Study on Seismic Countermeasures by Steel Pile Diaphragm Wall in Coastal Levee

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ABSTRACT

After the 2011 off the Pacific coast of Tohoku Earthquake, for existing coastal levees, performance-based seismic designs have been carried out to counter this level II earthquake with a Tsunami, and earthquake resistance measures are being implemented in preparation for The Nankai Trough earthquake which is expected to occur at 70% probability in the next 30 years. In this research, as earthquake countermeasures for the coastal levee against liquefaction of the foundation ground during the earthquake by the press-in type steel pile, setting of the required performance at the time of earthquake, examination of performance-based seismic design and countermeasure construction study were carried out. First, we estimated the extent of damage by the numerical analysis method that the levees suffered with inertia force and ground liquefaction accompanying the earthquake, then we found out the necessity for earthquake resistance measures with the levees according to the performance requirements. In this paper, we mainly describe difference in calculated behavior of the levee due to analytical modeling methods about components of the coastal levee such as sands embankment, wave-dissipating block and concrete revetment. In addition, we show effects of press-in type steel pile as earthquake countermeasures by the numerical analysis method.

Key words: *Coastal levee, seismic countermeasure, numerical method, liquefaction, press-in type steel pile*

1. Outline of the project

1.1. Background and objectives of the project

In recent years, in preparation for a huge earthquake by the Nankai Trough after the Tohoku Region Pacific Offshore Earthquake, seismic performance checks against level 2 earthquake motions and seismic countermeasures accompanying the check result are implemented in the existing coastal protection. In this research, in setting seismic performance countermeasures according to the seismic performance evaluation of the coastal levees and the seismic countermeasure accompanying the inspection result, considering the structure characteristics of the coastal levees, the surrounding environment including neighboring main roads, press-in type steel pile is shown as an example of the earthquake countermeasure method, the concept of the performance regulation against the seismic embankment earthquake, the concept of the seismic performance check, and the method of modeling the existing structure and countermeasure work in the analysis will be described.

1.2. Structure

On the coastal levee, the nourished beach in the front of the revetment is equipped with wave dissipating concrete blocks. In addition, the back side has main roads which are important in the living environment of the surrounding residents, and the other side is the residential area. **Fig. 1** shows the target section plan. The structure of the coastal levee is a special breakwater of the parapet type as shown in **Fig. 2**.

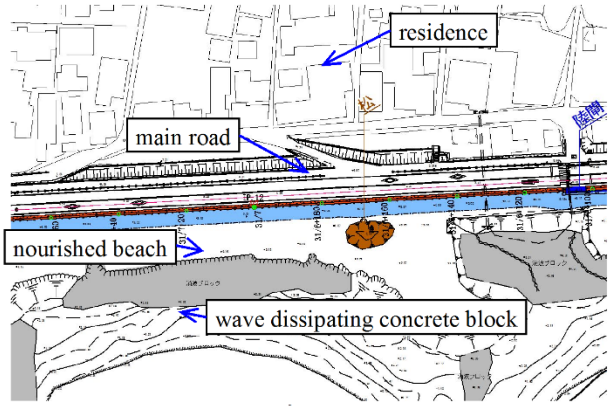

Fig. 1 Section plane

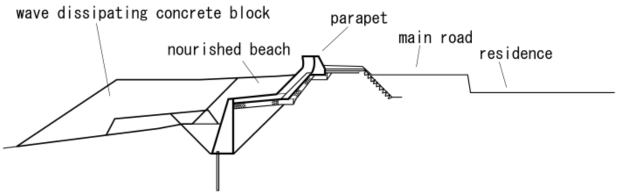

Fig. 2 Cross section

2. Concept of countermeasures against earthquake tsunami in coastal levees

2.1. Required performance

In the Nankai Trough, the Philippine Sea plate gets under the plate on the continental side, and the edge of the plate on the continental side is drawn. For this reason, the strain gradually accumulates, reaching the limit and trying to return to the original, then a trench-type huge earthquake accompanied by a tsunami occurs. The Nankai Trough Earthquake occurred at intervals of about once every 100 years, more than 70 years have passed since the last occurrence, and it is expected to occur with a probability of about 70% over the next 30 years. The Cabinet Office Central Disaster Prevention Council presented the idea of a new tsunami countermeasure on September 28, 2011, following the massive tsunami caused by the Great East Japan Earthquake that occurred on March 11, 2011. There are basically two levels assumed for building future tsunami countermeasures. First of all, the "largest class tsunami" that causes maximam damage, although its occurrence frequency is extremely low. This corresponds to the tsunami in the Great East Japan Earthquake. On the other hand, the occurrence frequency is higher than that of "the largest class tsunami", and the earthquake tsunami causing great damage, although the tsunami height is low, is called the planned tsunami. This tsunami occurs at frequencies of decades to hundreds of years.

