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Design and performance of the foundation of the tsunami protection wall at the Hamaoka Nuclear Power Station

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ABSTRACT: Following the disaster at Tokyo Electric Power Company's Fukushima Daiichi Nuclear Power Station induced by the 2011 Tohoku Earthquake, Chubu Electric Power Company has been implementing countermeasures in Hamaoka Nuclear Power Station (NPS) against potential mega-earthquakes and mega-tsunamis. For this purpose, the L-shaped Tsunami protection wall 14–16 m high above the site, which is at an elevation of 6–8 m above sea level, was constructed along the coastline around the site. The total length of the protection wall was 1,600 m and it was fixed to the underground walls, which were embedded in rock mass to a depth of 10–30 m. Foundation rock consists of intercalated mudstone and sandstone. The performance and uplift resistance of the underground walls embedded in rock mass is investigated using centrifuge shaking table experiments with a scale of 1/30. Furthermore, finite element analyses for the full-scale of the protection wall were performed under a base acceleration up to 2,000 gals. The author describes these studies regarding the very unique actually built protection wall against mega-earthquakes and mega-tsunamis and discusses its implication for important rock engineering structures for very similar dynamic conditions.

1 INTRODUCTION

1.1 *Overview of Hamaoka NPS*

Chubu Electric Power supplies electric power to the central part of the main island of Japan facing the Pacific Ocean. The Hamaoka NPS is located in Shizuoka prefecture, close to Mt. Fuji and located along the Pacific coast (Figure 1).

There are five nuclear power plants. Unit 1 and 2 are under decommissioning since 2009, and other 3 units are now waiting to restart. The total output of the remaining Units, 3, 4, and 5, is 3,617 MW.

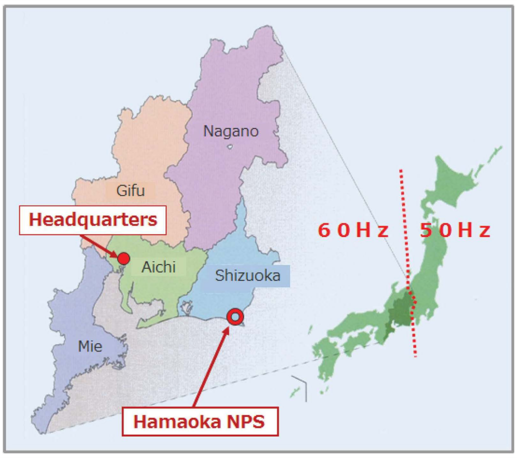


Figure 1. Location of Hamaoka NPS.

1.2 *Tsunami countermeasures at Hamaoka NPS*

When the 2011 Tohoku Earthquake occurred, the nuclear reactors of Tokyo Electric Power's Fukushima Daiichi NPS sensed massive seismic ground motions and automatically shut down. However, after the earthquake, tsunami waves higher than the station site arrived, flooding the site and buildings. Key facilities were made unusable, including seawater intake pumps for cooling and emergency generators. When batteries ran out, the power station lost its "cooling function". Consequently this led to a severe accident escalating to a massive discharge of radioactive materials.

Fukushima Daiichi was not fully prepared for the arrival of the tsunami nor the subsequent accident. To prevent a similar accident, we had promptly started safety improvement measures work, including tsunami countermeasures, after the accident.

We have applied a three phase strategy to tsunami countermeasures in the Hamaoka NPS (Table 1); "flooding prevention measures 1", "flooding prevention measures 2", and "enhanced emergency measures".

Firstly, "flooding prevention measures 1" are designed to prevent a tsunami flooding the sta-

Table 1. Three phase strategies to tsunami countermeasures.

Flooding prevention measures 1	Prevention of tsunami inundation of the station site
Flooding prevention measures 2	Prevention of tsunami flooding of buildings on the site
Enhanced emergency measures	Adopting multiple alternative means of electric power supply, water injection, and heat sink

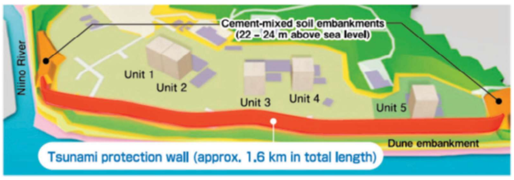


Figure 2. Perspective view of the tsunami protection wall and the cement-mixed soil embankments.

