

EFFICIENT LONG-PERIOD WAVE ABSORBING SUBMERGED MOUND STRUCTURE

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In many Japanese ports, it has been reported that long-period waves with periods of 30 to 200s cause serious problems in cargo handling. As a countermeasure, wave absorbing mounds installed on the harbor side of breakwaters have been constructed. The crown heights of such rubble mounds are almost equal to those of the caissons. Although such structures can be expected to provide some wave absorbing performance, they are too large to apply to specific site conditions. It is important therefore to reduce the size of the structure to apply to specific site conditions. In our study, a slightly submerged mound type long-period wave absorbing structure is proposed. A series of hydraulic model experiments was carried out to evaluate the wave absorbing performance in port and stability characteristics of the armor units against wave overtopping from the sea side. Throughout this study, it became clear that submerged structures display a higher wave absorbing performance compared with those of conventional structures. Moreover, a prediction formula for the stability number for Tetrapods covering these submerged mounds is proposed.

Keywords: slightly submerged wave absorbing mound; long-period wave; hydraulic model experiments

INTRODUCTION

In many Japanese ports, long-period waves with periods of 30 to 200s can cause serious problems in cargo handling. For example, Hiraishi et al. (1996) describes examples of suspension of cargo handling in ports such as Tomakomai and Shibushi. The origins of problems in cargo handling mainly come from resonances in the periods of long waves with that of the mooring system in the harbor. Hiraishi et al. (1997) conducted a series of field observations to investigate the mechanism of damages by long period wave in ports. The results of observations demonstrate that the long period waves are mainly induced as the non-linear bounded waves in grouping waves and that their resonance with the natural period of mooring system amplifies the surge motion. In 2002, The Ministry of Land, Infrastructure, Transport and Tourism (hereafter, referred to as MLIT) carried out interviews to uncover the actual situation of the long period wave problem. Results of these interviews can be found in the home page of MLIT. Some examples of the damage described in these interviews are as follows:

- Frequent interruptions of cargo handling in Onahama port
In Onahama Port in Fukushima prefecture, more than 100 ships were forced to suspend cargo handling for short durations from January to November 2001.
- Large amount of fiscal damage in Shibushi port
In Shibushi port in Kagoshima Prefecture, due to the influence of long-period waves generated from a typhoon in Okinawa main island, a ship collided with the quay. Fiscal damage for the quay reached about 650 million yen. There is no mention of the ship damage, but it must have been a considerable amount.
- Personal injury in Tomakomai port in Hokkaido
In Tomakomai port in Hokkaido, a personal injury, laceration of hand occurred. The injured party was tying additional ropes to secure the ship. At that moment, the rope broke and he was injured.

A fifth assessment report of IPPC was published at the time this paper was being written. The relationship between climate change and long-period waves in a harbor is not clearly defined. However, the wave height of long-period waves in a harbor is proportional to the value of the same outside the port. In the event that the scale and frequency of storms increases due to global warming, it is expected that the long-period waves inside the harbor will become more severe.

Countermeasures include improvements in the mooring system, extension of offshore breakwaters, establishment of warning systems for vessels and long-period wave absorbers in the harbor (e.g., Hiraishi and Hirayama, 2002). Yamada et al. (2005) and Hiraishi et al. (2009) conducted model and on-site studies of mound type wave absorbing structures installed on the harbor side of breakwaters. In this study, different types of rubble mound structures to reduce the energy of reflected long-period waves in the harbor were tested. When installing such structures on site, a suitable arrangement to take the required harbor tranquility into account, should be prepared. The discussion herein is concentrated

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on the wave absorbing performance of the mound type structure itself and the stability characteristics of the armor material covering such structures.

Although rubble mounds for conventional structures can be expected to provide some wave absorbing performance, a width of over 30m would be required to attenuate the effect of long-period waves (e.g., Hiraishi et al. 2009 and Matsuno et al. 2011). To accommodate effective port operations and the costs associated with this, it is therefore clearly important to reduce the size of the structure according to specific site conditions. The crown heights of conventional mound type structures are almost equal to those of the caissons behind them. In this study, a slightly submerged mound type long-period wave absorbing structure (hereafter referred to as a submerged mound) is proposed. This structure has a high performance, and its basic concept is to level the crest elevation to the water surface to establish high efficiency in energy dissipation on the surface of the crown of the mound. To clarify details on the wave absorbing performance, a series of hydraulic model experiments on wave reflection were carried out.

Because such mound structures are to be situated behind in a port's outer breakwater which faces the open sea, severe wave overtopping will inevitably be encountered (e.g., Hayakawa et al. 1998). For such a structure to be stable, it is necessary that the armor units in it be sufficiently stable to overcome wave overtopping. However, no design method for armor units in such a structure has been established yet. Therefore stability tests on armor units against wave overtopping from the sea side were conducted, with the aim of proposing a design formula for such armor units.

This structure also has a function of increasing the resistance against caisson sliding due to tsunami force. It also has the effect of retarding the time to scour out the mound during tsunami overflow. Mitsui et al. (2013) investigated such additional expected functions in a hydraulic model experiment.

WAVE ABSORBING PROPERTIES

Experimental Conditions

Wave flume setup. A series of experiments was conducted using a 50m-long, 1.0m-wide and 1.3m-deep wave flume equipped with a piston type wave generator. Figure 1 shows the wave flume setup. The long-period wave absorbing structure model was situated on a horizontal bottom modeling a uniform depth in the harbor. The wave reflection coefficient K_R was estimated based on Goda and Suzuki (1976) using instantaneous records of water surface elevation obtained by two wave gauges located at the center of the horizontal bottom keeping the gauge spacing one-fourth of a wavelength. Since monochromatic waves corresponding to the resonance frequency of the harbor are an important factor, regular waves with periods of 4.24 to 16.97s and heights of 0.5 to 3.0cm were used. The model scale was set to 1/50 based on the Froude number.

