Earthquake and Tsunami Disaster Preventive Measures for Sea Embankment at Usa Fishing Port, Kochi Prefecture

- Application Examples of Press-In Method to Steel Pipe Pile Installation -

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ABSTRACT

This paper presents a study on coastal disaster preventive measures for an earthquake and tsunami strike on the sea embankment installed on the shore of Usa Fishing Port, Kochi Prefecture, Japan. The facility is a slope type structure with a prefectural road constructed on its crown. Since there are private houses behind, consideration for noise and vibration impact on the surrounding areas is hence a crucial point during the work. Results of past studies reveal that the existing embankment does not partly meet crown height requirement against a tsunami by a possible concurrent strike of Tonankai and Nankai earthquakes. We extracted the following three methods as the disaster preventive measures considering impact on roads, and vibration and noise emissions: “Double sheet pile installation”, “Steel pipe pile installation” and “Counterweight fill (this last serves also for soil improvement)” and qualitatively compared them. Evaluation of the three options resulted in the selection of the steel pipe pile method for its advantages in site applicability, workability and economic advantages; optimum structural dimensions were established accordingly. Besides, “Press-in Method” for installing steel pipe piles was successfully adopted for its little environmental impact (noise and vibration) and easiness to work in a restricted space.

Key words: *coastal disaster preventive measures, sea embankment, steel pipe pile installation method, Press-in Method*

1. Outline of the project

1.1. Place

The targeted location for examination is a sea embankment installed on the Nii seashore that faces Tosa Bay, Kochi Prefecture and situated along the Black current line lying to the east of Hagimisaki of Tosa city.

Fig. 1 shows the target facility and its surrounding area.

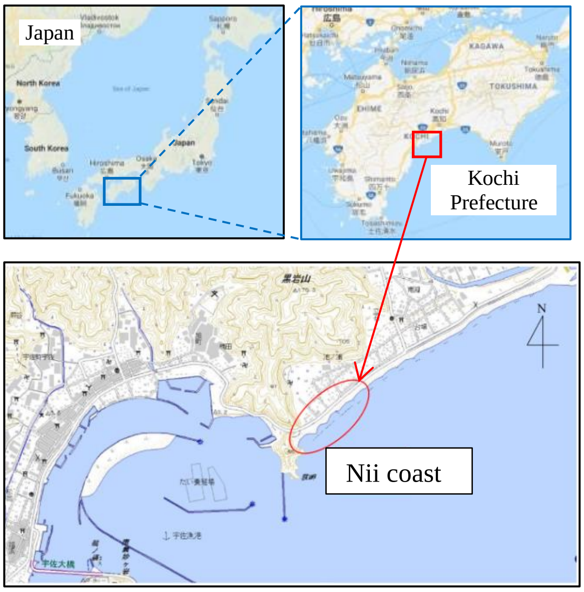


Fig. 1 Targeted location for examination

1.2. Background and objectives of the project

Learning from the Great East Japan Earthquake that occurred on March 11, 2011, Kochi Prefecture has been strengthening protections against earthquake and tsunami in preparation for a Nankai Trough earthquake anticipated to break out in the near future. Nankai Trough has had repeated big earthquakes at intervals of some 100 to 150 years. Having passed more than 70 years from the last earthquake, the outbreak probability of over Mw7.0 within the next 30 years is supposed to be “Around 70 % ”.

In the targeted Nii area of the coast of Usa Fishing Port, amidst the progress of development and construction of residential areas located in the coastal area, and in view of increasing damage by high-tide, embankment has been constructed by around 1963 in a manner to utilize the limited land areas and to bank the waterfront areas in front of the private houses. The original embankment has since been progressively upgraded to date in order to ensure the safety of life and preserve the property of people living in the hinterland against the possible strike of a megathrust and derived tsunami. This report presents a study on the earthquake and tsunami disaster preventive measures taken for the existing sea embankment installed on the coast of Nii area. Completed in 2014, this facility remains to this day.

2. Structural type and piling method

2.1. Site condition

The cross section of the initially constructed embankment in the target area is shown in **Fig. 2**. The facility is a slope type structure having a crown height of T.P. + 10.5 m. There is a prefectural road behind its superstructure, which makes it necessary to select a working method that considers that there will be no adverse impact on the road shoulder width of 0.5 m. In addition, private houses are built close behind the embankment, which requires selection of a work method that emits little vibration and noise annoyance.

Photo 1 section-wise statuses of the superstructure of the existing embankment and its hinterland.

