Wave overtopping and stability of armor units under multidirectional waves

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Abstract

This paper describes the wave overtopping along a seawall of a man-made island where nuclear power plants are projected to construct and the stability of armor units at the concave section of seawall. Effects of the wave directionality on them were investigated by using multidirectional wave basin not only for a design wave but also larger waves than the design one to ascertain the durability and margin of the safety of seawall.

1   Introduction

To extend siting area for power plants, a synthetic project for new siting technologies has been undertaken by Central Research Institute of Electric Power Industries(CRIEPI). One of the most promising technologies is to site them in a man-made island instead of on coastal land. A new design method for very important coastal structure of the man-made island has been proposed by Kajima(1994).

Concerning the power plants on the man-made-land, the wave overtopping and the stability of armor units for the new design concept has been investigated mostly with two-dimensional physical model tests (Kajima *et al.*,1993; Kaijma, 1994; Sakakiyama *et al.*, 1994a; Sakakiyama *et al.*,1994b). Because the man-made island will be constructed offshore, effects of the wave directionality on the wave

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overtopping and the stability of armor units are of great importance for rational design of the seawall.

Works on the wave overtopping recently preformed were aiming at design formula(de Waal and van der Meer,1992; Herbert *et al.*, 1994). Although multidirectional wave basins are available nowadays, research on the overtopping rate was carried out for oblique waves (Juhl and Sloth, 1994) and for two-dimensional coastal structure due to multidirectional waves(Hiraishi *et al.*, 1995).

Effects of the wave directionality on the overtopping rate and the stability of armor units were investigated in the wave field where the diffracted waves were caused by three dimensional coastal structure. To ascertain the durability and margin of the safety of seawall of the man-made island, not only a design wave but also larger waves than the design one were applied for the structure.

2 Design concept

To guarantee the safety of man-made island where a nuclear power plant is supposed to be constructed, a new design method was proposed by Kajima(1994). That design concept is based on the following two-step design: At first step, according to the current design method, coastal structure should be stable for a "fundamental design wave". That wave condition is determined with 100-year return period.

In addition to the present design concept, functions of the seawall are confirmed for a severer wave than the fundamental design wave height, where it is named the "functional check wave".

In this research, to demonstrate this two-step design method, design wave conditions are supposed as shown in Table 1. The deep-water significant wave height H_0=10m and wave period T=16s are taken as the fundamental design wave with 100-year return period.

For second step, a wave condition to check up functions of the coastal structure is determined: The wave height is one and a half times larger than that of the fundamental design wave, H_{F0}=1.5×H_{0D}=15m. Water depth of the man-made

Table 1 Design wave conditions

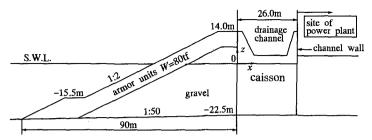| fundamental design wave		functional check wave
return period	100 years	roughly 1000 years
conditions in deep-water conditions		
wave height H_0(m)	10m	15m(=1.5×H_0)
wave period T_0(s)	16s	16s
conditions at man-made island		
wave height H_D(m)	10.3m	12.8m(=1.24×H_D)
wave period T_D(s)	16s	16s
armor units	stable	slight deformation
allowable overtopping rate	0.05m³/s/m	0.2m³/s/m

Fig.1 seawall covered with armor units

island is assumed as h=22.5m including the high tidal range. An incident significant wave height at the water depth h=22.5m applied in the tests ranges from the design wave height H_D=10.3m to H=1.24×H_D=12.8m.

Coastal structure should be stable for the fundamental design wave according to the current design method. However, it is allowed to get slight damage of the armor layer within keeping its functions.

Coastal structure to protect the man-made island consists of wave dissipating work and caisson on which drainage channel for overtopping water is constructed. Fig. 1 shows a cross section of the seawall and it is designed to be stable for the fundamental design wave condition with 100-year return period. The incident significant wave height for the fundamental design is H_D=10.3m at the water depth h=22.5m where the seawall is constructed. Armor units used in the present experiments is Tetrapod and its weight W=80tf which is determined by Hudson formula(K_D=8.2). A crest elevation of the seawall was estimated as h_c=14m above S.W.L. for an allowable overtopping rate, q_{da}=0.05m^3/s/m. For the functional check wave, the allowable overtopping rate was also given. But the crest elevation of the seawall is lower than that of the fundamental wave condition. For those given wave conditions, the relationship between the crest elevation and the overtopping rate was obtained by using the diagrams for wave overtopping proposed by Goda's(1985). Area inside the channel wall is site of the power plant. So extremely large amount of water due to the wave overtopping is not allowed in this area.