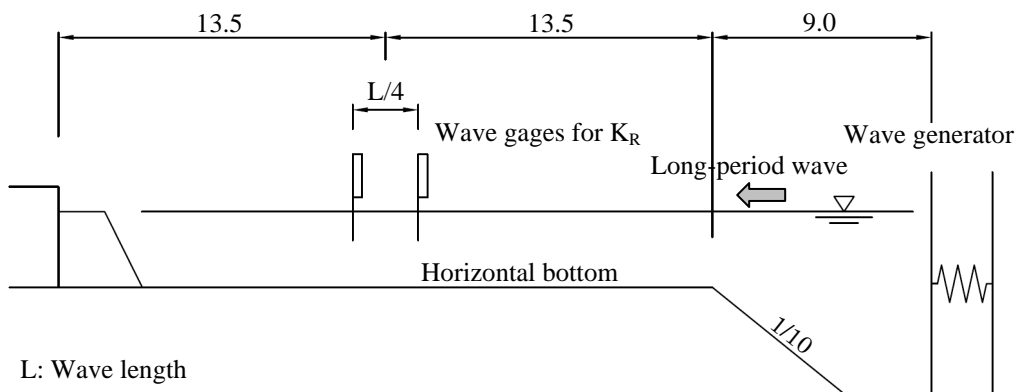


Figure 1. Wave flume setup (units in m).

Test sections. Conventional and submerged mounds were examined. In the initial stage, fundamental properties in the wave absorbing performance of both structures were investigated. After this, the characteristics of the submerged mound were analyzed in detail. The test sections in the initial stage are shown in Figure 2. The experimental conditions are shown in Table 1. The crown heights

above the water level for conventional mounds were set to 10.0cm. On the other hand, the crest elevation of the submerged mounds coincided with the water level. The rubble mound consisted of 0.4 to 1.6g stones. In the initial stage, the water depth h was 20.0cm. The widths of the mounds at the still water level were set to 60.0cm. Armor stones were used because the conventional mound type structures on site have used armor stones. Armor stones of 8.0g were used in a two-layer placement. In the second stage, h was 14.0 to 32.0cm. The widths of the mounds were set to 30.0 or 60.0cm. Tetrapods and X-Blocks were used as examples of wave dissipating concrete blocks and concrete armor blocks. Tetrapods, 14.5 to 235.1g, were used in a two-layer placement while X-Blocks of 16.2g were used in a one-layer placement. Figure 3 shows the geometry of the blocks.

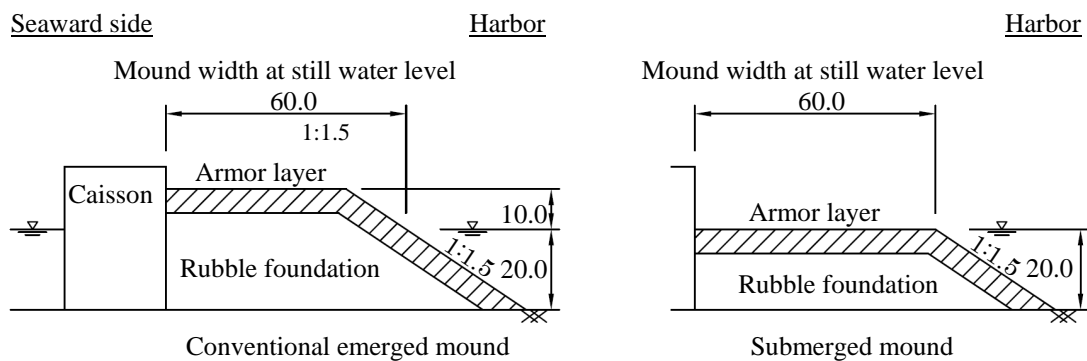


Figure 2. Test sections for wave absorbing properties in initial stage (units in cm).

	Model scale		Prototype scale	
Scale	1/50		1	
Wave period	4.24 - 16.97s		30 - 120s	
Wave height	0.5 - 3.0cm		0.25 - 1.50m	
Crown height above the water level	Emerged: 10cm Submerged: 0cm		Emerged: 5m Submerged: 0m	
Rubble foundation	0.4 - 1.6g		50 - 200kg	
Mound slope	1:1.5		1:1.5	
Water depth	Initial stage	Second stage	Initial stage	Second stage
	20cm	14, 20, 26 and 32cm	10m	7, 10, 13 and 16m
Mound width on the water level	60cm	30 and 60cm	30m	15 and 30m
Armor material	8.0g armor stones (two-layer)	14.5g } Tetrapods 60.5g } (two-layer) 121.0g } 235.1g }	1t armor stones (two-layer)	2t } Tetrapods 8t } (two-layer) 15t } 29t }
		16.2g X-Blocks		2t X-Blocks



Tetrapod



X-Block

Figure 3. Geometries of Tetrapod and X-Block.

Experimental Results

Difference between conventional and submerged mounds. Hereafter a prototype scale notation is used unless a particular explanation is given. Figure 4 compares the reflection coefficients of conventional and submerged structures shown in Figure 2. Both structures are covered with armor stones and the width at the still water level was 30m. The reflection coefficient of the submerged mound is smaller than that of the conventional type, independent of the wave period. As is often pointed out in the literature, for example Madsen (1983), the energy dissipation of a permeable breakwater takes place not only inside the porous structure but also on the surface of the structure due to friction. Since a submerged structure has a larger amount of surface area compared to that of a conventional structure, it is thought that the surface area of the submerged structure leads to effective energy dissipation due to friction there. Matsumoto et al. (2013) reproduced such effective energy dissipation of a submerged structure by numerical computation. The difference in the reflection coefficient becomes more pronounced with the decrease in wave period. Therefore in the rest of this paper, the focus will be on the submerged mound.