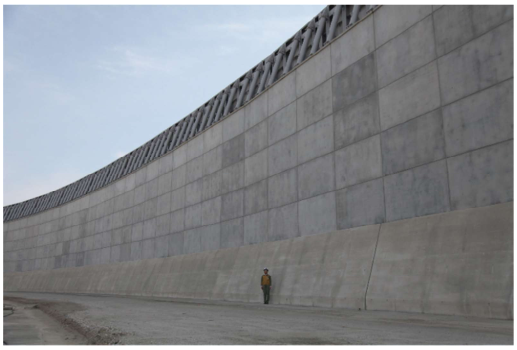


Figure 3. Tsunami protection wall.

Figure 4. Cement-mixed soil embankment.

Figure 5. Overflow prevention wall.

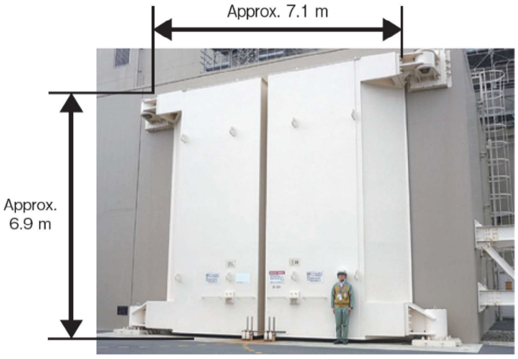


Figure 6. Reinforced protection door.

tion site. We constructed “tsunami protection wall” measuring 22 m above sea level, stretching approximately 1.6 km along the front side of the station on the ocean side. In order to prevent a tsunami from entering the station site from the sides, “cement-mixed soil embankments” with a height of 22–24 m above sea level are also constructed on the eastern and western edges of the site (Figures 2–4).

In addition, we built “overflow prevention walls”, approximately 4 m high, around water intake ponds which are linked to the sea via water intake tunnels (Figure 5).

Secondary, “flooding prevention measures 2” are designed to prevent buildings from flooding even if there is inundation in the station site. For preparedness against a tsunami higher than the tsunami protection wall, the pressure resilience and waterproof performance of exterior doors are reinforced by replacing reactor buildings’ waterproof doors with watertight doors and combining them with new tsunami protection doors for dual protection (Figure 6). Watertight doors are also installed at rooms that contain important facilities (Figure 7).

Finally, “enhanced emergency measures” referring to ensure the cooling function will work even if there is a situation like that at the Fukushima Daiichi NPS, as there will be multiple alternative means of cooling the reactor; electric power supplies, water injection, and heat sink.

In this paper, we explain the efforts about design and construction of the tsunami protection wall which is a major pillar of tsunami countermeasures of the Hamaoka NPS.

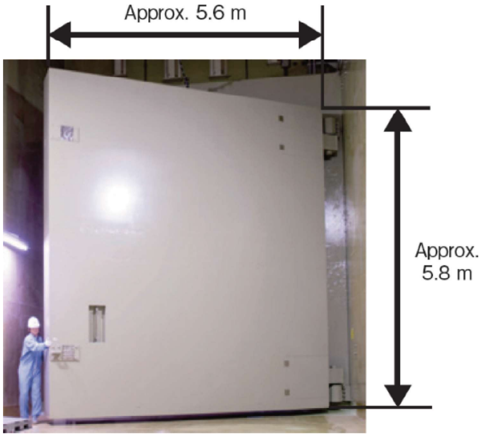


Figure 7. Watertight door.

