIP ROSE, Iai et al., 1992, 2011).

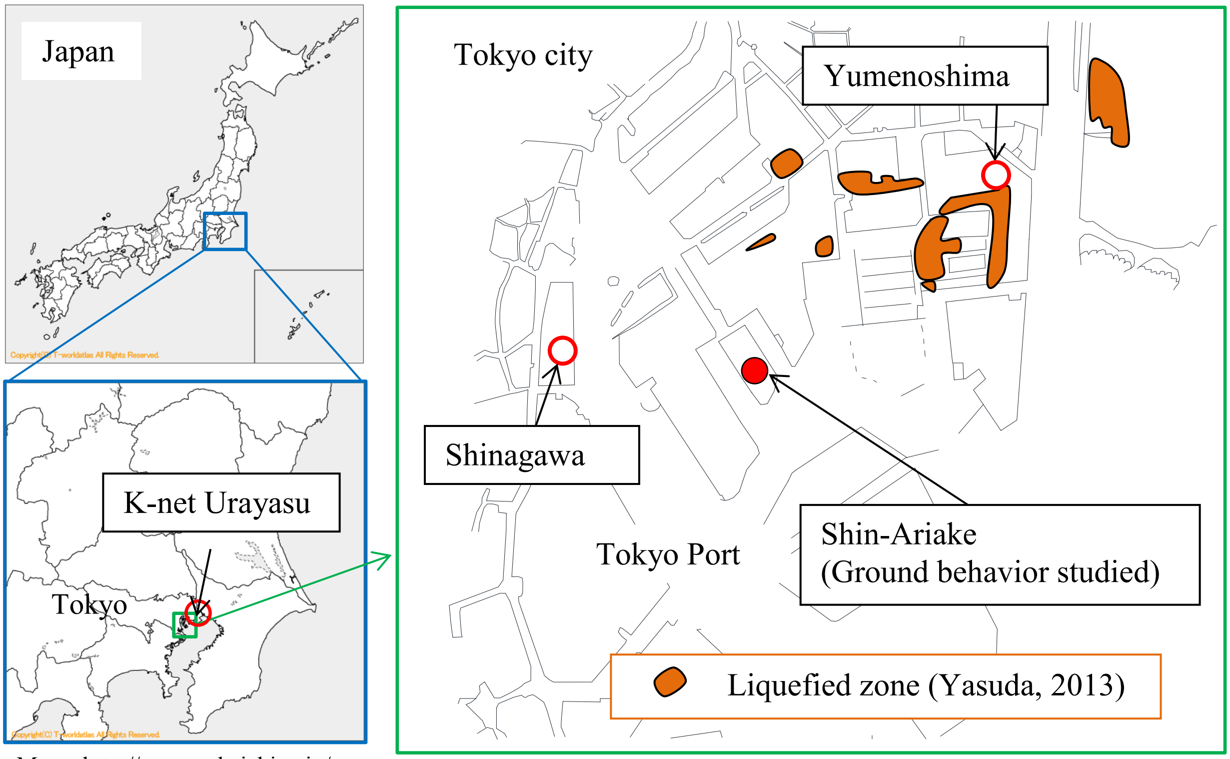
Seismic Ground Behavior during the Earthquake 3.11 at Tokyo Port

Recording Station and Recorded Motions

The location of recording stations around Tokyo Port is illustrated in Figure 1. The reported liquefied zone (Yasuda et al, 2013) at Earthquake 3.11 are shown in the figure as well. At the Shin-Ariake recording station, four set of seismograph are installed along the depth by Tokyo metropolitan government. Typical observed acceleration time histories in Earthquake 3.11 of main shock at GL-2.0m, GL-16.0m, GL-75.0m are depicted in Figure 2. PGAs are 1.38 m/s^2 (NS direction, GL-2.0m) and 0.67m/s^2 (EW direction, GL-75.0m).