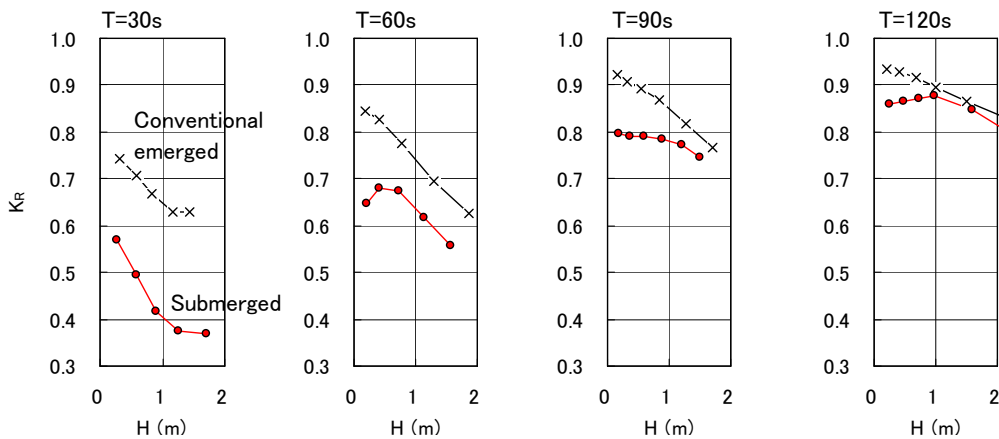


Figure 4. Reflection coefficients of stone armored mound.

Influence of armor material. Figure 5 shows the relationship between the wave period and the reflection coefficients of the submerged mound with various armor materials. When the wave period is below 80s, the K_R with Tetrapod covering is the smallest among all the structures, while that with the armor stone covering is the smallest against wave periods above 80s. The K_R with Tetrapod covering seems to converge to 0.9 within the range of this study. The change in K_R with X-Block covering with variation in the wave period is almost the same as that with armor stone covering. Also, the change in K_R with 8t Tetrapod covering is almost the same as that with 2t Tetrapod covering. Although the details are not shown, non-significance in the dependency of the size of the Tetrapods on K_R was observed. Because the Tetrapod covered mound can realize low wave reflection in a wide range of wave periods, the use of Tetrapods appears to be highly effective for this type of structures.

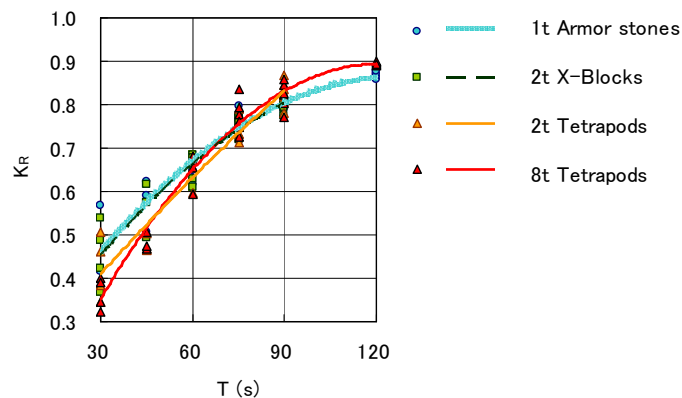


Figure 5. Relationship between wave period and reflection coefficient.

Influence of water level change. In the previous subsection, the usefulness of a submerged mound was clearly confirmed. The high wave absorbing performance for idealized conditions, *i.e.*, when the crest elevation coincided with the water level, was verified. However, in an actual site condition the structure would be exposed to water level variation due to tides. Accordingly, in the following section the influence of water level change is discussed.

In the coasts around Japan, generally speaking the difference between flood and ebb tide is less than 2.5m. Therefore in the experiments, a tidal variation of $\pm 1.5\text{m}$ was provided by changing the water depth from 8.5 to 11.5m against the fixed mound geometries, as shown in Figure 2. 1t armor stones and 8t Tetrapods were used for the conventional and submerged mounds as armor material. Figure 6 shows the relationship between the submerged depth of the crown d_s and the wave reflection coefficient K_R under the condition where the wave height is 0.5m. It is known that a long-period wave with wave height below 0.5m can cause a decrease in the efficiency of cargo handling, as shown in Hiraishi et al. (1996). The reflection coefficients of the conventional type increase with increase in water depth. On the other hand, those of the submerged type show a V-shaped distribution with a minimum value at the submerged depth of the crown of 0m. Deeper water level yields the reflected waves directly from the vertical wall of caisson, on the other hand, shallower water level diminishes the effectiveness of wave energy dissipation at the crest of the mound. Although K_R increases when the water level varies from its idealized position, the K_R of a submerged type indicates smaller values than those of a conventional type within the tidal range of $\pm 1.0\text{m}$. This confirms that the submerged mound with Tetrapod covering appears to be the ideal type of structure.