Coastal structure should keep its functions to protect the power plant inside of man-made island. Therefore, investigatation was also performed in this research from this point of view. Dimensions of drainage channel was determined with two-dimensional wave flume tests including large- and small-scale test(Sakakiyama *et al.*, 1994b).

3 Experimental method

The experiments were carried out by using a multidirectional wave basin(35.0m long, 45.0m wide and 1.1m deep) equipped with a 48-segmented wave maker(19.2m

wide) as shown in Fig. 2. A scale factor of physical model test is $\lambda=1/88.1$. A model of man-made island is 5.7m long in the direction on-offshore and 11.4m wide in the direction alongshore(500m by 1000m in full-size). It is protected with seawalls consisting of caissons covered with armor units as shown in Fig. 1. The model of man-made island was located as shown in Fig. 2 to make center lines of the wave generater and of a half section of offshore-side seawall on the same line. Measurement of wave overtopping was done in a half section of seawall. Guide walls were settled during the tests for both normal and obliquely incident waves. Uniformity of wave heights at the water depth of offshore-side seawall was checked parallel to the shoreline

Table 2 shows the experimental conditions. Three kinds of wave directional conditions were selected, CASE A: multidirectional wave with normal incident($\theta=0$ deg.), CASE B: uni-directional irregular wave($\theta=0$deg.) and CASE C: obliquely uni-directional irregular wave($\theta=30$deg.). The wave directionality is expressed with the spreading parameter for wave energy S_{max} defined by Goda(1985). When S_{max} is small, e.g., waves with $S_{max}=10$ corresponds to wind waves. The wave of which S_{max} tends to the infinity is a uni-directional wave. Multidirectional wave used in the test has $S_{max}=25$ which corresponds to swell at short decay distance with relatively large wave steepness(Goda, 1985). The wave height ranges from the design wave height($H_D=11.7$cm in model and $=10.3$m in full-size) to the functional check wave height($1.24H_D=14.5$cm in model and $=12.8$m in full-size). The wave period is constant sa $T=1.71$s(16s in full-size).
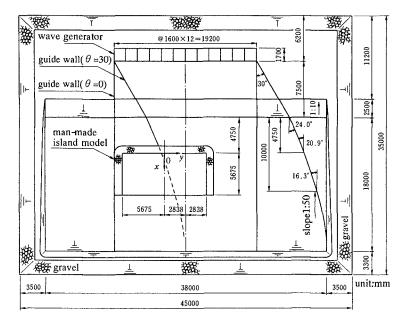

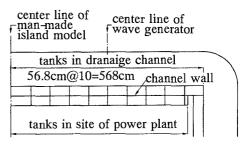Fig.2 Multidirectional wave basin and man-made island model

Table 2 Experimental conditions(scale factor $\lambda=1/88.1$)

CASE	wave directionality	S_{max}	θ (deg.)	wave height	wave period(s)
A	multi-directional	25	0	H_D	
B	uni-directional	∞	0	to	1.71
C	obliquely uni-directional	∞	30	$1.24H_D$	

H_D is design wave height $=11.7$cm
water depth at seawall is 25.5cm.

Fig. 3 shows an experimental set-up for measurement of the over-topping volume. Local change of the wave overtopping was measured at a half of the offshore-side sea-wall alongshore for both in the drai-nage channel and onto the site of power plant. A half length of the test section is divided into ten parts to make tanks for water storage. Channel wall as shown in Fig. 3

Fig.3 tanks to measured overtopping volume

was scaled down to be 15.9cm high. To store the overtopping water beyond the channel wall and onto the site of the power plant, 50-cm high wall was set to the tanks in site of power plant. It was caused mainly by spray.

A number of wave trains was 250 in one test run. Because amount of water beyond the channel wall was very small, tests were repeated three or more times under the same wave condition to get sufficient vol-ume of water. Mean overtopping rate was obtained from the mea-sured volume and the wave dura-
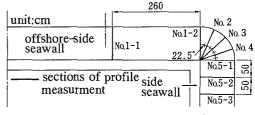

Fig.4 measuring sections of profiles

tion time. Physical values are converted to full-sized ones in the following.

