16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 852 (Abstract ID)

Registration Code: S-C1463122864

NUMERICAL SIMULATIONS ON CAUSE OF DAMAGE TO A SHEET PILE TYPE QUAY WALL IN SOMA PORT DURING THE 2011 OFF THE PACIFIC COAST OF TOHOKU EARTHQUAKE

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Abstract

Port structures were damaged over a wide area of eastern Japan by the 2011 off the Pacific Coast of Tohoku earthquake. Though many damages of quay walls were due to liquefaction of ground, some damages were supposedly caused by tsunami action in addition to earthquake ground motion. Tie wire ruptured and collapse of steel sheet pile wall and aepron happened at the quay wall of Soma Port, possibly affected by the tsunami. Effective stress finite element analysis taking account of earthquake motion and subsequent tsunami under drained condition was conducted in this study, to clarify factors of the collapsed sheet pile type quay wall.

As a result, it was found that damage of the sheet pile quay wall of Soma Port was light due to earthquake motion but was extended by subsequent tsunami action. Influence of tsunami action to the sheet pile quay wall was clarified with the analysis taking into account the tsunami water weight on the backfill ground; increase in residual water pressure as tsunami water weight on backfill ground more largely affected unstability of the sheet pile quay wall than vertical load acting on backfill ground surface as that.

Keywords:the 2011 off the Pacific of Tohoku earthquake, effective stress analysis, finite element method, tsunami, gravel drain

1. Introduction

The 2011 Earthquake off the Pacific Coast of Tohoku caused enormous damage to ports and harbors in eastern Japan. Damage to most quay walls stemmed from ground liquefaction, but some quay walls appeared to collapse due to seismic motion and tsunami behaviors. The Port of Soma in Fukushima Prefecture experienced large-scale collapse of steel sheet pile quay walls, perhaps as a result of not just seismic motion but also tsunamis[1]. This study investigates the causes of the damage to the quay walls.

Earthquake engineering for port structures on the practical level often uses FLIP[2][3], a seismic response analytical program using the effective stress method. To express undrained ground movements, the multispring model in FLIP is used as a ground model. This study, however, examines tsunami behaviors after seismic motion has ceased, and thus applies the cocktail glass model in FLIP[4][5] that can incorporate drainage conditions to investigate ground movements after the excess pore water pressure elevated due to seismic motion has dispersed. This study also uses the cocktail glass model to examine the impact of permeability of the ground behind the quay walls, which had been improved with the gravel drain method.

2. Quay Walls and Their Damage

2.1 Quay 1-4 of Wharf No. 1 at Port of Soma

Quay 1-4 of Warf No. 1 at Port of Soma, 280 meters in length, is of anchored steel sheet piles. The construction of the quay began in FY1980 and ended in FY1982. Figures 1 and 2 show the location and cross section. For the construction of the quay wall, steel sheet piles were installed on the ocean side first, and then the front ground was excavated. Among all the quay walls at Warf No. 1, the ground around the anchor coupled-pile on the rear reclaimed ground at Quay 1-4 alone was improved with the gravel drain work.

2.2 Damage to Quay 1-4

Photo 1 shows the damage to Quay 1-4 of Wharf No.1 caused by the 2011 Earthquake off the Pacific Coast of Tohoku. The steel sheet pile wall fell toward the sea, and the apron of about 30m in width severely collapsed, where the wire tying the anchor piling of H-steel with the head of the steel sheet pile wall ruptured. The damage of this kind was observed at Quay 1-4 only at Wharf No.1. Other quays along the same face line of Wharf No.1 experienced some deformation of the head of sheet piles, but, it seems, have no such serious damage such as collapse.

The tsunami inundation height at the Port of Soma was about 10m[6], and so tsunami behaviors might have a certain impact on the deformation of the quay. Tsunami behaviors could scour the seabed, but a survey after the earthquake found no obvious increase in the water depth in front of the steel sheet pile wall[7] and concluded that the damage was not aggravated by scouring there.

