

A METHOD FOR EVALUATING TSUNAMI LOADING ON SEAWALLS DURING OVERFLOW

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Seawall constructions are one of the main ways of protecting coasts against tsunamis. The estimation of tsunami loading on a structure is important for evaluating the fragility of seawalls. In this study, hydraulic flume tests were conducted to investigate the characteristics of tsunami loading when tsunamis flow over seawalls. Correlations between the loading on seawalls and the specific energy of the flow are investigated. A method for evaluating the maximum tsunami loading on the seawall is proposed. The approach can evaluate the maximum tsunami loading, even when tsunamis flow over seawalls.

Keywords: seawall, wave pressure test, wave pressure evaluation, Tsunami PRA

INTRODUCTION

Seawalls are important structures for preventing waves from reaching the important structures in the nuclear power station site. The tsunami PRA (Probabilistic Risk Assessment) can evaluate the total risk including the condition that a tsunami overflows the seawall. To conduct a fragility evaluation of the seawall, it is necessary to evaluate the wave pressure/wave force acting on the seawall, under the condition that the tsunami overflows the seawall. Although Asakura et al. (2002), Sakakiyama (2012) and Ikeya (2013) evaluated the wave pressure, there are few examples that examined the evaluation method of wave pressure under the condition that tsunami overflows the seawall.

The time historical characteristics of tsunami wave pressure acting on the seawall must be considered when evaluating wave pressure/wave force, as shown in Figure 1. The bore wave pressure is the pressure from the first impact of the tsunami on the seawall, and it significantly changes in a short time period. The wave pressure which exerts on the seawall after bore wave pressure is called the continuous wave pressure. The continuous wave pressure acts relatively for a long time. In this study, the wave pressure time history is divided into the bore wave pressure region and the continuous wave pressure region by the maximum wave height in the front of the seawall.

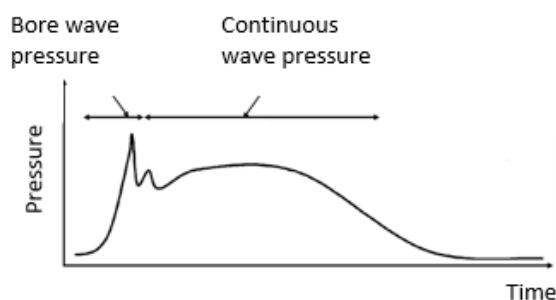


Figure 1. Wave pressure acting on a seawall.

Regarding the bore wave pressure, Ishida et al. (2015) confirmed that its effect on structures was, in general, lesser than that of the continuous wave pressure. Although the effect of the bore wave can become greater than that of the continuous wave when the specified multiple conditions are combined, this study focuses on the continuous wave pressure which, in general, has more serious effects on structures than the bore wave pressure.

Regarding the continuous wave pressure, the overall reduction of the wave pressure due to the overflow was studied in Taisei Corporation (2018) and Oda et al. (2018). This is because the water energy contributing to the wave pressure is released due to the overflow. Moreover, the equation to evaluate the maximum continuous wave force with overflow from the maximum continuous wave force acting on the other seawall which is high enough not to overflow was suggested.

Objective

To conduct the tsunami PRA appropriately, a systematic approach of the evaluation method of tsunami wave force/wave pressure acting on the seawall is developed. The tsunami height includes not only non-overflow height but also excessive overflow height.

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HYDRAULIC FLUME TESTS

To develop the evaluation method of tsunami wave pressure/wave force acting on the seawall, hydraulic flume tests with overflow were conducted, and the data related to the tsunami wave pressure was measured. The evaluation method of the design wave pressure was studied with the hydraulic flume tests of the design condition where the waves did not exceed the seawall by Ishida et al. (2016) and Toriyama et al. (2018). In this study, not only the design condition but also the condition where the waves are high enough to overflow the seawall. To evaluate the effect of continuous wave pressure when tsunami overflows the seawall, the run-up test and the wave pressure test were conducted.

The run-up test is a hydraulic flume test without seawall. Its purpose is to examine the wave characteristics by measuring the run-up water depth and the wave velocity. The wave pressure test measures the continuous wave pressure acting on the seawall by using the wave pressure gauges installed on the front side of the seawall. Its purpose is to investigate the effect of continuous wave pressure.

Schematic of the hydraulic flume

Figure 2 shows an overview of the test equipment for hydraulic flume tests. The total length of the flume was 47 m, and its width was 0.8 m. The scale of the seawall model was assumed to be 1/80 of the real size, and two seawalls with different heights were used. The dimensions of seawalls are 0.79 m in width and 0.008 m in thickness. There are two types of height of seawalls. The seawall height (0.275 m) of type A envisaged the situation that the wave overflows its height, and the seawall height (0.8 m) of type B envisaged the situation without overflow (Figure 3). The seawalls were located on the shoreline, which is 1.25 m inland or 2.5 m inland from the shoreline. The bed slopes were a zero slope and a 1/20 slope.

The capacity type of the wave height gauge was used. The electromagnetic wave velocity gauges were used on the ocean side, and the propeller type wave velocity gauge (V_8) and electromagnetic bottom wave velocity gauge (V_9) were used for the land side. Pressure gauges were installed at every 5 cm vertically on the seawall (Figure 3). 10 gauges were installed on the type A seawall and 20 gauges were installed on the type B seawall. The sampling interval of the wave height gauges and wave velocity gauges was 0.001 s (1000 Hz).

The wave types were solitary waves (Wc11–Wc13) and sine waves (Wc21, Wc22, Wc31, and Wc32). The additional solitary waves (Wc14 and Wc15) that break close to the shoreline were used. Figure 4 shows the wave height history at the point of WG₃.