Fig. 4 shows the lines(No.1-1 to No.5-3) along which profiles of the armor layer were measured with depth-meters. Basically the armor units in two-dimensional section(Line No.1-1 in Fig. 4) are stable for the normal incident design waves. Interest in the stability of armor units is concentrated on critical part, that is, concave section. Profiles were measured at initial and several stages in the wave duration up to 3000 times the significant wave period($t/T=0$, 250, 500, 1000 and 3000). "Damage level S" defined by van der Meer(1987) is used to quantify the deformation of the armor layer. Fig. 5 shows its definition. The damage level physically means a number which is proportional to armor units number in an eroded area per width of the nominal diameter of the armor units D_n.
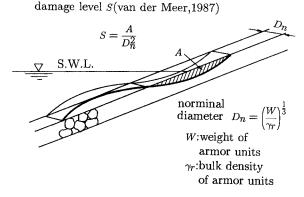
damage level S(van der Meer,1987)

$$S = \frac{A}{D_n^2}$$

S.W.L.

norminal diameter $D_n = \left(\frac{W}{\gamma_r}\right)^{\frac{1}{3}}$

W:weight of armor units
γ_r:bulk density of armor units

Fig.5 Definition of damage level

4 Overtopping rate

Fig. 6 shows distribution of the local overtopping rate along the seawall due to the multidirectional wave, where the local overtopping rate is defined as the discharge per unit length normal to the seawall. It was measured for several wave heights ranging from the design wave height(denoted by $H/H_D=1.0$) to the functional check wave height($H/H_D=1.24$). Because the man-made island has finite length, diffracted waves were generated from ends of the structure. They cause the wave height change along the seawall. Consequently they results in change of the overtopping rate as shown in Fig. 6. The diffracted waves consist
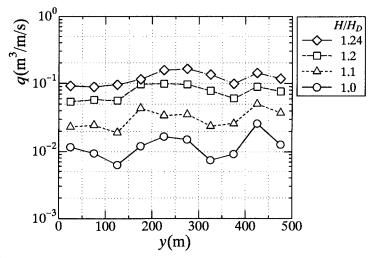
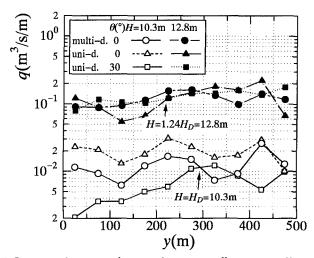Fig.6 Local overtopping rate along seawall due to multidirectional wave

Fig.7 Overtopping rate change along seawall on wave directionality

of plain waves and cylindrical scattered waves. The scattered wave propagates along the seawall not normal to that. Effect of the cylindrical scattered wave on the overtopping rate is smaller than that of the normal incident wave but not null. The diffracted wave causes an increase in the mean water level along the seawall as a function of wave height. A ratio of the maximum to minimum local overtopping rate is about four at the design wave height denoted with a hollow circle, $H/H_D=1.0$. The ratio decreases as the wave height increases. The ratio due to the functional check wave(with a diamond, $H/H_D=1.24$) is about two.

Dependency of the local overtopping rate on the wave directionality is shown in Fig. 7. The results of large waves shows the overtopping rate of uni-directional wave denoted with solid triangle fluctuates most, the second is uni-directional wave and the smallest due to the obliquely uni-directional wave. This tendency agree with the diffracted wave height distribution. Trend due to the design wave height slightly differs from the previous one. It is because that the overtopping rate is nonlinear to the wave height. The diffracted wave height due to the multidirectional waves is smoother than the uni-directional irregular wave ones.

Roughly speaking, the local overtopping rate fluctuates from a half to twice the mean value in space except the result of the obliquely incident wave with the design wave height. The diffracted wave from another corner of the model was not included from the experimental restriction. A ratio of the length of man-made island to the significant wave length is 4.5. It means that the man-made island is relatively long. Therefore, it is considered that the scattered wave from one corner dissipated at another. Experiments were also performed by capping the tanks in the drainage channel to investigate that the water can be drained through the channel. Outlets of the drainage channel were located at the landward ends of the side seawall. It was confirmed that most of water over the
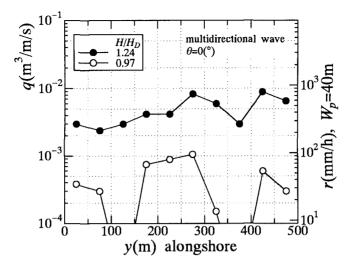
Fig.8 Overtopping rate in drainage channel

parapet was drained through the channel on the caisson. Maximum water depth in the drainage channel was about 4m. Comparing with the height of channel wall 10.5m, the channel has sufficient capacity for draining. Dimensions of the drainage channel was determined mainly to reduce the overtopping water onto the site of power plant.