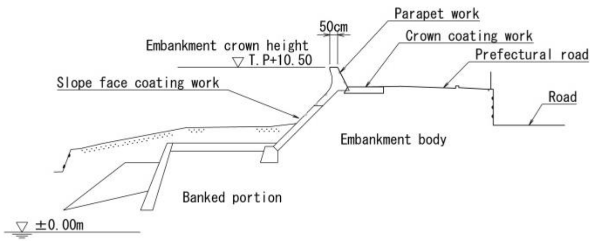


Fig. 2 Typical cross section of the initially constructed embankment

Superstructure section Hinterland section

Photo 1. Status of the initially constructed embankment

2.2. Design condition

(1) Elevation of the embankment after improvement :

T.P. + 10.5 m

(2) Tidal level :

Planned high tide : T.P. + 2.20 m

High water : T.P. + 0.92 m

Low water : T.P. - 0.97 m

(3) Extension after improvement : 565 m

(4) Design seismic coefficient : k_h

Design seismic coefficient shall be established by the thickness of alluvium and diluvium.

$k_h = 0.16$

(for 25 m or more of the above layers' thickness)

$k_h = 0.13$

(for less than 25 m of the above layers' thickness)

(5) Target earthquake ground motions :

A Level-2 earthquake - our target design earthquake - shall be the modeled Two Continuous Earthquakes (Tonankai earthquake coupled with Nankai earthquake: Mw8.6) defined as such by Central Disaster Prevention Council in 2003. **Fig. 3** shows the acceleration time history of input seismic waveform. The maximum acceleration of the earthquake motions is 404.99 Gal. The Level-2 earthquake is meant to be a maximal earthquake that can be considered for the targeted location.

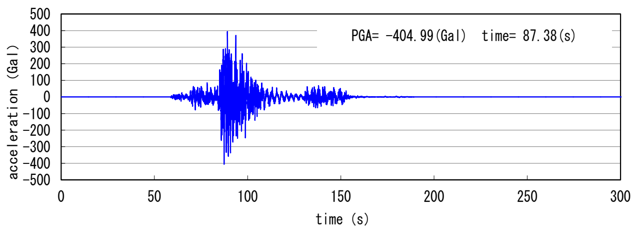


Fig. 3 Acceleration time history

6) Design tsunami water level : T.P. + 8.0 m

7) Crustal movement :

A crustal movement of -2.03 m is assumed to result from the modeled earthquake described in item (5) above at the sea embankment in Nii area.

2.3. Ground condition

Soil survey consisting of the five boring sites of Nos.SP60, SP200, SP320, SP400 and SP500 was conducted along the coastline of the targeted location for examination, the location of which is shown in **Fig. 4**.

The profile of soil layers is shown in **Fig. 5**. The emergent depth of the mudstone layer that plays the role of support layer tends to be increasingly greater in the eastward direction of the coastline. At the borehole SP60, soil layers above the mudstone layer are almost horizontally being composed of, in the descending order, Reclaimed soil, Alluvial sand 1, Alluvial gravel 2, Alluvial sand 2, Alluvial gravel 3, Alluvial clay 1, Alluvial sand 4 , Diluvial clay 1.

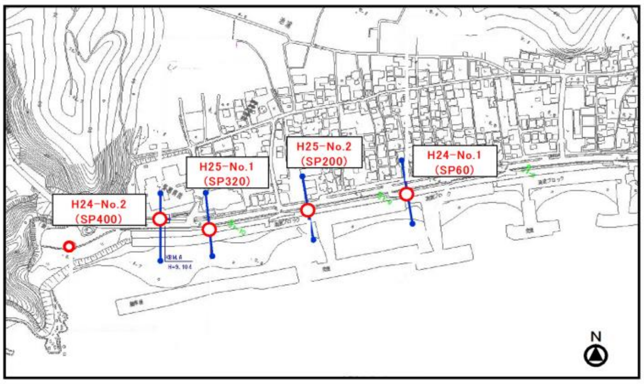


Fig. 4 Location of soil survey

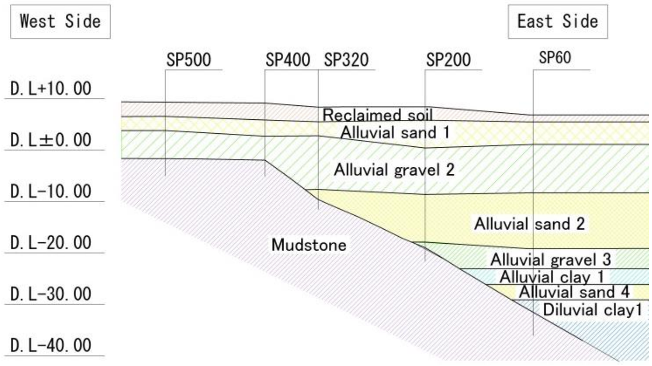


Fig. 5 Profile of soil layers

The results of Standard Penetration Test (SPT) conducted in the depth direction at the four boring sites of SP60, SP200, SP320 and SP400 are shown in **Fig. 6.**

