The proposal of countermeasures against level 2 earthquakes and tsunamis to -7.5m pier of Futami port in Chichijima islands of Ogasawara

by

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ABSTRACT

This paper describes an improved design for level 2 earthquake ground motion and level 2 tsunamis in the -7.5 m pier of Futami Port of Chichijima island of the Ogasawara Islands, about 1,000 km south-southeast of Tokyo.

1. Introduction

The coast of Japan is an area subject to frequent earthquakes and tsunamis. Since the Great East Japan Earthquake and the resulting tsunami in 2011, countermeasures against earthquakes and tsunamis have been implemented rapidly and extensively in Japan for facilities such as breakwaters and quay walls of commercial and fishing ports. Countermeasures are also being considered for port facilities on remote islands. Japan's Ogasawara Islands, a world heritage site, is located about 1,000 km for Japan’s main island, south-southeast of Tokyo, and consists of 30 large and small islands. Among these lies inhabited Chichijima island (population approximately 2,000), and Futami port serves as the only logistics mode and provides transportation for the flow of people, including tourists. A regular service is provided by a cargo and passenger ship, Ogasawara Maru (11,035 tons, 150 meters long), from Tokyo port on a round trip a week basis. In addition, since Futami Port is the only port on Chichijima, it is also required to fulfill the function of an emergency transportation facility in the event of a disaster such as an earthquake or tsunami. The main mooring facility, - 7.5 m quay is expected to withstand level 2 earthquakes and the resulting tsunamis.

In this paper, the result of seismic performance surveys and tsunami wave performance surveys on the quay of the pier structure are presented, and countermeasures to satisfy the required performance are discussed. For the seismic performance, dynamic analysis was conducted on the level 2 earthquake by the effective stress analysis FLIP program. On the other hand, for anti-tsunami performance, a tsunami simulation based on nonlinear long wave theory for level 2 tsunami was conducted and stability for the tsunami level was checked. Based on these results, a structure was proposed that can withstand level 2 earthquake motions and tsunami, and is superior in economic efficiency and constructability. Note that the level 2 earthquake ground motion is the earthquake ground motion having the greatest strength from the past to the present and the future at the designated design point. The level 2 tsunami indicates a tsunami occurring about once in 1000 years. Level 2 earthquake motions and tsunamis are applied as external forces to particularly important facilities in the port among port facilities.

Figure 1. Location of Ogasawara islands and map of Chichijima island

Photo 1. Bird’s-eye view of Futami port

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2. Earthquake and tsunami in the Ogasawara Islands

2.1 Countermeasures against tsunami and earthquakes in Japanese harbors

Since the East Japan Pacific Offshore Earthquake Tsunami that occurred in 2011, the Nankai Trough massive earthquake was set up as the largest class earthquake hypothesized in Japan and various disaster prevention measures have been taken against earthquakes and tsunamis.
For port facilities, concepts of level 1 earthquake and level 2 earthquake, level 1 tsunami and level 2 tsunami were introduced.
The level 1 earthquakes are highly likely to occur during the design and service period, based on the relationship between the reproduction period of the earthquake motion and the design service period of the facility at the site where the facility is installed, the approximate reproduction period is 75 years. The level 2 earthquake is the earthquake motion having the largest scale of the assumed earthquake ground motion at the site where the facility is installed.
Level 1 tsunami is a tsunami with a high possibility of occurring during the service period of the facility, and its occurrence frequency is from once in several decades to once in one hundred and fifty years. The protection target against the level 1 tsunami is that the necessary port functions can be used immediately after the disaster. The level 2 tsunami is the largest assumed tsunami at the site where the facility is installed, and its occurrence frequency is once every several hundreds to one thousand years. The protection target is the early restoration of the port function.
In the case of ordinary facilities, measures are taken for level 1 earthquake ground motion and level 1 tsunami, and for more important facilities, level 2 earthquake ground motions, level 2 tsunami measures are taken. Normally, for earthquakes and tsunamis exceeding level 1, it is fundamental to protect human life including implementing evacuation measures. When damage to facilities has a significant impact on economic activity, when damage is seriously affecting human life, and when facilities are important for disaster prevention measures will have to be taken against level 2 external forces.

2.2 Previous tsunami and earthquake in Chichijima islands

It is said that the Ogasawara Islands were discovered by the Japanese in 1593, and that people settled in the Ogasawara Islands around 1820. Since 1826, a number of earthquakes and tsunamis have been recorded in the Ogasawara archipelago.
Yoshinobu TSUJI (2006) [1)]has reported the record of the earthquake tsunamis that hit the Ogasawara archipelago. A list of tsunami and earthquake records is shown in Table 1. No serious damage is reported, but the data show that many tsunamis hit the region and that tsunamis countermeasures are necessary.

Table 1. List of tsunamis which hit the Chichijima Ogasawara islands

The date the tsunami occurred	The place the tsunami occurred	Details	The magnitude of the earthquake	The tsunami height along the Ogasawara coast
1826	Sea near the Ogasawara Islands	This tsunami hit Chichijima after a massive earthquake occurred in Chichijima.	?	6 m
1854	Eastern Sea of Japan	The epicenter off Tokaido was the Pacific side of Japan.	8.4	3 m-5 m
1872	Sea near the Ogasawara Islands	Damaged by flooding. According to local people, the tsunami hit 6 or 7 times	?	3 m

1896	Off Sanriku	22,000 people were killed by the tsunami in the Tohoku Sanriku region of Japan.	7.6	4 m
1923	Kanto southern part	At Futami Port, a tsunami with a height of 90 cm and a period of 30 minutes attacked several times.	7.9	3 m
1933	Off Sanriku	The tsunami arrived 90 minutes after the earthquake occurred, 145 minutes later the maximum tsunami came.	8.1	3 m
1944	East-west coast	Tsunami caused by the Tonankai earthquake.	7.9	3 m
1946	Off the Kii Peninsula Shikoku	Nankai Earthquake Flood under floor at Chichijima	8.1	3 m
1960	Chile South America	Tsunami caused by the Chile earthquake	9.2	4 m

2.3 Necessity of countermeasures against tsunami and earthquakes at ports of the Ogasawara Islands

The Tokyo Metropolitan Port Administration, which manages Futami Port, aims to prevent Chichijima from being isolated even when a level 2 earthquake or a level 2 tsunami occurs, and for this it was aimed to strong then the -7.5 m quay. This is because Chichijima Island is a remote island that is 1,000 km away from Tokyo and has no airport, so the -7.5m quay is the only facility to be utilized for the supply of living necessities, rescuing victims and the evacuation of islanders at the time of a disaster.