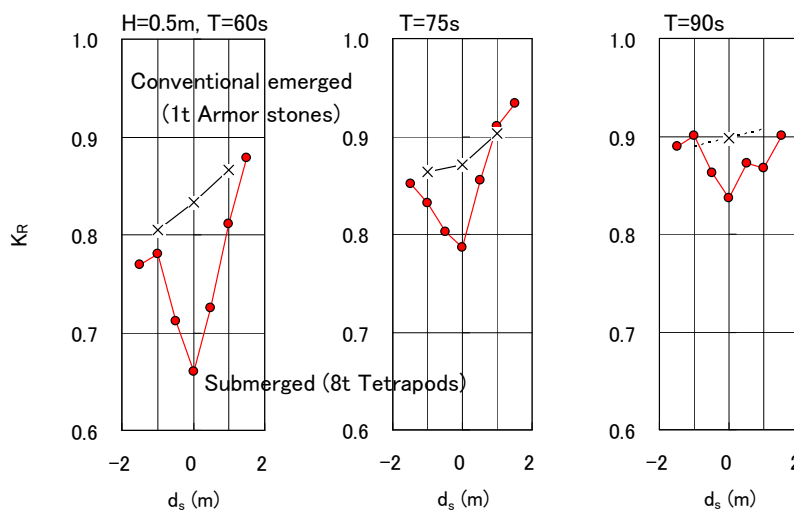


Figure 6. Relationship between submerged depth of crown and wave reflection coefficient.

Influence of water depth and mound width. Figure 7 shows the relationship between the equivalent mound width normalized by the wavelength B^*/L and the reflection coefficient K_R of the submerged mound with 8t Tetrapod armor layer for various combinations of water depths and the mound widths. This figure also shows the results of the conventional emerged mound structure covered with armor stones under the condition where h was 10m and B_M was 30m. A definition of B^* is found in the figure. Because the water particle motion of long-period waves is still present even near the sea bottom, the use of B^* , which includes the influence of water depth, is more appropriate when compared to the simple crown width of the mound B_M which is used for the current design of conventional mound type structures. It can be seen that K_R can be estimated by using B^*/L independently of the water depth and width of the mound.

Using this graph, when the K_R is 0.85, B^*/L can be graphically obtained to be 0.041 and 0.068 for the submerged and conventional mound structure, respectively. This would result in a 16.8m wide structure for the submerged case with Tetrapods, or a 32.8m mound for the conventional structure with armor stones. Table 2 summarizes the details of the calculation.

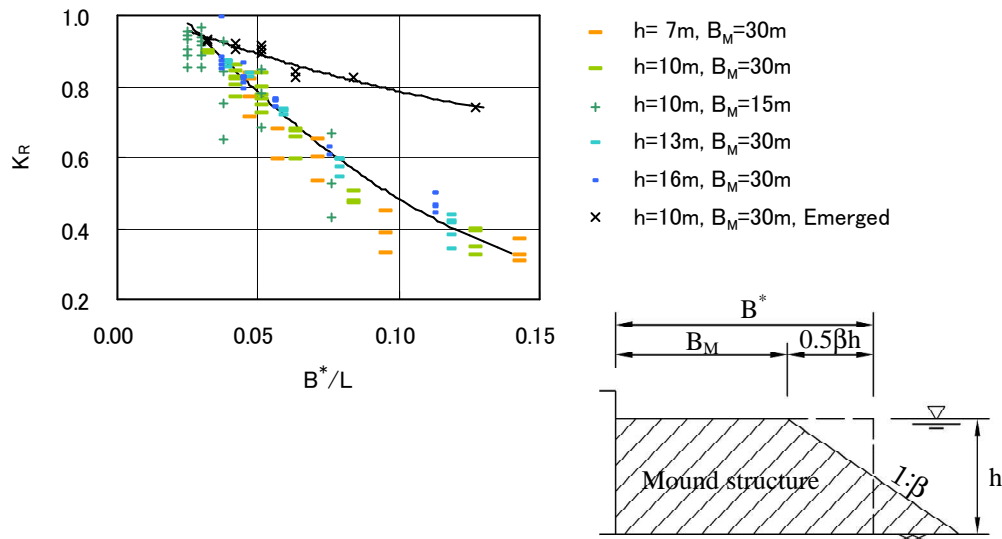


Figure 7. Relationship between B^*/L and wave reflection coefficient.

Structure type	K_R	Slope	h (m)	T (s)	L (m)	B^*/L	B^* (m)	B_M (m)
Submerged	0.85	1:1.5	10.0	60.0	593	0.041	24.3	16.8
Conventional						0.068	40.3	32.8

STABILITY AGAINST WAVE OVERTOPPING

Experimental Setup

A series of stability experiments were conducted using a 50m-long, 1.0m-wide and 1.3m-deep wave flume. The breakwater model with or without wave-dissipating works at the seaward side was situated on a 1/30 bottom slope. The test sections are shown in Figure 8, with the test conditions given in Table 3. The water depth was 34.0cm. The crown height of the caisson above the water level, h_c , was set to 9.0cm or 15.0cm, which is 0.6 or 1.0 times the design wave height for the stability of caisson H_D . The crown width of the caisson was 36.0cm. In order to develop a formula for the stability of Tetrapods, the mound width B_M was set to 30cm ($= 2H_D$) or 60cm ($= 4H_D$). The wave period was 1.98s, and the frequency spectrum of irregular waves was of the Modified Bretschneider-Mitsuyasu type.

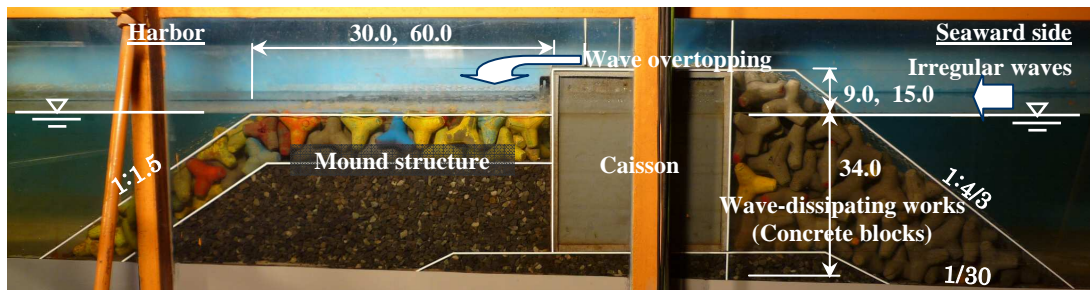


Figure 8. Test sections for stability experiments (model scale, units in cm).

