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Effects of Discount Rate and Various Costs on Optimal Design of Caisson Breakwater

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Abstract

In this study, the method developed by Goda and Takagi in 2000 for optimal design of a vertical breakwater caisson is extended to take into account the effects of discount rate and economic damage costs due to long-term harbor shutdown and temporal stoppage of harbor operation. The effect of discount rate is important only at smaller return periods where the damage to the caisson frequently occurs. Among the various costs, the initial construction cost and the economic damage cost due to long-term harbor shutdown caused by extraordinary sliding of caissons are found to be equally important in finding the minimum expected total lifetime cost. On the other hand, the rehabilitation cost and the economic damage cost due to temporal stoppage of harbor operation caused by excessive wave overtopping are not so important in the optimal design of the breakwater. In general, in smaller water depths the optimal return period and the corresponding optimal cross-section of the caisson are determined as those yielding the minimum expected total lifetime cost, while they are determined by the allowable expected sliding distance in greater water depths.

Keywords: *breakwaters, caissons, discount rate, economic damage cost, expected sliding*

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distance, expected total lifetime cost, optimal design

1. Introduction

Vertical breakwaters, along with rubble mound breakwaters, have been widely used to provide a calm basin for ships and to protect harbor facilities from rough seas, especially where the water depth is relatively large. The current deterministic design method is well established for not only the resistance of the upright section against sliding and overturning but also the bearing capacity of the rubble mound foundation and the seabed. In the deterministic design method, uncertainties in the magnitudes of loading on and resistance of the structure are supposed to be covered by a safety factor. Therefore, it is difficult to consider the uncertainties of each design parameter separately and to evaluate the relative importance of different failure modes, so that there is always a possibility to over- or under-design the structure. To overcome these shortcomings of the deterministic design, the reliability-based design method has been proposed. For a vertical breakwater, Burcharth and Sørensen (2000) established a partial safety factor system by summarizing the results of the PIANC (Permanent International Association of Navigation Congresses) Working Group 28, which was adopted by U.S. Army Corps of Engineers (2002) as well. This system belongs to what is called as Level 2 methods. On the other hand, performance-based design methods have also been developed, in which the expected sliding distance of a caisson of a vertical breakwater during its lifetime is estimated (Shimosako and Takahashi, 2000; Hong *et al*., 2004).

In order to properly apply the reliability-based design method, the probabilistic and statistical characteristics of the variables involved in the design should be known. The level of acceptable probability of failure (in Level 2 method) or expected sliding distance (in performance-based design method) of the caisson should also be specified a priori. A method being used to determine the acceptable probability of failure is to calculate the probability of failure of existing breakwaters that were constructed by the deterministic design method but did not suffer significant damage for a long time. Assuming that these breakwaters have enough reliability, the acceptable probability of failure may be taken as either the failure probability of the breakwaters or a somewhat smaller value. The same principle could be used for the acceptable expected sliding distance of a caisson. However, this method can be subjective in certain cases, because the sample breakwaters were designed and constructed neither in the same condition nor with the same safety level.

Recently, to cope with this problem, efforts are made to introduce the concept of optimization in the design of breakwaters, which considers their functionality and economics as well as their safety.

In the optimal design of a breakwater, it is designed so that the total lifetime cost (including initial construction cost, maintenance cost, rehabilitation and economic damage costs, and so on) becomes a minimum, while it fulfills a certain level of safety necessary for maintaining its functionality. In the case of a vertical breakwater, Burcharth *et al*. (1995) formulated a reliability-based design optimization procedure, in which the objective function models the cost of the caisson which is assumed to be proportional to the width of the caisson. The only design variable considered was the caisson width, though reliability analyses were made for sliding of the caisson, foundation failure in the rubble mound and rupture failure in clay. They assumed a specific water depth at the site. With the limited water depth it was not relevant with a high foundation as it would have caused larger wave impacts.

Voortman *et al*. (1998) proposed a more realistic procedure, in which the objective function consists of two parts that describe the construction costs and the expected costs of failure, respectively. Moreover, as design variables, the caisson height, the caisson width and the height of the rubble foundation were considered.

In the above-mentioned optimization studies, Level 2 methods were used in the reliability analyses. Recently, Goda and Takagi (2000) extended the performance-based design method of Shimosako and Takahashi (2000) by introducing the concept of the optimal return period for selection of design wave heights, and proposed a method to determine the optimal cross-section of a caisson that yields the minimum expected total lifetime cost within the allowable expected sliding distance. More recently, Burcharth and Sørensen (2006) used a similar approach by taking into account not only caisson sliding but also slip failure in rubble foundation and repair by placing mounds in front or behind the caisson.

In the present study, we extend Goda and Takagi’s (2000) model by taking into account the interest cost and the long-term change of the monetary values by inflation and others. Furthermore, the economic damage costs due to long-term harbor shutdown caused by extraordinary sliding of caissons and temporal stoppage of harbor operation due to excessive wave overtopping are also considered.

In the following section, the method for calculating the expected total lifetime cost is described. In Sec. 3, the procedure for determining the optimum cross-section of the caisson is described. In Sec. 4, some calculation examples are presented to examine the effects of discount rate, economic damage costs and water depth. In Sec. 5, sensitivity analyses are made for the discount rate and the criterion of caisson sliding distance for harbor shutdown. The major conclusions then follow.

2. Calculation of Expected Total Lifetime Cost

The total lifetime cost consists of the initial construction cost, rehabilitation cost, and economic damage costs due to long-term harbor shutdown or temporal stoppage of harbor operation. The maintenance costs are neglected in this paper because they are usually small compared with other costs. Many of the parameters for calculation of various costs are borrowed from Voortman (1998) with some modifications.

2.1 Initial Construction Cost

To calculate the initial construction cost, the length and cross-section of the breakwater and the costs of construction materials should be known. The total length of the breakwater is assumed to be 3,000 meters. The material costs used are 200 and 250 US$/m^3 for caisson and rubble mound, respectively, which are the values presently used in Korea.

2.2 Rehabilitation Cost

There are several modes of failure for vertical caisson breakwaters. Goda and Takagi (2000) examined the failure modes of the caisson breakwaters constructed in Japan over several tens of years, and concluded that the sliding of caissons comprises the majority of the cases of breakwater damage. Therefore, the sliding of the caisson is taken as the principal and only failure mode of vertical breakwaters in the present study. The Japanese breakwaters have low rubble mounds in general. The failure of rubble mound could also be important for caissons placed on high rubble mounds. Therefore, the present analysis and results may be limited to caissons placed on relatively low rubble mound over a strong seabed.

Goda and Takagi (2000) also proposed three simple models to estimate the rehabilitation cost, in which the rehabilitation cost increases with the sliding distance linearly, parabolically, or tangent-hyperbolically. Once the rehabilitation work starts, a great initial cost may be needed without regard to the sliding distance. Therefore, we adopted the third model as shown in Fig. 1, in which the cost increases rapidly with the sliding distance when the sliding distance is relatively small, and the rate of increase is reduced as the sliding distance increases. The model shown in Fig. 1 computes the rehabilitation cost, no matter how small the sliding distance is. In the present study, however, we introduced the threshold distance of 0.3 m, below which no rehabilitation work is made. The cost of rehabilitation normalized with the initial construction cost is then given by

$$C_r = \begin{cases} 0, & S \le 0.3\,\mathrm{m} \\ \tanh\left(3 \times \dfrac{S}{S_{MAX}}\right), & 0.3 < S \le S_{MAX} \\ 1, & S > S_{MAX} \end{cases} \tag{1}$$

where S is the accumulated sliding distance in meters, and S_{MAX} is the threshold sliding distance beyond which the caisson is judged as fallen from the mound and is given by

$$S_{MAX} = b + \frac{B}{2} \tag{2}$$

where b is the rear berm width of rubble mound foundation and B is the caisson width. Finally, the rehabilitation cost is calculated by multiplying the normalized cost C_r by the initial construction cost.

2.3 Economic Damage Cost Due to Harbor Shutdown

Even though the rehabilitation work should be made when the accumulated sliding distance exceeds 0.3 m, the breakwater still could maintain its function until the caisson slides much farther so that the harbor completely loses its function and is shutdown. In order to calculate the economic damage cost due to harbor shutdown, the accumulated sliding distance for harbor shutdown and the resulting economic cost should be determined.

