

TIME-DEPENDENT PERFORMANCE-BASED DESIGN OF CAISSON BREAKWATER CONSIDERING CLIMATE CHANGE IMPACTS

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A time-dependent performance-based analysis was conducted to analyze the influences of sea-level rise and wave-height increase due to climate change on caisson sliding of the breakwaters designed in different water depths. We used the Goda's spectral method to overcome the time-consuming problem in the calculation of the wave height at the breakwater site. In general, severe caisson sliding occurred when considering the climate change impacts. However, the influence of sea-level rise on the stability of caisson sliding is insignificant compared with that of wave-height increase. Especially, since the characteristics of caisson sliding are different depending on water depths, we have to establish countermeasure against these features for the design and maintenance of a caisson breakwater.

Keywords: caisson breakwater; caisson sliding; performance-based design; climate change impacts

INTRODUCTION

Vertical caisson breakwaters have been widely used since the monolithic caisson structures are effective in severe coastal environment. Among various failure modes of a vertical breakwater, the caisson sliding is dominant (Goda and Takagi 2000; Takahashi et al. 2000). Thus, in this study, only this failure mode is considered in the calculation of sliding distance. While various design methods have been developed for the structure, the performance-based design method has been recently adopted in the technical standard in Japan (OCDI 2009). This method is useful not only for the design process but also for the maintenance and operation because the displacement (i.e. sliding distance) and exceedance probability are calculated for the lifetime.

The vertical breakwater is influenced by climate change impacts such as sea-level rise and wave-height increase. Since the lifetime of most breakwaters is generally longer than several decades, the breakwater design should consider future coastal environments. Suh et al. (2012) conducted the performance-based design for the East Breakwater No. 4 cross-section at the Port of Hitachinaka in Japan. Because the cross-section was located in deepwater of 24.2 m (low water level), the sea-level rise rarely influenced the sliding of the caisson. Therefore, in this study, the breakwaters are fictitiously designed in various water depths both inside and outside surf zone and we analyze the influence of climate change impacts including sea-level rise. The water depth of wave breaking is located between 10.7 m and 13.4 m (mean sea level). The five cross-sections in water depths of 8, 12, 16, 20, and 25 m were designed by using a deterministic design method. The safety factor was used as 1.2 and both tides and storm surges were included to determine geometric variables.

CLIMATE CHANGE IMPACTS

In order to consider how the climate change impacts are related to the performance-based design of a breakwater, sea-level rise and wave-height increase are used. The prediction is made on the Pacific Ocean side of Japan (130-145°E, 25-40°) from 2000-2100 years. For future sea-level rise, the result of the A2 scenario of Mori et al. (2011) was used. Mori et al. (2011) estimated the sea-level rise using five different general circulation models with SRES (Special Report on Emission Scenarios) scenarios A1B and A2 of CIMP3 (Phase 3 of the Coupled Model Intercomparison Project). The mean value of sea-level rise calculated by A2 scenario in 2100 is 0.58 m and this value is slightly larger than the upper limit of the IPCC AR4 (Fourth Assessment Report) of 0.51 m. The mean value rapidly increases with time, and the more the mean value increases, the more the standard deviation increases. In other words, the uncertainty of sea-level rise increases with time as shown in Fig. 1.

Secondly, the extreme deepwater wave height distribution for future wave climate should be determined for the performance-based design. Suh et al. (2012) evaluated the Weibull distributions for the extreme wave height near the Port of Hitachinaka at the end of 20th and 21st centuries with the design variables given by Takata et al. (2003) as follows:

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$$F^*(H) = \left[1 - \exp \left\{ - \left(\frac{H - 4.65}{1.27} \right)^{1.0} \right\} \right]^{0.35} ; H_{50} = 8.30 \text{ m} \quad (1)$$

$$F^*(H) = \left[1 - \exp \left\{ - \left(\frac{H - 4.63}{1.72} \right)^{1.0} \right\} \right]^{0.46} ; H_{50} = 10.02 \text{ m} \quad (2)$$

where H_{50} is the significant wave height corresponding to the return period of 50 years.

