Journal of Korean Society of
Coastal and Ocean Engineers
Vol. 20, No. 2, pp. 176~183, April 2008

# Probabilistic Assessment of Wave Overtopping of Seawall at Busan, Korea
# 부산 신항 방파제의 월파 확률 평가

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**Abstract :** In this paper, three classical overtopping models: Owen model, Van der Meer & Janssen model and Hedges & Reis model were used to calculate the failure probability of wave overtopping of seawalls. Among of them, the Hedges & Reis model was regarded as a moderate method to analyze the failure probability of wave overtopping of seawalls and the probabilistic assessments of wave overtopping were carried out for a constructing seawall at Busan in Korea by Level II and Level III reliability methods. Considering the cost of construction, an appropriate crest level was proposed for a certain rate of wave overtopping at a lower failure probability.

**Keywords :** coastal structure, wave overtopping, seawall, probabilistic assessment

**요 지 :** 본 논문에서는 세 가지 모형(Owen 모형, Van der Meer & Janssen 모형, Hedges & Reis 모형)을 적용하여 월파의 확률적 평가를 수행하였다. 건설중인 부산 신항 방파제의 월파 확률 예측 값을 도출하는 데에 있어서 Hedges & Reis 모형이 월파에 의한 방파제 파괴 확률분석과 Level II와 Level III 신뢰성 해석을 이용하는 것으로 적합하여 건설비용의 절감을 위하여 파괴가능성이 낮은 월파의 천단고를 제안하였다.

**핵심용어 :** 월파, 방파제, 확률 평가

## 1. Introduction

In the design of the crest elevation of a seawall, usually a tolerable wave overtopping rate is applied (Owen, 1980; Goda, 1985; van der Meer, 1994; Allsop et al., 1996; Besley et al., 1998; Bruce et al., 2001). A number of different methods may be available to predict overtopping of particular structures under given wave conditions and water levels. Empirical methods use a simplified representation of the process presented in equations to relate the overtopping discharge to key wave and structure parameters. Extensive study has been done by Goda, Owen, Allsop, Van der Meer and many other researchers (Goda et al., 1975; Owen, 1982; Allsop, 1994; Van der Meer et al., 1998; Reis, 1998; Van der Meer, 2002). Owen (1980) developed a design equation relating the mean overtopping discharge to incident wave conditions, described by the significant wave height and the mean zero-crossing wave period, and the seawall's freeboard. Van der Meer and Janssen (1995) proposed a calculation method for wave overtopping of seawalls, which was recommended to use in coastal engineering of European countries. Based on empirical fitting to hydraulic model test results, Reis (1998) suggested a regression model (Hedges & Reis model) for wave overtopping discharge to execute the probabilistic assessment of wave overtopping of seawalls.

In this paper, Owen model, Van der Meer & Janssen model and Hedges & Reis model were used to analyze the failure probability of seawall overtopping, as well as the probabilistic assessments of wave overtopping of seawall was carried out for a constructing seawall in Busan New Port, Korea. Considering the cost of construction, an appro-

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priate crest level was also proposed for a certain rate of wave overtopping at a lower failure probability.

## 2. Level II and Level III Reliability Methods

Generally, the failure function of a structure can be expressed as $Z = g(X_1, X_2, ..., X_N)$, where $X_i$ ($i$ = 1, 2, ..., $N$) are the basic variables of the problem. For most practical applications, each basic variable $X_i$ is a random variable with a probability density function $f_{X_i}$. $Z$ is a function of the basic variables. Then, the probability of failure is written as follows:

$$P_f = \text{Prob.}(Z \le 0) = \int\int_{Z\le 0} \cdots f_{X_1, X_2, ..., X_N} dX_1 dX_2 \ldots dX_N \quad (1)$$

where $f_{X_1, X_2, ..., X_N}$ is the joint probability density function of $X_1, X_2, \ldots, X_N$. Except for some special cases, the above integrations cannot be performed analytically and have to be approximated in some way.