Takahashi *et al*. (2001) defined the accumulated sliding distance of 1.0 m as the collapse limit state in which extremely large sliding occurs such that the caisson falls off the rubble mound foundation. This value could be used as the criterion of sliding distance for harbor shutdown. However, it seems somewhat unreasonable to use 1.0 m without regard to the caisson width. In the present study, therefore, we recommend to use $0.1S_{MAX}$ as the criterion for harbor shutdown. According to the results of the present study, in the case where the water depth is 20 m and the design significant wave height is 7~8 m, S_{MAX} is calculated to be about 15 m. Therefore, the sliding distance for harbor shutdown is 1.5 m, which is not much different from 1.0 m proposed by Takahashi *et al*. (2001) as the collapse limit state. The smaller the water depth and wave height become, the smaller becomes S_{MAX}, so that $0.1S_{MAX}$ will be closer to 1.0 m. S_{MAX} in Eq. (2) was derived only in the geometrical viewpoint by assuming that the rubble mound is rigid and the pressures on the front and rear sides of the caisson are the same as the hydrostatic pressure. In the real situation where waves act on the caisson which is mounted on a rubble mound of granular material, the caisson may fall from the mound at a sliding distance much smaller than S_{MAX}. Note that S_{MAX} used as the criterion for caisson fall in Eq. (2) is only to calculate the rehabilitation cost using Eq. (1).

To calculate the economic damage cost due to harbor shutdown, a socio-economic analysis should be made. In this study, however, we just adopted the value used by Vrijling *et al*. (2000), US$ 555 million per event. This value originated from a similar study for a rubble mound breakwater (Delft University of Technology, 1995).

2.4 Economic Damage Cost Due to Temporal Stoppage of Harbor Operation

When wave overtopping is so severe that the transmitted waves agitate the inside of the harbor beyond a certain level, the harbor operation will be stopped temporarily. The threshold wave heights for cargo handling should be determined in consideration of the type, size, and cargo handling characteristics of the vessels. Overseas Coastal Area Development Institute of Japan (2002) recommends the significant wave height of 0.5 m as the threshold wave height for cargo handling for medium- and large-sized vessels. In the present study, we adopted this value as the criterion of transmitted wave height for temporal stoppage of harbor operation.

The wave transmission coefficient due to wave overtopping is calculated by

$$C_t = \begin{cases} \sqrt{0.25\left(1-\sin\dfrac{\pi}{2(\alpha_1+\alpha_X)}\left[\dfrac{h_c}{H_i}+(\beta+\beta_X)\right]\right)^2+0.01\left(1-\left[\dfrac{d+d_c}{h}+\gamma_X\right]\right)^2} \\ \qquad\qquad \text{for } \beta+\beta_X-(\alpha_1+\alpha_X)<\dfrac{h}{H_i}<\alpha_1+\alpha_X-(\beta+\beta_X) \\ 0.01\left(1-\left[\dfrac{d+d_c}{h}+\gamma_X\right]\right) \qquad \text{for } \dfrac{h}{H_i}\geq\alpha_1+\alpha_X-(\beta+\beta_X) \end{cases} \tag{3}$$

where h_c is the crest elevation, h the water depth in front of the breakwater, d the depth above the armor layer of the rubble foundation, d_c the vertical distance from the bottom of the caisson to the top of the armor layer of the rubble foundation, and H_i is the incident wave height, which was taken as the daily maximum significant wave height in the present study. The preceding equation was proposed by Heijn (1997) by adding the new variables α_X, β_X, and γ_X to Goda's (1969) model so that it can be used for different types of caissons. We used $\alpha_X=-0.9$, $\beta_X=-0.34$, and $\gamma_X=0$ proposed for a conventional caisson. We also used $\alpha_1=2.2$ and the value of β given in the Goda's (1969) report as a function of d/h.

The wave height inside the harbor is calculated by multiplying the daily maximum significant wave height H_i by the wave transmission coefficient C_t. If the calculated wave height is greater than 0.5 m, it is assumed that the harbor operation is stopped on that day. This may lead to an overestimation of the stoppage time. However, such an assumption is used because the economic damage cost is given per day. The waves diffracted through the harbor entrance are neglected, and the transmitted wave over the structure is assumed to directly influence the vessels at anchor. The economic damage cost due to stoppage of harbor operation is taken as US$ 750,000 per day, again by following Vrijling *et al*. (2000).

2.5 Conversion to Present Value

The above-mentioned rehabilitation cost and economic damage costs are the costs which will occur in the future and in different times in general. The monetary value changes with time. Therefore it is necessary to convert the costs to those at a certain reference time. It is customary to take the time of initial construction cost as the present time and convert all the future costs to those in the present time. To convert the future monetary value to the present value, the future cost is multiplied by the present value interest factor, i.e.

$$PV = \sum_{n=1}^{L} \frac{C_n}{(1+r)^n} \tag{4}$$

where PV is the present value of the future costs, C_n the cost which will occur after n years, $1/(1+r)^n$ the present value interest factor, r the real discount rate, and L is the service lifetime of the breakwater. The real discount rate is calculated by

$$r = \frac{1+i}{1+j} - 1 \approx i - j \tag{5}$$

where i is the nominal discount rate, and j is the inflation rate. In principle, the nominal discount rate is taken as the interest rate of long-term treasury bonds, but the bank interest rate is often used when the share of the bonds is not so great as to dominate the rate of interest.

Different discount rates are used depending on the importance and service lifetime of structures; the longer service lifetime, the lower discount rate, in general. In the present study, the real discount rate of 3.7% was used, which was calculated using the average values of the inflation rates and interest rates provided by Korea National Statistical Office and the Bank of Korea, respectively, for seven years from 1999 till 2005. The service lifetime of the breakwater was taken as 50 years.

3. Procedure of Optimal Design

In this paper, fictitious design cases are considered. However, the deepwater wave condition is assumed to be the same as that used by Kim and Suh (2006) for Donghae

Harbor breakwater located in the east coast of Korea. The cumulative probability distribution of the extreme wave height is given by the Weibull distribution:

$$F(X) = 1 - \exp\left\{-\left(\frac{X - 3.037}{1.493}\right)^{1.1}\right\} \tag{6}$$

where X stands for the annual maximum significant wave height. The wave periods are calculated by the relationship proposed by Goda (2001) as $T_s = 3.3 H_{s0}{}^{0.63}$, where H_{s0} and T_s are the deepwater significant wave height and period, respectively. The significant wave heights for the return periods of 50 and 100 years are 8.2 and 9.0 m, respectively, and the corresponding wave periods are 12.4 and 13.2 s.

In order to calculate the economic damage cost due to temporal stoppage of harbor operation, the cumulative probability distribution of the ordinary wave heights should be known. For this, we used the wave data observed by Korea Ministry of Marine Affairs & Fisheries (2001) for one year of 2000 at Gangneung wave observation station, which is close to Donghae Harbor. We assume that the wave data observed in 2000 represent the average wave climate in this area and the distribution of daily wave heights is the same in other years. The histogram of the probability density of the observed daily maximum significant wave heights are shown in Fig. 2, along with the Gumbel distribution whose parameters were estimated using the method of moments:

$$F(x) = \exp\left\{-\exp\left[-\left(\frac{x - 0.7}{0.56}\right)\right]\right\} \tag{7}$$

where x indicates the daily maximum significant wave height.

The procedure for optimal design of the caisson is explained in conjunction with the computational flow chart sketched in Fig. 3. First, using the extreme wave height distribution function given by Eq. (6) and the nominal design wave height with the return period of 50 years, a preliminary design wave height is picked up. The preliminary wave height is varied over a certain range by choosing the return period in a range of 0.5 to 2.0

times the service lifetime. If the optimal caisson design is not obtained within this range of wave height, then the range is expanded until the optimum design is obtained.

The variation of water level by tides, η_t, is represented with a triangular distribution, which extends from LWL ($\eta_t = 0$) to HWL ($\eta_t = \Delta\eta$), where $\Delta\eta$ is the tidal range. The effect of storm surge is also taken into account by adding 10% of the deepwater wave height to the tide level.

Once the offshore wave height, wave period, and water level are determined, the significant wave height at the location of the breakwater is calculated. In the present study, we assumed unidirectional random waves propagating normal to the shoreline on a plane beach of slope of 1/100. For this, we used Kweon *et al*.'s (1997) wave transformation model by setting the directional spreading parameter $s_{\max}$ to be 1,000. The variability in principal wave direction was not considered. For the calculated wave height, the caisson is designed according to the conventional deterministic design method with the safety factor of 1.2, and the initial construction cost is calculated.

The designed caisson is then subjected to simulated daily maximum waves over one year as shown in Fig. 3(b) Subroutine A. For each simulated daily maximum wave, the transmitted wave height is calculated by Eq. (3). If the transmitted wave height is greater than 0.5 m, it is assumed that the harbor operation is stopped on that day. The number of these days is counted during the 365 days of calculation, and it is used for the estimation of economic damage cost due to temporal stoppage of harbor operation for one year. This procedure is repeated for L years to obtain the expected cost.