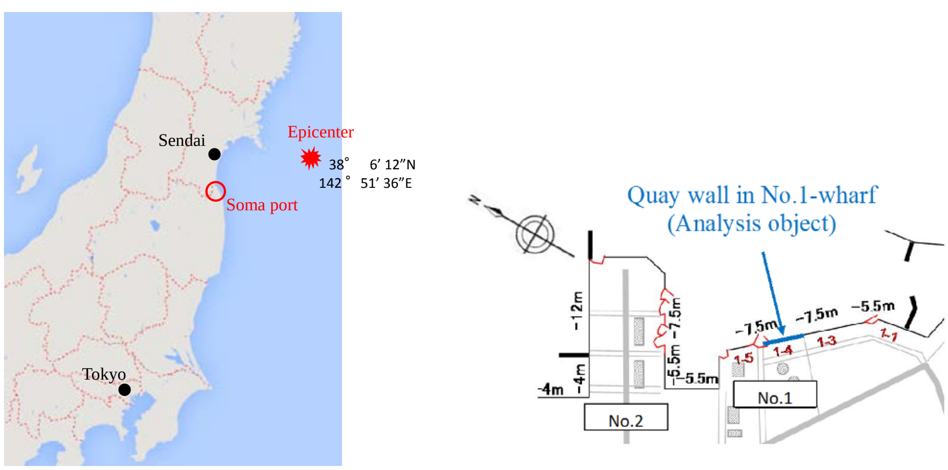


Fig. 1 – Locations of the quay walls for simulation

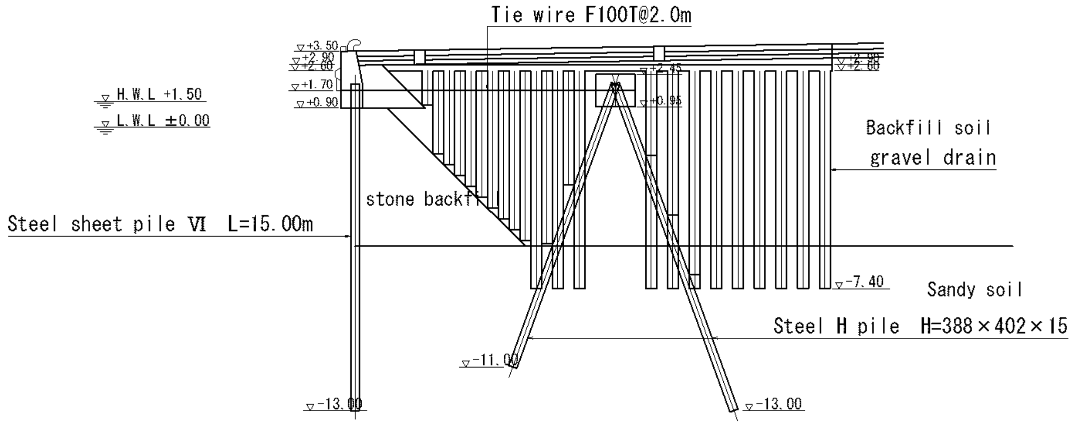


Fig. 2 – Cross sections of the quay walls for simulation

Photo 1– Damage to Quay 1-4

3. Analysis on the Damage from Seismic Motion

3.1 Setting of ground parameters

According to the boring data presented, the ground model for the analysis is assumed to consist of a reclaimed layer of 8.00m thick with an equivalent N-value of 8, and a sand layer of 7.25m thick with an equivalent N-value of 21. Ground parameters such as strength parameter and shearing rigidity have been set out according to a simplified setting method using the N-value and fine fraction content, F_c, of the boring data[8] (Table 1).

3.2 Setting of liquefaction parameters

Information about liquefaction properties of the ground concerned is required to set parameters for the cocktail glass model. However, liquefaction test has not been performed for the ground concerned, so the liquefaction strength curve has been estimated according to the simple judgement using N-values[9], which is proposed by Tokimatsu and Yoshimi. Table 2 shows the parameters of the cocktail glass models.

3.3 Modeling of gravel drain

For the ground, where the gravel drain work has been performed, an average coefficient of permeability for the ground including drains has been calculated in the light of the intervals of gravel drains, thickness of the drain work and other factors[10]. It is difficult to analyze the ground consolidated by the gravel drain work as a plane strain problem because it is a radial flow with the axial symmetry. Thus, the consolidation problem against the radial flow (Barron) has been replaced, as a proxy, by one dimensional consolidation with the equivalent consolidation rate (Terzaghi) by modifying the coefficient of permeability[11][12]. Table 3 shows the coefficients of permeability set out so far.