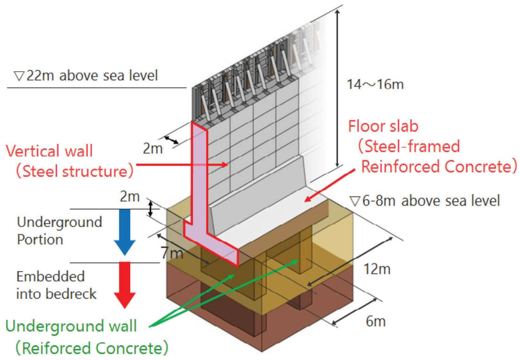


Figure 8. Structural overview of the tsunami protection wall.

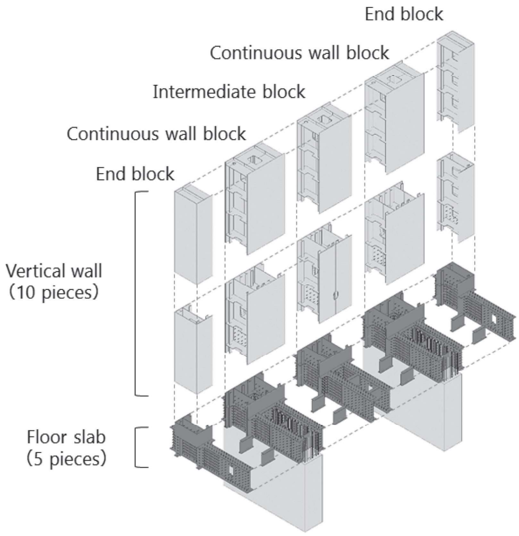


Figure 9. L-shaped wall (steel structure portion).

2 DESIGN OF TSUNAMI PROTECTION WALL

2.1 *Requirements for design*

When we started to design the tsunami protection wall, there were three requirements given below in consideration of the lessons in the disaster caused by the 2011 Tohoku Earthquake and the local conditions of the Hamaoka NPS.

- To withstand megaquakes and huge tsunamis, which may exceed paleo-quakes and paleo-tsunamis,
- To prevent large deformation against external forces far beyond the design force, and
- To be a slim structure that can be installed at the place with a limited width.

2.2 *Structural overview of the tsunami protection wall*

We had considered a structure satisfying above requirements and reached a conclusion that a combination of a wall, which had enough strength and resiliency, and a foundation, which supported the wall with high stability would be the most suitable.

As a result, we adopted a new structural system for seawalls. An L-shaped composite wall consisting of steel and steel-framed reinforced concrete was fixed to foundation of two underground walls of reinforced concrete that were embedded into solid bedrock (Figure 8). This structure provides an extra safety margin to seismic-resistant and tsunami-resistant design.

2.2.1 *L-shaped wall (upper structure)*

L-shaped wall consists of vertical wall and floor slab. To withstand huge tsunamis, it must have enough strength, but on the other hand, it is desirable to be lightweight to reduce the inertial force of megaquakes. As a result, the vertical wall was designed as steel structure which had high strength and resiliency and was lightweight. Moreover, to enhance seismic resistance, the structurally critical lower part of the vertical wall is filled with concrete.

The L-shaped wall stands 14–16 m high above the site, which is situated at an elevation of 6–8 m above sea level. A total of 109 blocks, each 12 m long, were constructed. For the steel structure portion of the wall, blocks were fabricated in a factory with one block consisting of 15 pieces that were transported to the site, connected, and erected using splice plates together with the use of about 14,000 high-strength bolts (Figure 9).

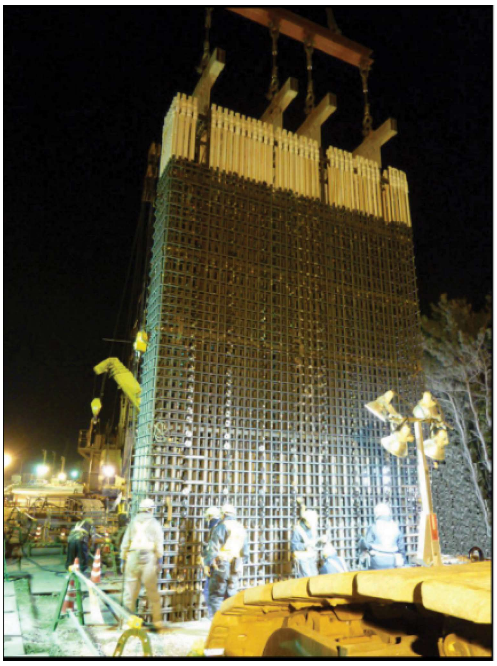


Figure 10. Underground wall (reinforced frame).