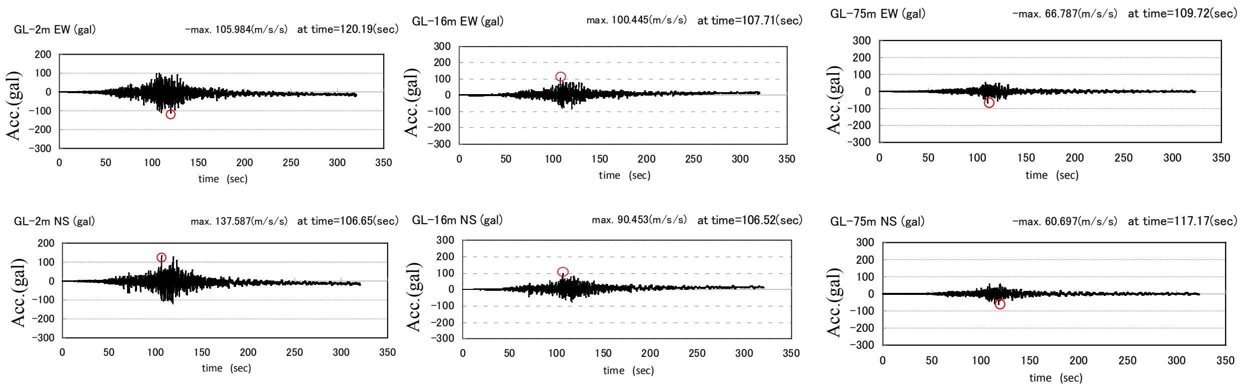
Reproduction of Stress Strain Hysteresis

To clarify the soil behavior during main shock in Earthquake 3.11, stress strain hysteresis were reproduced using recorded accelerations. Shear stress in the ground was calculated by considering acceleration time history and mass density of soil.Shear strains were assessed by horizontal displacement time histories which were derived by time integration of acceleration, and vertical distance between seismographs (CDIT, 1997). Typical stress strain hysteresis are illustrated in Figure 3. The ground between GL-2.0m to GL-16.0m includes liquefiable soil of bank and alluvial sandy layer (Ysu). GL-36.0m to GL-75.0m are at non liquefiable soil of diluvial layer. No significant nonlinear hysteresis are shown in stress strain at GL-2.0m to GL-16.0m, suggesting no liquefaction occurred in the ground.



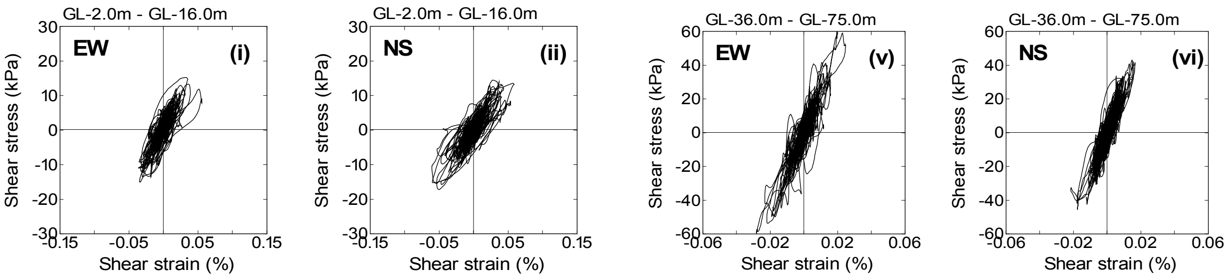
Maps: http://www.sekaichizu.jp/

Figure 1. Location of the recording station (Tokyo Port, Japan)



(a)GL-2.0m (b)GL-16.0m (c)GL-75.0m

Figure 2 Observed horizontal acceleration time histories at Shin-Ariake station in Tokyo Port



(a) GL-2.0m to 16.0m (b) GL-36.0m to 75.0m

Figure 3 Reproduced Stress strain hysteresis during main shock by observed acceleration time histories at Shin-Ariake recording station