3. Study of Level 2 tsunami countermeasure based on tsunami simulation

3.1 Tsunami analysis method in this study

In general, when checking the stability of a structure against a tsunami, the maximum water level at the front of the structure is calculated based on the tsunami simulation and the tsunami wave force is estimated by a reliable tsunami wave force formula.
In the technical standards of port facilities in Japan (2007)(translated in 2009[2)]), for the breakwaters of the gravity structure, Tanimoto's formula is recommended. And for gravity quay and steel sheet pile structure, a method for calculating the tsunami wave force using the water level difference is defined, but there is no clear mention for the pier structure.
Therefore, in this study, in the first stage, a two-dimensional planar tsunami simulation based on nonlinear long wave theory was carried out and the time series of the tsunami level on the front of the target structure was calculated.
In the second stage, the tsunami wave force acting on the pier was directly calculated from the numerical simulation by numerical wave trough tank using the time series of the tsunami level as the input condition.

3.2 Analysis and results based on tsunami simulation

3.2.1 Conditions of tsunami simulation

Numerical simulations of tsunami inundation were conducted using the non-linear shallow water wave theory. Basic equations are as below, and the numerical scheme is the Leap-Frog method.

$$\frac{\partial \eta}{\partial t} + \frac{\partial M}{\partial x} + \frac{\partial N}{\partial y} = 0 \tag{1}$$

$$\frac{\partial M}{\partial t} + \frac{\partial}{\partial x}\left(\frac{M^2}{D}\right) + \frac{\partial}{\partial y}\left(\frac{MN}{D}\right) + gD\frac{\partial \eta}{\partial x} - K_h\left(\frac{\partial^2 M}{\partial x^2} + \frac{\partial^2 M}{\partial y^2}\right) + \gamma_b^2 \frac{M\sqrt{M^2+N^2}}{D^2} = 0 \tag{2}$$

$$\frac{\partial N}{\partial t} + \frac{\partial}{\partial x}\left(\frac{MN}{D}\right) + \frac{\partial}{\partial y}\left(\frac{N^2}{D}\right) + gD\frac{\partial \eta}{\partial y} - K_h\left(\frac{\partial^2 N}{\partial x^2} + \frac{\partial^2 N}{\partial y^2}\right) + \gamma_b^2 \frac{N\sqrt{M^2+N^2}}{D^2} = 0 \tag{3}$$

where,

t:time

x,y; coordinates

η; water surface elevation

M,N; flux in x-direction and y-direction

h; still water depth

D; total depth $(D=h+\eta)$

g; acceleration of gravity

K_h; horizontal diffusion coefficient

γ_b;friction(=gn^2/D^3,n: friction coefficient of Manig)

Calculation method, setting conditions such as calculation range, natural conditions such as wave source and tide level are as shown in the following table 2.

The wave force of the level 2 tsunami acting on the pier was calculated and the stability check of the entire structure was carried out for the tsunami wave force acting on the whole structure of the pier.

The Japanese Cabinet Office published a report on the “Nankai Trough's Great Seismic Model Review Committee (Second Report)" (2012). The report includes an examination of a total of 11 tsunami fault models of the Nankai Trough massive earthquake: 5 "basic examination cases" and 6 "other derivation study cases". Each municipality on the coast of Japan will conduct a tsunami simulation using these tsunami fault models and implement damage estimation. The Tokyo Metropolitan Port and Harbor Bureau, which manages Futami Port, conducted a comparative study of 5 of the 11 cases and adopted Case 5. Case5 fault model is shown in Figure 2. In this case the large slip region and the super large slip region were set off the coast of Shikoku island and Kyushu island.

Table 2. List of Conditions of numerical simulations

Item	The contents	The values
Basic equation	non-linear shallow water wave theory	
The earthquake	Nankai Trough huge earthquake	
Bathymetry (mesh size) (see Figure 3)	Area 1, Area 2,Area 3, Area 4, Area 5, Area 6, Area 7	2,430m,810m,270m,90m,30m 10m, 5m
Calculation time step		0.10s
Calculation run time		12h
Tide level	H.W.L. L.W.L.	D.L.+1.10m D.L.±0.00m