Table 3. Test conditions for stability experiments.		
	Model scale	Prototype scale
Scale	1/50	1
Significant wave period	1.98s	14s
Significant wave height	6 - 18cm ($H_D=15\text{cm}$)	3.0 - 9.0m ($H_D=7.5\text{m}$)
Water depth	34cm	17m
Crown height of caisson above the water level	9cm ($=0.6H_D$) 15cm ($=1.0H_D$)	9cm ($=0.6H_D$) 15cm ($=1.0H_D$)
Width of caisson	36cm	18m
Mound slope	1:1.5	1:1.5
Mound width	30cm ($=2H_D$) 60cm ($=4H_D$)	15m ($=2H_D$) 30m ($=4H_D$)
Rubble foundation	0.4 - 1.6g	50 - 200kg
Armor stones (two-layer)	8.0g	1t
Tetrapods (two-layer)	60.5, 121.0 and 235.1g	8, 15 and 29t

The test started with small waves, which did not cause damage, with the wave height being gradually increased. The number of waves for each wave height rank was set to approximately 1000. After each wave attack, the section was not rebuilt.

In this study, damage to the armor material was defined in two ways. One is the deformation level S proposed by Van der Meer (1987) originally for armor stones on the slope of a rubble mound breakwater with a high crown. Eq. (1) shows the definition of the deformation level:

$$S = \frac{A_e}{D_n^2} \quad (1)$$

where A_e is an eroded area of cross section and D_n is the cubic root of the volume of the armor material. Photo 1 shows the damage of mound. Figure 9 shows the schematic of damage. The eroded area A_e was estimated by using a mechanical type topography profiler.

The other type of damage definition was the damage level N_0 aiming at a design formula for Tetrapods. Eq. (2) shows the definition of the damage level:

$$N_0 = \frac{nD_n}{B_F} \quad (2)$$

where n is the total number of dislocated Tetrapods and B_F is the width of the wave flume in which the experiments are carried out. The damage level N_0 represents the number of dislocated Tetrapods within a width D_n in the direction of the breakwater alignment. For this expression, damage was defined as when the Tetrapod had moved more than half its length from its initial position or rotated more than 45 degrees or lifted up more than its height. The number of dislocated Tetrapods was counted by visual observation and an analysis of photographs shot before and after wave attack.

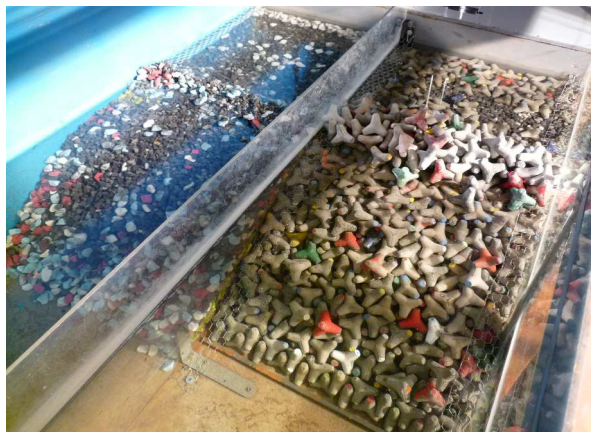


Photo 1. Damage of mound structure.

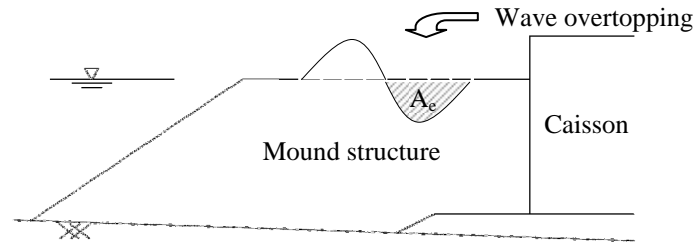


Figure 9. Schematic of damage.

Experimental Results

Damage progression on stone armored mound. Figure 10 shows the damage progression on the stone armored mound due to wave overtopping under the condition where h_c was $0.6H_D$. In this case, wave-dissipating works were installed at the seaward side of the breakwater. The x -axis shows the distance from the harbor side wall of the caisson, and the z -axis shows the height from the crown of the mound. The design wave height for the stability of caisson H_D was 7.5m. The amount of eroded area increased with an increase in wave height. If the mound width was $4H_D$, almost all the armor stones were washed away and the underlayer stones were seen around the position $x = 10$ m when the normalized wave height $H_{1/3}/H_D$ was 0.8. This was a very dangerous situation because it represented a situation where the underlayer was being extracted, which could lead to the collapse of the entire structure. As for the displaced armor and underlayer stones, they were deposited on the crown of the mound when $H_{1/3}/H_D$ was less than 0.8, and this displacement proceeded to the slope of the mound when it exceeded 0.8. If the mound width was $2H_D$, it was observed this situation on the crown and the slope when $H_{1/3}/H_D$ was 0.7, and the shoulder received serious damage because of direct impact of the wave overtopping when it was 0.8.

When $H_{1/3}/H_D$ became 1.0, i.e., after the design wave attack, the crown received serious damage and the toe of the slope moved approximately 5m from those initial positions in both cases. This advance of the toe of the slope is not desirable, as it can interfere with port navigation operations, especially for the case of the bigger vessels.