However, it is difficult to predict how the waves will change during 100 years. Suh et al. (2012) assumed that the scale parameter $A(t)$ and the mean rate $\lambda(t)$ increase linearly or parabolically as follows:

$$A(t) = 1.27 + 0.0045t, \quad \lambda(t) = 0.35 + 0.0011t \quad (3)$$

$$A(t) = 1.27 + 4.5 \times 10^{-5} t^2, \quad \lambda(t) = 0.35 + 1.1 \times 10^{-5} t^2 \quad (4)$$

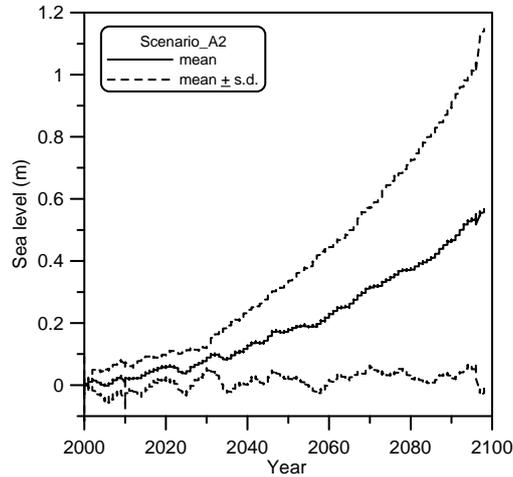


Figure 1. Temporal variation of projected sea-level rise on Pacific Ocean side of Japan by Scenario A2.

WAVES AT THE BREAKWATER SITE

When the deepwater wave height is determined, the wave height at the breakwater site should be calculated by using the wave transformation model including the wave breaking effect. In the performance-based design, computing time is very important because several thousands of simulations should be performed. To overcome this problem, a simple spectral method is used in this study because the depth-contour is almost straight and parallel to the coastline and the breakwater.

Goda's Spectral Method (Goda 1975; Goda and Suzuki 1975; Kweon and Goda 1996)

In the present study, a simplified method was developed as follows: (1) Goda's (1975) formula for wave height estimation within the surf zone, (2) Goda and Suzuki (1975) for the calculation of refraction coefficient by using the spectral method for irregular waves, (3) Kweon and Goda (1996) for the evaluation of non-linear shoaling coefficient by using the closure solution.

Goda (1975) proposed an approximate formula for nearshore wave height estimation including the surf zone as follows.

$$H_s = \begin{cases} K_s^{nl} H_0' & : h / L_0 \geq 0.2 \\ \min \{ (\beta_0 H_0' + \beta_1 h), \beta_{\max} H_0', K_s^{nl} H_0' \} & : h / L_0 < 0.2 \end{cases} \quad (5)$$

where H_s is the significant wave height at the breakwater site, H_0' is the equivalent deepwater

wave height corresponding to the significant wave height, K_s^{nl} is the nonlinear shoaling coefficient, h is the water depth, $L_0 (=1.56T_s^2)$ is the deepwater wave length, and $\beta_0, \beta_1, \beta_{\max}$ are the coefficients for approximate estimation of wave heights within the surf zone (Goda 1975).

In order to calculate the equivalent deepwater wave height $H_0' (=K_r^{\text{eff}} H_0)$ in Eq. (5), the refraction coefficient is needed. The refraction coefficient for irregular waves can be calculated by

$$K_r^{\text{eff}} = \left[\frac{1}{m_{s0}} \int_0^\infty \int_{\theta_{\min}}^{\theta_{\max}} S_0(f, \theta_0) K_s^2(f, h) K_r^2(f, h, \theta) d\theta_0 df \right]^{1/2} \quad (6)$$