### 2.1 Level II reliability method

Level II method is a semi-probability approach, in which approximation methods are used to transform the generally correlated and non-normal variables into uncorrelated and normal variables. Reliability indices are used as measures of the structure reliability. Non-linear failure functions are approximated using a tangent hyper-plane at some point (First Order Reliability Methods), using a quadratic approximation (Second Order Reliability Methods) or even high order approximations. The second and higher order methods considerably complicate the computations and, in many cases, the First Order Reliability Method (FORM) gives good approximations.

For the probability design method, the reliability index $\beta$ corresponding to the failure probability is generally used to express the safety degree or the reliability of the structure. Generally, linearization is performed by a truncated Taylor series expansion around some point, $X^*$, retaining only the linear terms in FORM. This results in the following approximation for $Z$:

$$Z \approx Z^* + \sum_{i=1}^{N} (X_i - X_i^*)\left(\frac{\partial Z}{\partial X_i}\right)^* \quad (2)$$

where $Z^*$ is the value of the function $Z$ at the point $X^*$ under consideration, and $\left(\frac{\partial Z}{\partial X_i}\right)^*$ is the partial derivative with respect to $X_i$, likewise evaluated at the point $X^*$.

The mean and variance of $Z$ are:

$$\mu_Z = Z^* + \sum_{i=1}^{N} (\mu_{X_i} - X_i^*)\left(\frac{\partial Z}{\partial X_i}\right)^* \quad (3)$$

$$\sigma_Z^2 = \sum_{i=1}^{N} \left[\left(\frac{\partial Z}{\partial X_i}\right)^* \sigma_{X_i}\right] \quad (4)$$

respectively. The probability of failure and the reliability index are again expressed by followings:

$$P_f = \text{Prob.}(Z \le 0) = \int_{-\infty}^{0} f_Z dZ = \Phi\left(-\frac{\mu_Z}{\sigma_Z}\right) = \Phi(-\beta) \quad (5)$$

$$\beta = \frac{\mu_Z}{\sigma_Z} \quad (6)$$

where $f_Z$ is the probability density function of $Z$, $\Phi(\cdot)$ is the cumulative function of standard normal distribution.

### 2.2 Level III reliability method

To validate the results of the Level II method, Level III calculations have to be carried out (Ang & Tang, 1984; Van der Meer, 1987). Level III method is called a full distribution approach. This method provides an exact probability analysis for whole structure systems, or structural elements, using full joint probability density functions including the correlations among the variables. A simple way to carry out a Level III analysis is to use existing software packages, such as @RISK (Palisade Corporation, 2002).

Monte Carlo Sampling is the traditional technique for using random numbers to draw samples from a probability distribution. However, an increase in the number of iterations is not always computationally convenient. The Latin Hypercube Sampling (LHS) method is designed to accurately recreate the input distribution, preserving the randomness of the traditional method while using fewer samples.

## 3. Probability Method Applied to Wave Overtopping of Seawalls

Since the late 1970s, there have been major advances in the prediction of wave overtopping (Goda et al, 1975; Owen, 1982; Allsop, 1994; Van der Meer et al, 1998; Reis, 1998). These studies concentrated on the prediction of mean overtopping discharge over unit length of seawall. The mean overtopping discharge per unit length of structure, $Q$ (m$^3$/s/m) was available to quantify the overtopping discharge. The overtopping discharge depends on wave and structure parameters, including the seawall freeboard, the crest geometry, the seaward slope, the significant wave height, the mean or peak wave period, the angle of wave attack measured from the normal to the structure, the water depth at the toe of the seawall, and the seabed slope. In general, the mean overtopping discharge per unit length of seawall, $Q$, depends upon the wave motion, the seawall profile and the foreshore characteristics:

$$Q = f(H_s, T_m, \varphi, R_c, \alpha, d_s, g, etc) \tag{7}$$

where $H_s$ is the significant height of the incident waves; $T_m$ is the mean zero-crossing wave period; $\varphi$ is the angle of wave approach measured from the normal to the seawall; $R_c$ is the seawall's freeboard (the height of the crest of the structure above the still-water-level); $\alpha$ is the angle of the seawall front slope measured from the horizontal; $d_s$ is the still-water-depth at the toe of the structure; and $g$ is the acceleration due to gravity (Fig. 1). In the figure, CL denotes crest level, TL denotes toe level and SWL is the still-water-level above datum.