The designed cross section of caisson is also subjected to simulated yearly storm waves over L years. In general, the sliding of a caisson is caused by large waves comparable to the design waves. Therefore, the annual maximum wave height is considered sufficient to be incorporated into the calculation. For each simulated yearly storm, the total sliding distance is calculated. The process of this calculation is the same as that of Hong *et al*. (2004) and briefly represented as Fig. 3(c) Subroutine B. In the figure, T is the wave period of an individual wave and τ is the storm duration, which was taken as 2 hours in this study. Goda and Takagi (2000) evaluated the rehabilitation cost on the basis of accumulated sliding distance for L years, because they did not take into account the discount rate and the economic damage cost due to harbor shutdown. In order to take these into consideration, however, it is important to find when the rehabilitation work is needed and when the harbor shutdown occurs. In the present study, the values of total sliding distance by yearly storms are accumulated, and if the accumulated value becomes greater than 0.3 m, the rehabilitation cost is calculated by Eq. (1) and the sliding distance is reset to zero. If the accumulated sliding distance is greater than $0.1S_{MAX}$, both the rehabilitation cost and economic damage cost due to harbor shutdown are calculated.

The process of L-year cycles is repeated 1,000 times, and the total lifetime costs and accumulated sliding distances thus obtained are added together to yield the expected values of total cost and sliding distance. This process is repeated for a number of return periods of different design wave heights. Finally, the cross-section of the caisson that yields the minimum expected total lifetime cost within the allowable expected sliding distance is searched. The corresponding return period is then the optimal return period.

To reduce the number of simulations (i.e. 1,000 times) in Fig. 3(a), the Latin hypercube sampling technique (McKay *et al*., 1979) was used. To briefly explain this technique, consider a variable Y that is a function of other random variables $X_1, X_2, \cdots, X_k$. The Latin hypercube sampling selects L different values from each of k variables $X_1, X_2, \cdots, X_k$ in the following manner. The range of each variable is divided into L non-overlapping intervals on the basis of equal probability. One value from each interval is selected at random with respect to the probability density in the interval. The L values thus obtained for X_1 are paired in a random manner (equally likely combinations) with the L values of X_2. These L pairs are combined in a random manner with the L values of X_3 to form L triplets, and so on, until L k-tuplets are formed. This is the Latin hypercube sample. The Latin hypercube sampling can be used when the variables are independent. In the present study, therefore, only the annual maximum wave height, friction coefficient between caisson and rubble mound, and the water level were sampled using the Latin hypercube sampling, while the conventional random sampling was used for other variables, in the calculation of sliding distance due to yearly storm waves. On the other hand, the Latin hypercube sampling was used for the daily maximum wave height and water level in the calculation of transmitted wave heights due to wave overtopping.

Fig. 4 shows the relation between the number of simulations and the expected damage cost (due to harbor shutdown and temporal stoppage of harbor operation) calculated with the Latin hypercube sampling and conventional random sampling. The design wave height of return period of 50 years and the water depth of 19 m at the design site were used. In Fig. 4 are shown the maximum, minimum, and average values of 10 calculation results of the expected damage cost obtained for the respective number of simulations along with the standard deviation. Assuming that the coefficient of variation (i.e. standard deviation divided by average) must be less than 0.1, about 1,000 simulations are enough if the Latin hypercube sampling is used, but the conventional approach shows the coefficient of variation greater than 0.2 even for the number of simulations of 10,000.

In Table 1 are given the design factors employed in the present study and their statistical characteristics, which were obtained based on Goda and Takagi (2000) and Hong *et al*. (2004).

The procedure explained above can be mathematically expressed as minimizing the cost function given by

$$I(\vec{z}) = I_0(\vec{z}) + \sum_{m=1}^{M}\left(\sum_{n=1}^{L}\left(\frac{C_R{}^{(n)}FN_R{}^{(n)}(\vec{z}) + C_S FN_S{}^{(n)}(\vec{z}) + C_O FN_O{}^{(n)}(\vec{z})}{(1+r)^n}\right)\right)\Bigg/ M \quad (8)$$

where $\vec{z}$ is the vector of design variables (crest elevation and caisson width), $I(\vec{z})$ expected total lifetime cost, $I_0(\vec{z})$ initial construction cost, $C_R{}^{(n)}$ rehabilitation cost in the *n*-th year, C_S economic damage cost due to harbor shutdown per event, C_O economic damage cost due to temporal stoppage of harbor operation per day, $FN_R{}^{(n)}(\vec{z})$ number of rehabilitation in the *n*-th year (0 or 1), $FN_S{}^{(n)}(\vec{z})$ number of harbor shutdown in the *n*-th year (0 or 1), $FN_O{}^{(n)}(\vec{z})$ number of days of stoppage of harbor operation in the *n*-th year, and M is the number of simulations (1,000 in the present study).

4. Illustrative Examples

In principle, the economic optimization of a breakwater is performed in such a way that the total lifetime cost is minimized. However, there can be a case in which the breakwater is damaged too many times during its lifetime if the breakwater is designed by the economic optimization principle alone. Therefore, we have to satisfy both the economic optimization and the condition to keep the expected damage below a tolerable limit. In this study, the optimal cross-section of a caisson is defined as the cross-section which yields the minimum expected total lifetime cost within the allowable expected sliding distance by following Goda and Takagi (2000). In the case where the point of minimum cost is not found within the allowable sliding distance, the cross-section at the allowable expected sliding distance is defined as the optimal cross-section. The allowable expected sliding distance has been proposed as 0.3 and 0.1 m respectively by Shimosako and Takahashi (2000) and Goda and Takagi (2000). In the present study, both of these are examined.

The breakwater is assumed to be installed parallel to the shoreline on a plane beach of slope of 1/100. Constant values were used for the height of the rubble mound from the seabed to the bottom of the caisson of 2.5 m, the height of foot-protection block and armor layer of the mound of 1.5 m, and the front and rear berm widths of the mound of 10 and 7 m, respectively. The crest elevation of the caisson was taken as $h_c = 0.6H_s$, where H_s is the design significant wave height at the location of the breakwater.

4.1 Effect of Discount Rate

In order to examine the effect of the discount rate, we included only the initial construction cost and the rehabilitation cost with or without consideration of the discount rate in the calculation, neglecting the economic damage costs. Fig. 5 compares the expected total lifetime costs with respect to the return period calculated with and without consideration of the discount rate in water depth of 19 m. The right ordinate indicates the expected sliding distance. The difference becomes undistinguishable as the return period increases because the cross-section of the caisson enlarges with the return period so that little damage occurs and consequently the effect of the discount rate disappears. The return period of the minimum total cost is 10 years in both cases, but the corresponding expected sliding distance is much greater than the allowable ones.

4.2 Effects of Economic Damage Costs

Fig. 6 shows the expected total lifetime costs calculated by including (1) only the rehabilitation cost, (2) the rehabilitation cost and the economic damage cost due to only temporal stoppage of harbor operation, and (3) the rehabilitation cost and the economic damage costs due to both temporal stoppage of harbor operation and long-term harbor shutdown. Of course, the initial construction cost was also included and the discount rate was considered in all the cases. The result of the first case must be the same as that calculated with consideration of the discount rate in Fig. 5. When the economic damage cost due to temporal stoppage of harbor operation is included in addition to the rehabilitation cost, its effect is not so significant, though the total cost somewhat increases at the smaller return periods where the crest elevation is relatively low to permit severe wave overtopping. The return period of the minimum total cost is again 10 years, being the same as that in the case when only the rehabilitation cost is included.

The effect of economic damage cost due to long-term harbor shutdown is so significant that the expected total lifetime cost increases largely especially at smaller return periods. The return period of the minimum total cost is about 25 years, and the corresponding expected sliding distance is smaller than the allowable values of 0.3 and 0.1 m. Therefore, the optimum cross-section of the caisson should be determined as that designed with the wave height of the return period of 25 years. The characteristics of the design parameters and the resulting caisson design are listed in Table 2 along with the conventional design. In the new design, the initial construction cost is reduced by 14% compared with the conventional design.

4.3 Effect of Water Depth

Fig. 7 shows the result in water depth of 14 m. The minimum total cost occurs at the return period of 10 years, and the corresponding expected sliding distance is smaller than the allowable values of 0.3 and 0.1 m. Therefore, the optimum cross-section of the caisson should be determined as that designed with the wave height of the return period of 10 years. The comparison of the design parameters between conventional and new design methods is given in Table 3. In the new design, the initial construction cost is reduced by 23% compared with the conventional design.