where $m_{s0} = \int_0^\infty \int_{\theta_{\min}}^{\theta_{\max}} S_0(f, \theta_0) K_s^2(f, h) d\theta_0 df$, $S_0(f, \theta_0)$ is the deepwater directional wave spectrum, $K_s(f)$ is the shoaling coefficient for regular wave based on linear wave theory, $K_r(f, \theta_0)$ is the refraction coefficient for regular wave with frequency f and deepwater wave direction θ_0 and h is the water depth. In actual calculation, the refraction coefficient of irregular wave is calculated by

$$K_r^{\text{eff}} = \left[\sum_{i=1}^M \sum_{j=1}^N (\Delta E)_{ij} (K_r)_{ij}^2 \right]^{1/2} \quad (7)$$

where the term $(\Delta E)_{ij}$ is expressed as

$$(\Delta E)_{ij} = \frac{1}{m_0} \int_{f_i}^{f_i + \Delta f_i} \int_{\theta_j}^{\theta_j + \Delta \theta_j} S_0(f, \theta_0) d\theta_0 df \quad (8)$$

in which $m_0 = \int_0^\infty \int_{\theta_{\min}}^{\theta_{\max}} S_0(f, \theta_0) d\theta_0 df$. In this study, the deepwater frequency wave spectrum and directional spreading function were taken as the modified Bretschneider-Mitsuyasu spectrum (i.e. Pierson-Moskowitz spectrum) and Mitsuyasu-type, respectively. Mitsuyasu-type directional spreading function is given as follows.

$$G(f, \theta) = G_0 \cos^{2s} \left(\frac{\theta - \theta_p}{2} \right), \quad G_0 = \left[\int_{\theta_{\min}}^{\theta_{\max}} \cos^{2s} \left(\frac{\theta - \theta_p}{2} \right) d\theta \right]^{-1} \quad (9)$$

in which s is a parameter related to the frequency with the principal parameter $s_{\max} (=25)$ (Goda 2010). For the Goda's approximate method, the shoaling coefficient is required. Thus, we use the nonlinear shoaling coefficient proposed by Kweon and Goda (1996) with Iwagaki et al.'s (1981) data as follows:

$$K_s^{nl} = K_s + 0.0015 \left(\frac{h}{L_0} \right)^{-2.87} \left(\frac{H_0'}{L_0} \right)^{1.27} \quad (10)$$

Kweon and Goda (1996) employed this functional form and adjusted the coefficients to approximate the Shuto's theory (Shuto 1974). Because the water depth contours are straight and parallel to the coastline near the Port of Hitachinaka, Snell's law is used for wave direction at the breakwater site. Hereafter, this method is called Goda's spectral method.

Uncertainty of Wave Transformation Model

Most wave transformation models have uncertainty in the calculation of waves at the breakwater site. The estimation error of Goda's spectral method is evaluated by the comparison with laboratory

experiments. The mean value and standard deviation of the ratio between experimental data and calculated data are expressed as

$$E\left(\frac{H_s^E}{H_s^C}\right) = \frac{\mu_{H_s}}{H_s^C} = 1 + \alpha_{H_s} \quad (11)$$

$$\sqrt{\text{Var}\left(\frac{H_s^E}{H_s^C}\right)} = \frac{\sigma_{H_s}}{H_s^C} = \gamma_{H_s} \quad (12)$$

where H_s^E and H_s^C are the experimental and calculated significant wave height, respectively. α_{H_s} and γ_{H_s} are the bias and the coefficient of variation of wave model's uncertainty.