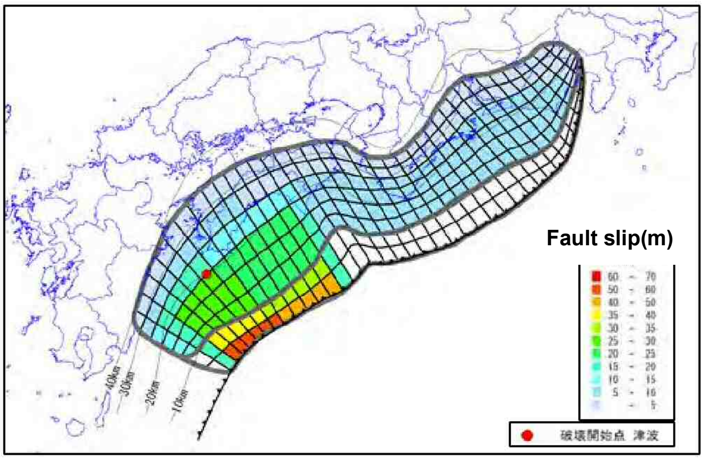


Figure 2. Case5 fault model

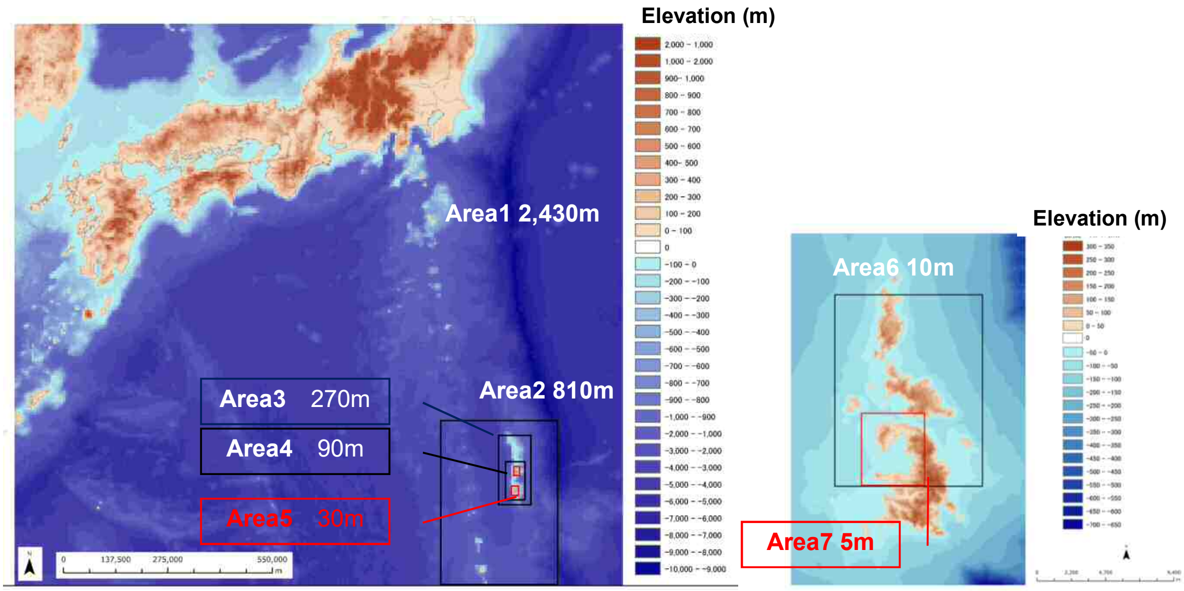


Figure 3. Calculation area（left:2,430m-30m, right:10m-5m）

3.2.2 Result of tsunami simulation

The result of the tsunami simulation, a distribution map of the maximum tsunami height around Chichijima, and a distribution map of the maximum inundation depth around Futami Port, are shown in the following Figure 4. A water level variation of time scale of point "★" is shown in Figure 5. According to the maximum water level distribution chart, it can be seen that the tsunami converges and increases in Futami Bay located on the western side of Chichijima, similarly in the southern bay in the west side and in the northern bay on the east side. The maximum inundation depth is around 5 m in the vicinity of Futami Port and it can be seen that inundation spreads to the mountain side mainly in Futami Port. According to the time series fluctuation of the tsunami water level, the first wave of the tsunami has reached Futami Port in about 1.5 hours after the occurrence of the tsunami, and it is found that the maximum water level is about 6 m at the third wave. Furthermore, although the tsunami of about 2 to 3 m repeatedly reached until 6 hours after the occurrence of the tsunami, the tsunami rapidly declined after 6 hours.

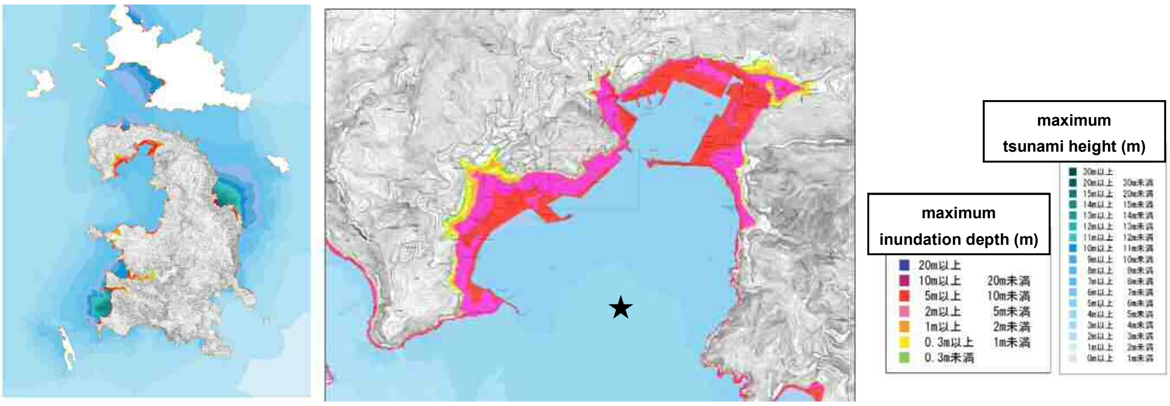


Figure 4. Maximum tsunami height distribution map and maximum inundation depth distribution map
(Left: Maximum tsunami height around Chichijima, Right: Maximum inundation depth map around Futami Port)

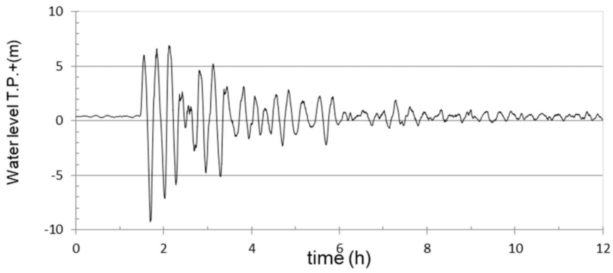


Figure 5.Time series water level variation of tsunami (at point ★ shown in Figure 4)