Much research on this subject has been done, and several formulas for predicting the quantity of wave overtopping are known. In this paper, some of the known overtopping models are given in short, and used to execute the probabilistic assessment of wave overtopping of Busan seawall.

### 3.1. Owen overtopping model

Owen (1980) studied the average wave overtopping discharge of seawalls and developed a design equation relating the mean overtopping discharge to incident wave conditions, described by the significant wave height and the mean zero-crossing wave period, and the seawall's freeboard. The average wave overtopping discharge for the normal incident wave can be expressed in the form of dimensionless:

$$\frac{Q}{T_m g H_s} = A\exp\left(-B\frac{R_C}{T_m\sqrt{gH_S}}\right) \tag{8}$$

where $A$ and $B$ are best-fit coefficients determined from experimental data. The scope of application of Eq. (8) is

$$0.05 < \frac{R_C}{T_m\sqrt{gH_S}} < 0.30 \tag{9}$$

Eq. (8) and Eq. (9) are called Owen overtopping model, which was widely used in the design of coastal engineering in the United Kingdom.

For the allowable discharge of wave overtopping TR (the target value), the failure function may be written as (Reis, 1998):

$$Z = TR - A\sqrt{\frac{2\pi g H_S^3}{s_m}}\exp\left[-e_B B\frac{\sqrt{s_m}R_C}{rH_S\sqrt{2\pi}}\right] \tag{10}$$

in which, $s_m = 2\pi H_S/gT_m^2$ is the deep water steepness calculated using $T_m$, model parameter $r$ is the roughness of the seawall front slope, model parameter $e_B$ is described the influence for overtopping by the slope degree.

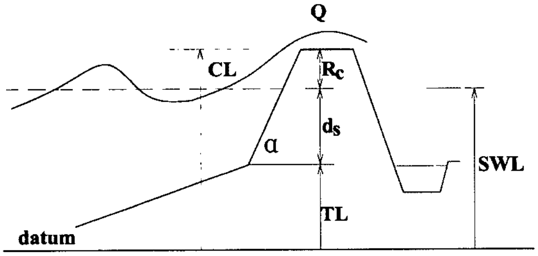


**Fig. 1.** Notation for seawall overtopping.

### 3.2 Van der Meer & Janssen Overtopping Model

Based on the research on the wave overtopping of seawalls, Van der Meer and Janssen (1995) presented a calculation method for prediction of wave overtopping discharge, called Van der Meer & Janssen model (V&J model). In the design of seawall in European countries, the V&J model was recommended to apply in coastal engineering. The corresponding failure functions were expressed as (Van der Meer & Janssen, 1995; Reis, et al., 2006):

$$Z = TR - \frac{0.06\xi_{OP}\sqrt{gH_S^3}}{\sqrt{\tan\alpha}}\exp\left[-A\frac{R_C}{r\xi_{OP}H_S}\right] \quad \text{for } \xi_{OP} < 2$$

$$Z = TR - 0.2\sqrt{gH_S^3}\exp\left[-B\frac{R_C}{rH_S}\right] \quad \text{for } \xi_{OP} > 2 \tag{11}$$

where TR is the mean permissible wave overtopping discharge per unit length of seawall; $A$ and $B$ are empirical coefficients; $\xi_{OP}$ is the surf similarity parameter or Iribarren number for random waves, $\xi_{OP} = \tan\alpha/\sqrt{2\pi H_S/(gT_P^2)}$; $g$ is the gravitational acceleration; $T_p$ is the wave period corresponding to the peak spectral density, $r$ is the seawall slope roughness.

### 3.3 Hedges & reis overtopping model

Based on equations which have been obtained from empirical fitting to hydraulic model test results, Hedges and Reis developed a regression model for evaluating the wave overtopping of seawalls.