Because the bottom of coastal region near the Hitachinaka Port is almost plane and mild slope, Mase and Kirby's (1992) experimental data were used. However, this experiment was conducted at slightly steep slope of 1:20 so that it is not the same test condition as that of Hitachinaka Port. Figure 2 shows the relative frequency of the estimation error of Goda's spectral model. The histogram of this model followed the normal distribution where the mean and standard deviation of the ratio are 0.94 and 0.06, respectively. These statistics of estimation error are similar to those of Kweon et al.'s (1997) model (Hong et al. 2004). The biases of the two models are the same, but the standard deviation of Kweon et al.'s model is larger than that of Goda's spectral model. As mentioned above, Goda's spectral method included the Goda's (1975) approximation formula to consider depth-induced wave breaking inside surf zone. The approximation formula also has an uncertainty to predict the wave height in surf zone depending on the bottom slope. When the bottom slope is 0.1 with the wave steepness 0.02, this formula slightly overestimated the wave height, but when the bottom slope is 0.01 with the same wave steepness, the wave height was considerably underestimated (Goda 1975). The mean of the ratio is evaluated as 1.04 for the bottom slope of 0.01. As a result, for Goda's spectral method proposed in the present study, the bias and coefficient of variation are used as 0.0 and 0.1 respectively as shown in Table 1.

Figure 3 shows the significant wave height calculated by the spectral method at the location of the breakwater in water depth of 16 m. In this case, the tidal level and deepwater wave direction are 0.0 and 0^0 respectively. Since the significant wave height at the breakwater site is considerably smaller than the maximum design wave height, the expected sliding distance will be not large in this water depth.

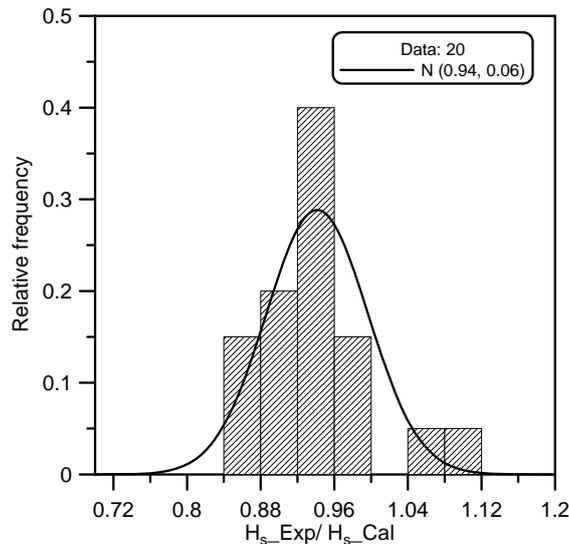


Figure 2. Relative frequency of ratio between the experimental data and calculated one by Goda's spectral method

Table 1. Uncertainties of wave transformation models			
Wave model	Bias(= α_{H_s})	CV(= γ_{H_s})	Remarks
Goda (1975)	-0.13	0.09	Unidirectional random waves normally incident to plane beach (Takayama and Ikeda1993)
Kweon et al. (1997)	-0.06	0.1	Unidirectional random waves normally incident to plane beach (Hong et al. 2004)
	-0.06 (truly, -0.04)	0.1	Directional random waves normally incident to plane beach or unidirectional waves with some principle wave direction or effect of variation of principle wave direction (Hong et al. 2004)
	0.0	0.1	Directional random waves (Hong et al. 2004)
Goda's approximation formula (1975)	+0.04	0.09	For bottom slope 1:100, the formula underestimated wave height compared with Goda (1975) model.
Goda's spectral method	-0.06	0.06	Mase and Kirby (1992), slope 1:20 Unidirectional random wave normally incident to plane beach (Numerical tests in Fig. 2)
Goda's spectral method (Present)	0.0	0.1	Directional random waves, the slope of Hitachinaka Port 1:100