Fig. 8 shows the result in water depth of 24 m. In this case, the minimum total lifetime cost occurs at the return period of about 35 years, but the corresponding expected sliding distance is about 0.45 m, which exceeds the allowable value of 0.3 m or 0.1 m. Therefore, the optimum cross-section of the caisson should be determined as that designed with the wave height of the return period corresponding to the allowable expected sliding distance. The comparison of the design parameters between conventional and new design methods is given in Table 4. In the new design, the initial construction cost is reduced by 6% if the allowable expected sliding distance is taken as 0.3 m, while it increases by 7% if the allowable expected sliding distance is 0.1 m.

The comparison of Figs. 6 to 8 shows that in smaller water depths the optimal return period and the corresponding optimal cross-section of the caisson are determined as those yielding the minimum expected total lifetime cost, while they are determined by the allowable expected sliding distance in greater water depths.

5. Sensitivity Analyses

In this section, sensitivity analyses are made for the discount rate and the criterion of caisson sliding distance for harbor shutdown. The variation of each cost with respect to the return period is also examined to see their relative importance.

5.1 Discount Rate

Fig. 9 shows the change of the expected total lifetime cost with the discount rate in water depth of 19 m. The expected total cost decreases as the discount rate increases. The effect of the discount rate disappears at larger return periods where the caisson is so stable that little damage occurs. The return period of the minimum total cost also decreases as the discount rate increases.

5.2 Criterion of Caisson Sliding Distance for Harbor Shutdown

The computations in Section 4 have been made with the criterion of $0.1S_{MAX}$ for harbor shutdown due to excessive sliding of caissons. However, this criterion will depend on the situations of each harbor and the subjective judgments of harbor authorities. Fig. 10 to 12 show the expected total lifetime costs calculated with different criteria of sliding distance for harbor shutdown in water depths of 14, 19, and 24 m, respectively. The expected total cost decreases as the criterion of sliding distance for harbor shutdown increases, especially at smaller return periods. The return period of the minimum total cost also decreases as the criterion increases. The effects of this criterion propagate further towards the larger return periods as the water depth increases.

Nevertheless, the optimal return period is determined by the allowable expected sliding distance in deeper waters, because the expected sliding distance is large there.

5.3 Comparison of Importance of Costs

To examine the relative importance of the various costs, the expected value of each cost is plotted as a function of return period in Fig. 13. The water depth is 19 m. The discount rate of 3.7% and the criterion of $0.1S_{MAX}$ for harbor shutdown were used. As expected, the initial construction cost increases with the return period, while other costs decrease with the return period. The economic damage cost due to temporal stoppage of harbor operation is the smallest. The rehabilitation cost is somewhat larger than this, but its effect on the total cost is still not significant. The initial construction cost and the economic damage cost due to long-term harbor shutdown are found to be equally important to determine the optimal return period and optimal cross-section of the caisson.

6. Conclusions

In this study, the method developed by Goda and Takagi (2000) for optimal design of a vertical caisson breakwater was extended to take into account the effects of discount rate and economic damage costs due to long-term harbor shutdown and temporal stoppage of harbor operation. The Latin hypercube sampling technique was used for some variables to obtain statistically more reliable results within reduced computing time.

The effect of discount rate is important only at smaller return periods where the damage to the caisson frequently occurs. Among the various costs, the initial construction cost and the economic damage cost due to long-term harbor shutdown caused by extraordinary sliding of caissons were found to be equally important in finding the minimum expected total lifetime cost. On the other hand, the rehabilitation cost and the economic damage cost due to temporal stoppage of harbor operation caused by excessive wave overtopping were not so significant in the optimal design of the breakwater. Therefore, a detailed economic analysis should be made for the initial construction cost and the economic damage cost due to harbor shutdown to apply the present study to a real situation. The economic damage costs due to long-term harbor shutdown or temporal stoppage of harbor operation may depend on the extent of a harbor; in general, the larger harbor extent, the larger costs. The values adopted in this study were estimated for a harbor of relatively large extent. An assessment of these costs may be necessary for a smaller harbor.

In general, in smaller water depths the optimal return period and the corresponding optimal cross-section of the caisson are determined as those yielding the minimum expected total lifetime cost, while they are determined by the allowable expected sliding distance in greater water depths. However, more detailed investigation may be necessary because only three specific water depths and one long-term wave statistics were used in this study.

In the present study, the initial costs not directly related to the construction, maintenance costs, and the dismantling costs were not included, which should be considered for more reliable design. Finally, though only the sliding failure of the caisson was considered in the present study, the overturning failure could be important especially when the return period is so small that the caisson width is small too. In this case, it is probable to design the caisson with less sand fill instead of reducing the caisson width. Then the initial construction cost and the optimal cross-section of the caisson may change. These should be considered to use the present method in the real situation.

Acknowledgements

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Table 1. Statistical characteristics of design factors

Design factor	Bias	Coefficient of variation	Distribution function	Remarks
Daily maximum wave height	0.0	0.15	Normal	Mean by Gumbel dist.
Offshore wave height	0.0	0.1	Normal	Mean by Weibull dist.
Water level	-tide amplit.	*	Triangular	-
Wave deformation	-0.06	0.1	Normal	-
Friction coefficient	0.06	0.1	Normal	$\mu = 0.6$ as the base
Individual wave height	*	*	Rayleigh	2 hours duration
Wave forces	-0.09	0.1	Normal	-
Storm surge	0.0	0.1	-	Standard is 10% of offshore wave height
Significant wave period	0.0	0.1	Normal	-
Period for an individual wave	0.0	0.1	Normal	-

Table 2. Comparison of caisson cross-section and initial construction cost between conventional and new methods in water depth of 19 m

Item	Conventional	New
Return period (year)	50	25
Offshore wave height $(H_{1/3})_0$ (m)	8.20	7.36
Local wave height $H_{1/3}$ (m)	8.13	7.17
Crest height h_c (m)	4.88	4.30
Caisson width B (m)	19.01	16.45
Initial construction cost (US $)	1.47×10^8	1.26×10^8

Table 3. Comparison of caisson cross-section and initial construction cost between conventional and new methods in water depth of 14 m

Item	Conventional	New
Return period (year)	50	10
Offshore wave height $\left(H_{1/3}\right)_0$ (m)	8.20	6.22
Local wave height $H_{1/3}$ (m)	7.68	6.14
Crest height h_c (m)	4.61	3.69
Caisson width B (m)	18.12	14.32
Initial construction cost (US $)	1.24×10^{8}	9.53×10^{7}

Table 4. Comparison of caisson cross-section and initial construction cost between conventional and new methods in water depth of 24 m

Item	Conventional	New	
		$S_E = 0.3$ m	$S_E = 0.1$ m
Return period (year)	50	40	66
Offshore wave height $(H_{1/3})_0$ (m)	8.20	7.93	8.53
Local wave height $H_{1/3}$ (m)	7.92	7.63	8.27
Crest height h_c (m)	4.75	4.58	4.96
Caisson width B (m)	17.50	16.53	18.74
Initial construction cost (US $)	1.51×10^8	1.42×10^8	1.62×10^8

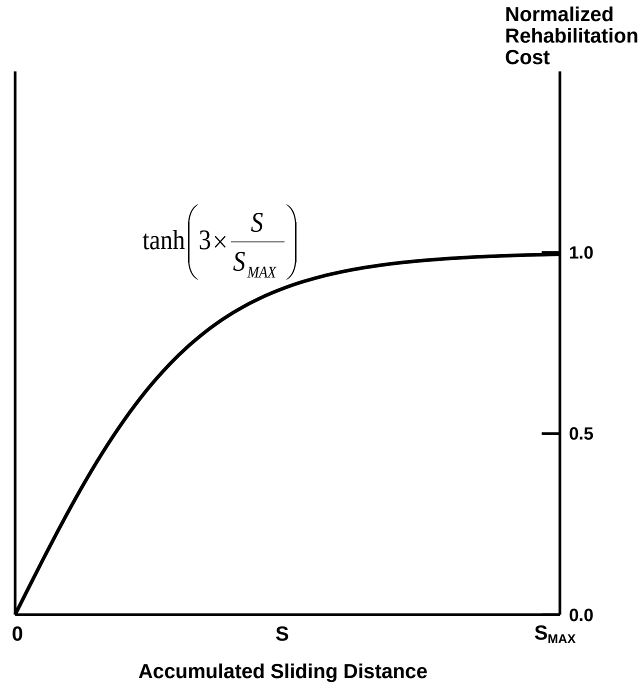


Fig. 1. A model for estimating rehabilitation cost as a function of total sliding distance (after Goda and Takagi, 2000)