The failure functions of Hedges & Reis overtopping model (H&R model) may be written as follows (Reis, 1998):

$$Z = TR - A\sqrt{g(CH_S)^3}\left[1-\frac{R_C}{rCH_S}\right]^{e_B B} \quad \text{for } 0 \le \frac{R_C}{rCH_S} < 1$$

$$Z = TR \qquad \text{for } \frac{R_C}{rCH_S} \ge 1 \qquad (12)$$

where $TR$ is the allowable overtopping discharge per unit length (the target value), $e_B$ is represented the degree of variability in $A$ and $B$.

The coefficient $A$ represents the dimensionless discharge over the seawall when the freeboard is zero. It is also clear that the coefficient $B$ is related, in the case of regular waves, to the shape of the function which describes the water surface variation on the seaward face of the wall. There will be a similar dependence on the detailed behavior of the water surface at the face of the wall in the case of random waves. The value of $C$ will depend upon the duration of the incident wave conditions and $C = R_{max}/H_S$ would best be determined from experimental data, where $R_{max}$ is the maximum run-up induced by the random waves. All three coefficients will be influenced by the seaward profile of the structure.

## 4. Probability of failure of a Seawall at Busan New Port, Korea

A seawall along the southern of Gaduk island nearby Busan New Port is being constructed. The seawall will shelter the new road which will be built and connects between Busan city and Geoje city. As a case study, the probabilistic assessments for safety of structure in terms of overtopping by wave action are performed for the deepwater section of the structure (Fig. 2).

### 4.1 Critical mean discharges in the analysis of Busan seawall

The overtopping criteria for design depend upon the structure's function and the degree of protection required, and upon the associated risks, taking into account the joint probability of wave heights and water levels. Based on the impressions of experts observing prototype overtopping, the values can be seen in the figure of critical mean overtopping discharges for use in design were determined as shown in Fig. 3 (CIRIA/CUR, 1991). It shows that discharges greater than about $2\times10^{-3}$ m³/s/m may damage embankment seawalls, whilst $5\times10^{-2}$ m³/s/m is approximately the critical discharge for seawalls with unprotected back slopes. For the seawalls, the range of critical mean discharges for the overtopping of seawalls runs from as lit-