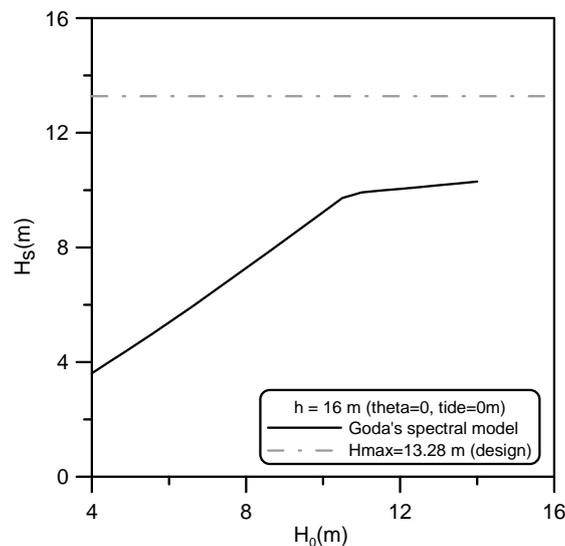


Figure 3. Significant wave height calculated by Goda's spectral method at the breakwater site in $h = 16$ m

BREAKWATERS DESIGNED BY DETERMINISTIC METHOD

In this study, the breakwaters are fictitiously designed in various water depths including both inside and outside surf zones in order to analyze the influence of climate change impacts. The water depth of surf zone is calculated by using the relationship between breaking depth d_b and breaker height H_b , proposed by U.S. Army (1984). The deepest breaking water depth is located between 10.7 m and 13.4 m below the mean sea level. Therefore, the breakwaters can be designed with five water depth conditions i.e., 8 m (inside surf zone), 12 m (wave-breaking zone), 16 m (outside surf zone), 20 m (outside surf zone), 25 m (far outside of surf zone).

The five cross-sections of breakwater were designed by using the deterministic design method of Goda and Takagi (2000) as shown in Table 2 and Fig. 4. The safety factor was used as 1.2 and

geometric variables included the astronomical and meteorological tides (storm surge, 10 % of deepwater wave height). H_{max} , H_s , T_s are respectively the maximum and significant wave height and wave period. h , h' , and d are the depths from sea bottom, caisson bottom, and foot protection block to H.W.L. (high water level) respectively, and h_c and B are the crest height and caisson width.

The significant wave height and period and geometric values are evaluated with the design tidal level (H.W.L. with $0.1H_0$). The elevation from L.W.L. (datum level) to the design tidal level is 2.33 m. The design deepwater significant wave height and wave period were 8.3 m and 14.0 s respectively, corresponding to the return period of 50 years without climate change impacts. Crest height is equal to $0.6 H_s$ above the design water level. Thickness of mound is taken as 20 % of the water depth, but at least the thickness should be more than 3.0 m (Goda and Takagi 2000). Height of foot-protection block and berm width are determined as 1.5 m and 8.0 m, respectively. The unit weight of upright section and sea water are 20.58 kN/m^3 and 10.09 kN/m^3 . Bottom slope is assumed as 1:100 as the same of existing breakwater. Figure 5 shows the design wave height and widths of caisson in different water depths with Table 2. The upper limitation of breaking height proposed by Goda (1974) is also included. We easily expected that the expected sliding distance will be small in water depths smaller than 16 m because the maximum wave height is limited by wave breaking.