In this study, we aim to prevent tsunami damage to coastal levees, targeting the planned tsunami by the Nankai Trough. From the viewpoint of protecting coastal levees from damage of the tsunami, the following two points were set as the performance required of the coastal levees after the earthquake. ① Tsunami does not overflow, ② Coastal levees maintain soundness after the earthquake.

2.2. Setting performance regulations for coast levees

In order to satisfy the required performance set in the previous section, the following performance was specified.

① How to confirm that the tsunami does not overflow

Confirm that the coastal levees height after the earthquake is higher than the design tsunami water level. The amount of settlement due to the earthquake is the sum of settlmetnt due to liquefaction and broad ground settlement. Evaluation of settlement by liquefaction shall be the average of three settlement amounts (δy1, δy2, δy3).

Here, the allowable liquefaction subsidence amount is 0.5 m because the current levee height is T. P. +10.5 m, the design tsunami water is level T.P + 8.0 m, and the broad ground settlement is -2.0 m.

② Confirmation that the coast levee is sound

Confirmation by horizontal displacement (ensuring continuous soundness of cut-off wall)

Maximum value of δX1 or δX2 ≤ 0.50 m

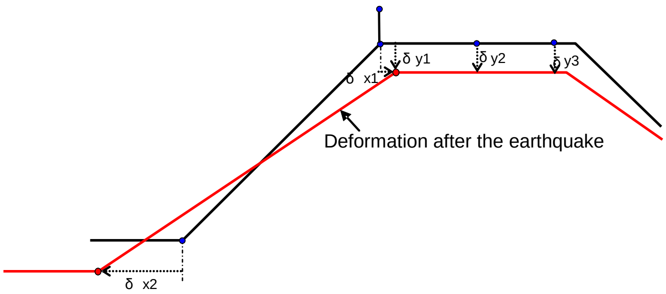

Fig. 3 Concept of performance regulation

3. Performance check of coast levees

As a performance check of the coastal levees, we show the results of liquefaction determination of foundation ground, simple evaluation of levee settlement amount and detailed evaluation of levee settlement amount.

3.1. Analysis conditions

The ground model to be studied is shown in **Fig. 4**. At this position, an alluvium layer (Ags layer, Ag1-m layer, As layer) that becomes a liquid layer is deposited just under the embankment.

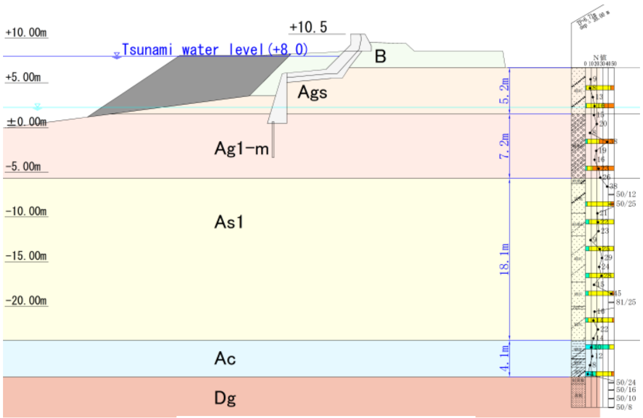

Fig. 4 Ground model

The parameters set from the soil test are shown in **Table 1**, **Table 2**. Also, as an indication of liquefaction, an average N value, RL_{20} is shown. RL_{20} is the liquefaction intensity ratio by cyclic triaxial test, it is the repetitive stress amplitude ratio when repetition or number of times is 20 and the amplitude of axial strain is 5 to 6%. As the value of RL_{20} is smaller, liquefaction tends to occur. N value converted to effective soil pressure 100 kN/m^2 is called converted N value, N_1. The relationship that RL_{20} is about 0.2 when N_1 is 10, and RL_{20} is about 0.4 when N_1 is 20 are shown in Specification for Highway bridge Part V seismic design. So, it can be said that the Ags layer and the Ag1 - m layer are small in RL_{20} and easily liable to be liquefied.