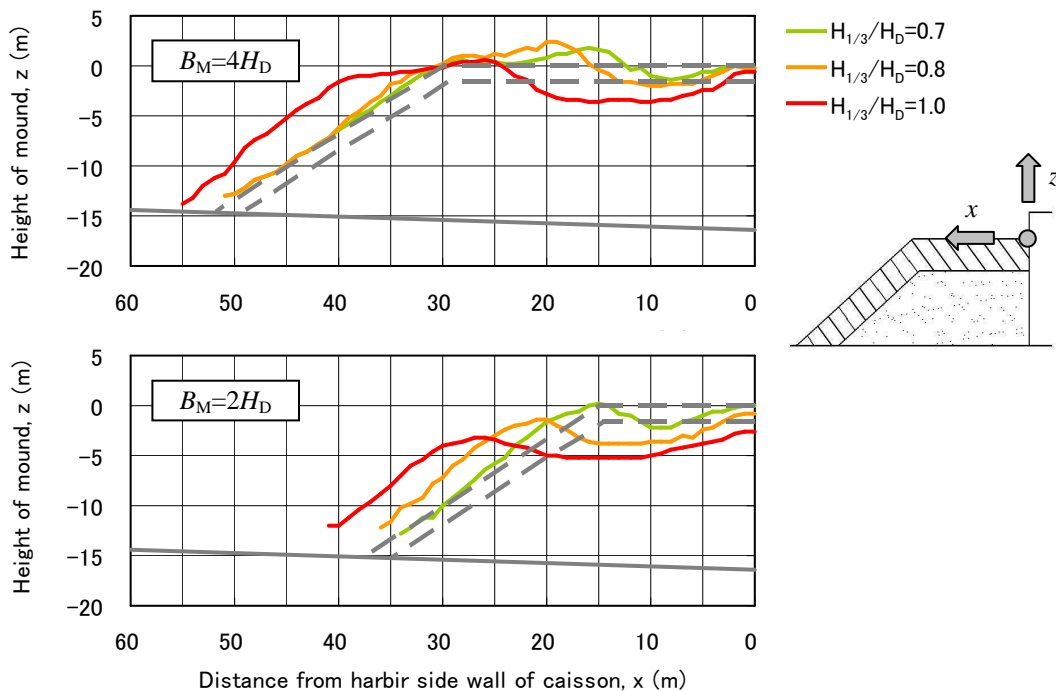


Figure 10. Change in mound shape with armor stones. ($h_c=0.6H_D$, with wave-dissipating works at the seaward side)

Influence of armor material. Figure 11 compares the erosion depth at the crown of the stone or Tetrapod armored mounds after the design wave attack ($H_{1/3}/H_D = 1.0$). Since H_D was 7.5m, the position $x/H_D = 4$ corresponded to $x = 30$ m, which is the shoulder of the slope. A huge amount of deformation was observed when stones were used as the armor, whereas the deformations were relatively small when Tetrapods with heavier weights were used. Specifically, the maximum normalized erosion depth of the armor stone section reached up to 5. Since the armor stones were placed in two layers, an erosion depth of more than $2D_n$ could cause a direct outflow of rubble stones. When Tetrapods were used, the maximum erosion depths were reduced to less than $2D_n$.

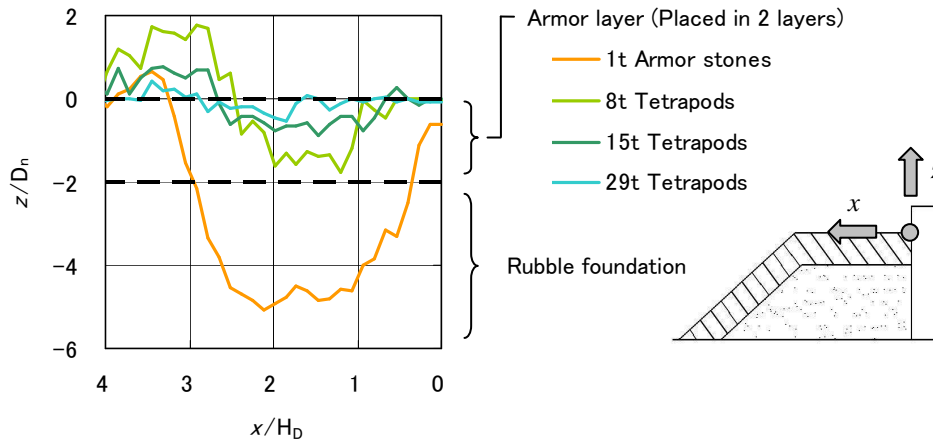


Figure 11. Erosion depth after design wave attack. ($h_c=0.6H_D$, with wave-dissipating works at the seaward side)

Influence of crown height of caisson and mound width. Figure 12 shows the relationship between the normalized wave height $H_{1/3}/H_D$ and the deformation level S of the 15t Tetrapod armored mounds. As shown in this figure, the deformation level decreases with the increase of the crown height of the caisson above the water level and the mound width. Also, the deformation level with wave-dissipating works at the seaward side became smaller than that without them, independent of the sectional condition.