3.3 Analysis and results based on numerical wave water tank

3.3.1 Conditions of numerical wave water tank

Currently, there is no established method to check the stability of the pier against the wave power of the tsunami. Also, the existing tsunami wave pressure equation cannot be applied.
Therefore, be using numerical wave water tank method for the numerical calculation, the tsunami wave force acting on the pier was directly estimated. Numerical wave water tank (CADMAS - SURF) is a numerical simulation model developed by Coastal Development Institute of Technology in 2001. The simulation model numerically solves the Navier-Stokes equation, which is a fundamental equation of a fluid, and the VOF（Volume of Fluid）method is used for the free surface processing method. This model can quickly and easily carry out numerical simulation of complicated phenomena accompanied by wave, flow, and ground interactions, and it is possible to directly calculate the tsunami wave force, with time series variation of the tsunami water level as an input condition. The seabed topography in the numerical wave motion tank was set taking into account the direction of the tsunami from the front of the -7.5 m quay wall shown in Figure 6. The range of the modeled seabed topography was set from the front of the -7.5 m quay wall to a depth of 25 m, and horizontally 100 m from the front of the -7.5 m quay wall. The water depth change with respect to the horizontal distance from the front of the quay wall is shown in the following figure below.
The lattice spacing of numerical wave water tank was set as ⊿z = 0.20 m for the vertical direction and as ⊿x = 4.00 - 0.2 m for the horizontal direction as the basis. These spacing was obtained considering 1/10 of wave height vertically and 1/100 of wave length horizontally. The ratio of lattice spacing for vertical direction vs horizontal direction lies among 1/10 and 1/5. For decay zones numerical wave

water tank, lattice spacing was expanded to reduce the calculation time.
The input waveform was set so as to reproduce the water level on the front of the quay in the tsunami simulation.
With reference to the water level time series at the front of the quay wall obtained from the tsunami simulation, two ways of water level adjustment for the flow velocity matrix (i.e. the water level variation distribution matrix and the long wave approximation matrix) were tried. As a result, matrix data using the input waveform as a long wave approximate waveform was used.
The outline of the analysis model for pier structures is as follows.

Model 1: An analysis model to express a steel pipe pile as an upright wall was set up in order to calculate an external force acting as a push wave on a steel pipe pile of a jetty.

Model 2: An analysis model with just the upper part was set up in order to calculate the external force acting as lifting pressure on the upper part of the pier. The main calculation conditions are as shown in Table 3.

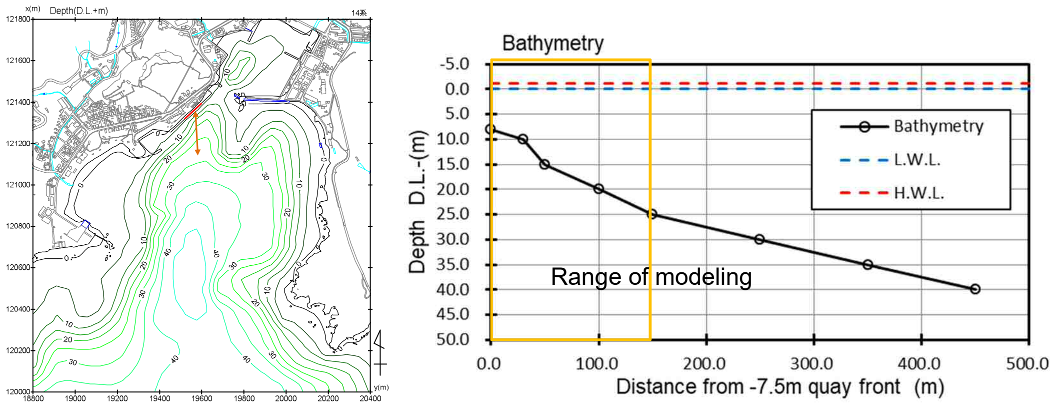
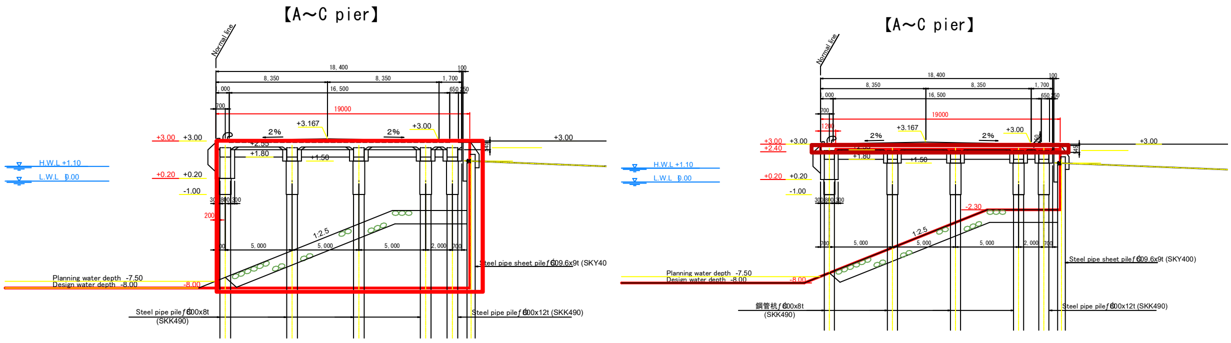


Figure 6. -7.5 m quay offshore topography (left: plane view, right: cross-sectional view)



Model 1 Model 2

Figure 7. Image diagrams of pier structure modeling

Table 3. List of conditions of numerical wave moving tank simulations

Item	Conditions
Input waveform	Waveform adjusted to long wave approximation from the tsunami simulation waveform. The maximum water level and the period are adjusted to 8 m and of about 16 minutes respectively
Calculation time	Period 16 minutes: 1000 s
Time step of calculation	Automatic, initial value 0.001 s, minimum value 1×10^{-8} s
Physical property value	Density 1030kg/m^3, Kinematic viscosity coefficient 1×10^{-6}m^2/s Gravitational acceleration 9.8m/s^2
Difference scheme	VP-DONOR 0.2

3.3.2 Result of calculation

As examples of numerical calculation results by numerical wave water tank (CADMAS - SURF), snapshots of waveform and velocity vector are shown in Figure 8. These figures show the state of overflow of the tsunami at about 450 s at which the water level is at its maximum.
Numerical calculations were performed with numerical wave water tank (CADMAS - SURF), and the wave power of each Model 1 and Model 2 were calculated. Results of the numerical calculation are obtained in result is the wave force per unit area, and the wave force acting to the steel pipe pile is calculated considering the actual diameter.
The horizontal wave force is the force acting in the direction of the pier from the offshore side and is about 65 kN/m^2. The vertical wave force is the force applied from the lower part of the pier to the upper work direction, and the force is 34 kN/m^2 the vertical downward negative pressure. The load distribution map is shown in Figure 9.