The earthquake ground motion to be considered was a seismic waveform assumed from the Central Disaster Prevention Council in the Tonankai / Nankai earthquake published in 2003. The maximum acceleration was 461 gal, the duration was 120 seconds, the top surface of the base was the input position. The input ground motion waveform is shown in **Fig 5**.

Table 1. Analytical parameters of the ground part1

soil layer	N-value	RL_{20}	wet density ρ (t/m^3)	porosity n	velocity of S-wave Vs (m/s)	Young's modulus E (kN/m^2)	standard-initial shear modulus Gma (kN/m^2)	Poisson's ratio v
B	12		1.9	0.45	230	–	1.63E+05	0.333
Ags	12	0.21	1.9	0.45	230	–	1.63E+05	0.333
Ag1-m	21	0.25	2.0	0.45	230	–	1.04E+05	0.333
As1	28	0.28	1.9	0.45	260	–	9.39E+04	0.333
Ac	9		1.7	0.55	290	–	8.88E+04	0.333
block	–		1.9	–	–	3.36E+04	–	–
parapet	–		2.3	–	–	2.35E+07	–	0.167
Dg	–		2.2	–	640	–	–	–

Table 2. Analytical parameters of the ground part2

soil layer	angle of shear resistance φf (°)	cohesion c (kN/m^2)	hysteresis damping hmax	parameters of liquefaction					
				φp (°)	W1	p1	p2	c1	S1
B	41	0	0.300	–	–	–	–	–	–
Ags	41	0	0.300	28.0	2.500	1.200	0.700	1.900	0.005
Ag1-m	41	0	0.300	28.0	3.500	0.700	0.700	2.000	0.005
As1	40	0	0.300	28.0	8.500	1.200	0.900	1.900	0.005
Ac	30	0	0.200	–	–	–	–	–	–
block	–	–	–	–	–	–	–	–	–
parapet	–	–	–	–	–	–	–	–	–
Dg	–	–	–	–	–	–	–	–	–

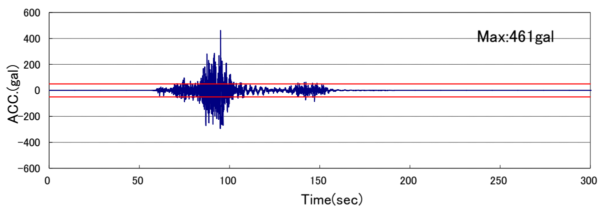

Fig. 5 Input ground motion on the base surface

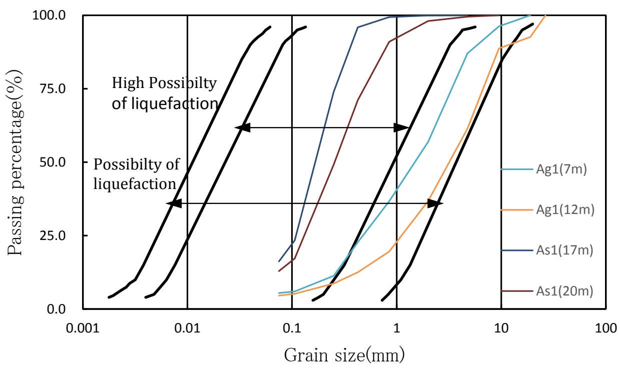

Fig. 6 Liquefaction determination by grain size

3.2. Liquefaction determination

(1) Determination by Grain size distribution of soil

If the range of the particle size curve is within the range of "possibility of liquefaction", it is judged to liquefy. **Fig. 6** shows the particle size accumulation curves of Ag1 layer and As1 layer. From this, it is judged that there is a possibility of liquefaction occurring in these layers.

(2) Determination by N-value

As judged by grain size, it was judged that "possibility of liquefaction", so liquefaction was judged by using N value by boring and earthquake acceleration. **Table 3** shows the liquefaction determination result.