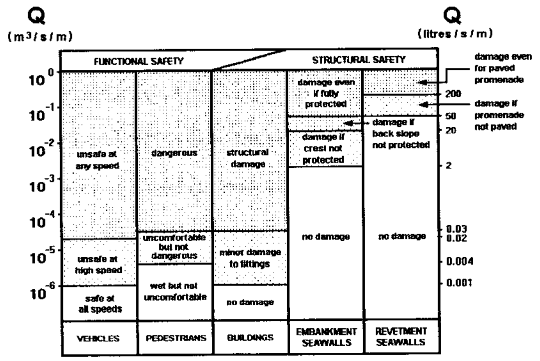


**Fig. 3.** Critical mean overtopping discharges for use in design (CIRIA/CUR, 1991).

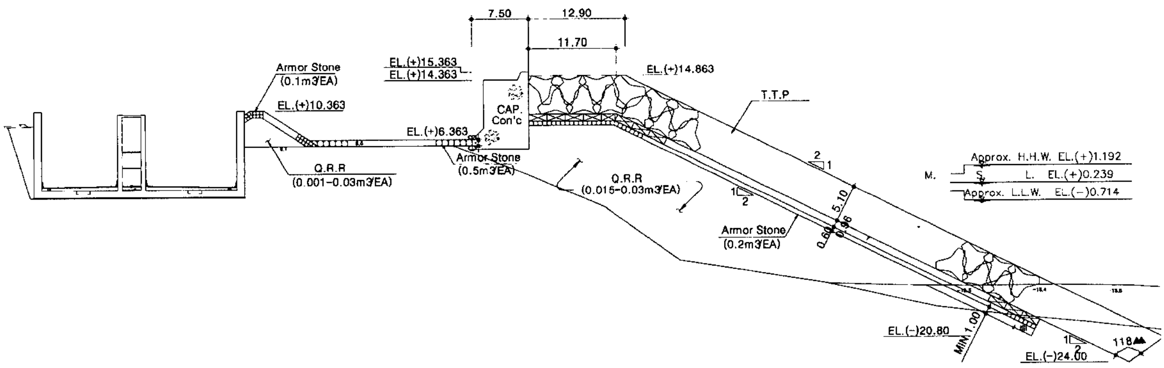


**Fig. 2.** Dimension of seawall of Busan New Port (TIWTE, 2006).

tle as $10^{-3}$ m³/s/m to about $2\times10^{-1}$ m³/s/m depending on the protection of crest and back slope. Therefore, we calculated the probability of failure for three different wave overtopping discharges of $10^{-1}$, $10^{-2}$ and $10^{-3}$ m³/s/m, respectively.

### 4.2 Probabilistic assessment of wave overtopping of Busan seawall

Assuming that the extreme wave height at the location of the seawall is described by the Weibull distribution, the significant wave height and period for 100yr return period are respectively taken as $H_S = 7.70$ m, $T_S = 15.0$ s. The crest level of the seawall is 15.36 m, toe level is -14.02 m, and the seawall slope is 1:2. The normal distribution is taken for the tidal level at Busan, of which mean is 0.645 m, and standard deviation is 0.3369 m (Choi, et al. 2002) (Fig. 4).

Using the Owen model, V&J model and H&R model, the failure probabilities of wave overtopping of Busan seawall had been carried out by Level III method (Fig. 5).

Comparing of the results by different overtopping models, the values obtained by Owen model are generally greater than other models and the failure probabilities by H&R model are mostly closed to that of V&J model for different allowable overtopping rates. Therefore, the H&R model was mainly used to calculate the probabilistic assessment of wave overtopping of Busan seawall in the following.

For each critical mean discharge, the crest level was calculated for different failure probability by level II method using PARASODE. PARASODE was developed for assessing the safety of wave overtopping of seawalls by Reis (1998). It is a Level II program for assessment of safety of coastal structures using FORM by Owen model and H&R model. According to the critical mean discharges, the fitting curves of crest level were carried out for the probability of failure by Level II method (Fig. 6).

Fig. 6 shows that the probability of failure decreases as the crest level increases, as expected, and the values of both of Owen model and H&R model were coincided in the case of $10^{-1}$ m³/s/m.

In validating the accuracy of the Level II analysis results by Level III, Latin Hypercube Sampling was used and 30000 simulations were made to obtain a convergent result. The failure probabilities of the structure for different allowable overtopping discharges by two different methods are shown in Fig. 7.

We calculated the results between the two methods in Fig. 7, it is known that the values of Level II are generally less than those of Level III method. This is because the H&R failure function has higher nonlinear degree, and the truncation error will be greater for the larger curvature of the function at the design point in FORM. In other words, the failure probabilistic assessment of risk using FORM on the seawall at Busan will be underestimated.

In order to find an appropriate crest level of the seawall, we calculated the failure probabilities of wave overtopping for the crest level from 15.0 m to 17.0 m by Level III methods using H&R model (Table 1).

For the Busan seawall, a certain rate of wave overtopping at permissible discharge of 0.01m³/s/m is safe according to the critical mean overtopping discharges in the use of design. Considering the cost of construction, a moderate crest level, such as 15.50 m, should be chosen for a certain rate of wave overtopping at a lower failure probability.