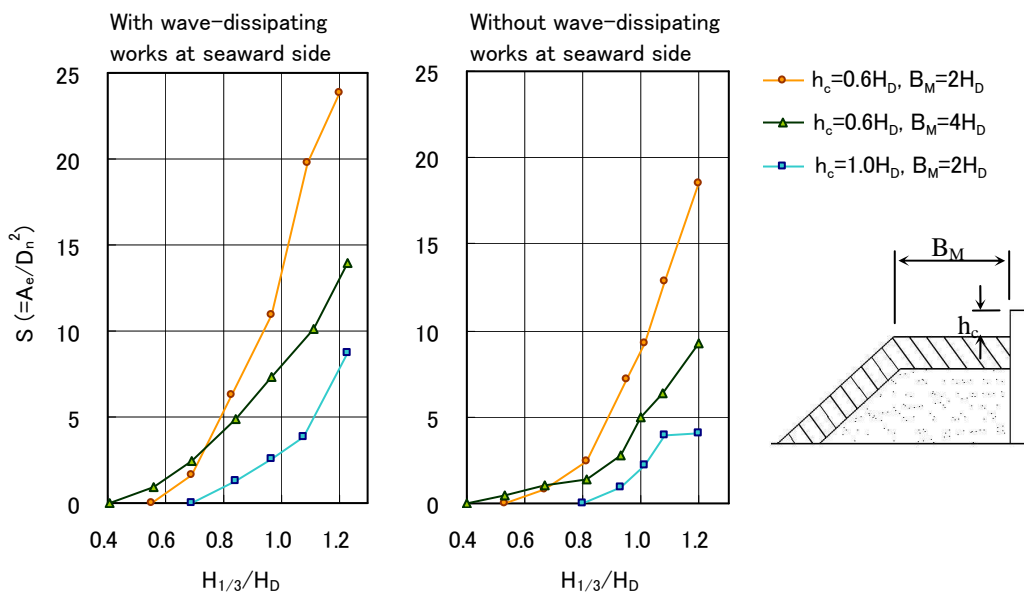


Figure 12. Deformation level of 15t Tetrapod armored mound.

Stability number for Tetrapods. To establish a prediction formula for stability number for Tetrapods used in the Hudson formula, the empirical equation proposed by Hanzawa et al. (1996) for Tetrapods in a horizontally composite breakwater was modified by introducing factors k_a , k_b and k_c . These factors represent the effects of enhanced stability due to energy loss during wave overtopping. Eqs. (3) and (4) show the original Hanzawa formula and the proposed one, respectively:

$$N_S = C_H \left\{ 2.32 \left(\frac{N_0}{N^{0.5}} \right)^{0.2} + 1.33 \right\} \quad (3)$$

$$N_S = C_H \left\{ 2.32 k_a \left(\frac{N_0}{N^{0.5}} \right)^{0.2 k_c} + 1.33 k_b \right\} \quad (4)$$

where N_S is the stability number, C_H is the reduction coefficient for wave breaking ($= 1.4/(H_{1/20}/H_{1/3})$), $H_{1/20}$ is the one-twentieth highest wave height and N is the total number of waves. The modification factors k_a , k_b and k_c were set as shown in Table 4 in such a way that the predicted stability numbers fit the experimental ones. Figure 13 compares the results from Eq. (4) with the experimental data employed when C_H was 1.0. The experimental stability number is calculated as $H_{1/3}/\Delta D_n$, in which, $\Delta = \rho_t/\rho - 1$, ρ_t and ρ denote the density of Tetrapods and the water, respectively. Figure 14 gives a correlation of the predicted and experimental stability numbers. The predicted stability number with the modification factors agreed fairly well with the experimental results.

Wave-dissipating works at seaward side	h_c/H_D	B_M/H_D	k_a	k_b	k_c
With	0.6	2.0	1.069	1.647	3.000
	0.6	4.0	1.483	1.789	
	1.0	2.0	1.884	1.970	
Without	0.6	2.0	1.647	1.346	
	0.6	4.0	1.358	1.549	
	1.0	2.0	2.724	1.564	

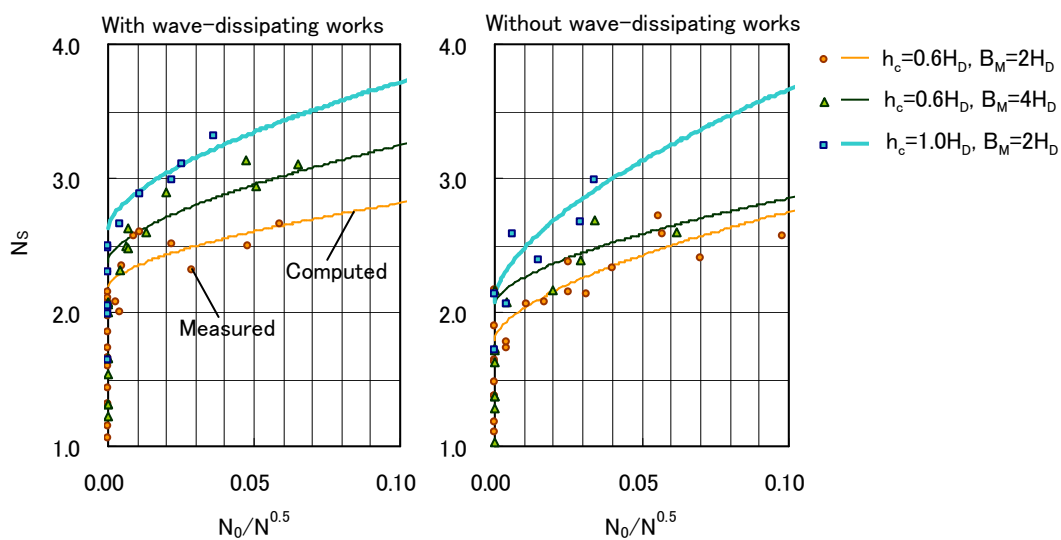


Figure 13. Comparison of measured and computed stability numbers.