Table 3. Liquefaction determination by N-value

―― : water level

Layer	depth (m)	wet unit weight γ (kN/m³)	N-Value	fine fraction content FC(%)	average grain daiameter D50(mm)	Dynamic shear strength ratio R	Seismic shear stress ratio L	Resistivity against liquefaction F_L	
Ags	1.3	19.0	3						
	2.3	19.0	5						
	3.3	19.0	7						
	4.3	19.0	6						
	5.3	20.0	16						
	6.3	20.0	18	3.4	1.80	0.272	0.239	1.14	0.83
	7.3	20.0	25	9.2	1.80	0.325	0.275	1.18	
	8.3	20.0	29	9.2	1.80	0.370	0.299	1.24	
	9.3	20.0	15	3.7	0.81	0.231	0.313	0.74	
	10.3	20.0	13	3.7	0.81	0.210	0.323	0.65	
	11.3	20.0	19	3.7	0.81	0.249	0.329	0.76	
	12.3	20.0	9	2.9	1.50	0.168	0.333	0.50	
	13.3	18.5	16	5.5	0.78	0.219	0.334	0.66	
	14.3	18.5	16	5.5	0.78	0.215	0.335	0.64	
Ag1m-u	15.3	18.5	10	2.1	3.20	0.160	0.333	0.48	0.59
	16.3	18.5	22	2.1	3.20	0.233	0.330	0.71	
	17.3	18.5	21	3.7	12.00			–	
	18.3	18.5	21	3.7	12.00			–	
	19.3	18.5	12	3.7	12.00			–	
Ag1m-l	20.3	18.5	26	4.2	4.20	0.231	0.315	0.73	0.81
	21.3	18.5	35	5.9	2.30	0.278	0.310	0.90	
	22.3	18.5	29	5.9	2.30	0.249	0.305	0.81	
	23.3	18.5	30	7.2	3.30	0.242	0.300	0.81	
	24.3	18.5	29	7.2	3.30	0.234	0.295	0.79	
As1	25.3	18.5	25	28.4	0.13	0.270	0.290	0.93	0.96
	26.3	18.5	21	28.4	0.13	0.247	0.285	0.86	
	27.3	18.5	23	33.8	0.11	0.266	0.280	0.95	
	28.3	18.5	23	33.8	0.11	0.263	0.275	0.96	
	29.3	18.5	29	33.8	0.11	0.291	0.270	1.08	
	30.3	18.5	19	32.1	0.11	0.233	0.264	0.88	
	31.3	17.0	23	32.1	0.11	0.252	0.259	0.97	
	32.3	17.0	22	32.1	0.11	0.245	0.253	0.97	
	33.3	17.0	23	15.0	0.15	0.209	0.247	0.85	
	34.3	17.0	22	15.0	0.15	0.202	0.241	0.84	
	35.3	17.0	33	15.0	0.15	0.244	0.235	1.04	
	36.3	17.0	14	15.0	0.15	0.160	0.229	0.70	
	37.3	17.0	24	27.3	0.14	0.233	0.223	1.05	
	38.3	17.0	19	27.3	0.14	0.208	0.217	0.96	
	39.3	17.0	24	38.7	0.10	0.252	0.211	1.20	
	40.3	17.0	18	38.7	0.10	0.221	0.204	1.08	

If the F_L value is less than 1, liquefaction occurs, and if it is small, the degree of liquefaction is intense. F_L value is the resistivity to liquefaction and is obtained by **Eq. (1)**.

$$F_L = R/L \tag{1}$$

F_L: the resistivity to liquefaction

R: dynamic shear strength ratio obtained by **Eq. (2)**

L: the shear strong stress ratio at the time of earthquake obtained by equation (3)

$$R = c_w R_L \tag{2}$$

$$\left.\begin{array}{l} \text{small and medium scale earthquakes and trench type large earthquakes} \\ \quad c_w = 1.0 \\ \\ \text{large scale earthquake with active fault} \\ \quad c_w = \begin{cases} 1.0 & (R_L \le 0.1) \\ 3.3R_L + 0.67 & (0.1 < R_L \le 0.4) \\ 1.0 & (0.4 < R_L) \end{cases} \end{array}\right\} \tag{2.1}$$

$$\left.\begin{array}{l} R_L = 0.082\sqrt{0.85N_a + 2.1)/1.7} \\ \qquad\qquad\qquad\qquad (N_a < 14) \\ R_L = 0.082\sqrt{\dfrac{N_a}{1.7}} + 1.6 \times 10^{-6} \cdot (N_a - 14)^{4.5} \\ \qquad\qquad\qquad\qquad (14 \le N_a) \end{array}\right\} \tag{2.2}$$