Overtopping water onto the site of power plant is caused by splash. Spray is not avoidable by the channel wall with reasonable height. Critical situation due to the wave overtopping is flood into the power plant. Therefore, it is required to estimate discharge in the site to design a drainage pumping system.

Fig. 8 shows the local overtopping rate onto the site. The right axis of ordinate shows the equivalent precipitation. A distance of drainage W_p is assumed as 40m from the channel wall which was estimated by observation in the experiments. Although the scale factor of this experiment is small and an accuracy for such small amount of water should be taken into account, the equivalent precipitation due to the design wave fluctuates from less than 10mm/h to 100mm/h. A mean equivalent precipitation over the seawall is $O(10)$mm/h. It is acceptable. Owing to the functional check wave, it will be about 500mm/h. It is equivalent to rainfall by a very strong typhoon. Drainage pumping system should be reinforced by double within this range of W_p=40m for both the wave overtopping as well as the rainfall.

Fig. 9 shows an effect of the wave directionality on the overtopping rate which is averaged rate over the offshore-side seawall. The axis of abscissas indicates the incident significant wave height. The uni-directional wave with the principal wave direction θ=0deg. causes the largest overtopping rate, the multi-directional wave with θ=0 deg. medium and the oblique uni-directional wave with θ=30 deg. the smallest. This results of the comparison between the multidirectional

and uni-directional waves agrees with that obtained by Hiraishi *et al.*(1995). The overtopping rate due to the multidirectional waves is smaller than that due to the uni-directional waves by about 30%. Results obtained with two-dimensional tests overestimates the overtopping rate when it will be applied for offshore structure in the multidirectional waves. The obliquely incident uni-directional waves produces the smallest overtopping rate among three wave directionality. Under such conditions, wave dissipating work becomes relatively long on wave run-up, although its slope becomes milder. This effect is valid for the multidirectional waves which come from the various directions.
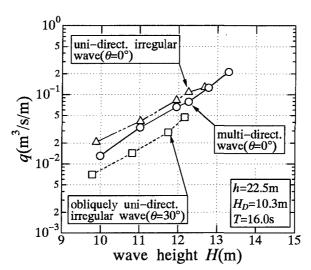

Fig.9 Overtopping rate q dependent on wave directionality

5   Stability of armor units

Fig. 10 shows the profiles of deformed armor layers due to the multidirectional wave with the design wave height at $\alpha=67.5$ deg. of the concave section. Under the design wave conditions for three wave directionality, the multidirectional wave gives the largest damage to the profile change although the differences are small. It is seen from the difference between the initial($t/T=0$) and final($t/T=3000$) profiles that small area just below the still water level was eroded and accumulated close to by down the slope. This eroded are is quantified with the damage level $S \approx 4$. Although the area eroded is small, it is not allowed under the current design concept. The armor units used in the experiments correspond to the maximum one available in practice. So it is necessary to do further work to make alternative ways to satisfy the present design method, i.g., to use high density armor units or to use more stable armor units.

Fig. 11 shows the profiles due to the functional check wave at the same section. The armor units initially placed near the still water level were removed by much severer waves than the design wave. In this figure, an eroded area is seen but accumulated one is not. Change of area does not balance between the initial and final profiles. It is because the armor units were removed in the wave propagation direction, not down the slope. The damage level S at the final profile at t/T=3000 is about 11. For the functional check wave as well as the design wave, the damage does not reach the crest level of armor layer.

Figures 12 and 13 show the damage level change dependent on the time of wave duration. Deformation occurs at an early stage of the wave duration. This tendency is same as the previous results obtained using two dimensional tests(Kajima et al., 1994). The damage level due to the design wave height reaches at most five at α=45(deg.) denoted with the solid square as shown in Fig. 12. The damage level of α=67.5(deg.) at t/T=3000 is four of which the profiles were shown in Fig. 10. It is considered that the damage level around five indicates small damage
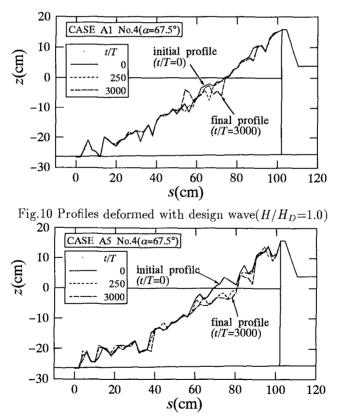

Fig.10 Profiles deformed with design wave(H/H_D=1.0)

Fig.11 Profiles deformed with functional check wave(H/H_D=1.24)

from the geometrical point of view.