Ground Model for Effective Stress Dynamic Analysis

Ground Model and Determination of Model Parameters

The effective stress model used in this study is composed of two models. One is the multiple shear mechanism for nonlinear stress strain relationship of soils (Towhata and Ishihara, 1985). Basic idea of this mechanism is the simple shear mechanism in any direction of shear plane in soil. The effect of rotation of principal stress axes, which is known as characteristic nature of soils, is considered in this model. Also the behaviors of soil at the anisotropic consolidation can be realistically simulated (Iai et al., 1992). Principal model parameters are initial shear modulus, maximum damping coefficient, shear resistance angle and cohesion.

The other is the model concerning generation of excess pore water pressures. A stress-dilatancy relation (Iai et al., 2011) is incorporated in the multiple shear mechanism for cyclic behavior of sand. The contractive component of dilatancy is in proportion to the cumulative shear strain. This model has ten particular parameters, designating the control of contractive and dilative behavior, as well as volume compressibility after soil liquefaction. Solving the governing equations of the model with seepage flow equation in a coupled manner (Zienkiewicz and Bettess, 1982), it becomes possible to calculate pore water pressures in soil. Volumetric strains of soil accompanied by dissipation of excess pore water pressures are simulated under the condition of given permeability of soil (e.g. Tamari et al, 2009).

The ground model at the recording station was developpped by refering to the boring log near the site (Tokyo metropolitan government) and the geological cross section in Tokyo Port (Tokyo Geotechnical Consultant Association, 2004). The ground model is illustrated in Table 2. The water table is assumed to be at GL-3.7m, liquefiable soil deposit to be the bank (Elevation of 2.165m to -3.135m) and alluvial sandy layer (Ysu, Elevation of -3.135m to -5.635m). Model parameters for soil were assessed by refering to literal parameters and technical reports (Ishihara et.al, 1989, TGCA, 2004). Parameters for phisical characteristics, dynamic deformation characteristics and permeability are summarized in Table 2. Dilatancy of soil was taken into account for bank and alluvial sand (Ysu) below water table as liquefaction characteristics. Volume compression characteristics are specified by relative density using relationship with maximum shear strain (Ishihara, Yoshimine, 1992). Permea-bilities are estimated based on the soil classification descrived in boring log. Parameters for liquefaction and volume compression are specified as best assessed parameter set by numerical simulation of undrained cyclic shear test. Soil parameters derived are summarized in Table 2.

Verification of the Numerical Ground Model

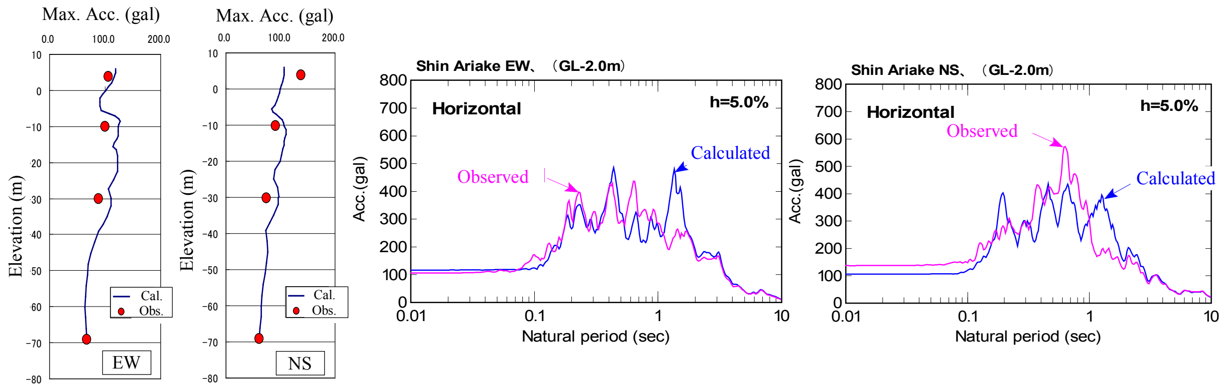
To verify the developed ground model, earthquake response analysis is conducted applying observed base motion (GL-75.0m). Distribution of calculated maximum accelerations and their response spectrum are depicted in Figure 4 with observed values. The calculated maximum accelerations show good agreement with observed values with difference of 10 to 30%. The maximum ratio of excess pore water pressure (=1 $-$ σ_m'/σ_{mo}', where σ_{mo}': effective mean stress) is calculated as about 0.1, being consistent with no liquefaction at the site. The calculated acceleration spectrum at GL-2.0m is very similar to observed. These calculation results suggest the developed grand model is considered reasonable.