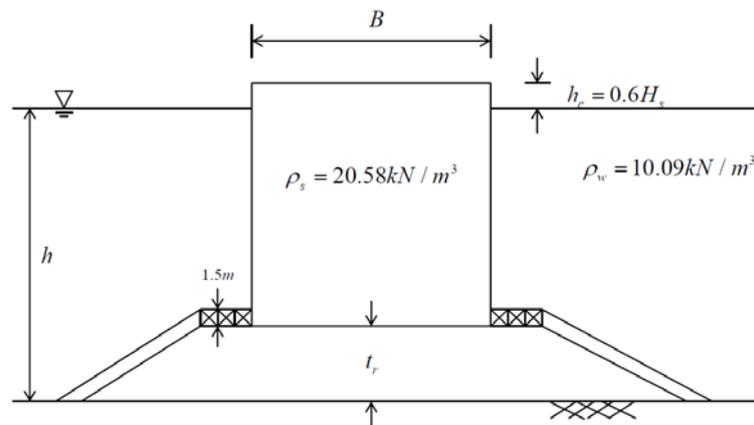


Figure 4. Typical cross-section of vertical breakwater

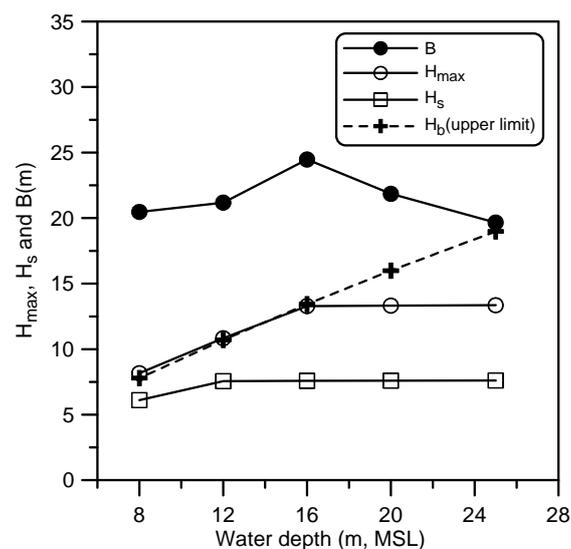


Figure 5. Design wave heights and widths of caisson in different water depths

h (MSL)	H_{max}	H_s	T_s	B	h	h'	d	h_c	Remarks
8	8.16	6.11	14.0	20.47	9.58	6.58	5.08	3.67	Inside surf zone
12	10.83	7.56	14.0	21.18	13.58	10.58	9.08	4.54	Wave-breaking zone
16	13.28	7.58	14.0	24.47	17.58	14.06	12.56	4.55	Outside surf zone
20	13.32	7.59	14.0	21.85	21.58	17.26	15.76	4.55	Outside surf zone
25	13.35	7.61	14.0	19.66	26.58	21.26	19.76	4.57	Existing structure

Description	X_i	α_{X_i}	γ_{X_i}	References
Offshore wave height	various	0.0	0.1	Shimosako and Takahashi (2000)
Significant wave period	various	0.0	0.12	Suh et al. (2010)
Wave transformation	various	0.0	0.1	Shimosako and Takahashi (2000)
Horizontal wave force	various	-0.09	0.19	Takayama and Ikeda (1993), Kim and Takayama (2003)
Vertical wave force	various	-0.23	0.20	Oumeraci et al. (2001)
Friction coefficient	0.6	0.06	0.16	Takayama and Ikeda (1993), Kim and Takayama (2003)

Case	Sea-level rise (SLR)	Wave-height increase (WHI)	Remarks
1	X	X	Nothing
2	O (A2)	X	SLR only
3-1	X	O (linear)	WHI only
3-2	X	O (parabolic)	WHI only

PERFORMANCE-BASED ANALYSIS

The vertical breakwaters designed by the deterministic method are analyzed by using the performance-based design method. With this method, the expected sliding distance and exceedance probability can be calculated for each breakwater. Sliding distance of caisson breakwater is calculated by using Shimosako and Takahashi's (2000) method considering time-dependent load variables (i.e. sea-level rise, deepwater wave-height increase) and the uncertainties of design variables in Table 3 were used. Total number of simulations is 50,000 and the expected sliding distance is calculated as the ensemble-average of the sliding distance in each simulation. Especially, Latin hypercube sampling method in the Monte-Carlo simulation was used to select the deepwater wave height in the calculation flow. Table 4 shows the test cases to examine the influence of climate change impacts. The primary aim in this study is to investigate the influence of sea-level rise (A2 scenario) in different water depths. Case 1 is the case in which no climate change impacts were considered. For Case 2 and 3, the influence of sea-level rise and wave-height increase are assessed separately.

Figure 6 shows the expected sliding distance at various water depths without consideration of climate change impacts. The expected sliding distance is very small when the water depth is less than 16 m as described in the previous section in relation to Fig. 5. In water depths deeper than 16 m, the expected sliding distance rapidly increases with water depth. This is because the maximum wave height is not limited by wave breaking in water depth deeper than 16 m.