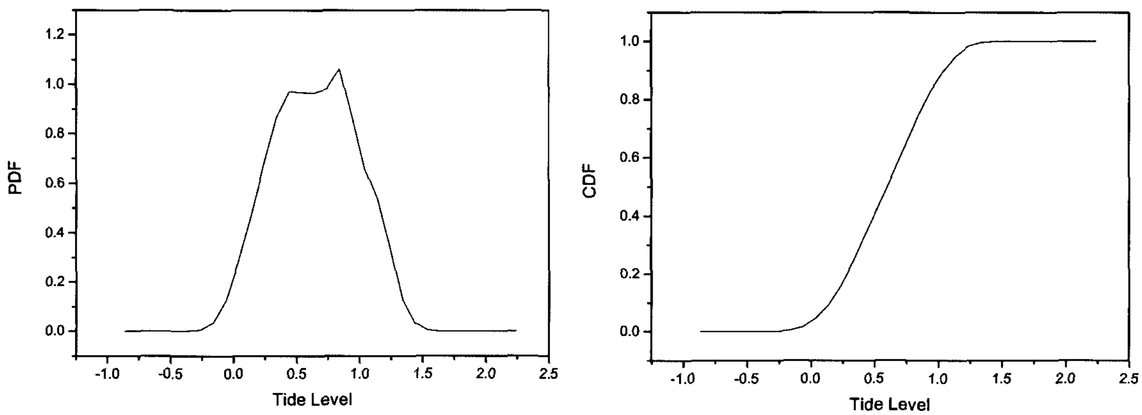


**Fig. 4.** Distribution of tide level at Busan Port.

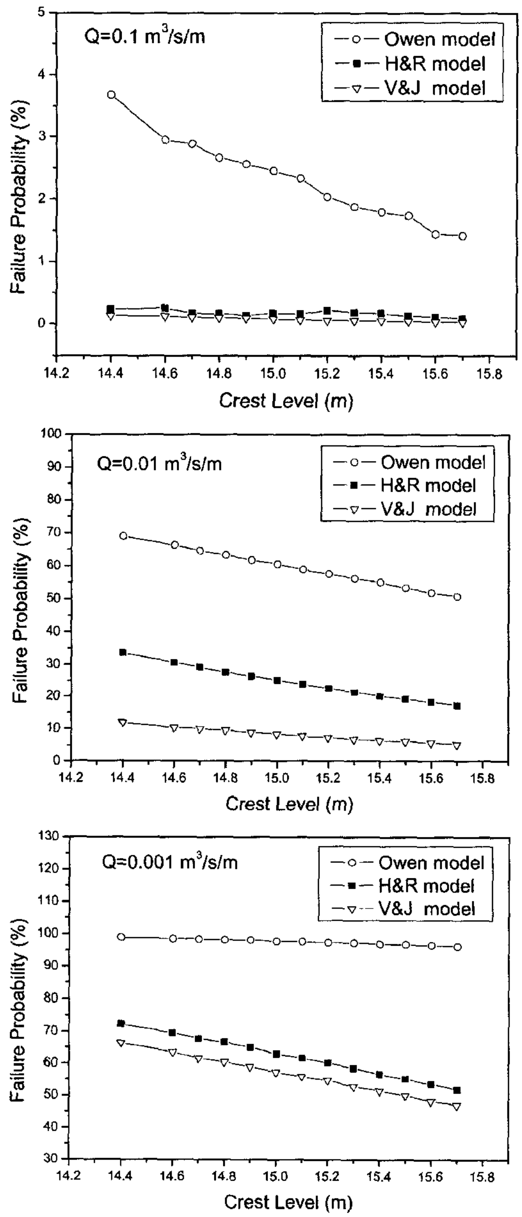


**Fig. 5.** Crest level plotted against probability of failure for different values of the permissible discharge (Q) by Level III method. (Owen, V&J, H&R model).

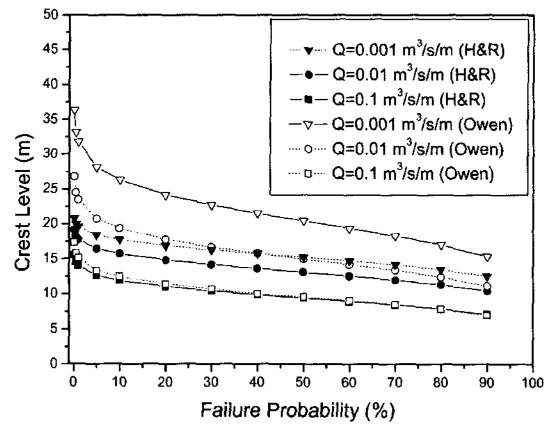


**Fig. 6.** Crest level (above datum level, Unit: meter) plotted against probability of failure (percentage) for different values of the permissible discharge (Level II method).