C_w: Correction coefficient by earthquake ground motion characteristics

R_L: liquefaction intensity ratio by cyclic triaxial test

N_a: Corrected N value in consideration of influence of granularity

$$L = r_d k_{hgL} \sigma_v / \sigma_{v\prime} \tag{3}$$

$$r_d = 1.0 - 0.0015x \tag{3.1}$$

r_d: Reduction coefficient in the depth direction of the shear stress ratio at the time of earthquake

k_{hgL}: Standard value of design horizontal seismic intensity used for judgment of liquefaction (ground surface: Approximately 0.4～0.8)

$\sigma_v / \sigma_{v\prime}$: The value obtained by dividing the total overpressureof the depth of x from the ground surface by the effective loading pressure

From this, it is judged that the Ags layer, the Ag1m layer, the As layer are liquefied and the Ag- u layer which is the upper layer in the Ag layer is considerably liquefied.

3.3. Simple evaluation of levee subsidence amount

In order to predict the amount of deformation of the coastal Levees at the time of the earthquake with high accuracy, it is necessary to carry out highly accurate seismic diagnosis for each individual facility at a great cost and time. Therefore, there is a method of executing simulation under various conditions in advance, and performing seismic diagnosis by making the calculation result of the deformation amount into a database. Here, by comparing the condition of the facility with the database, the deformation amount of the coastal levees at the time of the earthquake is calculated. This method is called a chart type seismic diagnosis.

As a condition for the simple evaluation, the ground height, liquefaction layer thickness, N value, revetment gradient, earthquake motion (speed PSI value of the time history waveform shown in **Fig. 5**) were set using the simplified evaluation model shown in **Fig. 7**. As a result of the calculation, subsidence due to liquefaction of 0.8 m occurred at the top of the parapet. This resulted in exceeding the permissible liquefaction sinking amount of 0.5 m, so it is carried out by the time history response analysis.

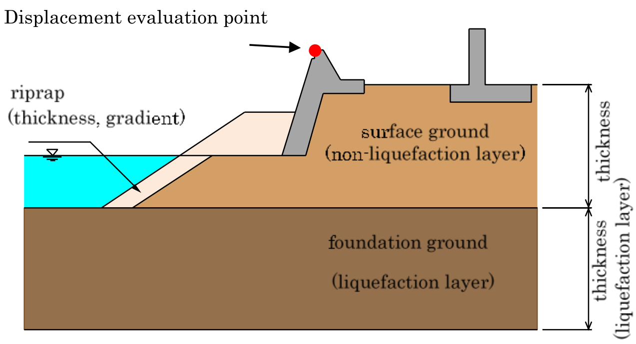

Fig. 7 Model in Simple Evaluation

3.4. Detailed evaluation of levee subsidence amount

Apply the time history seismic response analysis method as detailed evaluation. Because the foundation ground structure is liquefied, we evaluate the deformation of the coastal levees using the effective stress method (analysis code: FLIP).

As a preliminary analysis, **Fig. 8** shows the result of modeling the wave dissipating concrete block with linear elements. Focusing on the deformation mode, the coastal levees are deformed by being pulled by the horizontal displacement of the block. Actually, the block and the ground are discontinuous, and such deformation is unthinkable. For seismic performance, we focus on the horizontal / vertical displacement of the coastal levees, so we do not consider the deformation of the block and consider it as the node mass.