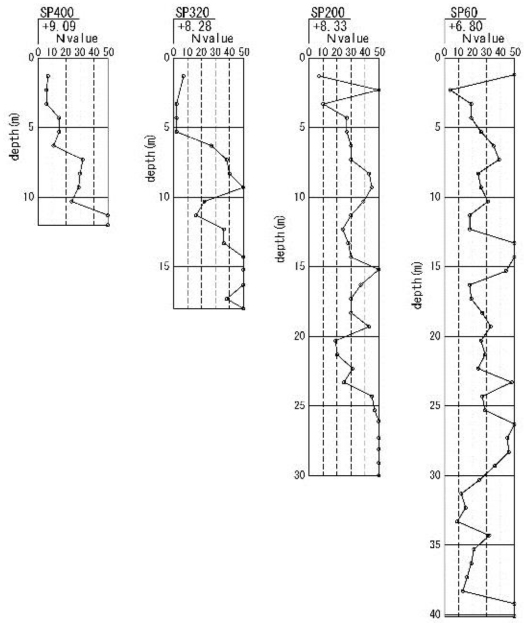


Fig. 6 SPT N Boring data

The engineering base layer was set to be the one in which the soil defined as SPT (N value) ≥ 50 and Shear Wave Velocity (Vs) ≥ 300 m/s is intersected continuously in the depth direction.

2.4. Performance code

The verification approach for the embankment on the coast of Usa Fishing Port suffering a Level-2 earthquake is based on the following :

- Whether the embankment height after earthquake exceeds the water level of design tsunami (i.e., no overflowing)
- Whether the embankment continues to function as such after earthquake (i.e., faceline of the embankment remains continuous)

The performance requirements for the embankment consist in no impairment of the soundness of the facility against Level-2 earthquakes and to definitely maintain its sea embankment function of preventing tsunami-induced overflow. The performance codes corresponding to the performance requirements are described below :

(1) Residual horizontal displacement :

The displacement shall not exceed the thickness of the embankment superstructure

Max｜δ_{X1} (horizontal displacement at the crown), δ_{X2} (horizontal displacement at the slope toe)｜ ≤ B (thickness of the embankment superstructure of 50 cm)

The reasons for setting the allowable value of horizontal displacement to be less than 50 cm are as follows: If residual horizontal displacements exceed the thickness of the embankment superstructure of 50 cm, an outflow of earth and sand can occur at the joints of the embankment, which makes it impossible to maintain the disaster preventive function as an embankment. In addition, it is necessary to ensure stability against slide of the embankment that can be caused by the relative horizontal and/or vertical displacements at the crown and the slope toe of the facility.

(2) Residual vertical displacement :

The crown height of the embankment shall be maintained above the design tsunami water level after earthquakes, meaning no overflowing of a tsunami lower than the design tsunami.

Current embankment height - (Residual settlement*[1] + Settlement due to drainage*[2]) + Crustal settlement) > Water level of design tsunami

*1: Residual settlement obtained by a two-dimensional earthquake response analysis

*2: Settlement due to drainage calculated from the relative gravity and the excess pore water pressure ratio

(3) Relative displacement:

Water tightness and continuity of the embankment shall be ensured.

Relative horizontal displacement $\Delta X = |\delta_{X1} - \delta_{X2}| \leq B$

Relative vertical displacement $\Delta Y = |\delta_{Y1} - \delta_{Y2}| \leq B$

(4) Angle of inclination :

Angle of inclination $\Delta\theta \leq 8.0^{\circ}$ (The stipulated value of Kochi prefecture)

The above are illustrated as a pattern diagram in **Fig. 7**.