Table 1 – Ground parameter

		Bs	As	mudstone	engineering base
Density	ρ(t/m^3)	2.00	2.00	1.73	-
Reference confining pressure	$\sigma'ma$ (kN/m^2)	98	98	98	-
Fine fraction content	F_c(%)	10.0	0.0	0.0	-
Shear resistance angle	φ(°)	39.6	41.7	-	-
cohesion	C(kN/m^2)	-	-	1427.7	-
Elastic shear modulus at a confining pressure	G_{ma}(kN/m^2)	73885	131714	394352	-
Bulk modulus at a confining pressure	K_{ma}(kN/m^2)	192680	343489	1028407	-
Maximum damping coefficient	h_{max}	0.24	0.24	0.2	-
Porosity	n	0.45	0.45	0.45	-
Poisson's ratio	ν	0.33	0.33	0.33	-
S wave velocities	V_s(m/s)	-	-	-	550
P wave velocities	V_p(m/s)	-	-	-	1990

Table 2 – Liquefaction parameter

		Bs	As
Limit of contractive component	$-\varepsilon_d^{cm}$	0.15	0.15
Parameter controlling contractive component	$\gamma\varepsilon_{dc}$	0.71	0.18
Parameter controlling dilative and contractive components	$\gamma\varepsilon_d$	0.15	0.20
Parameter controlling initial phase of contractive component	q_1	1.00	1.00
Parameter controlling final phase of contractive component	q_2	1.00	0.23
Upper bound for hysteretic damping factor	h_{maxL}	0.24	0.24
Power index of bulk modulus for liquefaction analysis	l_k	2.00	2.00
Reduction factor of bulk modulus for liquefaction analysis	γ_k	0.5	0.5
Small positive number to avoid zero confining pressure	s_1	0.005	0.005
Parameter controlling elastic range for contractive component	c_1	1.91	4.90

Table 3 – Coefficient of permeability parameter

	gravel drain				average degree of consolidation	time coefficients			Terzaghi drainage distance	thickness of the improved layer	coefficient of permeability	
	intervals of gravel drains	drain pile diameter	equivalent effective catchment diameter	de/dw		Barron		Terzaghi			original ground	converted coefficient
	d	d_w (m)	d_e (m)	n	U (%)	F(n)	T_h	T_v	H (m)	L (m)	k (m/sec)	k'_H (m/sec)
Bs	1.25x1.25m	0.5	1.41	2.83	60	0.469	0.054	0.286	0.61	5.50	5.10E-05	4.15E-03
As	1.25x1.25m	0.5	1.41	2.83	60	0.469	0.054	0.286	0.61	3.80	5.10E-05	1.98E-03

3.4 Input motion

For the seismic motion, the study has adopted 2E wave estimated on the engineering foundation bed at the strong earthquake observation point "Soma-G" at the Port of Soma after the 2011 Earthquake off the Pacific Coast of Tohoku[13]. As shown in Fig. 3, seismic motion converted in the direction perpendicular to the face line of the quay has been adopted as the input seismic motion.

3.5 Analytical model

Figure 4 shows the analytical model, where the front steel sheet pile (V_L-type) and anchor coupled-pile (H-steel: 388x40x15) are assumed to be a nonlinear beam element, and the tie wire is assumed to be a nonlinear spring element in the light of axial rigidity and yield load (413kN/wire). The anchor coupled-pile is also assumed to be a friction pile for modeling[14][15].

3.6 Analytical results

Figure 5 shows the residual deformation and residual excess pore water pressure ratio. Horizontal displacement at the point of ground surface behind the sheet pile is about 20cm in the multi-spring model and cocktail glass model considering the drains. Residual deformation and residual excess pore water pressure ratio in Figure 5 have revealed that an increase in the excess pore water pressure at the drained area is curbed in the case where a large coefficient of permeability is adopted to make drainage performance high.

These results suggest that damage to the quay concerned by seismic motion is not considerable, and that the actual damage, complete collapse of the steel sheet pile wall, might be caused by other factors.