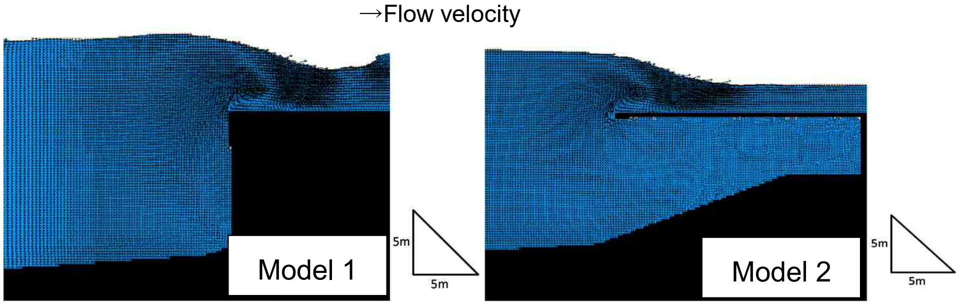


Figure 8. Water level and flow velocity just before the peak of the tsunami at the position of just in front the quay wall (tide: H.W.L.)

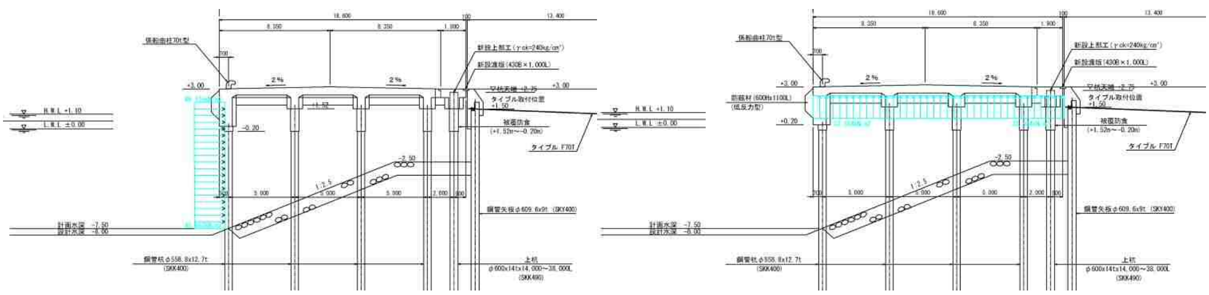


Figure 9. Horizontal direction wave force(left), and vertical wave force wave force(lift pressure) (right)

3.4 Stability check of the entire -7.5m pier against the tsunami

At present, there is no established method for checking the stability of the pier against the wave power of tsunamis. Also, the existing tsunami wave pressure equation cannot be applied here. Therefore, the two-dimensional frame calculation was carried out from the tsunami wave force calculated by the numerical wave water tank described in section 3.3 and the stability check of the pier in case the level 2 tsunami would arrive carried out. The results as shown in Table 4 reveal that the pier will keep stable for the level 2 tsunami.

Table 4. List of Level 2 tsunami check result

The state of action of the load	Direction of facility inspection	Element	Items to be checked	Verification result
The accidental state of the level 2 tsunami push wave	Pier cross section direction	Steel pipe pile 1	Stress (compression)	0.67≦1.000 OK
			Displacement	21.027(mm) OK
			Supporting force (indented)	R=353.925≦ Ra=2905.953（kN）OK
			Pile head moment Md/Mu	0.345≦1.000 OK
The accidental state of the level 2 tsunami lift pressure	Pier cross section direction	Steel pipe pile 3	Stress (tension)	0.053≦1.000 OK
			Displacement	0.442(mm) OK
			Supporting force (withdrawal)	R=146.226≦ Ra=1132.253（kN）OK
			Pile head moment Md/Mu	0.001≦1.000 OK
		Superstructure	Stress (bending)	8.458≦65.126 OK

4. Analysis of level 2 earthquake by dynamic analysis and examination of countermeasure works

4.1 The estimation of earthquake induced liquefaction

The soil structure of the ground where the -7.5m quay wall is installed consists of an embankment soil layer, a clayey soil layer, a gravel soil layer, a sandy soil layer, and a bedrock layer, i.e. the support layer from the surface.
The list of ground constants is shown in Table 5.
To check the response to the earthquake, first the study of liquefaction was conducted.
According to the liquefaction judgment flow shown in “The liquefaction countermeasure handbook”, the judgment is to be conducted in 3 stages: a method based on particle size in the first stage, a method using the equivalent N value and the equivalent acceleration in the second stage, and a method using the result of the repeated triaxial test in the third stage. As a result, most of the targets were judged to liquify (See Figure 10) .
According to the aforementioned determination method, the As layer clearly liquefied.
When the As layer liquefies, it was judged that the pore water pressure raised in the As layer will be propagated to the upper Ac layer, resulting in liquefaction of the Ac layer.
However, since the Ac layer contains as much as 25% clay, the Ac layer can be expected not to liquefy by itself.
As a result, it was decided that liquefaction countermeasures be executed only for the As layer.