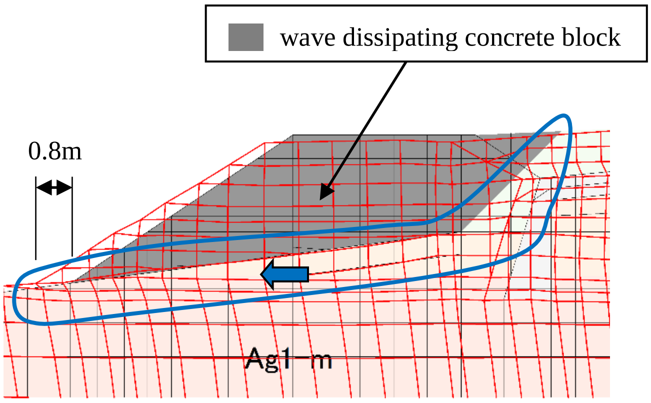

Fig. 8 Deformation of the block as liner element

On the modeling of concrete and soil of coastal levees, the authors have been studying dynamic interaction between soil elements and concrete elements. A joint element is provided at the boundary between concrete and soil to make a model that appropriately expresses dynamic interaction. **Fig. 9** shows the analysis model of current levee.

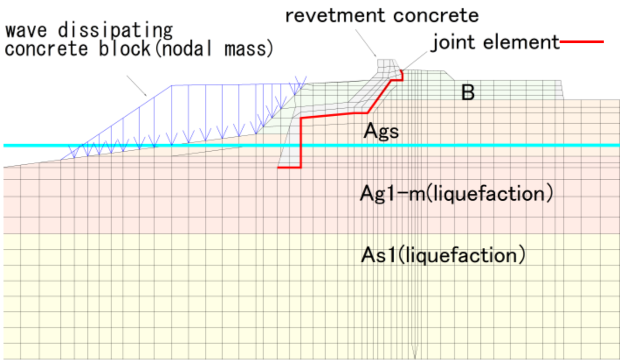

Fig. 9 Analysis model of the current levee

The analysis results of the current coast levees are shown in **Fig. 10.** Focusing on the deformation around the revetment, the liquefaction layer becomes fluidized and the embankment on the back of the revetment settles. On the other hand, the nourished beach in the front of the revetment works in a direction to suppress the flow of seawall to the sea side. As a result, the parapet revetment moves with rotation, the horizontal displacement is 0.71 m (tolerance is 0.5 m) and the vertical displacement is 0.7 m (tolerance is 0.5 m). The current levee does not satisfy the performance after the earthquake.

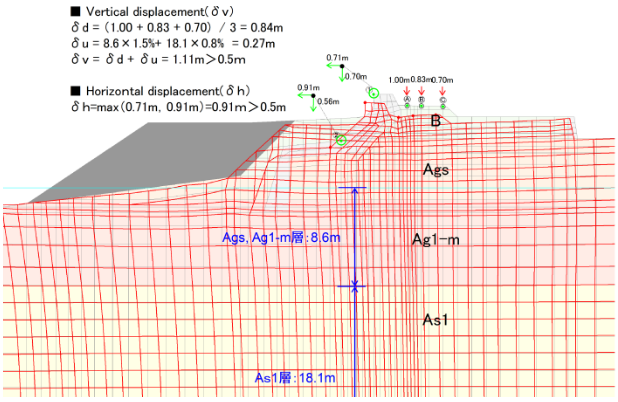

Fig. 10 Analysis results of the current coast levees

4. Study of countermeasures

4.1. Primary selection of countermeasures

As countermeasures for suppressing ground deformation caused by liquefaction, it is conceivable to (1) inhibit liquefaction by ground improvement, (2) inhibit deformation by steel piles, (3) raise the embankment, (4) adding wave dissipating concreter blocks as suppressing deformation of coastal levees.

Suppression of liquefaction caused by ground improvement is not improved from the viewpoint of economic efficiency and construction feasibility because it will be improved immediately under the embankment where the influence on the prefectural road traffic function is great. When adopting leve raising, the necessary raising height is 20 cm, but it is difficult to secure the soundness (horizontal displacement) of the coast levee. For this reason, we selected (2) suppression of deformation by steel pilels and (4) adding wave dissipating concreter blocks as countermeasures.

4.2. Study of countermeasures by adding blocks

Fig. 11 shows an analytical model to which blocks are added. These blocks are also taken into account by the node mass.

The analysis result is shown in **Fig.12**. The horizontal displacement is 0.92 m, and the vertical displacement is 0.84 m. The deformation is the same as the current section, and the deformation suppression effect by the wave-blocking block is not obtained.