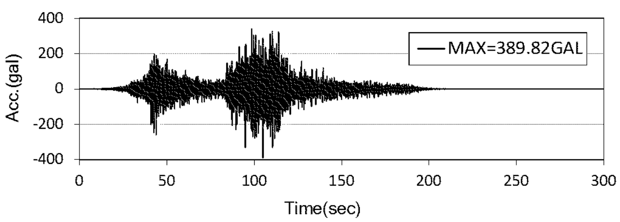


Fig. 3 – Input motion

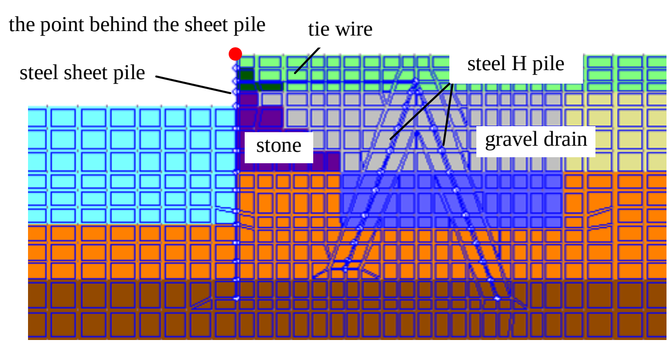
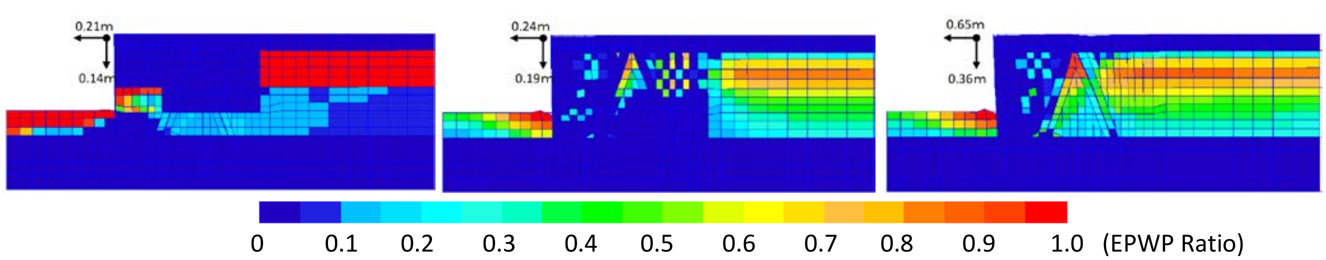


Fig. 4 – Analytical model



(a) multi-spring model (non-liquefaction layer) (b) cocktail glass model (considering drains) (c) cocktail glass model (not considering drains)

Fig. 5 – The residual deformation and residual excess pore water pressure ratio

4. Analysis in the light of Seismic Motion and Tsunami Behavior

4.1 Cases to be examined

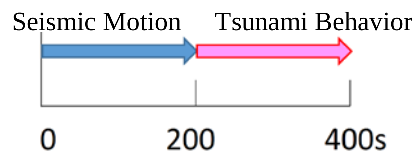
The analysis of the deformation of the quay in the previous chapter has revealed that the quay concerned might collapse mostly due to reasons other than seismic motion. Possible factors that might affect the collapse of the quay include tsunamis. Possible tsunami behaviors that could deform the quay are water flows and weight, but it is difficult to examine the impact of water flows on soil structures by a finite element analytical method. Thus, this study has introduced a cocktail glass model in FLIP to investigate the impact of the weight of tsunami water that overflew to the rear ground of the quay on the deformation of the quay.

Table 4 – Simulation cases

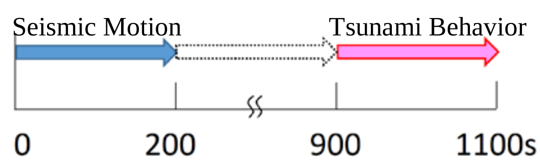
	Tsunami Behavior
Case_0	none
Case_1	distributed load
Case_2	forced pressure
Case_3	forced pressure and not considering drains

Table 4 shows cases examined. Case_0 assumes that the deformation was affected by seismic motion only. Case_1 assumes that the weight of tsunami water flowing into the rear ground acted as a distributed load. Case_2 considers the weight of tsunami water flowing into the rear ground as in Case_1 but assumes that the weight acted as forced pressure of the water table on pore water. Case_3 adopts the same assumption on the weight of tsunami water flowing into the rear ground as Case_2, but has investigated the effect of drains at the time of tsunamis by calculation using the coefficient of permeability of the original ground in no consideration of drains behind the quay.