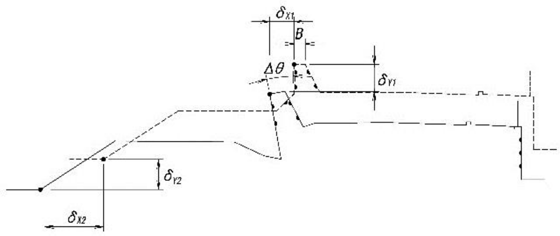


Fig. 7 Pattern diagram of displacements

2.5. Evaluation of seismic capacity

Seismic capacity against a Level-2 earthquake was evaluated for the initially designed structure of the embankment by conducting a two-dimensional effective stress analysis (FLIP) and on the basis of the afore-mentioned performance codes. FLIP (Finite Element Analysis Program of Liquefaction Process) is an effective stress-based earthquake response analysis program having a large number of application records for analyzing disaster-affected port structures and conducting seismic capacity verification, the applicability and reliability of which are well acknowledged. For a model of shearing stress - shearing strain, the program enlarges and uses a multi-spring model which consists of a large number of virtual hyperbolic springs.

For an excess pore water pressure generating model, the Iai model was employed, which, using liquefaction fronts, expresses effective stress courses under a non-drainage condition. For the establishment of soil parameters to be used for the analysis, the soil data obtained at borehole Nos. SP60, SP200, SP320 and SP400 out of the afore-mentioned five boring sites were used.

As a result of the seismic capacity verification, the cases setting soil parameters based on the boring surveys at Nos. SP200, SP320 and SP400 were proved to satisfy the seismic capacity. The result of the case, however, setting soil parameters based on the borehole No. SP60 satisfied the required capacity in respect to the slope, but failed to satisfy with the displacements exceeding the allowable values in both horizontal and vertical displacement at the crown of the embankment.

<u>Seismic capacity versus vertical displacement</u>

Current crown height : T.P. + 10.46 m

Settlements :

Crustal settlements : 2.03 m

Residual settlements : 0.73 m

Settlements due to drainage : 0.27 m

Total settlements : 3.03 m

Crown height after settlement (T.P. +10.46m - 3.03m) :

T.P. + 7.43 m < T.P. + 8.00 m (Failed)

<u>Versus horizontal displacement</u>

Displacement at the crown : 26 cm

Displacement at the bottom : 102 cm

Relative displacement : 76 cm > 50 cm (Failed)

<u>Versus slope</u>

Angle of inclination: 6.4 º < 8.0 º (Satisfied)

2.6. Structural type

2.6.1. Setting work sections

Taking into account the verification result of seismic capacity in the preceding sub-clause **2.5**, the unevenness of the engineering base layer and the hinterland conditions of the embankment, target work sections were established as follows :

- Section A

(length: 200 m : representative Borehole. No. SP60)

- Section B

(length: 250 m : representative Borehole. No. SP200)

- Section C

(length: 115 m : representative Borehole. No. SP320)

Out of the above, Section A that proved to need earthquake-resistant reinforcement was selected as the example of the target improvement work.

2.6.2. Selection of target work methods

As stated before, there are a prefectural road on the embankment and private houses in its hinterland. With current local situations taken into account, several target work methods were extracted that consider impact on roads, as well as vibration and noise annoyance on the houses.

Measures for liquefaction are roughly classified into the following methods: “Soil improvement”, “Restraining and dissipation of pore water pressure”, “Restraining of shearing deformation” and “Restraining of displacement resulting from liquefaction”, the last option of which was sorted out as the work method under this study because of the following advantages :

- Enables space-saving work
- Facilitates execution of works with less vibration and noise emissions
- Has little impact on the surrounding ground
- Has little impact on the prefectural road behind the embankment

As the “methods for restraining displacements in the wake of liquefaction”, the following three options were selected :

1) Double sheet pile installation method (**Fig. 8 (1)**)
   This is a method to install piles at both sea and land sides of the embankment, with the upper part of the piles connected mutually using tie material to restrain deformation of the ground and the embankment due to liquefaction.
2) Steel pipe sheet pile installation method (**Fig. 8 (2)**)
   A method to install steel sheet piles to reduce fluidity of a liquefied layer, thereby restraining the displacements of the embankment.
3) Combination of counterweight fill method and soil improvement (**Fig. 8 (3)**)
   A method to strengthen the frontal part of the embankment by placing a mound (comprising e.g. wave breaking blocks and rubble) in order to minimize the displacement of the facility in case the soil of its hinterland is liquefied, thereby restraining displacements.