Table 1 Summary of model parameters of dynamic deformation characteristics

Soil profile		Elevation (m)	Thickness of strata (m)	N value	γt (kN/m^3)	Vs (m/sec)	Elastic shear modulus at a confining pressure of (σ ' ma) Gma(kPa)	Reference confining pressure (kPa)	Poisson's ratio	Maximum damping coefficient Hmax	Shear resistance angle ϕ (°)	Cohesion C (kPa)	Coefficient of permeability k (m/sec)	Approximate location of seismograph
		5.865												
Bank	Sand	2.165	3.70	7	18.0	140	71402	98.0	0.33				1.0×10^{-5}	GL-2.0m
Bank	Sand	−3.135	5.30		19.1	175	71402	98.0	0.33	0.240	39.1	0.0	1.0×10^{-5}	
Ysu	Silty sand	−5.635	2.50	5	18.2	160	48218	98.0	0.33	0.240	38.2	0.0	1.0×10^{-5}	
Ycu	Sandy silt	−7.835	2.20	3	15.0	138	27876	98.0	0.33	0.150	0.0	40.0		
Btc	Silt	−16.535	8.70	6	15.0	172	39383	98.0	0.33	0.150	0.0	56.7	1.0×10^{-7}	
Tsm	Silty fine sand	−19.935	3.40	25	18.0	280	114020	98.0	0.33	0.200	40.8	0.0	1.0×10^{-6}	
Tcl	Silt	−29.835	9.90	10	16.0	209	51515	98.0	0.33	0.150	0.0	98.0	1.0×10^{-7}	GK-16.0m
Tg	Gravel	−31.935	2.10	100	21.0	421	252783	98.0	0.33	0.240	45.2	0.0	3.0×10^{-3}	
Ecu	Sandy silt	−34.935	3.00	30	16.0	313	103126	98.0	0.33	0.150	0.0	296.7	1.0×10^{-7}	
Esu-silt	Silty fine sand	−39.135	4.20	30	21.0	274	98697	98.0	0.33	0.150	0.0	303.3	1.0×10^{-7}	
Esu	Fine sand	−63.000	23.87	113	21.0	330	233357	98.0	0.478	0.240	44.7	0.0	1.0×10^{-5}	
Egu	Gravel	−69.135	6.14	—	21.0	560	672000	505.2	0.478	0.240	44.7	0.0	3.0×10^{-3}	
Km	Rock			—	21.0	560	—	—	0.430	—	—	—	Impervious	GL-36.0m

Table 2 Liquefaction Parameters

Model Parameters		**Bank**	**Ysu**
Phase Transformation angle ϕ_p (°)		28.0	28.0
Parameters for dilatancy	ε_d^{cm}	0.68	0.74
	$r_{\varepsilon dc}$	0.958	0.928
	$r_{\varepsilon d}$	0.68	0.74
	q_1	1.0	1.0
	q_2	0.5	0.6
	q_4	1.0	1.0
	q_{us}	—	—
	r_γ	0.2	0.2
	rrm_{tmp}	0.5	0.5
	S_1	0.005	0.005
Parameters for volume compressibility	l_k	2.0	2.0
	r_k	0.147	0.135
Parameter for limit of liquefaction	c_1	1.52	1.53



