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Analysis on the stability of rubble mound breakwater and wave overtopping phenomenon

Kyu-Han Kim, O-Gi Kwon, Kyu-Tae Shim

Abstract: ***This study, by Physical Model Test, has reviewed the displacement of blocks and wave overtopping due to the external force change when constructing breakwater as a counter facility in the area of Nghi Son of Vietnam, where the construction of a Refinery & Petrochemical Complex is planned. The front of the breakwater is coated with two layers of RAKUNA-IV, which is concrete armor unit. 100yr Return Period Tidal Level (C.D+5.45m) and 100yr Return Period Storm Wave (H_{m0}: 3.7m,T_p: 11.8sec) are set for the design wave condition to take into account the influence of sea-level rise caused by global warming. The test revealed that the stability of the blocks against the occurrence of incident waves is affected by wave run-up at the front of the breakwater, wave overtopping discharge and JET flow between blocks. Also, it found that there was displacement of blocks at the crest and rear slope of the sections without any coping concrete along with local exposures of core stone. Therefore, cap concrete on the breakwater was installed to decrease wave overtopping discharge and overflow velocity, which resulted in the increase of stability of the armor unit, when Damage Parameter of Nod, which is suggested by The Rock Manual 2007 was applied and the value of EurOtop 2007 was employed as Allowable Overtopping Rate.***

Keywords: Armor unit, Physical model test, Rubble mound breakwater, Stability, Wave overtopping

1. Introduction

1.1. The aim of the study

Most of such facilities as Nuclear Power Plants and Thermoelectric Power Plants are being constructed on the coast because the cooling system can be efficiently operated, the cost of the transport of raw materials is low and because such a location confers a geographical advantage. A large-scale breakwater is constructed at the front sea area of those facilities in order to maintain the tranquility of the harbor environment and defend against high waves like storm surges or tsunamis. However, such breakwaters constructed for wave control purposes have been damaged as the displacement of wave-dissipating blocks at the front and the overturning of coping concrete to the rear side occurs due to abnormally high waves whose magnitude is getting bigger. Accordingly, a constant stability should be secured at counter facilities for the normal operation of power plants. As the climate becomes more erratic and sea-levels increase and the frequency of sudden high-waves becomes more common, the occurrence of damage according to different scenarios needs to be considered and various situations should be anticipated in order to protect the stability of the structure.

This study, through a physical model test which is very effective for the review of structure stability and wave deformation inside harbors, sought to determine the best ways for securing breakwater stability and harbor tranquility at a breakwater which will be constructed at the front seaside of a petrochemical complex so that facilities for the entry and departure of vessels can be secured. In addition, in order to prevent any possible damage in the future and enhance reliability, the appropriateness of the design and the correlations reflected in the construction were examined.

1.2. Literature study

Studies on structure stability have been conducted in different ways to prevent any unexpected disaster in advance. Representative examples of measures undertaken include repairing or reinforcing existing facilities and installing new structures after field investigations, numerical simulations, and hydraulic experiments. However, this study focused on the research trends regarding experiments because its main purpose was to review the stability of the breakwater by a physical model test. For the examination of the breakwater, tests on the stability of the coping concrete and on the wave dissipation block mounted at the front of the breakwater and the destruction of the rear slope due to wave overtopping have been conducted. The stability of the head part has also been investigated for the establishment of a reinforcement plan suitable in the case of any damages caused by diffraction or other factors. These kinds of studies have been conducted by many researchers, one of which was performed by Shinde et al [1] in the form of a physical model test for the examination of the stability at a rubble mound breakwater erected to expand a fishing port. In this test the proper weight of the wave dissipation block which maintains stability against any changes in the incident waves at each section was examined. Andersen et al [2] and Van der Meer et al [3] conducted various systematic investigations on eroded areas where rubble mounds with a slope of 1:1.5 are reshaped as a result of plunging-type waves and surging-type waves. Comola et al [4] investigated damage patterns and the progression thereof caused by wave period changes and multi-directional irregular waves at rock armored breakwater roundheads. Based on the results of their study , they suggested that the head part is mostly destructed between 70°~135° and that the width of a single unit ±3 experiences the greatest impact at the still water level. On the other hand, Uchida et al [5], Morikawa et al[6], Ogura et al [7], Jikuhara et al [8] and others employed a 3d model test to investigate the stability of breakwater wings and discontinuous sections against the incident waves with a steep slope of over 60°.