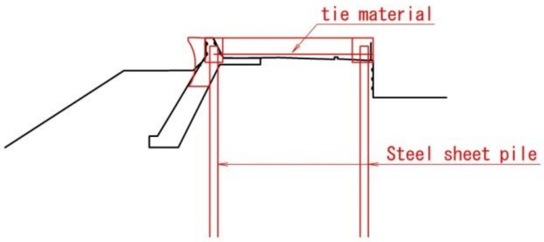


Fig. 8 (1) Double Sheet Pile Installation Method

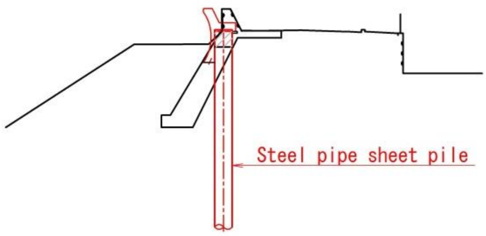


Fig. 8 (2) Steel Pipe Sheet Pile Installation Method

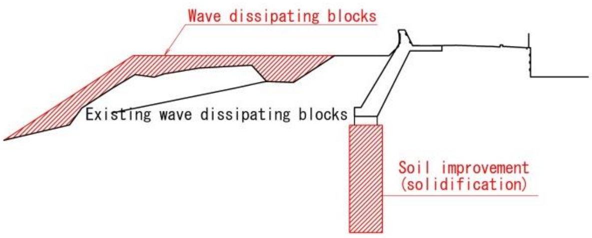


Fig. 8 (3) Counterweight Fill Method

2.6.3. Selection of work method

For each of the above three pre-selected work methods, seismic capacity, workability, applicability to the work site and economic efficiency were evaluated, the results of which are shown in **Table 1.** Since implementation of the double sheet pile installation method involves traffic restriction on the prefectural road, it can leave problems to be solved with respect to workability. The counterweight fill method may cause the facility to fall down in case hinterland soil is liquefied.

The steel pipe sheet pile installation method was eventually sorted out because of its advantages over the others in economic efficiency in addition to seismic capacity and workability.

Table 1. Selection of work method

	Double Sheet Pile	Steel Pipe Sheet Pile	Counter-weight Fill
Seismic capacity	○	○	△
Workability	△	○	○
Applicability	△	○	○
Economy	1.13	1.00	1.17
Evaluation	△	○	△

< Legend > ○: excellent; △: fair

2.7. Design of steel pipe wall

2.7.1. Verification of static stability

Verification of static stability by adopting the steel pipe sheet pile method confirmed that the generated sectional force and the displacements, both calculated by the frame structure analysis, come below the allowable values.

1) Steel pipe pile dimensions and embedment length

The steel pipe pile dimensions and the embedment depth were set as follows :

- Steel pipe pile dimensions: φ 1000 mm × t 10 mm
- Crown height of steel pipe piles: T.P. + 9.30 m
- Bottom end of embedded steel pipe piles : T.P. – 10.0 m

The embedment length of the steel pipe pile was established by the below-mentioned formula using an index called characteristic value β which appears in the deflection curve of beams on the elastic foundation.

$$L = 2/\beta$$

where

β: Characteristic value (= $(k_H \cdot D / (4EI))^{1/4}$ (1/m)

k_H: Coefficient of horizontal subgrade reaction (kN/m^3)

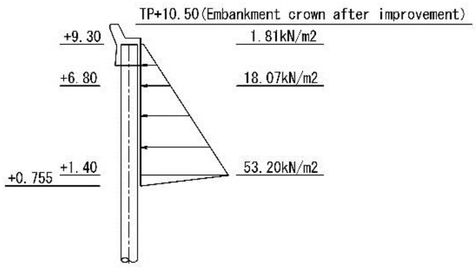
D: Unitary width of steel pipe wall (m)

E: Elastic modulus of steel pipe (kN/m^2)

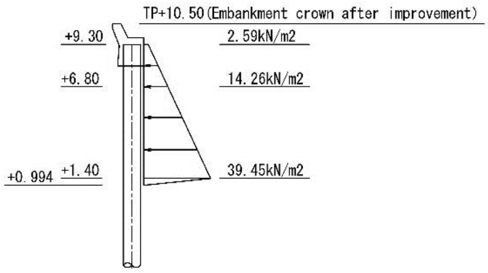
I: Second moment of area of steel pipe (m^4)

2) Loads acting on steel pipe wall

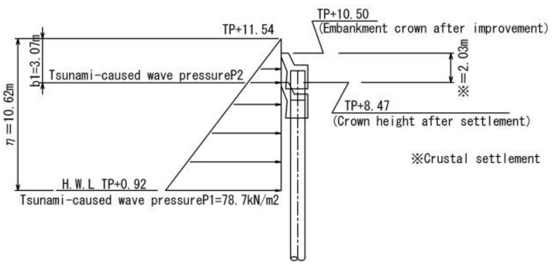
Fig. 9 illustrates images of loads derived from soil pressure under normal condition and at earthquake, and tsunami waves that act on the steel pipe walls. Considering also scouring effects caused by ocean waves and typhoons, a design ground elevation was set as T.P. + 1.4 m at the bottom end of the existing embankment.