The settings of run-up test cases were shown in Table 1. Those of the wave pressure test cases were shown in Table 2.

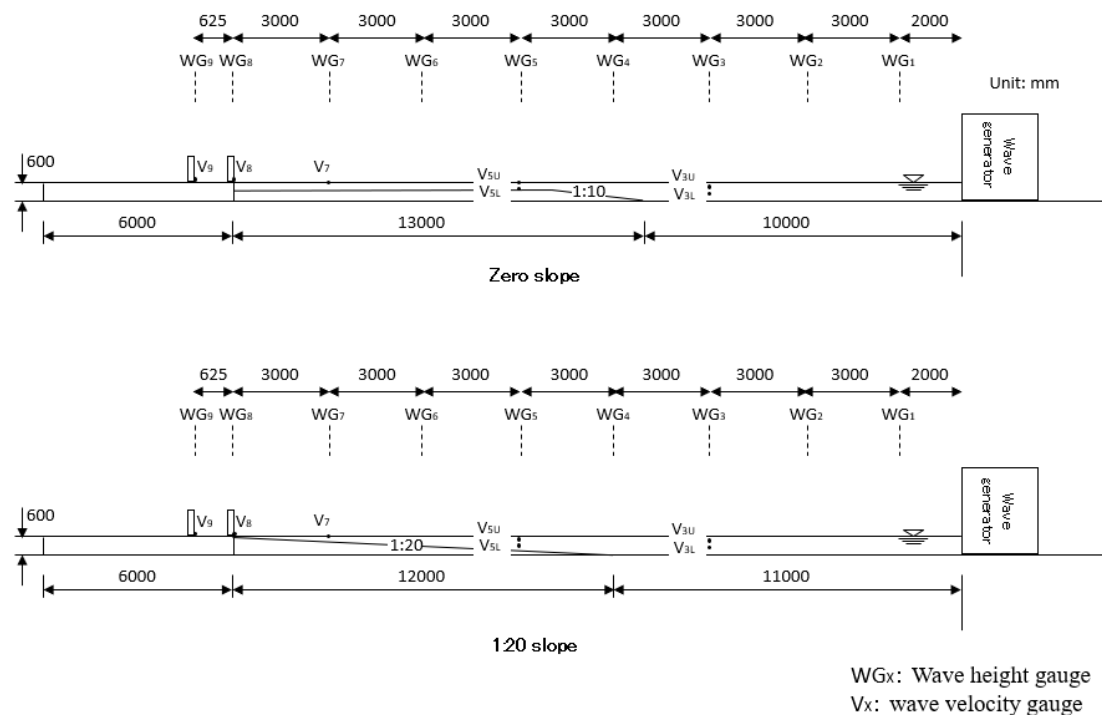


Figure 2. Schematic of hydraulic flume.

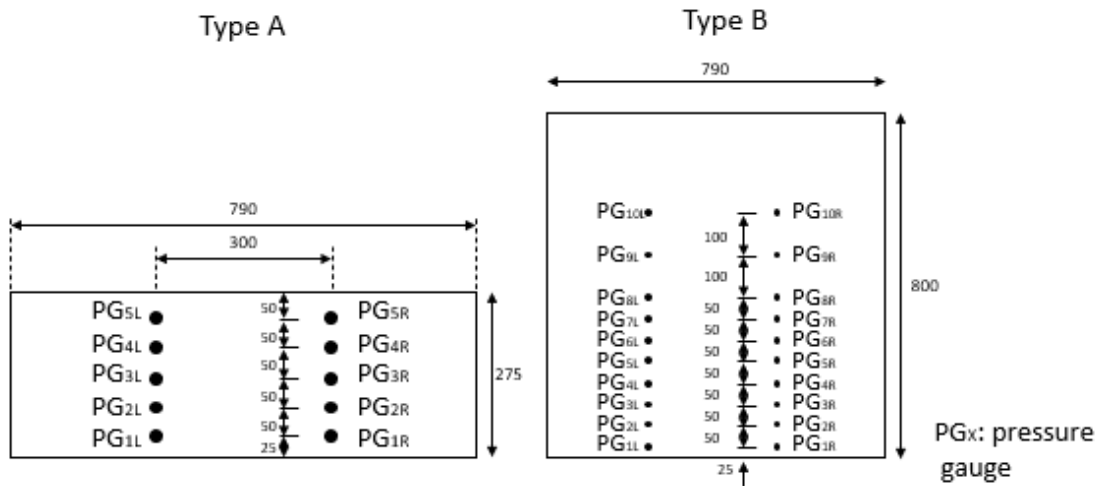


Figure 3. Position of gauges on seawall.

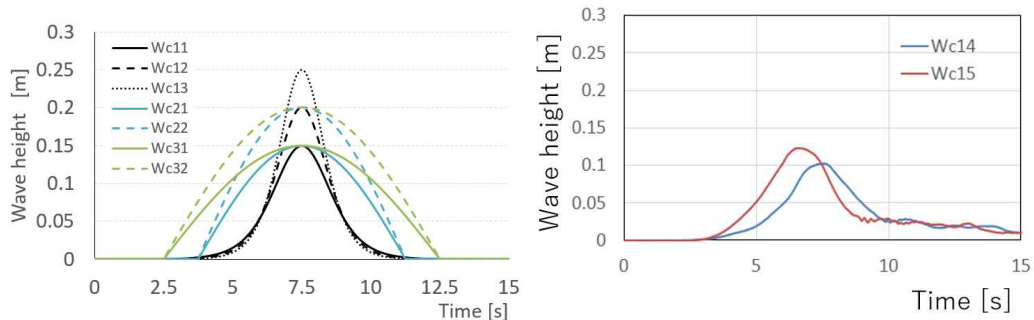


Figure 4. Waveforms for flume tests: solitary waves (Wc11 ~ Wc15) and sine waves (Wc21, Wc22, Wc31, Wc32).