(a)Distribution of maximum accelerations (b) Acceleration response spectra (GL-2.0m)

Figure 4 Comparison of calculated and observed accelerations

Reproduction of Ground Motion at base for Main shock and Aftershock

The earthquake motion both main shock and aftershock at the base is reproduced for numerical study. The observed accelerations at the surface of recording station K-NET Urayasu (NIED), and at GL.-89.48m of Yumenoshima (Tokyo metropolitan government) in Tokyo Port are used for calculation. Location of recording stations is shown in Figure 1. The outline of ground models used is presented in Figure 5. The model of Yumenoshima was developed by Ishihara (1989), K-NET Urayasu by Urayasu municipal government (2011). The depth of the base are defined as GL-50.2m at Yumenoshima, GL-42.8m at K-NET Urayasu respectivery.

One dimensional response analysis program “SHAKE” (Schnabel et al, 1972) and “DYNEQ” (Sugito, 1995, Yoshida et al, 1996) are used for calculation, in which frequency dependency is taken into account in “DYNEQ”. Caluculated base acceleration time historis and renponse spectra are depicted in Figure 6 taking EW direction as a typical result. The acceleration time history by DYNEQ seems to be consistent with real phenomena in terms of maximum acceleration of mainshock (M=9.0) and aftershock (M=7.7). The estimated acceleration time histories at the base are shown in Figure 6(b).

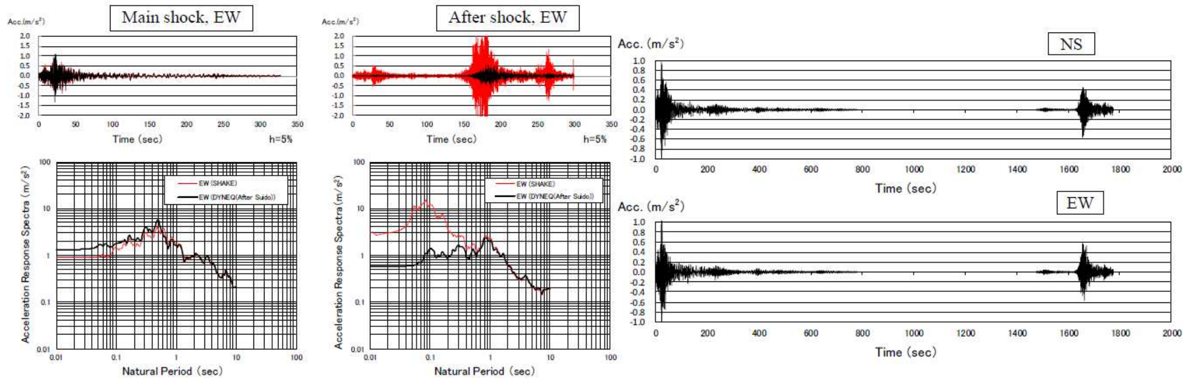
Site Name		Yumenoshima		
Surface Elevation (T.P.m)		—		
Water Level (GL.-m)		—		
Soil	Depth of Lower End	Thickness	Unit Weight	Shear Wave Velocity
-	(m)	(m)	(kN/m3)	(m/s)
Bs1	5.4	5.4	18.0	230
Bs2	9.0	3.6	19.1	130
As1	13.7	4.7	18.2	170
As2	20.0	6.3	18.2	220
Ac1	26.0	6.0	17.0	150
Ac2	40.0	14.0	15.0	150
Ac3	44.2	4.2	15.0	170
Ac4	45.5	1.3	18.7	250
Ac5	50.2	4.7	16.9	250
Dg1	59.5	9.3	21.0	560
Dg2	73.8	14.3	18.7	330
Dg3	83.6	9.8	17.9	330
Gr	89.5	5.9	21.0	560
Gr(Base)	—	—	21.0	560

(a) Yumenoshima (for Mainshock)