(i) Under normal condition



(ii) At earthquake



(iii) Wave pressure by tsunami

Fig. 9 Loads acting on steel pipe walls

3) Results of calculation

Table 2 shows the verification results of static stability.

Table 2. Verification results of static stability

Load condition	Stress (N/mm^2)	Displacement at the crown (cm)
Normal	$106.3 \leq 140$	$6.8 \leq 50.0$
Earthquake	$146.3 \leq 210$	$9.5 \leq 50.0$
Tsunami	$195.8 \leq 210$	$8.5 \leq 50.0$

2.7.2. Verification of dynamic deformation

Verification of dynamic deformation related to the steel pipe sheet pile installation resulted in the following : the generated sectional force and the displacements, both calculated by the FLIP analysis, come below the allowable values.

1) Parameters for the analysis

Parameters used for the analysis are shown in **Table 3**.

Table 3. Parameters for analysis

Name of Layer	Wet density ρ (t/m^3)	Reference effective confining pressure σ_{ma} (kN/m^3)	Reference initial shear modulus G_{ma} (kN/m^2)	Cohesion c (kN/m^2)	Shear resistance angle φ (°)
Alluvial sand 1	1.84	98.0	66700	-	39.8
Alluvial gravel 2	1.84	98.0	180100	-	41.9
Alluvial sand 2	2.04	98.0	180100	-	41.9
Alluvial gravel 3	2.04	98.0	128600	-	40.5
Alluvial clay 1	1.26	272.0	91800	162.5	-
Alluvial sand 4	2.04	98.0	128600	-	40.5
Diluvial clay 1	1.29	310.0	335800	200.0	-
Mud stone	1.02	-	-	-	-

2) Analytical model

The analytical model is shown in **Fig. 10.**

3) Results of analysis and evaluation

Residual displacements and distribution of excess pore water pressure ratios are shown in **Fig. 11** and **Fig. 12**, respectively.

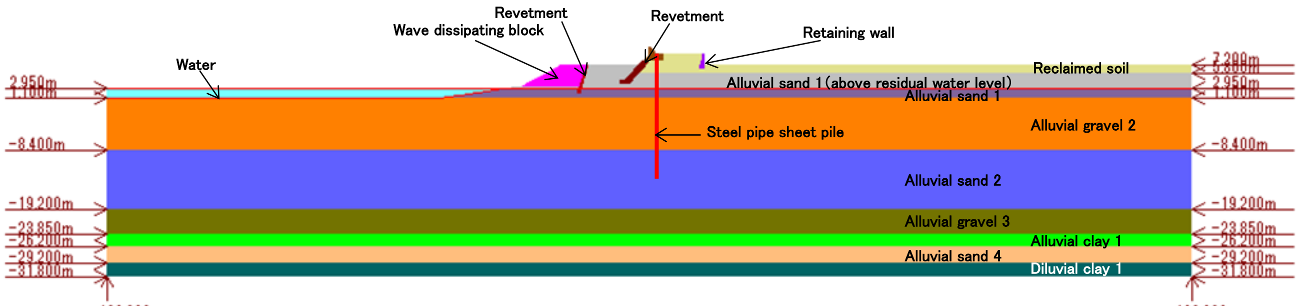


Fig. 10 Analytical Model

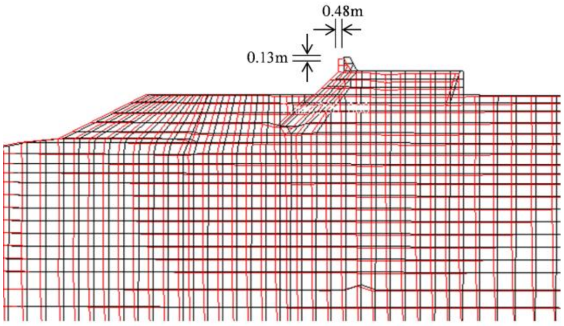


Fig. 11 Residual displacements (SP60)