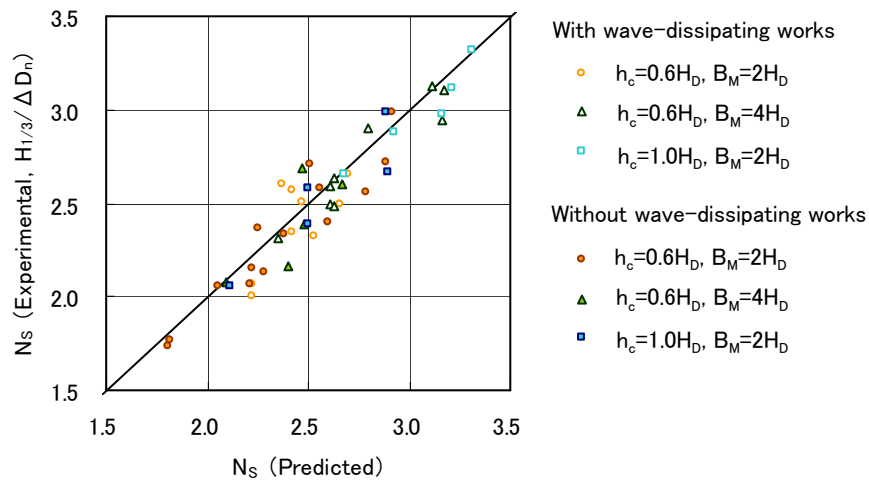


Figure 14. Correlation of predicted and experimental stability numbers.

For example, if the h_c was $0.6H_D$ with wave-dissipating works, and the B_M was $2H_D$ as shown in figure 15, the prediction formula for the stability number for Tetrapods covering the submerged mound structure installed on the harbor side of breakwaters are shown in Eq. (5). Figure 16 shows the required mass of Tetrapods, M , for submerged mound installed on the harbor side, and for wave-dissipating works at the seaward side, under the conditions where $C_H = 1.00$, $N = 1000$, $\rho_r = 2.30t/m^3$ and $\rho = 1.03t/m^3$. The required mass of Tetrapods was calculated depend on the damage level N_0 .

$$N_s = C_H \left\{ 2.48 \left(\frac{N_0}{N^{0.5}} \right)^{0.6} + 2.19 \right\} \quad (5)$$

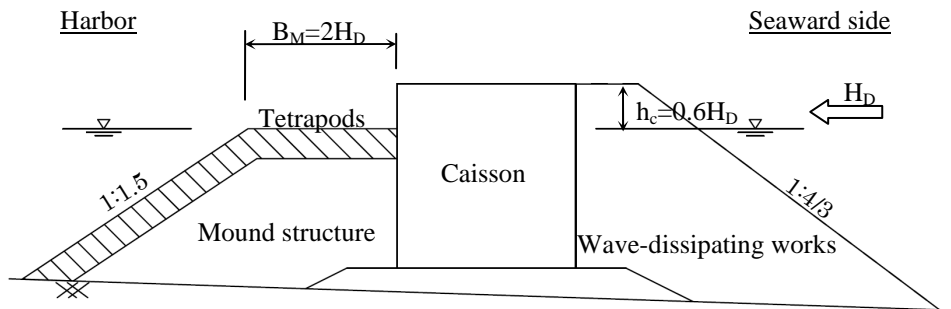


Figure 15. Example of design section.

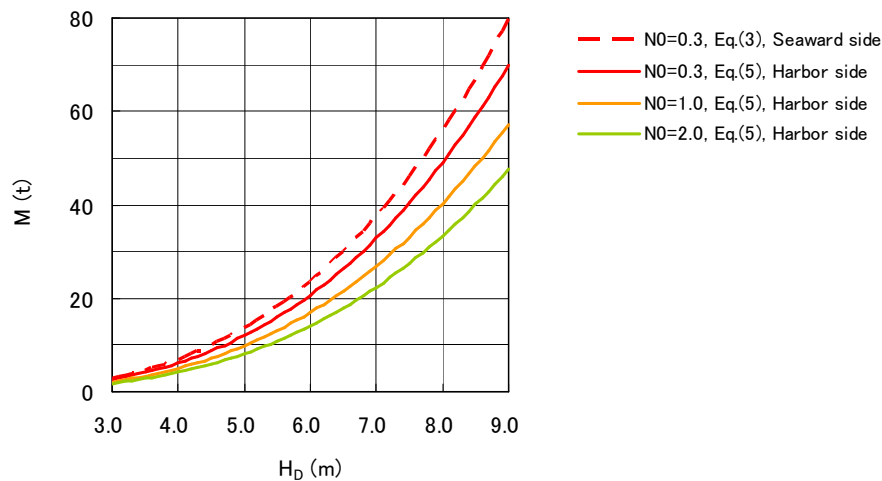


Figure 16. Required mass of Tetrapods.

CONCLUSIONS

As a countermeasure to problems in cargo handling, a submerged mound type long-period wave absorbing structure installed on the harbor side of breakwaters is proposed. The reflection coefficients of the structure were smaller than those of conventional type of rubble mound absorbing structures, independent of the wave period. This can result in a structure with a smaller size than conventional absorbing structures, and contribute to the reduction in cost of such countermeasures.

It is thought that wave overtopping from the seaward side could cause great deformation in such a structure for the case when armor stones are used. Therefore, a submerged mound type structure with concrete armor was proposed. To establish the prediction formula for the stability number for Tetrapods used in the Hudson formula, a modification to the existing formula of Hanzawa et al. (1996) was proposed. New prediction formulas for the stability number of Tetrapod used for this structure were hereby developed.

Based on the results of this study, the actual design of the submerged mound type structure is now possible for a variety of conditions. By using the proposed design diagram with an equivalent width of the mound structure taking the effect of the water depth into account, B^* , the required mound widths can be obtained. Also, the required mass of Tetrapods can be calculated by the harbor side prediction formula for the submerged mound structures.

In this study, a mound width was fixed to 2 or 4 times the design wave height. Also, an elevation of the caisson above the still water level was fixed to 0.6 or 1.0 times the design wave height. Experiments with the mound width of 3 times the design wave height are currently being conducted by taking into account an impact position of the wave overtopping. Furthermore, the ones with the crest elevation of 0.4 times the design wave height are also being conducted by taking into account the sea level rise caused by global warming. Results of these experiments will be published in other papers.

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