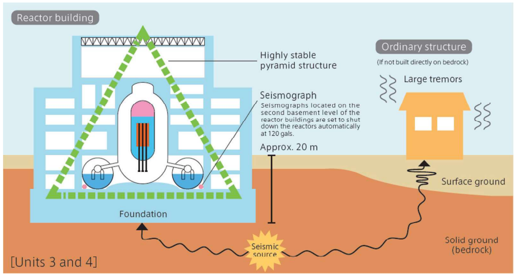


Figure 11. Basic seismic design of Hamaoka NPS.

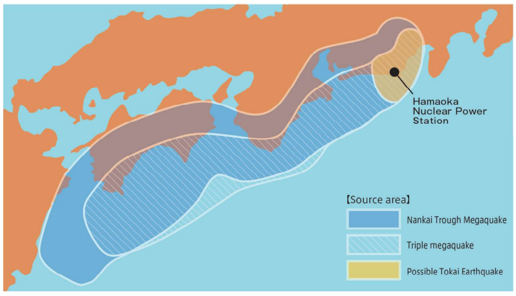


Figure 12. Anticipated Nankai Trough Megaquake.

2.2.2 *Underground wall (foundation)*

The size of the underground wall is 7 m in width, thickness 1.5 m, and approximately 10–30 m deep.

To withstand megaquakes and huge tsunamis, Large-diameter reinforcing steel, such as D51steel, is mainly used (Figure 10), and the underground wall is embedded in foundation rock consisting of intercalated mudstone and sandstone.

218 underground walls in total were constructed at 6 m intervals and arranged so that they were perpendicular to the vertical wall. Special excavators were used to drill to the designated depth. After erecting reinforced frames assembled at the site, highly fluid concrete was cast.

2.3 *Concept of seismic-resistant design*

As the Hamaoka NPS is within the hypocentral region of the anticipated Tokai Earthquake, the station has been built with a conservative seismic design from the very start of its construction (e.g. highly stable pyramid-like structure, built directly on bedrock) (Figure 11). Similarly, about the tsunami protection wall, seismic resistance was enhanced by embedding the foundation into bedrock. The seismic structural design of the protection wall was based on a response analysis against the design earthquake ground motions for the Nankai Trough Megaquake that is expected to be even greater than the triple megaquake (the Tokai, Tonankai, and Nankai) that had been anticipated before the 2011 Tohoku Earthquake occured (Figure 12).

2.4 *Concept of tsunami-resistant design*

The protection wall is also designed to withstand a huge tsunami produced by the Nankai Trough Megaquake.

The design tsunami height was set at 22 m above sea level in front of the protection wall. The design wave force for the wall was set in reference to "the results of the research of Asakura et al. (2000)".

To verify the design wave force for the wall, we carried out a wave force experiment in a large wave-generating channel with the topography of the site reproduced at a scale of 1/40. As a result, it was verified that the wave force used for the design of the protection wall was sufficiently larger so that it was on the conservative side.

3 VERIFICATION BY EXPERIMENT

3.1 *Centrifugal model experiment*

For seismic response, the ground including the protection wall and the dune embankment was reproduced by fabricating a laminar shear box test specimen with a scale of 1/30. A shaking-table experiment under a 30G field using a centrifugal loading device was carried out to study the behavior of the protection wall during an earthquake (Figure 13).

The test setup is shown in Figure 14. The ground model, including the dune embankment, was made in a laminar share box (inside dimensions 1,950 mm *600 mm *655 mm) by using sandy soil in the station site. The bedrock was cement-mixed soil. About the tsunami protection wall model, the underground wall and the floor slab were made of aluminium that unit volume weight was close to concrete, and the vertical wall was steel.