Figure 6 illustrates the analytical procedure. In the case where the tsunami arrival time is not taken into account (Figure 6(a)), simulation has been performed to firstly apply seismic motion on the quay and then apply tsunami behaviors immediately after the seismic motion is over. It has been assumed that the tsunami behaviors gradually increase the height of tsunami water once they started to act, and that the water level reaches 5m in height in 200 seconds.



(a) Tsunami arrival time is not taken into account



(b) Tsunami arrival time is taken into account

Fig. 6 – Analytical procedure

In the case where the tsunami arrival time is taken into account (Figure 6(b)), simulation has assumed that it takes time after the completion of the seismic motion until the arrival of tsunami. The arrival time at the Port of Soma has been estimated at about 9 minutes after the 2011 Earthquake off the Pacific Coast of Tohoku occurred[6]. This study has investigated the time of drawback, which is believed to have an impact on the deformation of the quay, and where the tsunami water overflows into the rear ground but the sea surface level in front is normal. Therefore, in reference to the results of tsunami simulations performed for the Port of Kamaishi and other research findings[6], and in the light of the time of fluctuation of the water level, the study has generated tsunami behaviors approximately 15 minutes after the outbreak of the earthquake.

4.2 The results of the analysis

1) Case where the tsunami arrival time is not taken into account

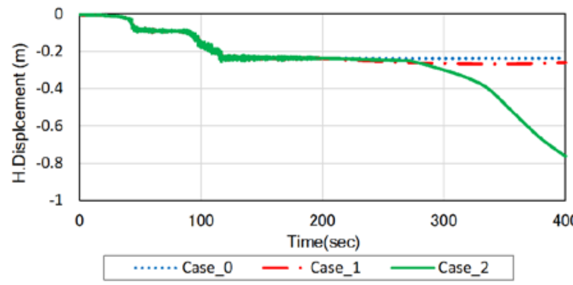
Figure 7 show the deformation time history of the ground behind the sheet pile due to seismic motion and tsunami behaviors (see Figure 4). There is no substantial difference between the results of Case 1, where the weight of tsunami water is taken into account as a vertical load, and Case 0, where only seismic motion (no tsunami behaviors). On the other hand, in Case 2, where the weight of tsunami water flowing into the rear ground is taken into account as forced pressure, the quay has started to deform considerably when about 300 seconds has passed, and the ground behind the sheet pile has continued deforming even the analysis has ended (400 seconds). This proves that the impact is greater if the weight of tsunami water is considered as forced pressure on pore water.

Figure 8 shows that there is not much difference in the excess pore water pressure ratios among all the cases at the time when the analysis ends.

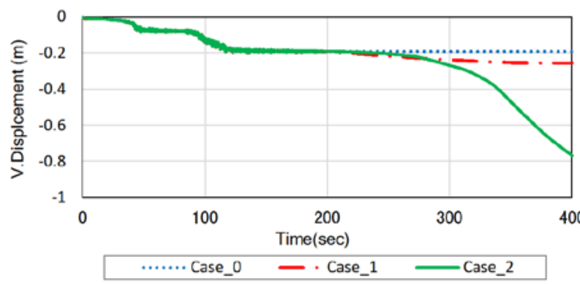
Figure 9 shows effective stress path. In Case 1, even if the height of tsunami water is raised to 5m, the stress path go upward to the right at both points ① and ② behind the front sheet pile, increasing the effective stress. In Case 2, on the other hand, the effective stress lowers after tsunami behaviors starts to have an impact. At the point ②, in particular, the effective stress continues lowering immediately after passing across the phase transformation line and before reaching the failure line, and then increases. The transition from compressive volume changes to expansive volume changes occurs along the phase transformation line. [16].