Test condition					
Test	Bed slope	Seawall location (m)	Input wave height (at 15 m point from the shoreline: m)		Number of repetitions
Run-up test	0 (Water depth at shoreline 0.3m)	None	Solitary wave (Wc11~Wc15)	0.1, 0.125, 0.15, 0.2, 0.25 (5 waves in total)	3
			7.5s Sine wave (Wc21, Wc22)	0.15, 0.2 (2 waves in total)	3
			10s Sine wave (Wc31, Wc32)	0.15, 0.2 (2 waves in total)	3
	1/20 (Water depth at shoreline 0m)	None	Solitary wave (Wc11~Wc13)	0.15, 0.2, 0.25 (3 waves in total)	3
			7.5s Sine wave (Wc21, Wc22)	0.15, 0.2 (2 waves in total)	3
			10s Sine wave (Wc31, Wc32)	0.15, 0.2 (2 waves in total)	3

Table 2. Settings of wave pressure tests.					
Test conditions					Number of repetitions
Test	Bed slope	Seawall location (m)	Input wave height (at 15 m point from the shoreline: m)		
Wave pressure test	0 (Water depth at shoreline 0.3m)	Shoreline 0.625	Solitary wave (Wc11~Wc15)	0.1, 0.125, 0.15, 0.2, 0.25 (5 waves in total)	Type A: 3 Type B: 15
			7.5s Sine wave (Wc21, Wc22)	0.15, 0.2 (2 waves in total)	Type A: 3 Type B: 15
			10s Sine wave (Wc31, Wc32)	0.15, 0.2 (2 waves in total)	Type A: 3 Type B: 15
	1/20 (Water depth at shoreline 0m)	Shoreline 0.625	Solitary wave (Wc11~Wc13)	0.15, 0.2, 0.25 (3 waves in total)	Type A: 3 Type B: 15
			7.5s Sine wave (Wc21, Wc22)	0.15, 0.2 (2 waves in total)	Type A: 3 Type B: 15
			10s Sine wave (Wc31, Wc32)	0.15, 0.2 (2 waves in total)	Type A: 3 Type B: 15

Run-up test results

Using the run-up test results, the parameters that represent the wave characteristics, that is., the Froude number (Fr) and the specific energy (E) are evaluated. The Froude number (Fr) is a dimensionless number that represents the ratio of the inertial force of the fluid to gravity, and it can be evaluated by the run-up water depth (η) and wave velocity (v) where the seawall stands, as shown in Eq. 1.

$$Fr = v / \sqrt{g\eta} \quad (1)$$

Where g means the gravitational acceleration. The specific energy (E) means a total energy per unit weight of water, while the dimension of the specific energy (E) is length. It is a parameter for non-viscous and steady flow based on the Bernoulli's theorem, and is expressed in Eq. 2.

$$E = 0.5v^2/g + \eta \quad (2)$$

Figure 5 shows the examples of time histories of run-up water depth (η), Froude number (Fr), and specific energy (E) at each seawall location. The Froude number (Fr) and the specific energy (E) become maximum when the tsunami reaches the seawall, and they decrease as the time passed.

At the shoreline position of the zero slope, the Froude numbers keep constant around 0.7. It is inferred that when tsunami overflows in case of the zero-slope shoreline, the control section with $Fr = 1$ appears at the point slightly on the land side from the shoreline (downstream side). Therefore, almost constant Froude number (Fr) stays near the shoreline in cases of zero-slope.