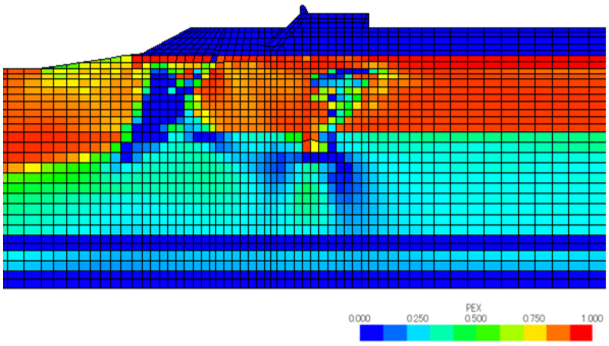


Fig. 12 Distribution of excess pore water pressure ratios (SP60)

The outcome of the FLIP analysis for the target structure is detailed below, which confirms that in the case of steel pipe sheet pile method, the post-quake embankment crown height resulting in T.P. + 8.03 m satisfies the required seismic capacity by exceeding the allowable value of T.P. + 8.00 m.

<u>Seismic capacity versus vertical displacement</u>

- Current crown height : T.P. + 10.46 m
- Settlements :

  Crustal settlements : 2.03 m

  Residual settlements : 0.13 m

  Settlements due to drainage : 0.27 m

  Total settlements : 2.43 m
- Crown height after settlement (T.P. +10.46 m -2.43 m) :

  T.P. + 8.03 m ≧ T.P. + 8.00 m (Satisfied)

<u>Versus horizontal displacement</u>

Displacements at the crown : 48.6 cm

Displacements at the bottom : 49.6 cm

Relative displacements : 1.0 cm < 50.0 cm (Satisfied)

<u>Versus slope</u>

Angle of inclination : 0.04 ° < 8.0 ° (Satisfied)

2.7.3. Inter-pile treatment

As shown in **Fig. 13**, the interval of installing the steel pipe piles was set at 1.18 m in consideration of the 0.18 meter clearance between the clamps of the piling equipment. In addition, for the sake of tsunami water shielding property and prevention of soil flow-out from inside the embankment by the drawback of a tsunami, equal angle steel (H: 200 mm x B: 200 mm x t: 20 mm) was set double between the piles. The length of the angle steel was determined to be 1 m below the existing embankment bottom end of T.P. + 1.4 m. **Fig. 14** shows an image of installing the equal angle steel between the piles.

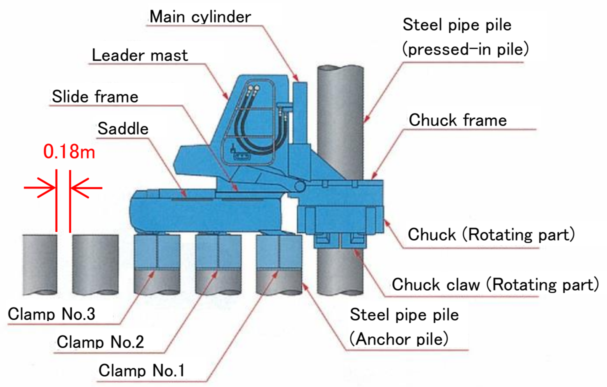


Fig. 13 Interval of clamps mounted on the piling equipment

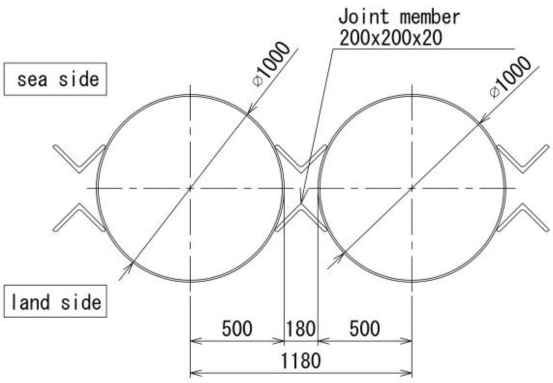


Fig. 14 Image of setting equal angle steel for tsunami water shielding

2.8. Piling method

As the steel pipe pile installation method, Press-in Method by Silent Piler was selected, equipment which allows work in a narrow space that has private houses close behind the embankment and emits less vibration and noise. Furthermore, in view that the existence of large size grains of φ 400 mm is confirmed in the rudaceous layer of the project site, determination was made of the rotary cutting press-in method. The space required for the work is handled by the assortment of a unit runner to mount the power unit, clamp crane to locate the piles, and pile runner to deliver the piles. Being self-propulsion, these units facilitate work execution in narrow spaces. An image of the rotary cutting press-in method is provided in **Fig. 15**.