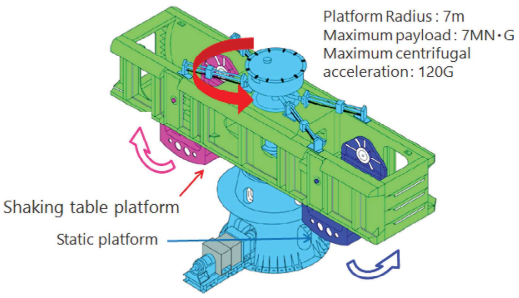


Figure 13. Centrifugal loading device.

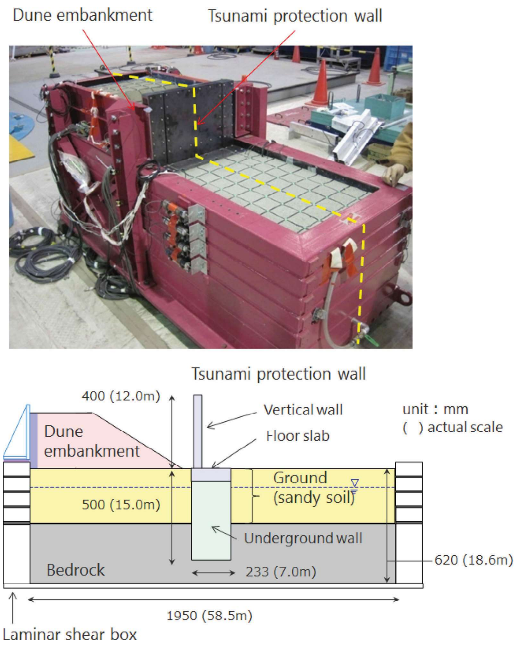


Figure 14. Test setup (centrifuge model).

The results of the experiments are shown in the following. The acceleration response spectrum of input wave is shown in Figure 15, and the strain response of the underground wall is shown in Figure 16. These show that the response of the underground wall was within an elastic range, although the input acceleration was corresponding to maximum of 2,000 gals in actual scale.

In addition, the maximum of earth pressure acting on the bottom of the underground wall is shown in Figure 17. The pressure was very small in comparison with the ultimate bearing capacity of the bedrock (21,100 kN/m²). Therefore the bedrock has enough support capacity.

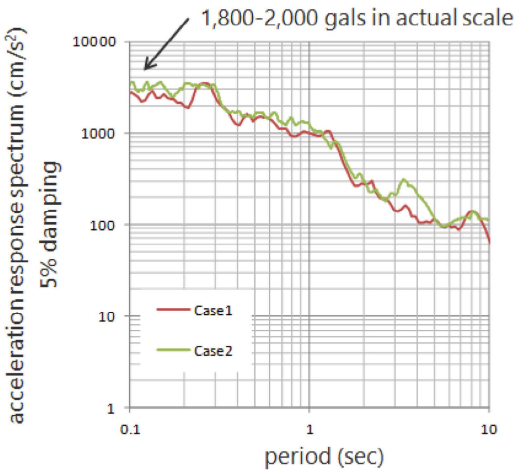


Figure 15. Acceleration response spectrum of input wave.

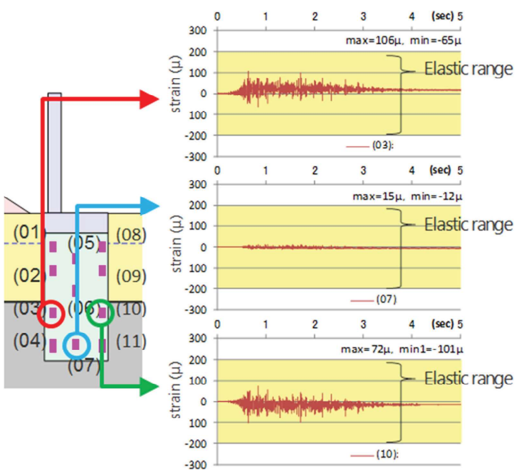


Figure 16. Strain response of the underground wall.