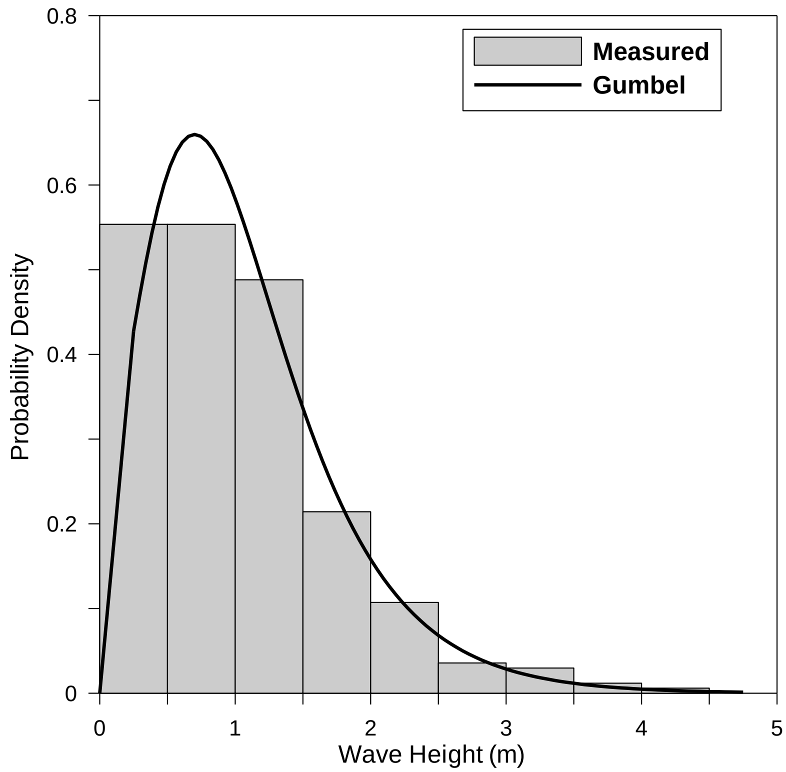
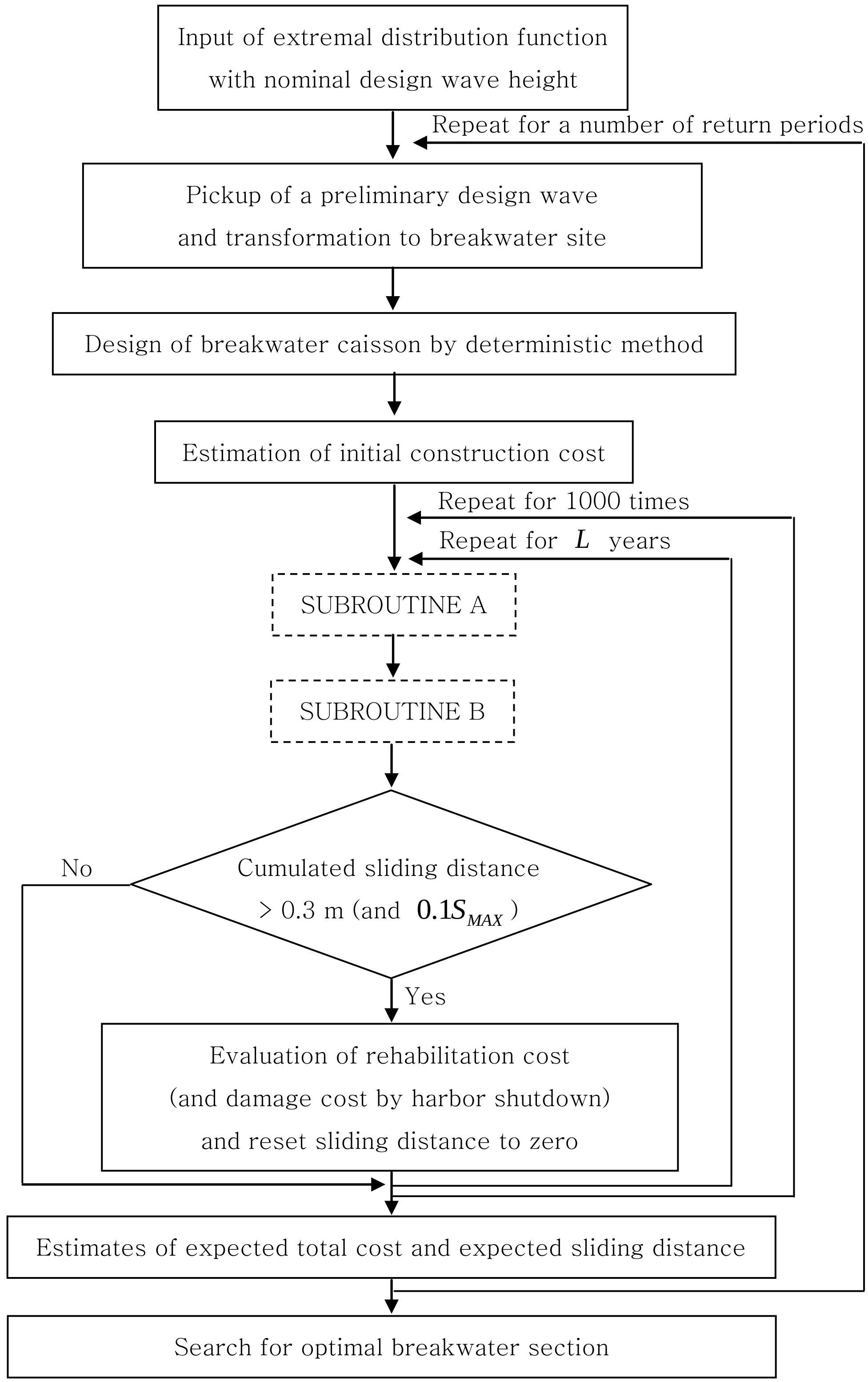
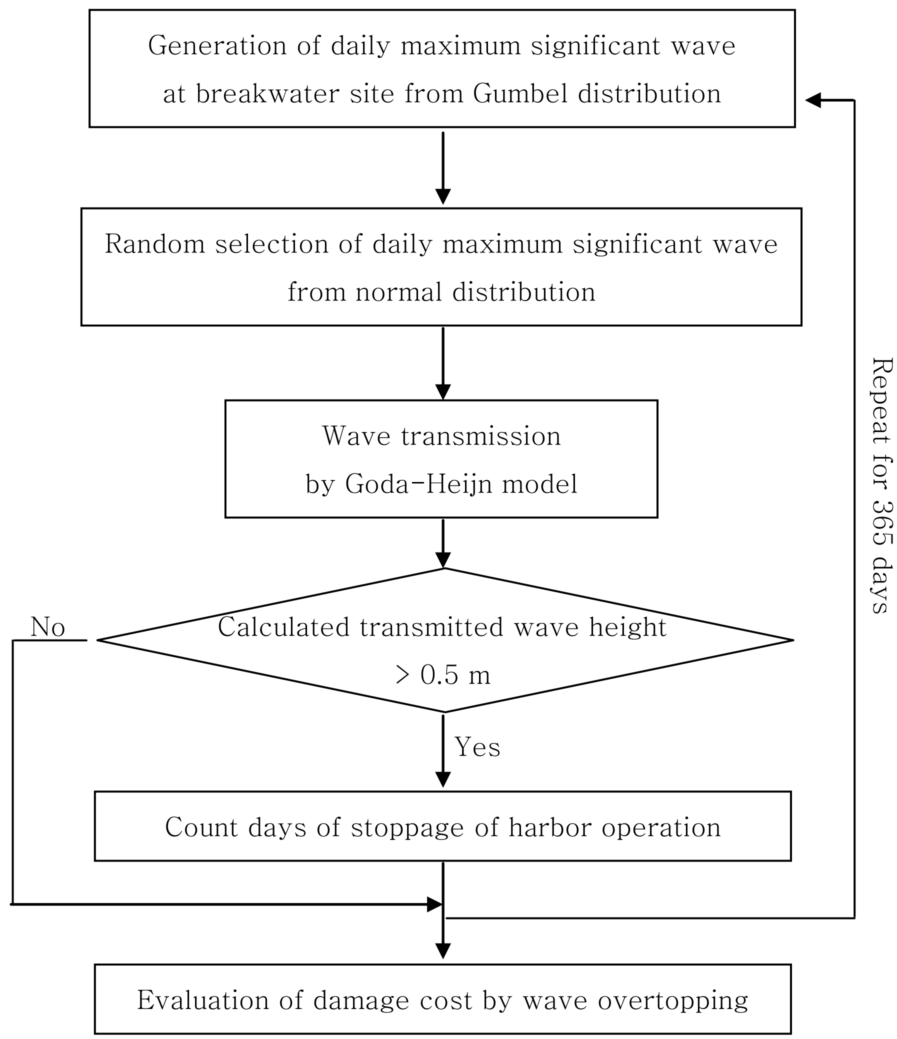