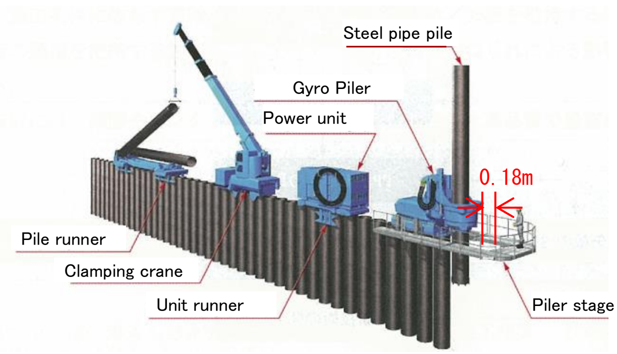


Fig. 15 Image of the rotary cutting press-in method

3. Press-in piling

3.1. Outline of the piling work

The press-in piling work for the steel pipe piles carried out in the Section A working area referred to in **2.6.1** above is outlined below.

(1) Work period

August 2014 to June 2016

(2) Steel pipe piles installation work

Dimensions of the pile: φ 1000 mm x t 10 mm x L 19.5 m

Material : SKK400

Number of piles installed : 170

(3) Equipment used

Gyro Piler GRV1226 (SP5) was used as the press-in equipment at the actual piling work, of which the main specification is provided in **Table 4** below.

Table 4. Main specification of Gyro Piler

Main Specification	
Press-in force	2600kN
Pull-out force	2800kN
Running torque	1200kN・m
Stroke	1300mm

3.2. Piling data

(1) Productivity

Pressing in 170 piles took a total of 101 days, averaging 1.68 piles a day.

(2) Measured data

Press-in data were automatically measured at and during the press-in work. **Fig. 16** illustrates examples of press-in force and distribution of torque in the depth direction that were measured automatically.

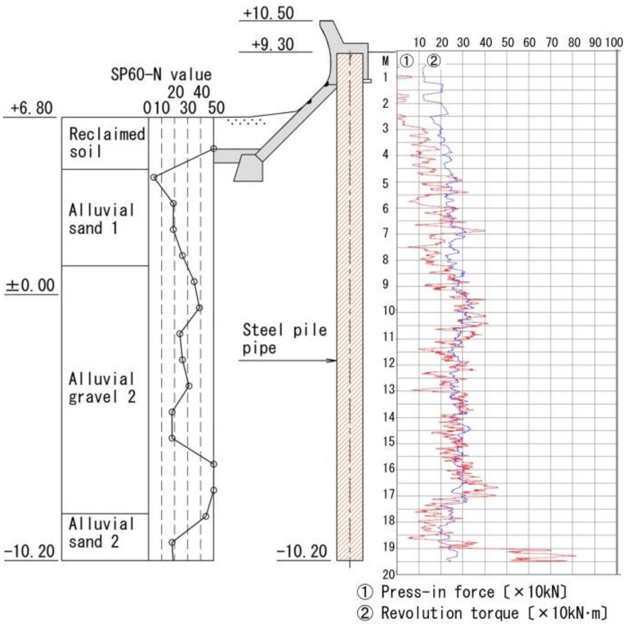


Fig. 16 Examples of press-in piling related data

3.3. Encountered difficulties

Since private houses are crowded in the surrounding area of the project site, monitoring vibration and noise emissions was necessary during the execution of the work. Accordingly, instruments were set up to measure noise and vibration so that the operators can develop work by monitoring values of noise and vibration. Careful consideration was given to the instruments setting up locations in such a way that the local residents also can check the degrees of noise and vibration. With all these measures duly taken, vibrations and noises emitted during the work have all gone below the standard values established by the prevailing regulations (noise: 85dB (A); vibration: 75dB).

4. Concluding remarks

As an earthquake and tsunami disaster preventive measure for the existing sea embankment of a slope type structure, driving steel pipe sheet piles by Press-in Method was carried out. The work site was a narrow area with a prefectural road constructed behind the superstructure of the embankment and private houses located in its hinterland. Application of Press-in Method by the Silent Piler, however, enabled completion of the improvement work within the prescribed period without having a vibration and noise impact during the execution of the work. In addition, the residents there have reported to us that the installation of steel pipe sheet piles in the embankment has helped to eliminate vibrations on their houses by ocean waves dashing against the coast, which taught us that there was a collateral effect brought by the selected work method.

Many a sea embankment is usually located having public roads and private houses close behind. The authors would appreciate it, if the measure hereby taken would serve as a reference in studying earthquake and tsunami preventive measures at sites having similar conditions as the present.

5. Acknowledgments

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