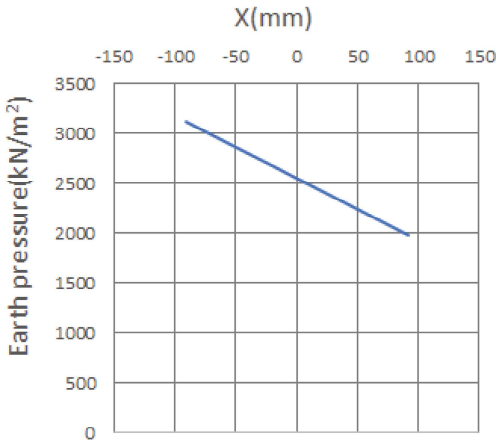


Figure 17. Earth pressure acting on the bottom of the underground wall.

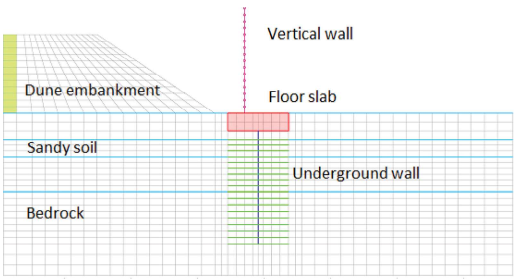


Figure 18. Analysis model (non-linear FEM).

3.2 *Simulation analysis of experiment*

We carried out simulation analyses of the experiment by dynamic response analysis, the same method of the seismic structural design of the tsunami protection wall.

The analysis model is shown in Figure 18. We modelled the test specimen by using finite element method. Specifically, ground including bedrock and floor slab were modelled by plane strain element, and vertical wall and underground wall by beam element.

Properties of the structure model were unit volume weight, moment of inertia of area, area, and Young's modulus and they were set in order to reproduce weight and characteristic value of experimental model. Regarding the bedrock, initial shear modulus was evaluated from E = 1000qu with unconfined compression strength (qu) of the test specimen that was obtained at the time of model preparation. The nonlinearity of deformation characteristics of the ground model during shaking was considered by the modified General Hyperbolic Equation (GHE) model. As an example, the deformation property of the bedrock consisting of intercalated mudstone and sandstone is shown in Figure 19.

The results of the simulation analyses are shown in the following. The maximum distribution of subgrade reaction acting on the underground wall side is shown in Figure 20. Significant subgrade reaction acts on the bedrock, and this result shows a good agreement with the experimental data in accordance with the experiment.

For the support performance of the bedrock, stress components σ_x, σ_y and τ_{xy} acting on the ground elements around the underground wall are shown in Figure 21. Large stress occurs relatively on the bedrock around the underground wall. However, these values are enough smaller than the strength of ground, so that we assume that the underground wall was safety supported by the bedrock.

The shear stress-strain hysteresis loop of the element No.4647, in which the value of τ_{xy} is largest in

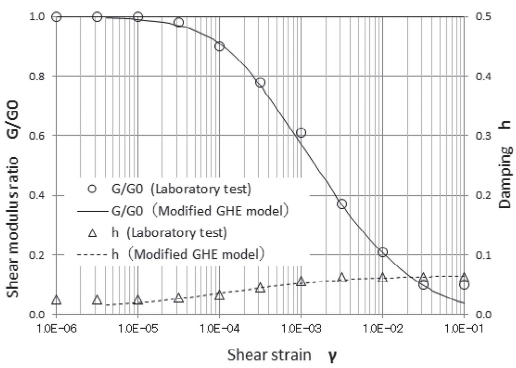


Figure 19. Deformation property of the bedrock defined by modified GHE model.