Table 5 – Residual displacement (in cases not taking into account tsunami arrival time)

	H. displacement (m)	V. displacement (m)
Case_0	0.24	0.19
Case_1	0.26	0.23
Case_2	0.76	0.77



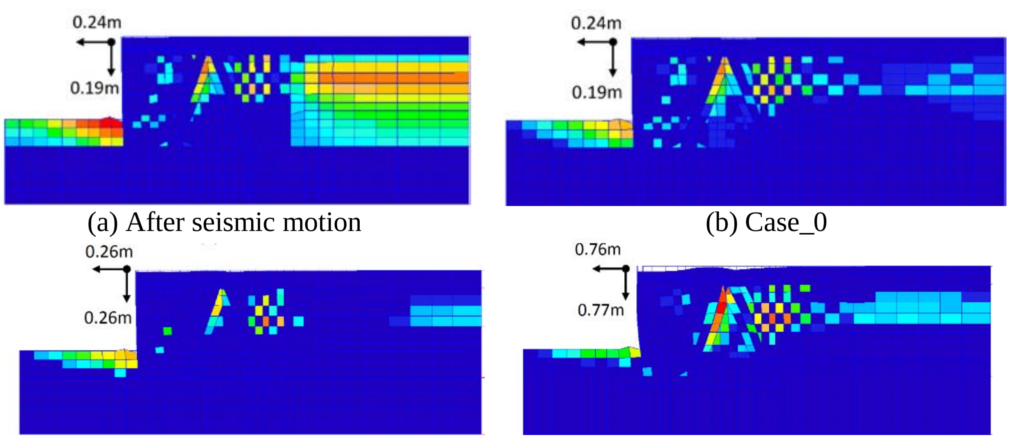
(a)H. displacement



(b)V. displacement

Fig. 7 – The deformation time history

As shown in Figure 10, the axial force on the tie wire reaches the yield load in around 320 seconds, and the ground behind the sheet pile is sharply displaced. Then, because of the breakdown of the tie wire, the sheet pile becomes unable to support the rear ground, and the internal stress on the rear ground reaches the failure line, resulting in a sharp increase in displacement.