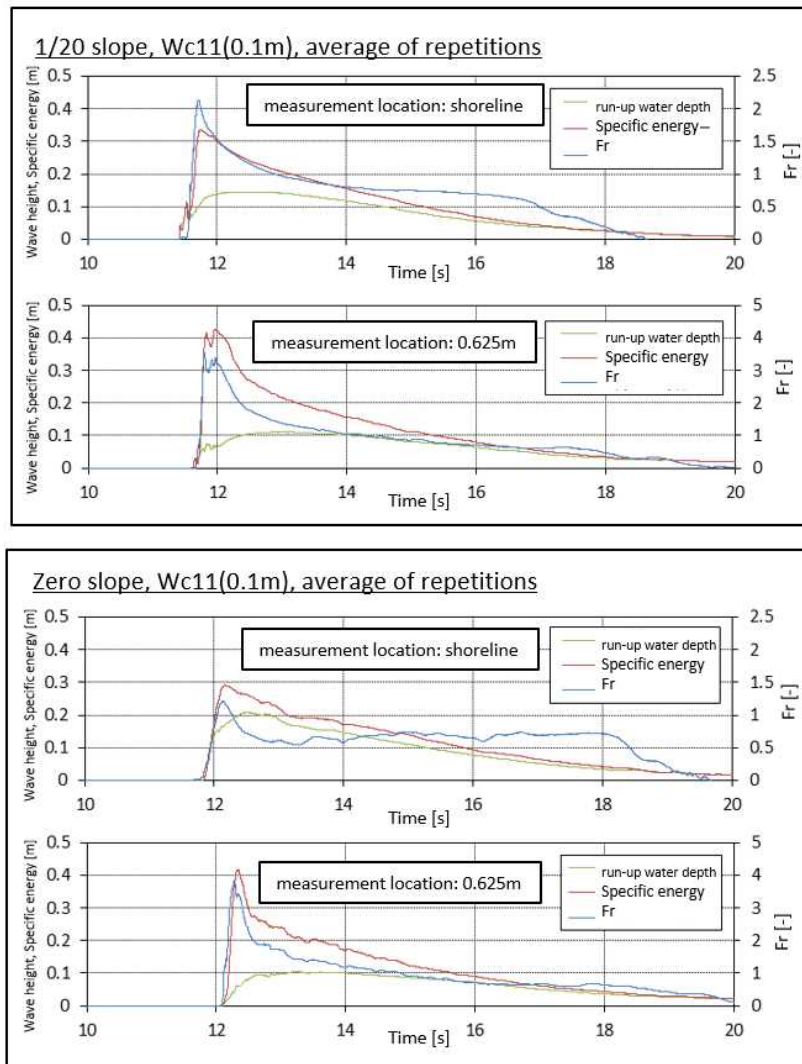


Figure 5. Time histories of wave height, Froude number, and specific energy (Wc11, average of repetitions, upper 2 figures: 1/20, lower 2 figures: zero).

Analysis based on wave pressure test results

Figure 6 shows an example of time histories of the wave height in front of the seawall ($\eta(t)$) and wave force ($F(t)$) calculated from the wave pressures acting on the seawall. While the time history of the water height in front of the type A seawall is similar to that of the type B seawall, the maximum value of the type A seawall is smaller than that of the type B seawall. That is because the water energy contributing to the increase of the wave height is released due to the overflow. As for wave force ($F(t)$), similar results are obtained.

Figure 7 shows the pressure distributions when the continuous wave forces become maximum. While the wave pressure distributions of both types are similar, the wave pressure of type A seawall is smaller than that of the type B seawall due to the overflow.

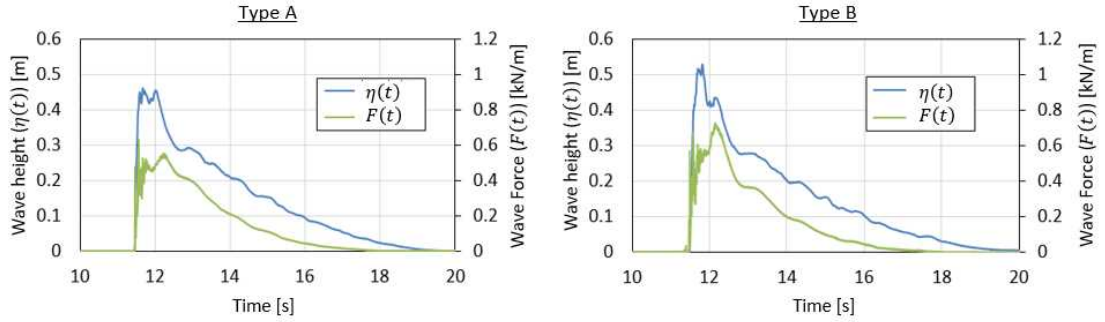


Figure 6. Time history of wave height and wave force (1/20 slope, shoreline, Wc11, average of repetitions).

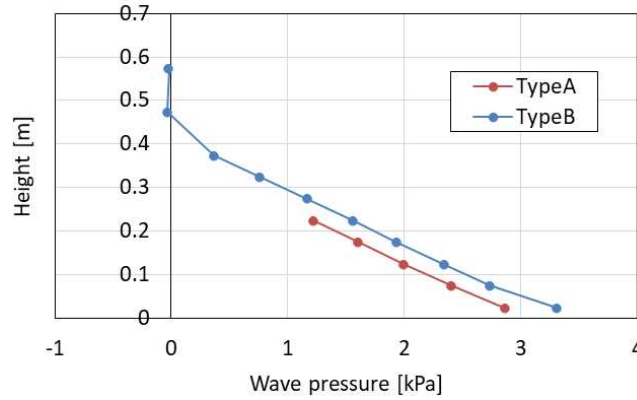


Figure 7. Wave pressure distributions when continuous wave forces become maximum (1/20 slope, shoreline, Wc11, average of repetitions).

EVALUATIONS FOR TSUNAMI WAVE PRESSURE

Evaluation method of the maximum continuous wave force without overflow

To develop the evaluation method of the maximum continuous wave force without overflow, the previous study for the design evaluation method is re-evaluated.

In the study, Toriyama et al. (2018) showed the equation to evaluate the maximum continuous wave force for the design. The design equation is obtained from the theoretical equation using the Froude number (Fr_E) and the water depth coefficient (α_E) at the time when the maximum specific energy (E_{max}) occurs, as shown in Eq. 3. The water depth coefficient (α_E) is a non-dimensional number relating to the maximum continuous wave pressure (P_{bottom}) acting on the bottom of the seawall. The water depth coefficient (α_E) represents the ratio of the maximum continuous wave pressure (P_{bottom}) to the hydrostatic pressure which is equivalent to the run-up water depth (η_E), as presented in Eq. 4.