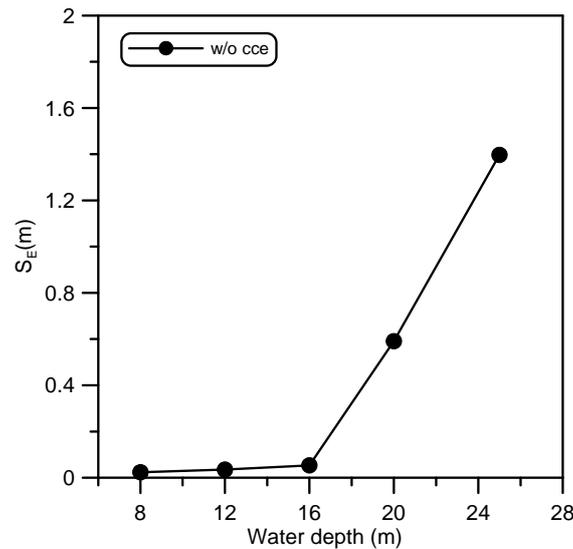


Figure 6. Expected sliding distance versus water depth without consideration of climate change impacts

h (MSL)	w/o	SLR	WHI (parabolic)	WHI (linear)	Safe or unsafe
8	0.023	0.026	0.028	0.032	Safe
12	0.035	0.048	0.047	0.065	Safe
16	0.053	0.066	0.087	0.123	Safe
20	0.591	0.664	0.963	1.451	Unsafe
25	1.397	1.432	2.167	3.214	Unsafe

RESULTS AND DISCUSSION

Figure 7 and Table 5 show the expected sliding distance calculated with or without consideration of climate change impacts for the breakwaters located in various water depths. The breakwaters located in water depths less than 16 m are assessed to be safe during the lifetime of 50 years regardless of climate change impacts because the maximum wave heights are limited by wave breaking. When the water depth is greater than 16 m, the expected sliding distance exceeds the allowable sliding distance of 0.3 m in all the conditions. Comparison between Case 1 and Case 2 shows that the influence of sea-level rise is negligible inside surf zone ($h = 8$ m) and far outside surf zone ($h = 20, 25$ m), while it is important near the breaker zone ($h = 12, 16$ m). Especially, in the area of $h = 12$ m, the influence of sea-level rise is almost the same as that of parabolic increase of wave height.

The influence of wave-height increase (Case 3-1 and 3-2) becomes more significant as the water depth increases. It is expected that the influence of wave height is negligible inside surf zone because large waves cannot reach the breakwater due to depth-limited wave breaking. Inside surf zone ($h = 8, 12$ m), however, the influence of wave-height increase is greater than that of sea-level rise, especially for the case of linear increase, probably because the waves of moderate height reach the breakwater without breaking.

In summary, outside surf zone, the influence of wave-height increase becomes more significant, while that of sea-level rise becomes negligible, as water depth increases. Inside surf zone, the influences of both wave-height increase and sea-level rise diminish as water depth decreases, but the influence of wave-height increase is greater than that of sea-level rise. Without showing the results, we just mention that the exceedance probability shows similar trends as the expected sliding distance.