4.3. Study of countermeasures by steel piles

(1)Place steel pile in front of the levees

Fig. 13 shows an analytical model where steel piles are placed in front of the levees. The steel pile has a diameter of 800 mm and a length of 16 m. The steel pile penetrates into the As layer which is hardly liquefied as compared with the Ag layer.

The analysis result is shown in **Fig. 14**. The horizontal displacement is 0.75 m, and the vertical displacement is 0.42 m. Vertical displacement decreases and it is less than 0.5 m which is the allowable value, but horizontal displacement is larger than 0.5 m, and seismic performance is not satisfied.

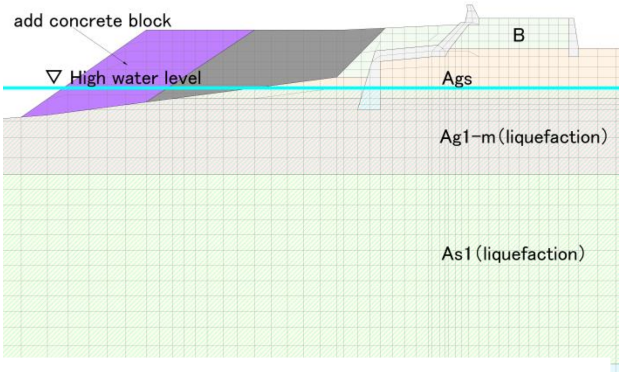

Fig. 11 Analysis model of adding blocks

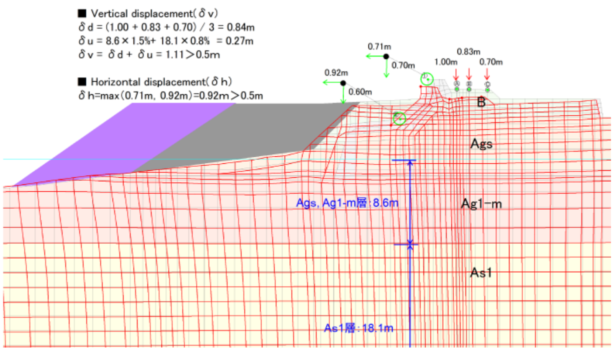

Fig. 12 Analysis results of adding blocks

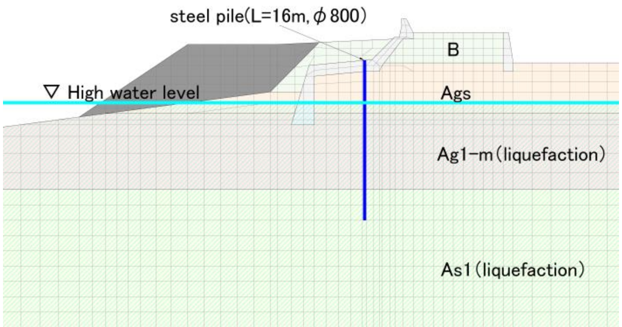

Fig. 13 Analysis model of steel pile in front of the levees

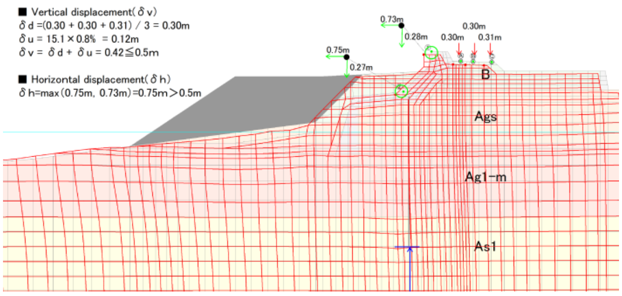

Fig. 14 Analysis results of steel pile in front of the levees

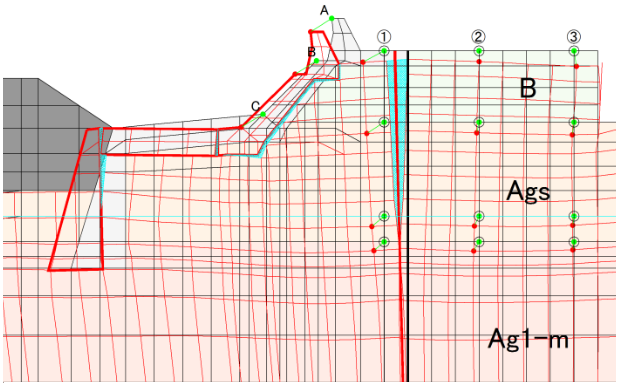

Fig. 15 Analysis results of steel pile at levee crown disconnect with parapet

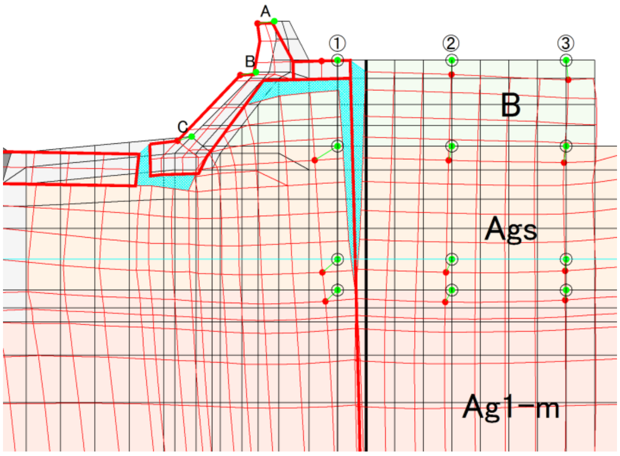

Fig. 16 Analysis results of steel pile at levee crown connect with parapet

(2)Place steel pile at the levee crown

As a preliminary analysis, a steel pile was placed behind a concrete parapet. The steel pile is not connected with the parapet. As shown in **Fig. 15** , it was found that the ground on the front side of the steel pipe greatly moved in the horizontal direction.