Table 5. List of ground constants

Symbol	Soil classification	N value	Underwater weightγ (kN/m^3)	Wet weight γ(kN/m^3)	Adhesive force C(kN/m^2)
E	Embankment soil layer	12	10	18	0
Ac	Viscous soil layer	3	7.2	17.2	-0.69(Z)+8.59
Ag	Gravel soil layer	20	9.8	19.5	0
As	Sandy soil layer	14	8.6	18.2	0
Tb	Bedrock layer	50	10	20	0

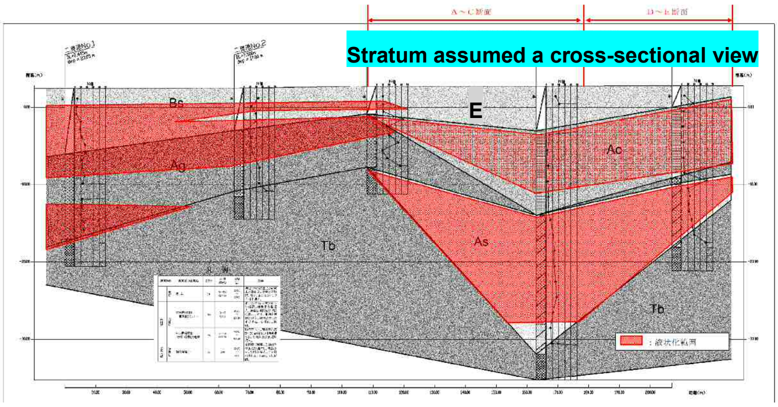


Figure 10. Result of liquefaction layer determination

4.2 Analysis and results by FLIP

4.2.1 Outline of analysis method

Response of the -7.5 m quay against level 2 earthquake motion and an examination of the countermeasures against Level 2 earthquake motion were carried out based on dynamic analysis by FLIP.
Here, FLIP stands for Finite element analysis program of Liquefacqtion Process. The FLIP programs include the constitutive model called cocktail glass model, three-dimensional effective stress analysis with various structural elements, and two-dimensional large deformation/finite strain effective stress analysis. And the program is applicable to the analysis of clayey materials.

4.2.2 Result of the analysis for the current facility

In order to clarify the situation of seismic performance of the current facility, analysis by FLIP was carried out.
The results of the FLIP analysis of are shown in Table 6 and Table 7. The residual displacement amounts in the normal and horizontal direction are less than the minimum allowable value 1,000 mm for both the jetty and the seawall behind the jetty (Table 6). However, regarding the stress of the steel pipe pile, it was revealed that the maximum moment generated reached the total plastic moment and did not satisfy the earthquake resistance performance.
The model diagram, distribution diagrams of residual displacement amounts, excess pore water pressure, and shear strain are shown in Figure 12, which compares with the case with countermeasures.

Table 6. List of residual deformation amount in the horizontal direction

Item checked	Location of check	Residual deformation amount in the normal horizontal direction (mm)	Tolerance of deformation amount (mm)	Judgment result
Pier	crown height	-687.0	1000	OK
	Installation ground on the seabed	-669.8	-	-
sea wall	crown height	-528.4	1000	OK
	Installation ground on the seabed	-672.1	-	-

Table 7. List of result of analysis of stress of steel pipe pile

Elements pile row	Elements	Maximum moment(kN・m)	Total plastic moment(kN・m)	Moment ratio Mp/Md	Judgment result
a	Maximum value	291.2	291.2	1.000	NG
b	Maximum value	290.9	290.9	1.000	NG
c	Maximum value	291.4	291.4	1.000	NG
d	Maximum value	292.9	292.9	1.000	NG
e	Maximum value	481.1	481.1	1.000	NG
Comprehensive evaluation		NG			

4.3 A Study on Countermeasures against Level 2 Ground Motion

4.3.1 Extraction of countermeasure method

The results of the analyses on 4.1 and 4.2 show the necessity of countermeasures for liquefaction. The following items are prerequisite when conducting a study on liquefaction measures at this facility.

- Terms of use: This quay is the only quay on the island for logistics to and from mainland Japan, and the construction works must be carried out while using the facilities. Also, on the front of the -7.5 m quay, the vessel is often moored 3-4 days per week.
- Geographical condition: Since the island is located more than 1,000 km from the mainland of Japan, it takes time and expenses to carry construction materials and machines to the island.
- Soil condition: Basically, a silt layer is deposited in the upper layer and a sand layer is deposited in the lower layer. The composition of the soil layer differs greatly from place to place in the sectional direction of the quay.
- Condition behind the facility: Behind the soil protection steel piles, tie material is installed at a pitch of 2 m in the normal direction of the quay.

Based on the prerequisites described above, the liquefaction countermeasure methods to be studied must result in only small ground deformation, must not affect the existing structure, and must not affect ships' berthing or navigation. 4 candidate methods were proposed and compared. The followings are cited as countermeasure construction methods: method 1: cement based soil improvement method, method 2: chemical liquid injection system ground improvement method, method 3: reinforcement method using a structure, and method 4: drain construction method, are cited as a countermeasure construction method.