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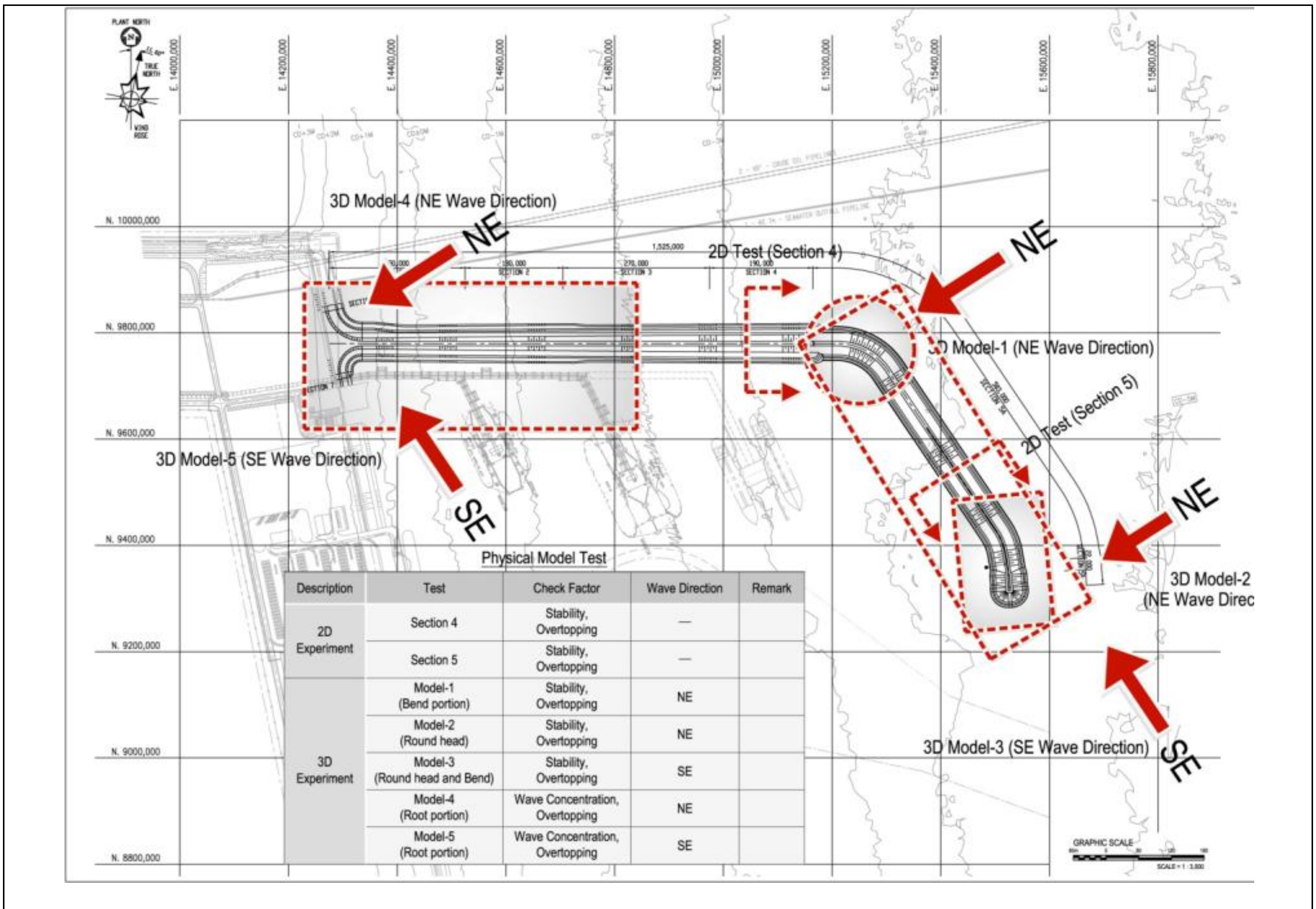
Kunisu et al [9] and others carried out a study on stability and wave pressure on structures by installing wave dissipation blocks at the front by decreasing the width between trunk parts connected to the head part of caisson breakwater as a plan for cutting down construction cost. Based on Van der Meer [10]; Van der Meer and Pilarczyk's [11] formula and with the results of a series of physical model trials, Melby [12]; Melby and Kobayashi [13] proposed an empirical formula that allows the prediction of mean damage progression with regular wave events, however the formula is limited to new structures because of the zero initial damages assumption. To avoid this shortcoming Melby and Kobayashi [14], [15] proposed two new methods. In these new methods the damage in every event only depends on the incremental time, thus the predicted damage progression is too sensitive to the wave height change. Using these results and additional tests, Melby and Kobayashi [16] developed a new model. The model's equations are based on the maximum momentum flux in the incident waves and explicitly include the effect of water depth at the toe of the structure. Melva et al [17] centers on this aspect and proposes two new approaches analyzing its accuracy by comparing real and theoretical damage progression on the main amour layer, with the model (Melby and Kobayashi [16]) used. Kim et al [18] analyzed the phenomenon of stability and wave overtopping with variation of incidence waves and water level through hydraulic model test when the wave dissipation blocks are coated randomly on the front side of the rubble mound breakwater. In the test, irregular waves using JONSWAP spectrum was applied. As a result of the test, it is confirmed that displacement of block is closely related to the change of tide levels and is affected by the installation type of the blocks. However, this study, unlike any of those mentioned, attempted to understand the rate of block-displacement caused by wave overtopping due to changes in external forces.