Fig. 2. Comparison between Gumbel distribution and histogram of probability density of observed daily maximum significant wage heights


Input of extremal distribution function
with nominal design wave height
Repeat for a number of return periods
Pickup of a preliminary design wave
and transformation to breakwater site
Design of breakwater caisson by deterministic method
Estimation of initial construction cost
Repeat for 1000 times
Repeat for L years
SUBROUTINE A
SUBROUTINE B
No
Cumulated sliding distance
> 0.3 m (and $0.1S_{MAX}$)
Yes
Evaluation of rehabilitation cost
(and damage cost by harbor shutdown)
and reset sliding distance to zero
Estimates of expected total cost and expected sliding distance
Search for optimal breakwater section

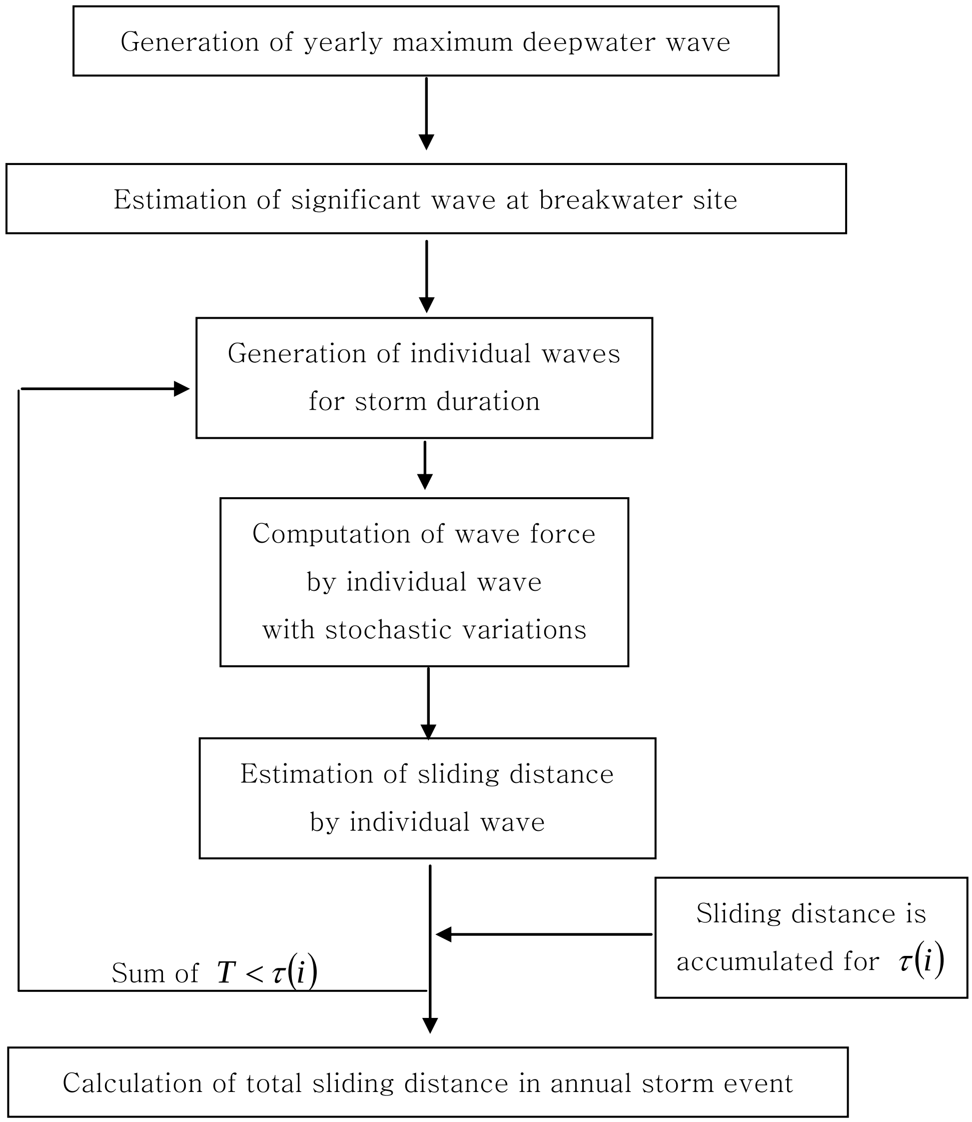
(a) Computational flow

Fig. 3. Computational procedure for optimal breakwater design



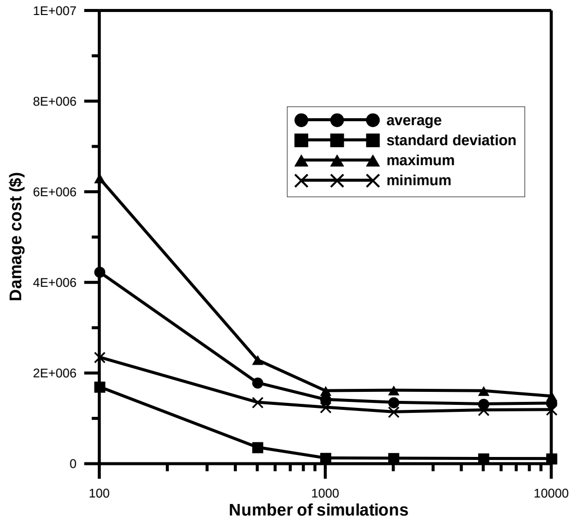
(b) Subroutine A

Fig. 3 (Continued)

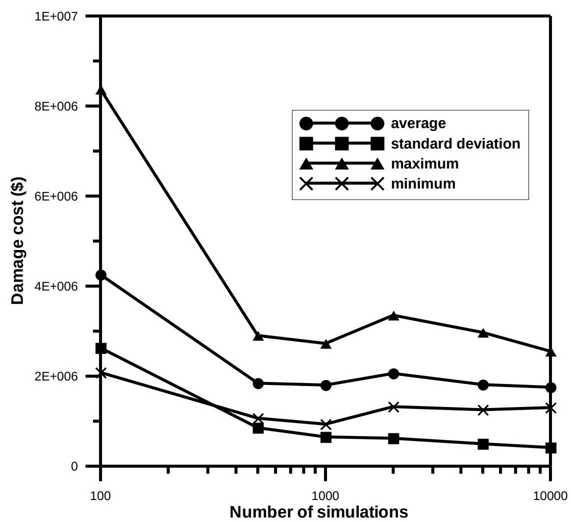


(c) Subroutine B

Fig. 3 (Continued)



(a)



(b)

Fig. 4. Calculated economic damage cost versus number of simulations: (a) Latin hypercube sampling, (b) Conventional random sampling

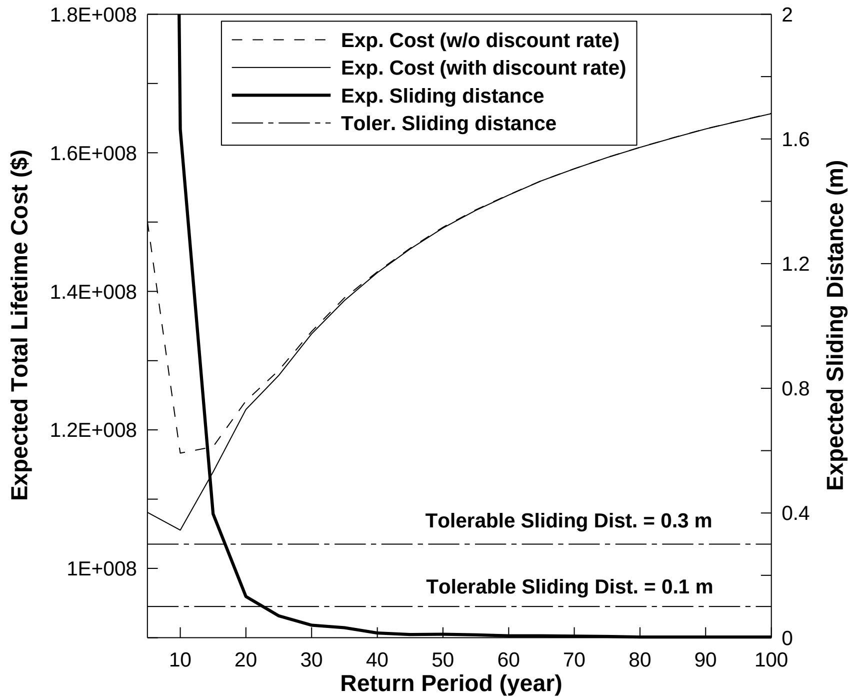


Fig. 5. Curves of expected sliding distance and expected total lifetime costs computed by including only initial construction cost and rehabilitation cost with and without consideration of discount rate in water depth of 19 m