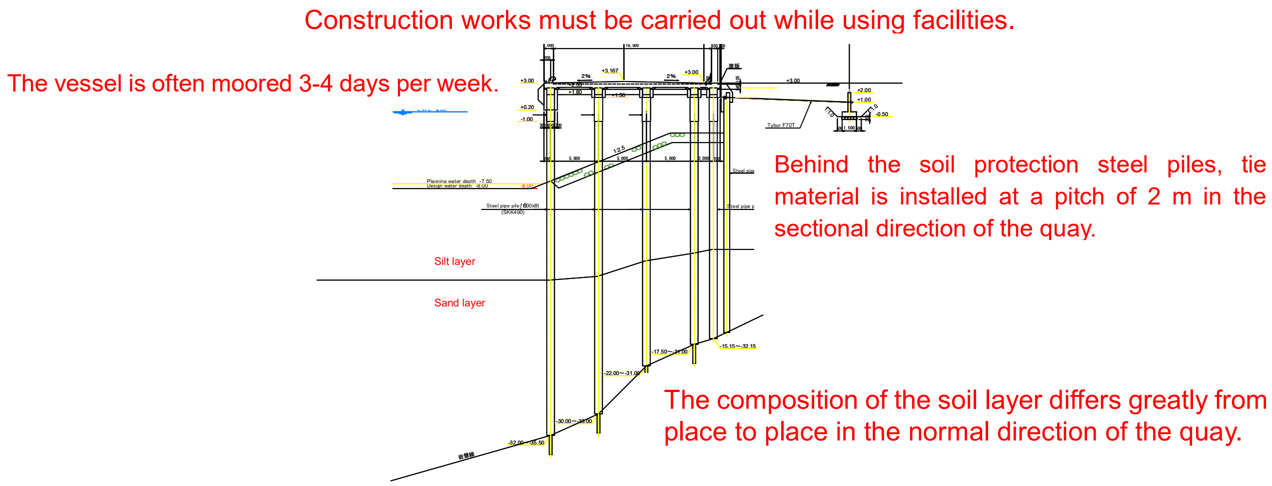
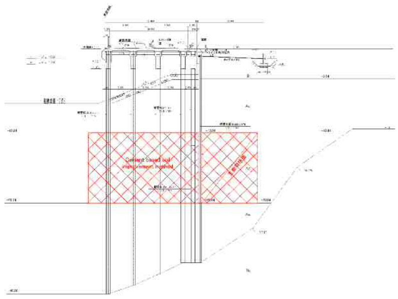
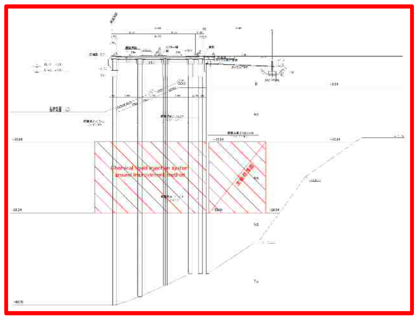
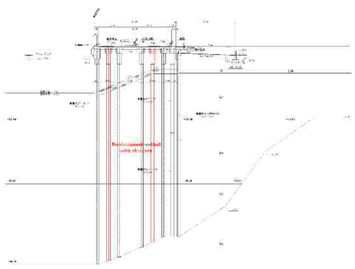
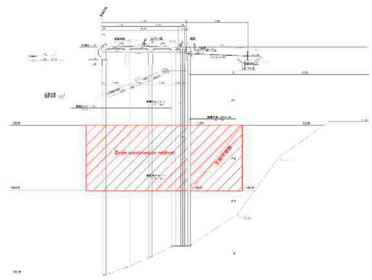


Figure 11. Image sketch of constraints on construction

Table 8. Comparative chart of structural form

Method 1: Cement based soil improvement method	**Method 2: Chemical liquid injection system ground improvement method (Permeation solidification treatment)**
	
Outline of construction method: The ground behind the steel sheet piles is prevented from liquefaction by using the cement-based soil implement system. The stress acting on the soil retaining pile and steel pipe piles is thus reduced and stabilized.	**Outline of construction method:** The ground behind the steel sheet piles is prevented from liquefaction by using a liquid chemical system to solidify and improve the soil, the stress acting on the sheet piles and the steel pipe piles is reduced and stabilized. The method cannot be applied to ground with a high content of fine grains. The application range is fine particle content up to 40% fine particle content.
Evaluation: This method directly prevents liquefaction of the ground, which is a factor of deformation, so that earthquake resistance can be reliably secured. However, due to the occurrence of sludge in the construction work, it is necessary to carry out the primary mud treatment at the site, and further final treatment in the mainland Japan is necessary. Because of the cost for treatment of sludge, this method was judged unadaptable.	**Evaluation:** This method directly prevents liquefaction of the ground, which is a factor of deformation, so that earthquake resistance can be reliably secured. It is possible to execute construction works at night, and the construction works can be executed without affecting service even when constructing on the quay. Furthermore, this method only injects a chemical solution, and no sludge is generated.
Method 3: Reinforcement method using structure	**Method 4: Drain construction method**
	
Outline of construction method: In this method, by placing piles on the ground behind the sheet piles, it is expected to suppress the deformation of the ground and reduce the stress on the sheet piles and the steel pipe piles.	**Outline of construction method:** This method prevents liquefaction by installing drain piles in the ground behind the sheet pile, and reduces the stress on the sheet pile and the steel pipe pile.
Evaluation: It is necessary to excavate the ground behind the sheet piles. Therefore, it is not possible to use the cargo handling yard during the construction period. Also, in order to prevent plastic failure of the steel pipe piles, it is necessary to make the steel pipe piles of the back ground more than φ 900. Therefore, construction becomes expensive.	**Evaluation:** Judging from the grain size distribution of the liquefaction area, this method was judged unadaptable.

4.3.2 Study of countermeasures by FLIP

From the comparative study described in the previous section, the penetration solidification treatment method was selected as a countermeasure construction method. A model for the selected penetration solidification method was made and the analyses of the level 2 earthquake motion by FLIP were coducted. The model diagram and the check result are shown in Figure 12. The residual displacement amount was within the allowable range. The stress degree of the steel pipe piles was also within the allowable range.