Site Name		K-net Urayasu		
Surface Elevation (T.P.m)		2.54		
Water Level (GL.-m)		2.5		
Soil	Depth of Lower End	Thickness	Unit Weight	Shear Wave Velocity
-	(m)	(m)	(kN/m3)	(m/s)
Bs	2.5	2.5	17.0	160
	5.2	2.7	17.0	110
As1	7.2	2.0	18.0	150
	10.8	3.6	18.0	160
	14.2	3.4	18.0	110
Ac1	23.7	9.5	16.0	110
	27.8	4.1	16.0	170
	36.8	9.0	16.0	140
Nac	39.6	2.8	17.0	150
	42.8	3.2	17.0	180
Kys(Base)	—	—	19.0	300

(b) K-net Urayasu (for Aftershock)

Figure 5 Ground Model at recording stations

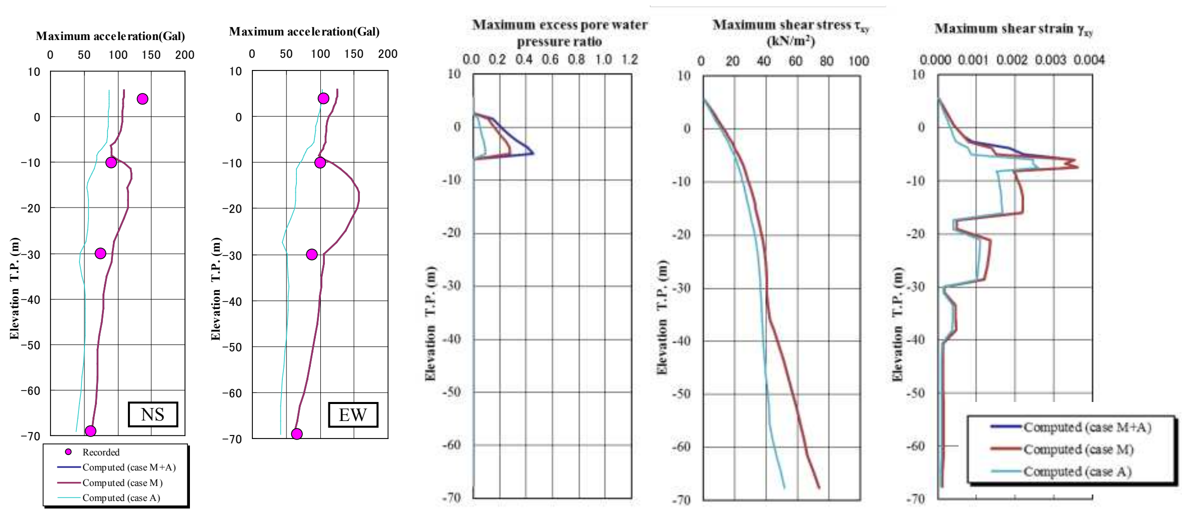


(a) Comparison of base earthquake motions (b)Accerelation time histories (DYNEQ)

Figure 6 Reproduced earthquake motion at the base

Dynamic Effective Stress Analysis focusing on the Impact of Aftershock

A series of dynamic effective stress analysis were conducted considering buildup and dissipation of excess pore water pressure with total duration time of 180 minutes (10800 seconds), including 30 minutes (1800 seconds) for main shock and aftershock, and 150 minutes (9000 seconds) for pore water dissipation (case M+A). In addition, the analyses with input of only main shock (case M) and only after shock (case A) are also carried out for comparison. Calculated maximum response values through the depth are illustrated in Figure 7. First of all, calculated maximum horizontal accelerations are reasonably consistent with observed values through the depth in both EW and NS direction. It is seen that the maximum acceleration in case M+A and case M are completely same, indicating maximum acceleration occurs during main shock. However, the maximum excess pore water pressure in case M+A of about 0.4 is greater than in case M of about 0.3. This implies that the maximum excess pore water pressure can appear during aftershock as presented in Figure 9(a).