Figure 8 shows the previous test results and theoretical curve which is equivalent to Eq. 3. The error bars represent the variation of the 15 repetitions under the same test condition. The theoretical curve is similar to the regression curve derived from the previous hydraulic flume test results by the least square method. The logarithmic standard deviation ($\sigma_{\alpha_E} = 0.217$) is evaluated as the variation of the test results from the theoretical equation Eq. 3.

$$\alpha_E = 0.50Fr_E^2 + 1 \quad (3)$$

$$\alpha_E = \frac{P_{bottom}}{\rho g \eta_E} \quad (4)$$

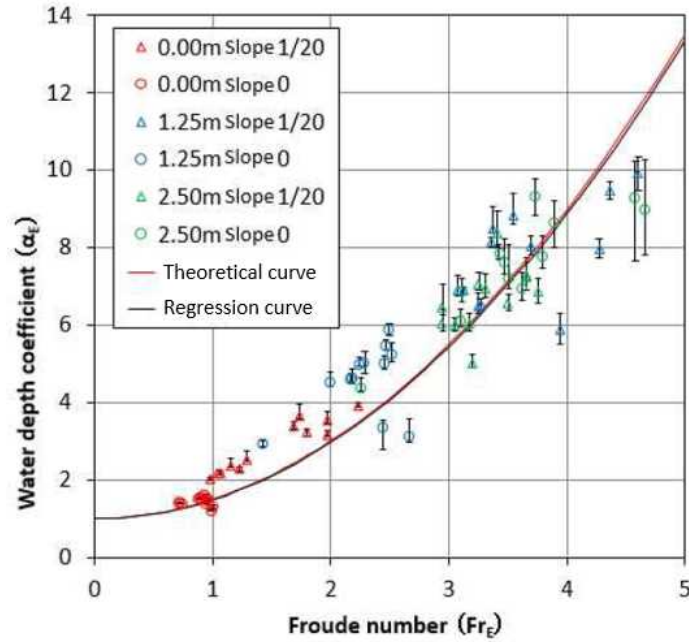


Figure 8. Relationship between Froude number (Fr_E) and water depth coefficient (α_E).

The theoretical equation Eq. 3 is applied to evaluate the continuous wave force without overflow (F_S) as expressed in Eq. 5. The continuous wave force without overflow (F_S) can be evaluated by the run-up water depth (η_E) and the wave velocity when the specific energy become maximum from the run-up test.

The logarithmic standard deviation ($\sigma_{\alpha_E} = 0.217$) includes the test results variation induced by the difference of the bed slope, the seawall location and the input waveform by the previous hydraulic flume test. In this study, it is assumed that the continuous wave force without overflow (F_S) had the uncertainty of the logarithmic standard deviation ($\sigma_{\alpha_E} = 0.217$).

$$F_S = 0.5\rho g(\alpha_E\eta_E)^2 \quad (5)$$

Evaluation method of the maximum continuous wave force with overflow

To develop the evaluation method of the maximum continuous wave force with overflow, the theoretical consideration is conducted by using test results and previous study.

As shown previously, wave pressure distribution with overflow is smaller than that without overflow. The pressure distribution reduction effect due to overflow is represented by the reduction factor (β). The water depth factor with overflow (α'_E) is represented as the product of reduction factor (β) and water depth coefficient without overflow (α_E).

$$\alpha'_E = \beta\alpha_E \quad (6)$$

The reduction factor (β) can be evaluated from the study by Taisei Corporation (2018). In the previous study, the wave height rate was theoretically derived. The wave height rate is the ratio of the maximum wave height in front of the seawall to the maximum wave height in front of the seawall which is high enough to avoid overflow. The wave height rate is equivalent to the reduction factor (β) because the maximum wave height in front of the seawall is evaluated as the multiplication of the run-up water depth and the each water depth factor.

The reduction factor (β) is expressed by the Eq. 7 using the maximum wave height in front of the other seawall which is high enough to avoid overflow (η^*) and the seawall height (h_d).

$$\beta = \frac{2 + \sqrt{3\left(\frac{\eta^*}{h_d}\right)^2 - 2}}{3\frac{\eta^*}{h_d}} \quad (7)$$

The Eq. 7 needs to evaluate the maximum wave height in front of the seawall (η^*) by the flume test or the analysis with seawall while the continuous wave force without overflow (F_s) can be evaluated only the run-up test results by Eq. 5.

Here, it is assumed that the maximum wave height in front of the seawall (η^*) is equivalent to the maximum specific energy (E_{max}) at the seawall location on the basis of the energy conservation. Except for the zero slope and shoreline condition, the relationship between the maximum wave height in front of the seawall (η^*) and the maximum specific energy (E_{max}) is nearly equal as shown in Figure 9. Therefore, the maximum wave height in front of the seawall (η^*) is replaced to the maximum specific energy (E_{max}) which can be evaluated only from the run-up test results. Note that with the condition where the seawall located on the shoreline of the zero slope, the vertical wave velocity accelerates because the shoreline formation and a large splash is generated that does not contribute to the wave pressure. Therefore, the maximum wave height (η^*) reaches about twice as large as the maximum specific energy (E_{max}). This must be considered when evaluating the wave forces under the condition where the seawall is located on the shoreline of the zero slope.

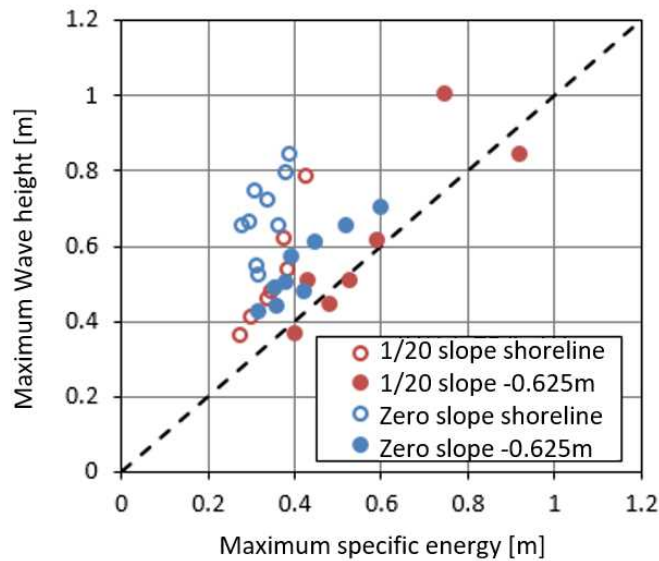


Figure 9. Relationship between the maximum specific energy and the maximum wave height.