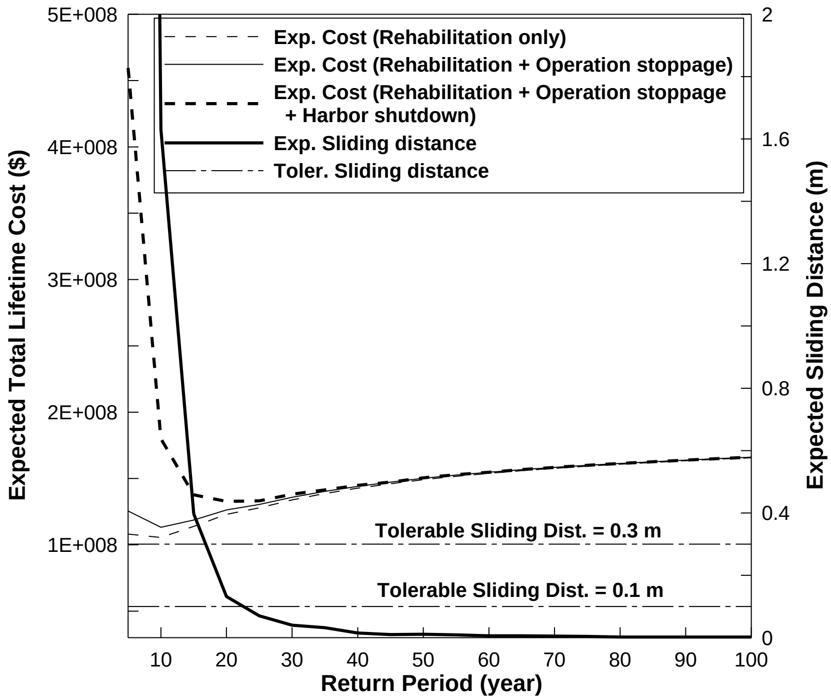


Fig. 6. Curves of expected total lifetime costs for showing the effects of economic damage costs due to temporal stoppage of harbor operation and long-term harbor shutdown in water depth of 19 m

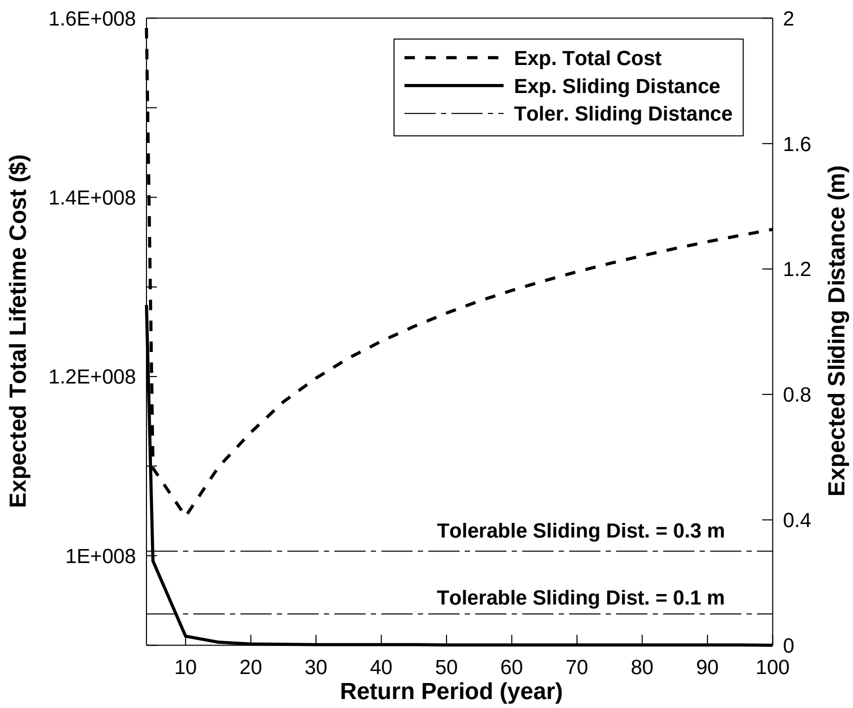


Fig. 7. Curves of expected sliding distance and expected total lifetime cost in water depth of 14 m

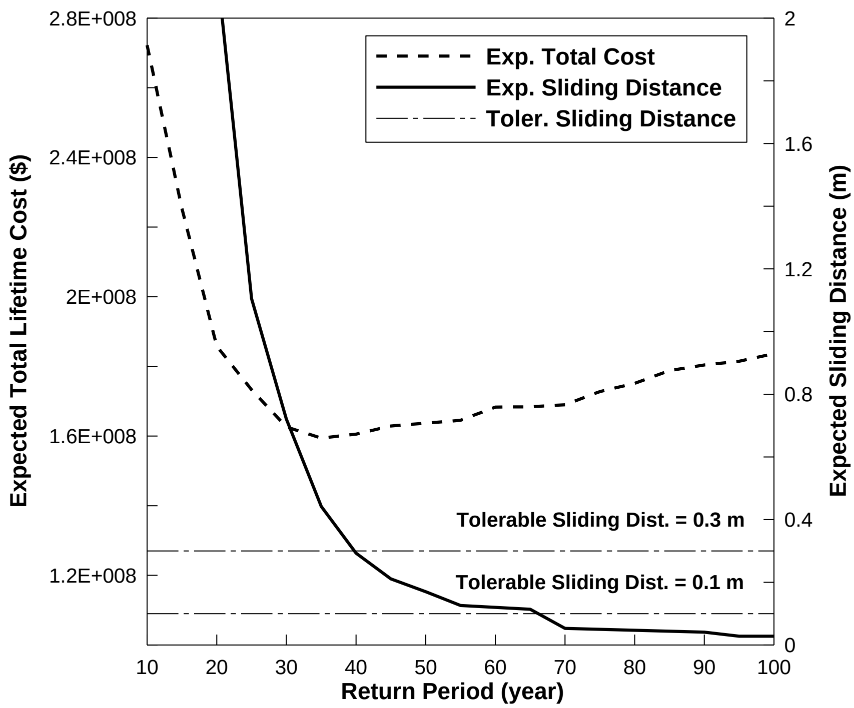


Fig. 8. Curves of expected sliding distance and expected total lifetime cost in water depth of 24 m

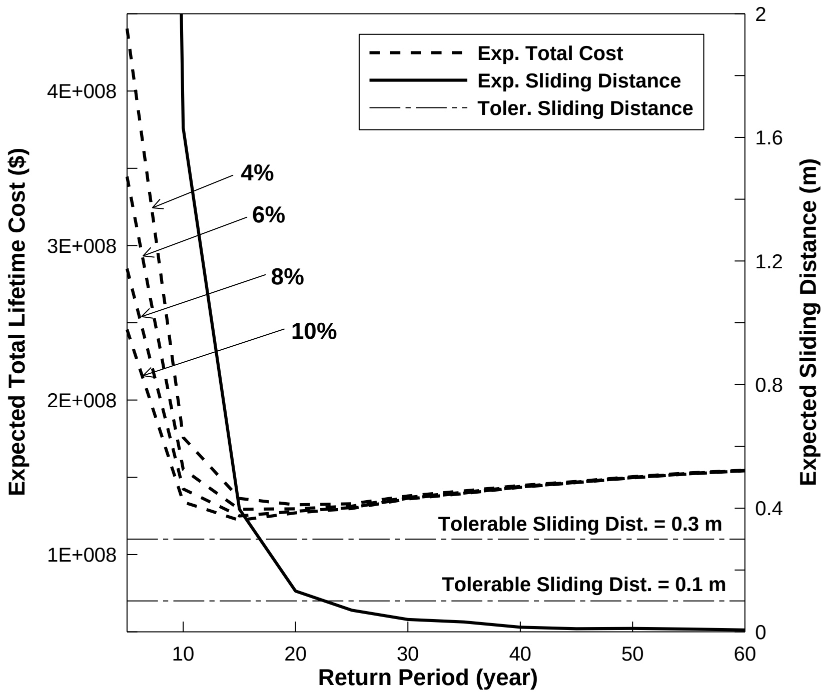


Fig. 9. Expected total lifetime costs calculated with different discount rates in water depth of 19 m