Fig. 13 shows the damage level change due to the functional wave. The most serious damage occurred at α=67.5(deg.) of which profiles were given in Fig. 11. The damage due to the larger wave also occurred at the early stage of wave duration. The damage level does not increase significantly compared with the results of the design wave height. However, it was observed that more armor units were removed by the functional check wave than the design wave. They were carried away in the direction of wave propagation not down the slope. Some of the removed armor units might fill up the eroded part at the different sections. Consequently, the obtained damage levels are rather small compared with the results of the design wave height.
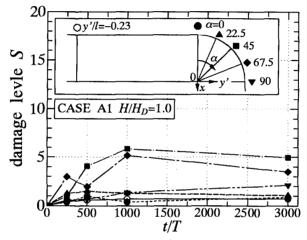

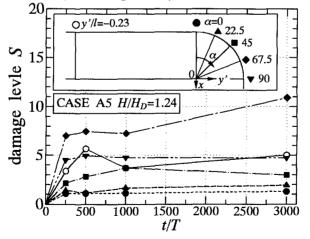Fig.12 Time history of damage level (multidirectional wave, Case A1)

Fig.13 Time history of damage level (multidirectional wave, Case A5)

The hollow circle in Figures 12 and 13 shows the damage level at the offshore-side seawall where the wave motion is expected as two-dimensional. Its damage level is very small for the design wave height but somewhat large due to the functional check wave.

In this experiment, the wave was applied for 3000 times the significant wave period which is equivalent to more than 13-hour duration. It is expected that the extremely large wave does not last such long time. Therefore, the damage level will be reduced by taking account of the wave duration. It is required for estimation of the damage level in practical use that the duration time should be considered as a function of the wave height and period.

Fig. 14 shows the damage level at the final stage(t/T=3000) around the concave section, where the effect of the wave directionality is compared for the design wave height. The damage level around the center of the concave section has peaks for three different kinds of wave directionality. Although it is seen that the damage level due to the design wave height of multidirectional wave is the largest, which is up to five, it is not concluded that the effect of the wave directionality is significant by considering the magnitude of damage which are small. The damage level at the side seawall is invisible.

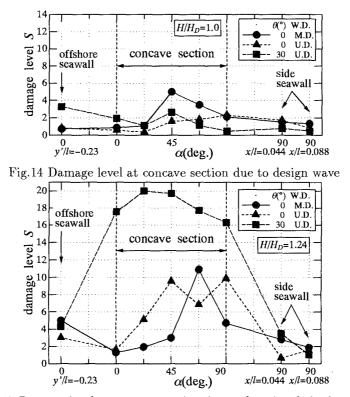Fig.14 Damage level at concave section due to design wave

Fig.15 Damage level at concave section due to functional check wave

Fig. 15 shows the result by the functional check wave height. Larger waves than the design wave cause more serious damage as expected. The obliquely uni-directional irregular wave caused the most serious damage around the whole concave section and the damage level becomes 20. However, it is considered that such condition resulted in an overestimated one. The normal incident waves caused the maximum damage level about ten for both the multidirectional and uni-directional waves. As the results of two normal incident waves, the wave directionality does not influence the damage at the concave section.

Considering repairability of the damaged section, it have to be estimate an allowable damage level for the severer wave than the design wave. Under both the design and functional check conditions, the deformation of the armor layer does not reach the crest elevation. It is important that damage does not reach the crest elevation in order to keep functions of seawall. Otherwise seawall collapses in a short time like the result of the obliquely incident wave.

6   Conclusions

The effect of the wave directionality on the overtopping rate was confirmed by tests using the multidirectional wave basin. It is caused by the diffracted waves generating at the concave section of the man-made island. The durability of the armor layer is confirmed for both the design wave and the larger wave than it. The margin of the safety can be expected to a certain degree against the larger wave than the design wave. The following s are main conclusions obtained by the present work.

Conclusions:
   1 The multidirectional waves give smaller overtopping rate than uni-directional waves for normal incident condition. Oblique wave gives the smallest.
   2 The overtopping rate was estimated for both in the drainage channel and onto the site of power plan. The later case is important for the safety of power plant. The equivalent precipitation can us information to design the drainage pumping system in the site power plant.
   3 No serious damage at the concave section of seawall was confirmed for keeping seawall's functions.
Future works:
   1 To make a model to estimate the overtopping rate with a numerical model to simulate wave height around an man-made island.
   2 To make no damage at concave section for the design wave. Alternative ways are to use high density armor units and to use more stable armor units.

Acknowledgements

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