Figure 10 shows the relationship between the reduction factor (β) and the overflow rate (E_{max}/h_d) that is the ratio of the maximum specific energy (E_{max}), which is equivalent to the maximum wave height in front of the seawall (η^*) to the seawall height (h_d). The theoretical curve of Eq. 7 is shown as well. While there is some uncertainty, the test result shows the tendency to decrease along with the theoretical curve. The condition where the overflow rate (E_{max}/h_d) exceeds 1 means the tsunami overflows the seawall.

Assuming that the reduction factor (β) follows the logarithmic normal distribution, the logarithmic standard deviation ($\sigma_\beta = 0.085$) from the theoretical equation is evaluated. The uncertainty is induced by the differences that the maximum specific energy (E_{max}) is replaced to the maximum wave height in front of the seawall (η^*), and the uncertainty of the theoretical equation of the reduction factor (β) itself.

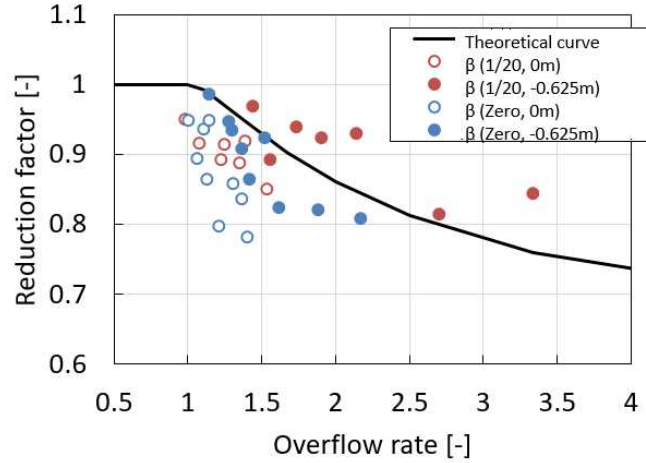


Figure 10. Relationship between the overflow rate and the reduction factor.

Because the water depth coefficient with overflow (α'_E) is a product of two logarithmic normal distributions, the water depth coefficient (α_E) and the reduction factor (β). Its median value becomes $\beta\alpha_E$ in Eq. 8 and the logarithmic standard deviation becomes square root of sum of them $\sqrt{\sigma_{\alpha_E}^2 + \sigma_{\beta}^2}$.

The continuous wave force with overflow (F'_S) can be evaluated by Eq. 9 using the water depth coefficient with overflow (α'_E) and the run-up water depth when the specific energy become maximum (η_E) as shown in Figure 11. In this study, the uncertainty of the continuous wave force with overflow (F'_S) has the uncertainty of the logarithmic standard deviation ($\sqrt{\sigma_{\alpha_E}^2 + \sigma_{\beta}^2}$).

$$\alpha'_E = \beta\alpha_E = \frac{2 + \sqrt{3 \left(\frac{E_{max}}{h_d}\right)^2 - 2}}{3 \frac{E_{max}}{h_d}} \cdot (0.5Fr_E^2 + 1) \quad (8)$$

$$F'_S = \rho g (\alpha'_E \eta_E) h_d - 0.5 \rho g h_d^2 \quad (9)$$

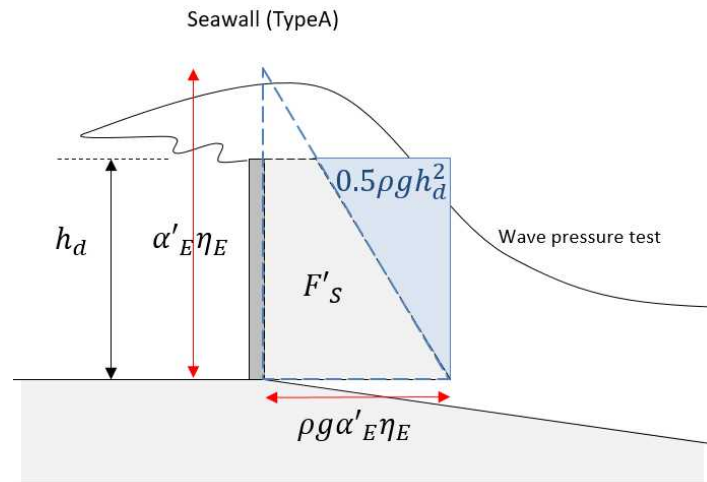


Figure 11. The continuous wave force with overflow.

Evaluation procedure of the maximum continuous wave force

To develop the systematic approach to evaluate the maximum continuous wave force, the equations in the previous sections are combined and the evaluation flow is proposed.