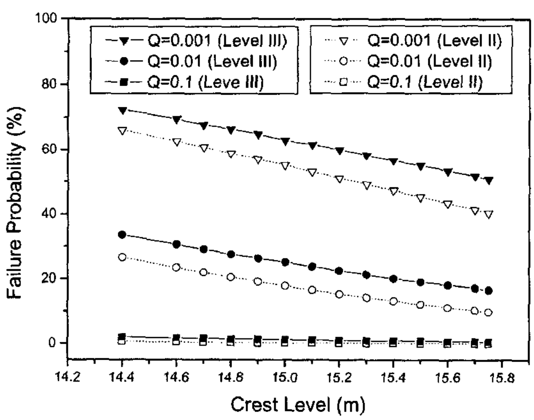


**Fig. 7.** Failure Probabilities of wave overtopping of Busan seawall by Level II and Level III methods (H&R model).

### 4.3 Considering rising of mean sea level

Because climatic changes over the next century may cause a significant rise in sea levels, it is important to ensure that the coastal structure may provide an appropriate level of protection. Assuming that the mean sea level may rise up to +1.0 m in the nextcentury, the probability of failure of Busan seawall also can be obtained for the

**Table 1.** Failure probability of wave overtopping of Busan seawall (H&R model)

| Crest Level | Q=10$^{-1}$ m$^3$/s/m | Q=10$^{-2}$ m$^3$/s/m | Q=10$^{-3}$ m$^3$/s/m |
|---|---|---|---|
| 15.0 m | 1.364% | 25.23% | 63.22% |
| 15.5 m | 0.835% | 19.08% | 55.32% |
| 16.0 m | 0.580% | 14.52% | 46.54% |
| 16.5 m | 0.410% | 10.84% | 38.95% |
| 17.0 m | 0.302% | 7.955% | 31.87% |

**Table 2.** Failure Probabilities of Busan seawall after sea level rise of 1.0 m

| Model | $Q=10^{-1}$ m³/s/m | $Q=10^{-2}$ m³/s/m | $Q=10^{-3}$ m³/s/m | Crest Level |
|---|---|---|---|---|
| Owen model | 3.597% | 67.48% | 98.74% | 15.5 m |
| V&J model | 0.106% | 10.97% | 64.89% | 15.5 m |
| H&R model | 1.945% | 31.94% | 71.13% | 15.5 m |

allowable overtopping discharges at crest level of 15.50 m (Table 2).

Table 2 shows that the failure probability of wave overtopping of Busan seawall will be increased in the case of the tidal levels (MSL) rising up to 1.0 m. For example, designing the Busan seawall at the crest level of 15.5 m, the annual failure probabilities of the seawall for the allowable overtopping discharges $10^{-1}$ m³/s/m by H&R model may respectively reach to 1.945%. However, the present coastal structure satisfies the engineering requirements according to the critical mean overtopping discharges of use in the design (CIRIA/CUR, 1991).

## 5. Conclusions

Three classical overtopping models: Owen model, V&J model and H&R model were used to perform the probabilistic assessment of Busan seawall in this paper as a case study. The H&R model was regarded as a moderate overtopping model and used to calculate the probabilistic assessment of seawall at Busan by Level II and III methods. The failure probability obtained by FORM was generally less than those of Level III. The truncation error would be significant for the higher order nonlinear failure function. Therefore, the assessment of failure probability using Level II on the Busan seawall will be underestimated.

Based on the critical mean overtopping discharges for use in design, an appropriate crest level, CL=15.5 m, is proposed for the design of seawall at a lower failure probability, considering the hypothetical mean sea level rising up to 1.0 m in next century.

## Acknowledgements

The authors would like to acknowledge the financial support of the BK21 Division for U-CITY Construction, Sungkyunkwan University, Korea. Thanks are also due to Dr M.T. Reis for providing us with PARASODE code.

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Received December 21, 2007
Accepted April 1, 2008