(a) After seismic motion (b) Case_0 (c) Case_1 (d) Case_2

Fig. 8 – The residual deformation and residual excess pore water pressure ratio

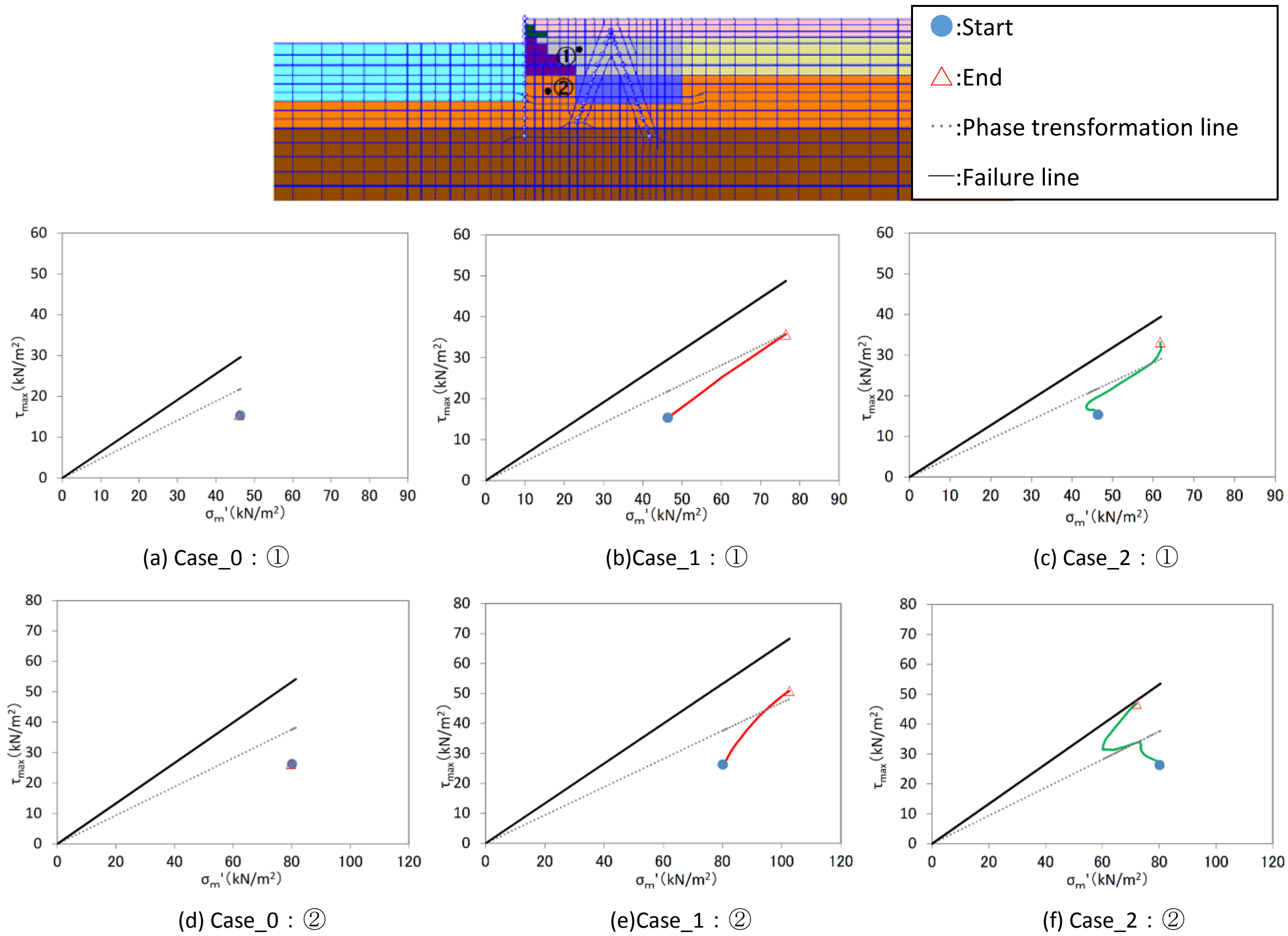


Fig. 9 –Effective stress path

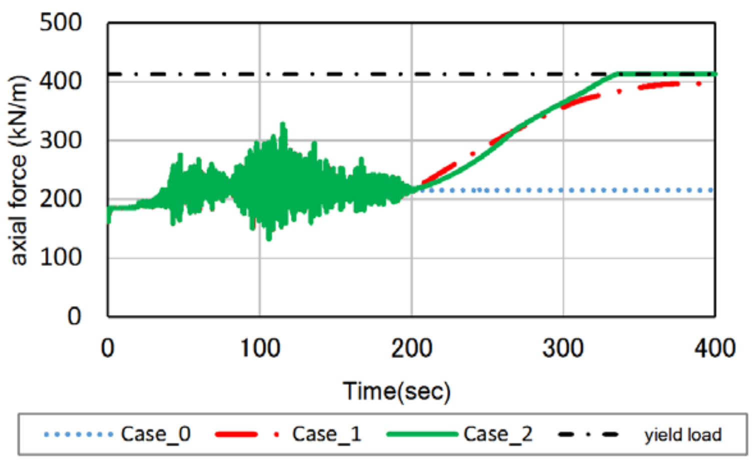


Fig. 10 – The axial force of the tie wire

2) Case where the tsunami arrival time is taken into account

Table 6 shows the residual displacement of the ground behind the sheet pile in cases taking into account tsunami arrival time (see Figure 4). In comparsiton with Table 5, particularly in Cases 0 and 1, the degree of deformation in the light of the tsunami arrival time is similar to that in cases where the tsunami arrival time is not taken into account, and the residual deformation in Case 2 is slightly smaller.

The study has found no particular difference, either, in the time history of excess pore water pressure ratios and effective stress path among cases where the tsunami arrival time is taken into account and is not.

3) Influence of gravel drain in the event of tsunami behavior

The results of the analysis given in the previous sections have revealed that the weight of tsunami water has a greater impact on the quay stability concerned in this study if it acts as forced pressure on the water table, rather than as a vertical load on the ground surface. In this section, Case 2, where forced pressure is applied on the water table, has been compared with Case 3, where the value of the original ground is used for the coefficient of permeability at the drained area and the effect of drains is ignored.

As shown in Table 7 (b) and Figure 10, in Case 3, displacement is greater because of the accumulated excess pore water pressure around the anchor pile without the drain, and is much greater after the event of tsunami behaviors.