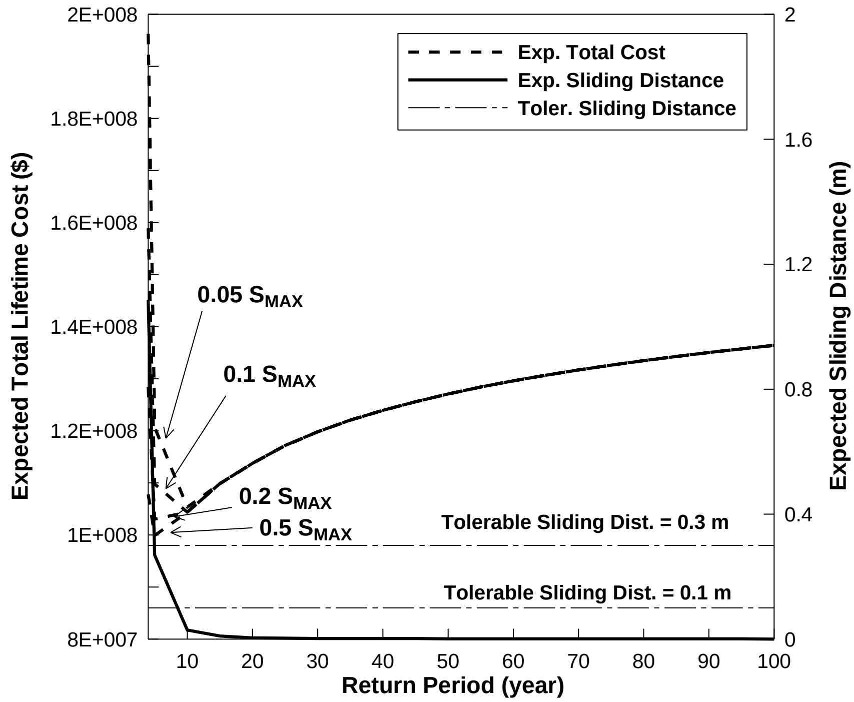


Fig. 10. Expected total lifetime costs calculated with different criteria of caisson sliding distance for harbor shutdown in water depth of 14 m

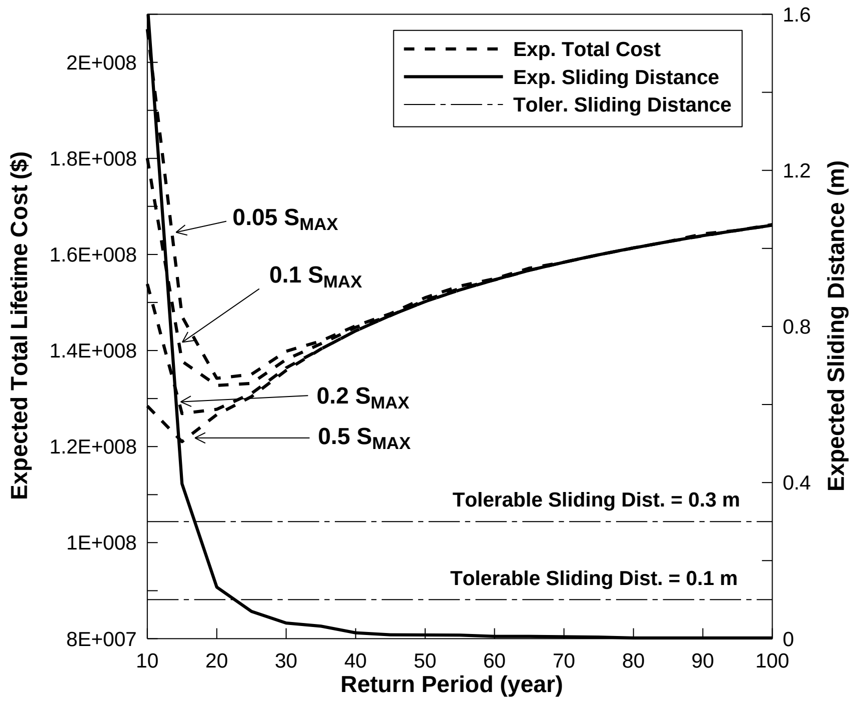


Fig. 11. Expected total lifetime costs calculated with different criteria of caisson sliding distance for harbor shutdown in water depth of 19 m

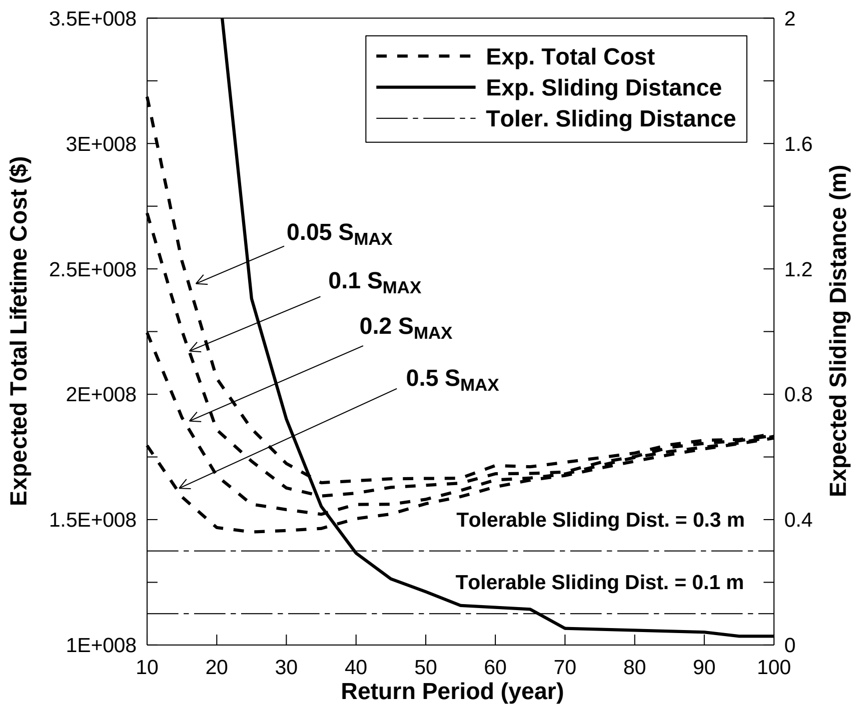


Fig. 12. Expected total lifetime costs calculated with different criteria of caisson sliding distance for harbor shutdown in water depth of 24 m

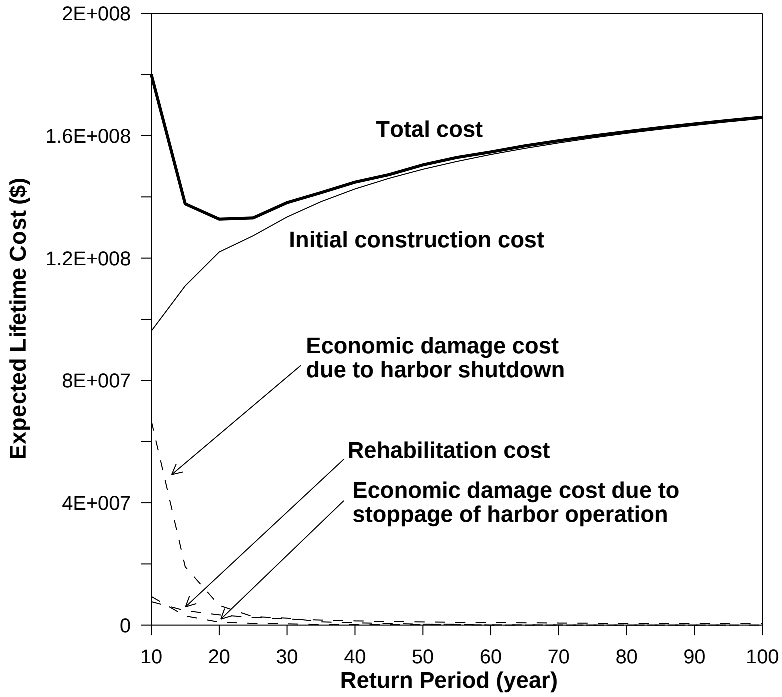


Fig. 13. Comparison of various costs for breakwater design in water depth of 19 m