The Eq. 5 which represents the continuous wave force without overflow (F_S) and the Eq. 9 which represents the continuous wave force with overflow (F'_S) are combined. The continuous wave force without overflow (F_S) has the uncertainty of the logarithmic standard deviation (σ_{α_E}), and the continuous wave force with overflow (F'_S) has the uncertainty of the logarithmic standard deviation ($\sqrt{\sigma_{\alpha_E}^2 + \sigma_{\beta}^2}$).

$$\begin{cases} F_S = 0.5\rho g(\alpha_E\eta_E)^2 & \dots (0 \leq E_{max}/h_d \leq 1) \\ F'_S = \rho g(\alpha'_E\eta_E)h_d - 0.5\rho gh_d^2 & \dots (1 < E_{max}/h_d) \end{cases} \quad (10)$$

These equations have two variables, the Froude number (Fr_E) and the maximum specific energy (E_{max}). These variables are combined by using the overflow rate (E_{max}/h_d) using the relationship between the maximum specific energy (E_{max}) and Froude number (Fr_E) as shown in Eq. 11 derived from Eqs. 1 and 2.

$$E_{max} = (0.5Fr_E^2 + 1)\eta_E \quad (11)$$

$$\begin{cases} F_S = \frac{\rho gh_d^2}{2} \left(\frac{E_{max}}{h_d} \right)^2 & \dots \left(0 \leq \frac{E_{max}}{h_d} \leq 1 \right) \\ F'_S = \frac{\rho gh_d^2}{2} \left(\frac{1}{3} + \frac{2}{3} \sqrt{3 \left(\frac{E_{max}}{h_d} \right)^2 - 2} \right) & \dots \left(1 < \frac{E_{max}}{h_d} \right) \end{cases} \quad (12)$$

While the evaluation method of logarithmic standard deviations is not considered well, in this study, the uncertainties evaluated from the test results, ($\sigma_{\alpha_E} = 0.217$) and ($\sigma_{\beta} = 0.085$) are used.

The evaluation procedure of the maximum continuous wave force is summarized as follows:

1. The run-up water depth and wave velocity at the seawall location is evaluated by the run-up test or analysis.
2. The maximum specific energy (E_{max}) at the seawall location and the overflow rate (E_{max}/h_d) from the seawall height (h_d) are evaluated.
3. The maximum continuous wave force without overflow (F_S) is evaluated, when the overflow rate (E_{max}/h_d) does not exceed 1.
4. The maximum continuous wave force with overflow (F'_S) is evaluated, when the overflow rate (E_{max}/h_d) exceeds 1.

The flow chart to evaluate the maximum continuous wave force is summarized as shown in Figure 12.

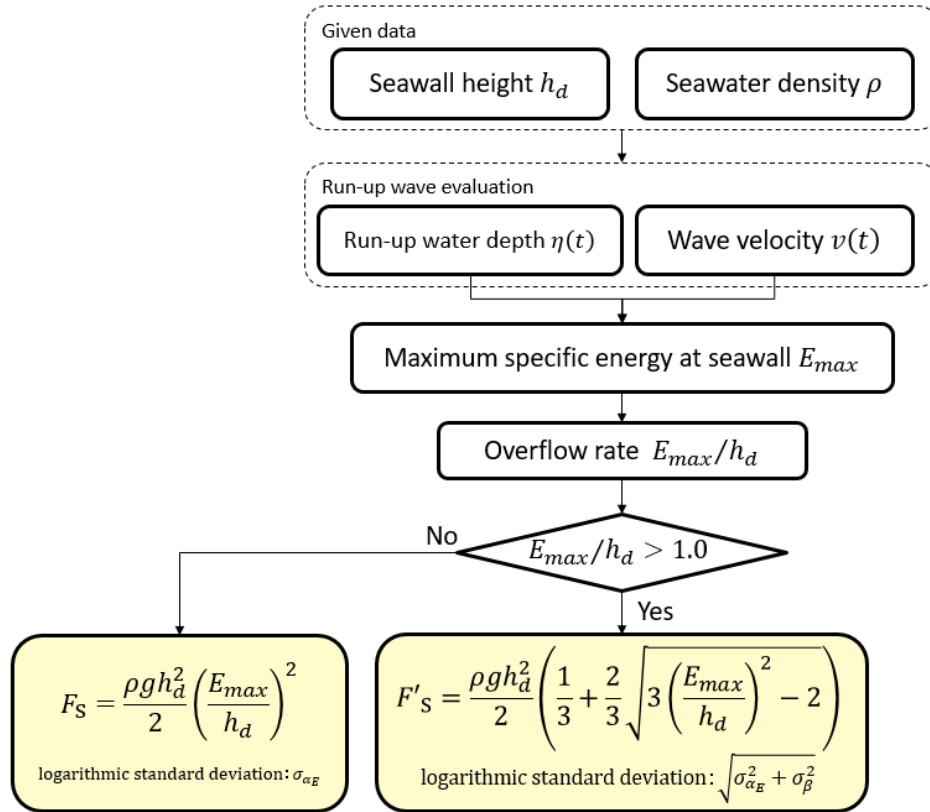


Figure 12. Flow chart to evaluate the maximum continuous wave force.

Continuous wave force evaluated by using dimensionless numbers

The test results to the theoretical equation using the approach discussed in the previous paragraph are compared. The dimensionless maximum continuous wave forces (G_S) are compared with logarithmic standard deviation, (σ_{α_E}) and ($\sqrt{\sigma_{\alpha_E}^2 + \sigma_{\beta}^2}$). The uncertainties evaluated from the test results, ($\sigma_{\alpha_E} = 0.217$) and ($\sigma_{\beta} = 0.085$) are used.

$$G_S = \begin{cases} \left(\frac{E_{max}}{h_d}\right)^2 & \dots \left(0 \leq \frac{E_{max}}{h_d} \leq 1\right) \\ \frac{1}{3} + \frac{2}{3} \sqrt{3 \left(\frac{E_{max}}{h_d}\right)^2 - 2} & \dots \left(1 < \frac{E_{max}}{h_d}\right) \end{cases} \quad (13)$$

Figure 13 shows the comparison of the theoretical equation based on the run-up evaluation and the wave pressure test results expressed as the dimensionless maximum continuous wave forces (G_S).