Next, the analysis results of a steel pile at levee crown connect with parapet are shown in **Fig. 16**. Vertical displacement and horizontal displacement were suppressed, but since cavities occurred under the revetment, it is considered that the revetment will actually be destroyed.

From the above, it is difficult to expect the function of the existing parapet after the earthquake. Therefore, we decided to set up a new parapet on the steel pipe as shown in the **Fig. 17**.

The analysis result is shown in **Fig. 18**. The horizontal displacement is 0.49 m, and the vertical displacement is 0.17 m. Both of them satisfied the allowable displacement, which resulted in being able to prevent the damage caused by the tsunami.

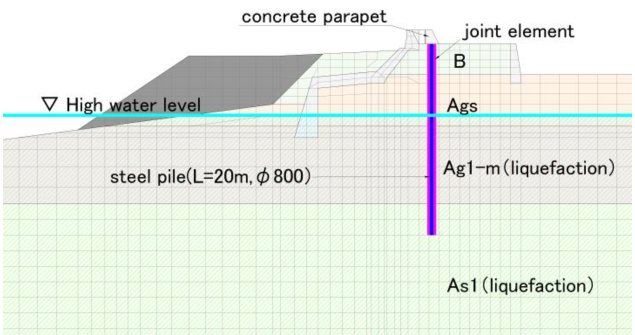

Fig. 17 Analysis model of steel pile with parapet

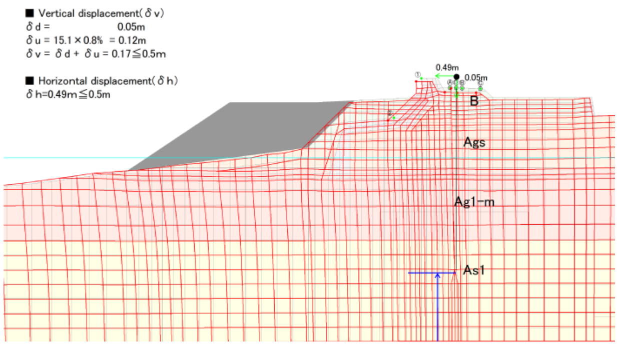

Fig. 18 Analysis results of steel pile with parapet

5. Concluding remarks

We examined tsunami countermeasures against the coastal levees which are restricted by selection of countermeasures because there are arterial roads and houses. As a result, the method of placing the steel pile at the top of the levees with parapet was the only way to satisfy the performance regulations. The construction of this steel pile was planned to be carried out by the gyropress method in order to secure the service of the road.

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