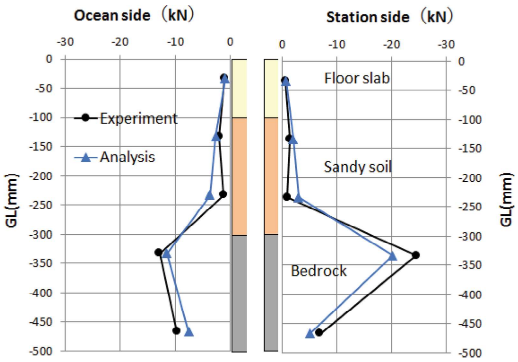
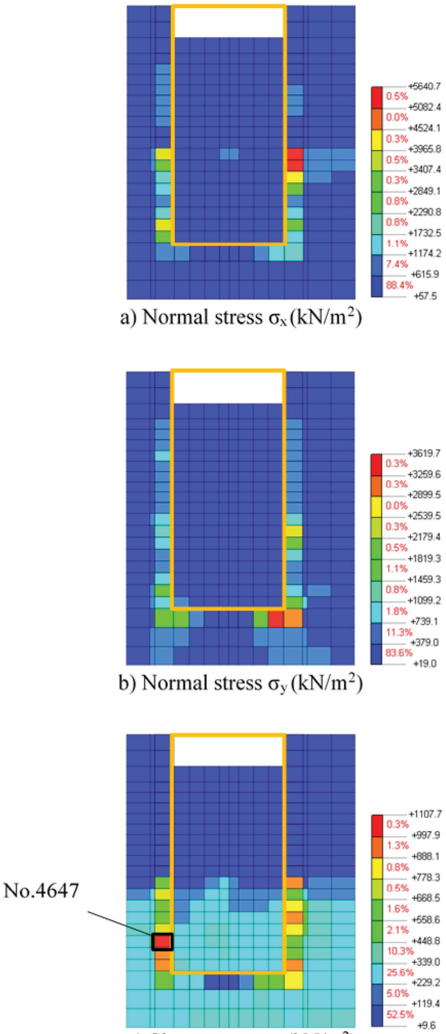


Figure 20. Subgrade reaction acting on the underground wall.



a) Normal stress σ_x (kN/m^2)

b) Normal stress σ_y (kN/m^2)

c) Shear stress τ_{xy} (kN/m^2)

Figure 21. Normal and shear stress distribution in ground around the underground wall.

Figure 21, is shown in Figure 22. The stress-strain relations are approximately linear, and the maximum amplitude of shear strain is not too large (about 0.1%), so that we assume that seismic performance of the bedrock has an enough safety margin.

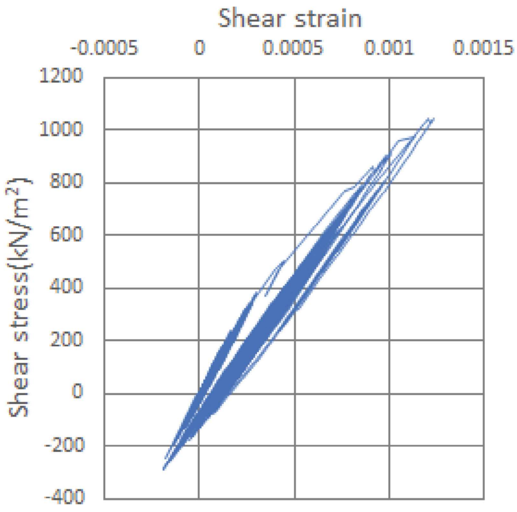


Figure 22. Shear stress-strain hysteresis loop (element No.4647).

4 CONCLUSION

In this paper, we explained a concept of seismic and tsunami-resistant design against mega-earthquakes and mega-tsunamis about the tsunami protection wall at the Hamaoka Nuclear Power Station.

About the seismic resistance of the foundation, underground wall and bedrock, it was verified by centrifugal model experiment and numerical analysis.

Tsunami countermeasures and serious-accident countermeasures in addition to the tsunami protection wall are implemented at the Hamaoka NPS as scheduled. We continue to make all-out efforts to steadily enhance the safety of the Hamaoka NPS, thereby bringing a sense of safety and security to local communities and the rest of the nation.

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