Table 6 – Residual displacement (in cases taking into account tsunami arrival time)

	H. displacement (m)	V. displacement (m)
Case_0	0.24	0.19
Case_1	0.25	0.23
Case_2	0.75	0.74

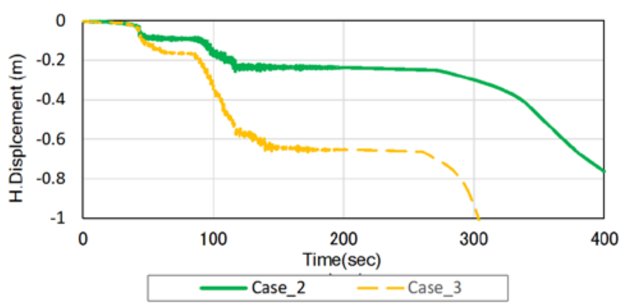
Table 7 – Residual displacement

(a) After seismic motion（0～200s）

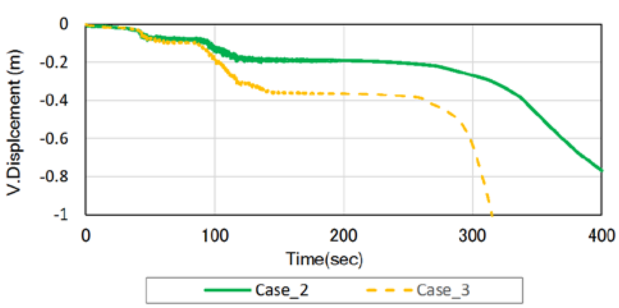
	H. displacement (m)	V. displacement (m)
Case_2	0.24	0.19
Case_3	0.65	0.36

(b) After tsunami behavior（200～400s）

	H. displacement (m)	V. displacement (m)
Case_2	0.76	0.77
Case_3	11.61	13.17



(a)H. displacement



(b)V. displacement

Fig. 12 – The deformation time history

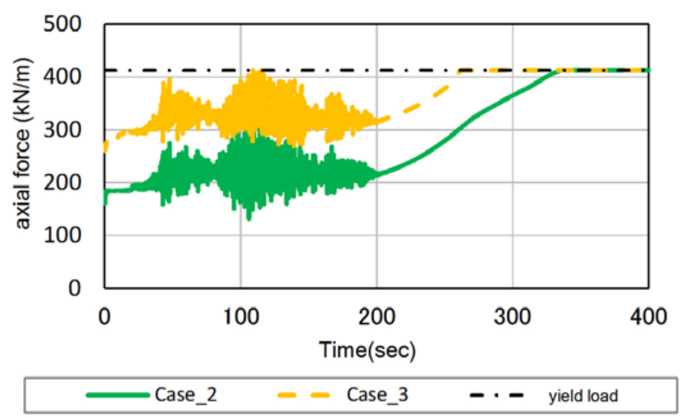


Fig. 11 – The axial force of the tie wire

5. Conclusions

This study has attempted reproductive analyses with seismic response analytical program, FLIP, using the effective stress method, on factors considerably damaging the steel sheet pile quay where even the tie wire was broken at the time of the 2011 Earthquake off the Pacific Coast of Tohoku. The study has confirmed the following facts through the analysis in consideration of the impact of tsunami behaviors and drains after seismic motion.

- The results of the analysis on the impact of seismic motion have revealed that seismic motion did not considerably damage Quay 1-4 of Wharf No. 1 at Port of Soma, the reasons being that the ground behind the quay had been improved by gravel drain work, and that no conspicuous liquefaction had occurred on the rear ground.

- The results of the analysis in the light of the weight of tsunami water on the rear ground have confirmed that tsunami behaviors after seismic motion had a greater impact on the steel sheet pile wall and rear ground if the weight was given as forced pressure on pore water in the light of the increase in residual pressure behind the wall, rather than as a vertical load on ground surface. If the weight of tsunami water was given as forced pressure, the effective stress lowered, which caused the axial force of the tie wire to reach the yield load and the displacement of the ground behind the sheet pile to expand. These findings are consistent with the actual damage to the quay.

- As for the influence of gravel drain, this study has confirmed that the deformation of the quay was smaller if the gravel drain work was applied (to increase the coefficient of permeability at the drained area), compared to the case where it was not applied (with the coefficient of permeability of the original ground), and also compared to the case where no drain work was applied and liquefaction occurred after tsunami behaviors arose. On the other hand, in the light of the height of tsunami water, 5m, on the rear ground, the deformation was greater if the weight of tsunami water was given as forced pressure on pore pressure, rather than a vertical load on the soil skeleton. At the same time, if the drain work was performed, the water pressure was more likely to be passed to the ground and affect the degree of deformation.

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