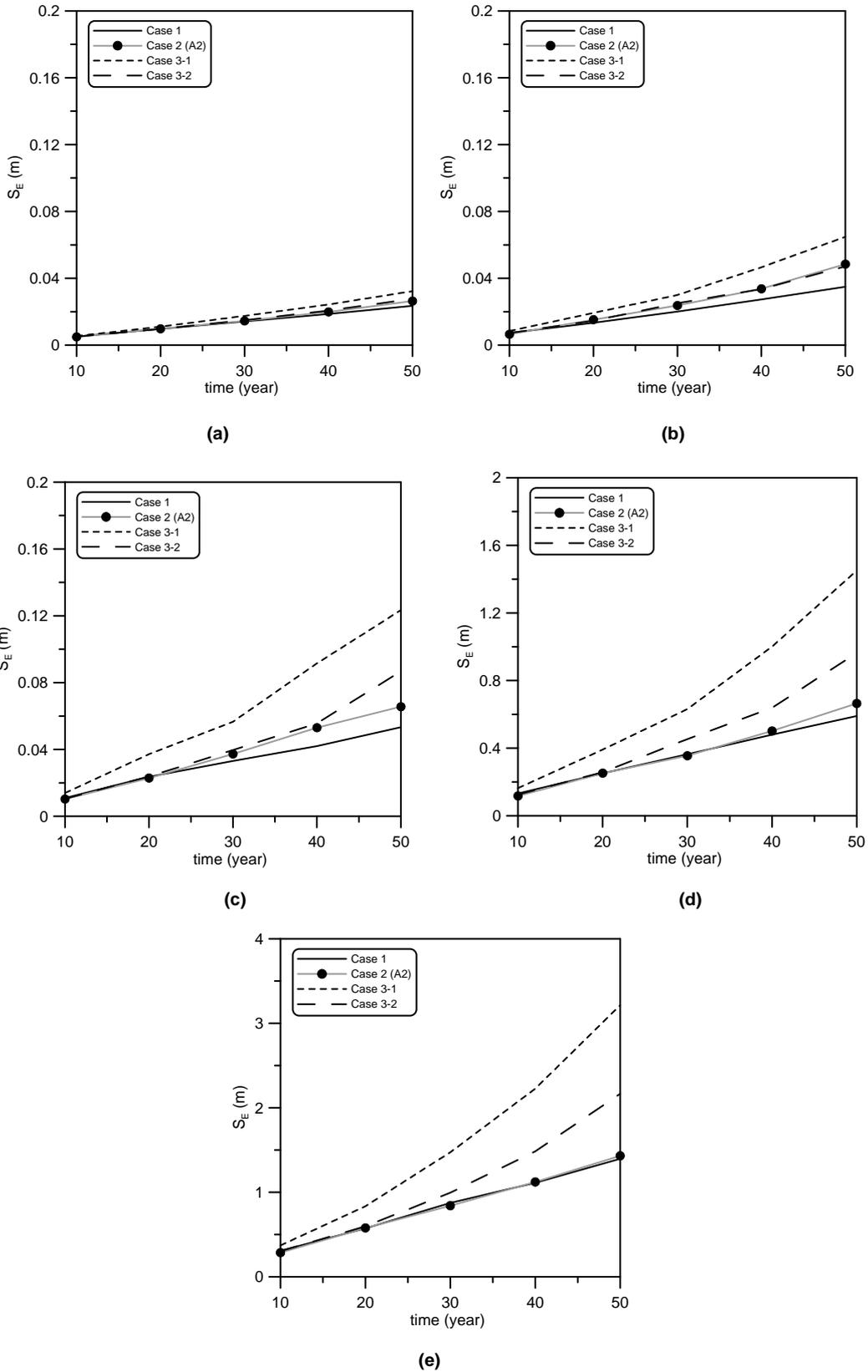


Figure 7. Expected sliding distance versus time at various water depths (MSL);
 (a) $h = 8$ m, (b) $h = 12$ m, (c) $h = 16$ m, (d) $h = 20$ m, (e) $h = 25$ m

CONCLUSION

We analyzed the influence of sea-level rise and wave-height increase due to climate change on the stability of caisson sliding of the breakwaters fictitiously designed in both inside and outside surf zones. In order to consider time-variant loads, a performance-based design method was improved. In the calculation process of this method including Monte-Carlo simulation, the Goda's spectral method as a wave transformation method was used to overcome a time-consuming problem. The sea-level rise rarely influenced the caisson sliding both inside surf zone and far outside surf zone. However, since the influence of wave-height increase is larger in deepwater, it is necessary that the design and the maintenance of breakwater is carefully conducted.

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