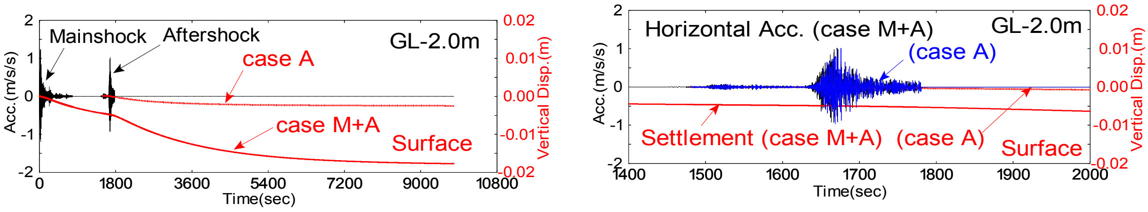


(a) Horizontal Acc. (b) ratio of E.P.W.Pr., Shear stress, Shear strain(EW direction)

Figure 7 Maximum response distributions

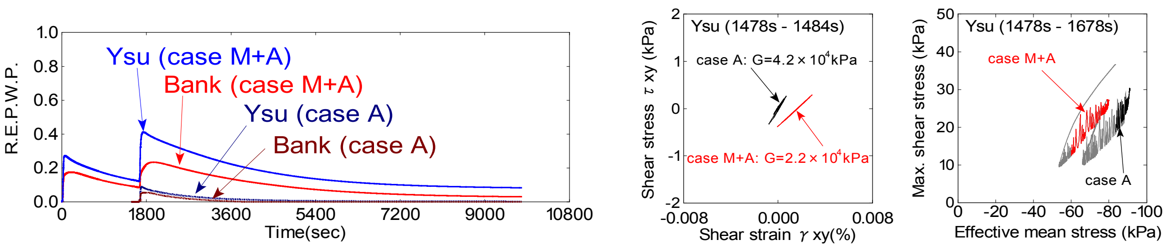
The time histories of horizontal acceleration at the GL-2.0m and settlement (vertical displacement, upward as positive) are depicted in Figure 8. Settlement is calculated to be

0.005m just before the aftershock and 0.018m finally. It suggests that about half of all the settlement can occur due to aftershock, supporting observations by Fukutake et al (2013). To clear the initial state of soil at the beginning of aftershock, secant shear modulus is calculated from the stress strain hysteresis as shown in Figure 9(b). The secant shear stiffness are G=2.2 $\times 10^4$ kPa in case M+A, and G=4.2$\times 10^4$ kPa in case A, implying about 50% of degradation of shear modulus. The stress path is illustrated in Figure 9(c). This indicates the effective mean stress before aftershock is reduced to about 80 kPa from the initial of 90kPa. The degraded shear stiffness seems to cause large shear strain, leasing to buildup of excess pore water pressure during aftershock.



(a) All duration of calculation (b) After shock (enlarged time scale)

Figure 8 Time histories of Acceleration and Settlement (EW component)



(a) Ratio of excess pore water pressure (b) Stress Strain hysteresis (c) Stress path

Figure 9 Time histories and hysteresis

Conclusions

The numerical study is conducted on the seismic ground behavior at the reclaimed ground at Shin-Ariake in Tokyo Port where no liquefied is reported at the Earthquake 3.11 to clarify the effect of aftershock on liquefaction. Following conclusions are derived:

(i) The stress strain hysteresis derived by recorded accelerations suggestd no liquefaction occurrence at the site during the mainshock in Earthquake 3.11.

(ii) By the dynamic effective stress analysis, it was found that the maximum pore water pressure and maximum shear strain appeared during aftershock not in main shock.

(iii) The ground which experienced the mainshock seems to become susceptive against liquefaction due to residual pore water pressure and degradetion of shear stiffness.

In engineering point of view, consideration of not only mainshock but also aftershock must be very important for assessment of excess pore water pressures in liquefiable soil layer.

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