The test results are scattered around the theoretical equation. Although there are some uncertainties, the maximum continuous wave forces can be estimated from the run-up evaluation using the evaluation method discussed in the previous paragraphs. The deviation value at overflow rate 3.3 is assumed as the underestimated value because test results overestimated the maximum wave velocity measured by the electromagnetic velocity gauges at the flume bottom.

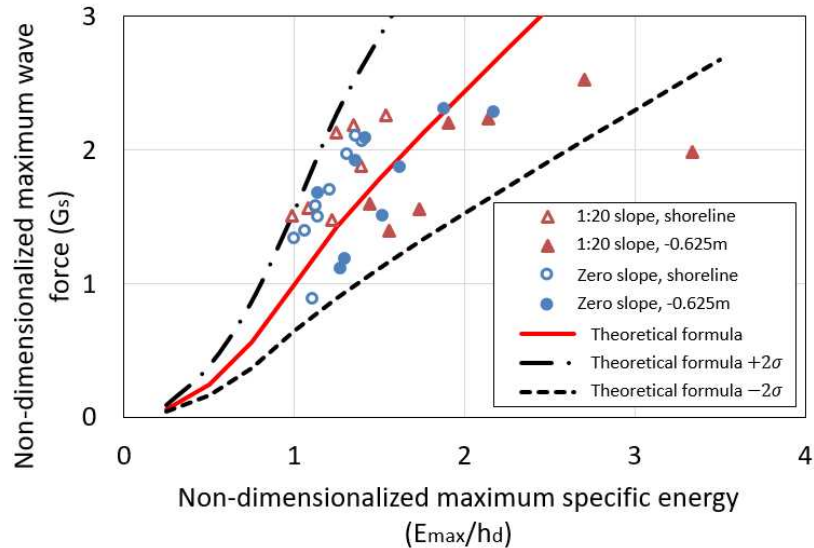


Figure 13. Relationship between the overflow rate and the dimensionless maximum continuous wave force.

CONCLUSIONS

In this study, to conduct the tsunami PRA appropriately, the systematic approach to evaluate the continuous wave force/pressure acting on the seawall installed at nuclear power plants is discussed. The equation to evaluate the maximum continuous wave force is theoretically derived from the study by Taisei Corporation (2018) and Oda et al. (2018), Ishida et al. (2016), Toriyama et al. (2018). The maximum continuous wave force can be evaluated by using the run-up evaluation results. Moreover, the systematic approach to evaluate the maximum continuous wave force is developed. The flow of evaluation is summarized as follows:

1. The run-up water depth and wave velocity at the seawall location is evaluated by the run-up test or analysis.
2. The maximum specific energy (E_{max}) at the seawall location and the overflow rate (E_{max}/h_d) from the seawall height (h_d) are evaluated.
3. The maximum continuous wave force without overflow (F_s) is evaluated, when the overflow rate (E_{max}/h_d) does not exceed 1.
4. The maximum continuous wave force with overflow (F'_s) is evaluated, when the overflow rate (E_{max}/h_d) exceeds 1.

Future challenges are to develop the evaluation of uncertainties included in the maximum continuous wave force.

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REFERENCES

- Asakura, R., K. Iwase, T. Ikeya, M. Takao, T. Kaneto, N. Fuji and M. Ohmori. 2002. The tsunami wave force acting on land structure, *Proceedings of 28th International Conference on Coastal Engineering*, ASCE, 1191-1202.
- Sakakiyama, T. 2012. Tsunami pressure on structures due to tsunami inundation flow, *Journal of JSCE B2 (Coastal Engineering)*, Vol68 No2, pp. I_771-I_775.
- Ikeya T. 2013. On the Hydraulic Mechanism of Sustained Tsunami Wave Pressure Acting on Land Structures, *Journal of JSCE B2 (Coastal Engineering)*, Vol69 No2, pp. I_816-I_820.
- Ishida N., Moritani H., Nakamura H., Iijima T. and Kawauchi H. 2015. Effects of Bore Pressure of Tsunami on Seawall, *NRA Technical Report Series*, NTEC-2015-4001, Regulatory Standard and Research Department Secretariat of Nuclear Regulation Authority (S/NRA/R). (in Japanese)

- Taisei Corporation. 2018. The hydraulic flume tests to develop the evaluation method for the Tsunami design and the fragility of the seawall (The effect of the wave pressure), *Proceedings of 36th Conference on Coastal Engineering*, pp. 7-11 – 7-12. (in Japanese)
- Oda Y., Honda T. and Omata T. 2018. EXPERIMENTAL STUDY ON TSUNAMI FORCES ACTING ON VERTICAL SEAWALLS UNDER OVERFLOW, *Proceedings of 36th Conference on Coastal Engineering*, structures. 37.
- Ishida N., Moritani H., Azuma K., Toriyama T. and Nakamura H. 2016. Water Depth Coefficients for Evaluating Tsunami Pressure on Seawall, *NRA Technical Report Series*, NTEC-2016-4001, Regulatory Standard and Research Department Secretariat of Nuclear Regulation Authority (S/NRA/R). (in Japanese)
- Toriyama T., Azuma K., Moritani H. and Ishida N. 2018. A METHOD FOR EVALUATING THE MAXIMUM TSUNAMI LOADINGS ON SEAWALLS, *The consignment research report in 29th year of the Heisei period*, structures. 40.