Figure 12. Result of dynamic analysis by FLIP

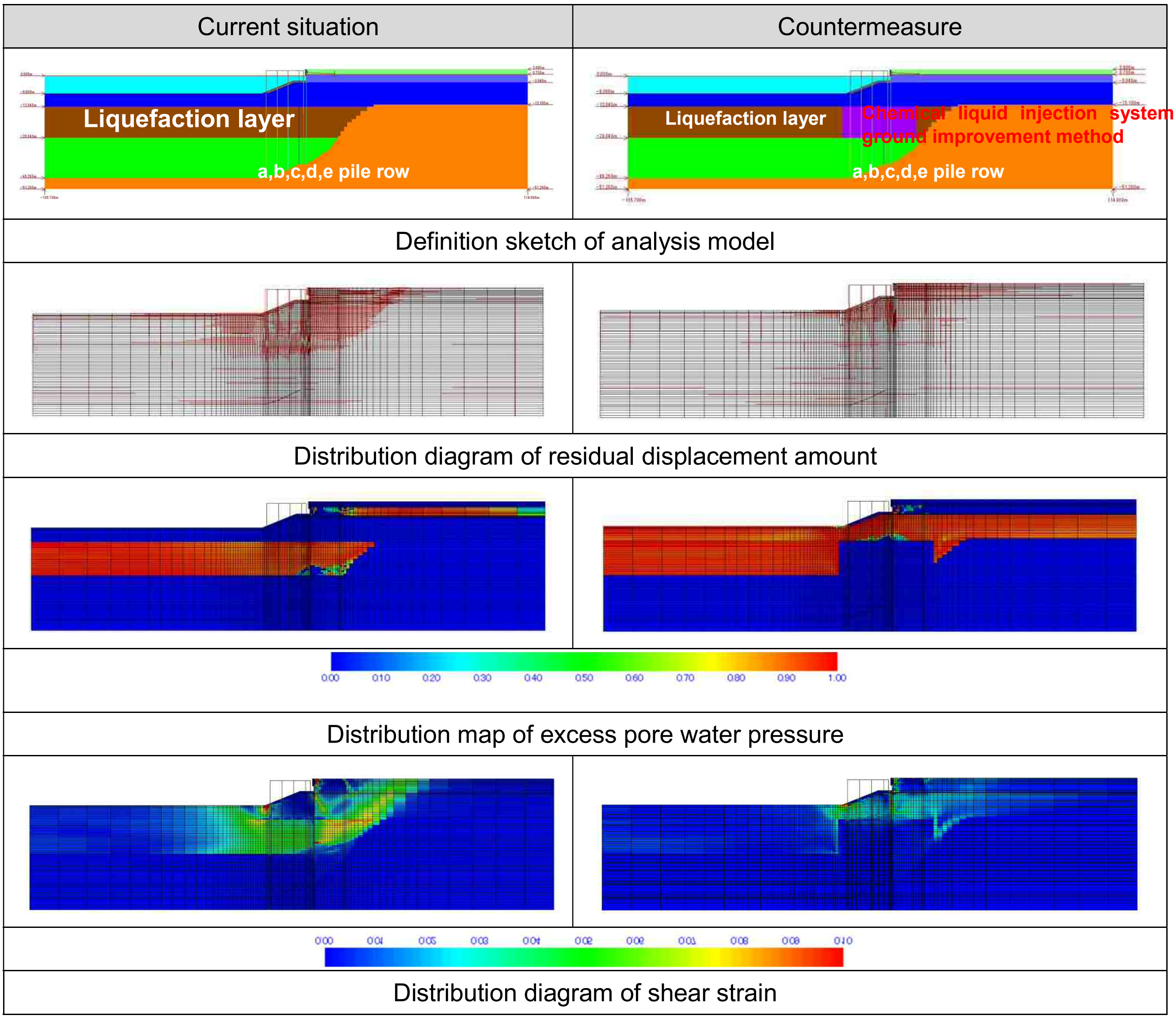


Table 8. Result of inspection of residual displacement amount against level 2 earthquake motion

Element	Position	Current situation	Countermeasure	Tolerance of deformation amount(mm)	Judgment
		Residual deformation amount in the horizontal direction(mm)			
Pier	crown height	-687.0	-172.1	1000	OK
	Installation ground on the seabed	-669.8	-130.7	-	-
Sea wall	crown height	-528.4	-201.2	1000	OK
	Installation ground on the seabed	-672.1	-174.2	-	-

Table 9. List of Result of inspection of stress of steel pipe pile

	Current situation				Countermeasure			
Pile row	Maximum moment (kN・m)	Total plastic moment (kN・m)	Moment ratio Mp/Md	Judgment	Maximum moment (kN・m)	Total plastic moment (kN・m)	Moment ratio Mp/Md	Judgment
a	291.2	291.2	1.000	NG	201.0	294.5	0.682	OK
b	290.9	290.9	1.000	NG	-186.0	294.3	0.631	OK
c	291.4	291.4	1.000	NG	-393.0	393.0	1.000	NG
d	292.9	292.9	1.000	NG	-357.3	394.1	0.906	OK
e	481.1	481.1	1.000	NG	-495.0	529.1	0.935	OK
	Comprehensive evaluation NG				Comprehensive evaluation OK			

4.4 Chemical liquid injection system ground improvement method (Permeation solidification treatment)

The application range of the penetration solidification treatment method is ground having a fine particle content of 40% or less. In the present study, the soil layers to which the penetration solidification treatment method can be applied mainly in the As layer are the B layer, the Ag layer, and the As layer.
The range of construction on the land side and the sea side of the permeation solidification treatment method was determined based on analysis by FLIP. The improvement strength was 70kN/m^2. The improvement rate was set to 100%.

4.4.1 Study on construction method

Improvement work on the quake resistant quays of Futami Port will be under construction in about 10 years. An example of a one-year construction plan is shown below. The engineering type will be Machine Transfer → Drilling Work → Sleep Injection Process → Infiltration Solidification Process Injection → Check Boring → Machine Removal.
Machine loading, marine transport from Tokyo Port to Futami Port is about one thousand kilometers one way.
In the drilling work, 10 holes are drilled in 1 m ^{2} of the reinforced concrete member of the apron part and 100 mm in diameter. At that time, careful attention should be paid to the position of the reinforcing bar of the reinforced concrete member.
The quality of ground improvement is ensured by check boring.

5. Conclusions

The conclusions in this study are as follows.
1) The level 2 tsunami to be designed for is the tsunami of the Nankai Trough massive earthquake. Based on the tsunami simulation for the level 2 tsunami by the tsunami fault model of the Nankai Trough massive earthquake, the maximum water level and the minimum water level at Chichijima Futami Port were calculated. From the results of the water level and the minimum water level, the stability of the accidental condition against the level 2 tsunami was examined, and it was revealed that it possesses sufficient strength.
2) In addition, anti-tsunami resistance performance of the elements of the piers was examined by the two-dimensional numerical wave motion channel, and it became clear that the structure has sufficient resistance.
3) For the accidental state of the level 2 earthquake, dynamic analysis by FLIP was carried out, and the structure which ensures the stability (an allowable displacement in the horizontal direction of within 1 m) was examined. Based on the results, a ground improvement as follows was proposed as an earthquake-resistant reinforcement construction method; a ground improvement plan by the injection solidification method (medicinal fluid penetration solidification processing method) which suppresses liquefaction of the ground by stiffening the ground at the lower part of the facility with a permeable chemical liquid, reducing the stress to the steel material.
4) At present, detailed examination of the countermeasure construction method focusing on the construction plan is being carried out based on the ground improvement plan utilizing the injection solidification method (chemical infiltration solidification processing method).

References:

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