2. Set-up of experiment condition

2.1. Target area and external force for testing

The study target area is Nghi Son on the east coast of Vietnam where construction of a large petrochemical refinery is planned. The wave hindcast based on observation results in Figure 2 found that NE and SE waves are predominant at the front sea of Nghi Son and incident high waves would affect the structural stability. The purpose of the construction of the breakwater is to ensure that cargo work can be done safely, and an increase in wave overtopping can negatively affect the tranquility of the harbor and the motion of ships. Therefore, the occurrence of wave overtopping is a very important factor. Based on the resulting data, as shown in the overall 3D layout plan in Figure 1, it was possible to determine the hydraulic characteristics and stability of marine structures that would be constructed. Considering various situations that might occur during loading and unloading work after the completion of structure construction, waves of a 1 year return period and those of over a 100 year return period were applied. In addition, based on the consideration that sea-levels will rise, scenarios of high water levels and wave heights with a safety margin of more than 120% of the 100-year return period were examined. In order to fully gain an understanding regarding the stability of the structure based on the experiment, the construction cost among various scenarios was also considered with the 100-year return period (Wave 4) model employed. Table 1 shows the external forces associated with each scenario. Since the wave height differs at every section, the value of the range from the head part to the root part was applied.



Description	Test	Check Factor	Wave Direction	Remark
2D Experiment	Section 4	Stability, Overtopping	—	
	Section 5	Stability, Overtopping	—	
3D Experiment	Model-1 (Bend portion)	Stability, Overtopping	NE	
	Model-2 (Round head)	Stability, Overtopping	NE	
	Model-3 (Round head and Bend)	Stability, Overtopping	SE	
	Model-4 (Root portion)	Wave Concentration, Overtopping	NE	
	Model-5 (Root portion)	Wave Concentration, Overtopping	SE	

Fig 1: 3D-Layout plan for physical model test and construction

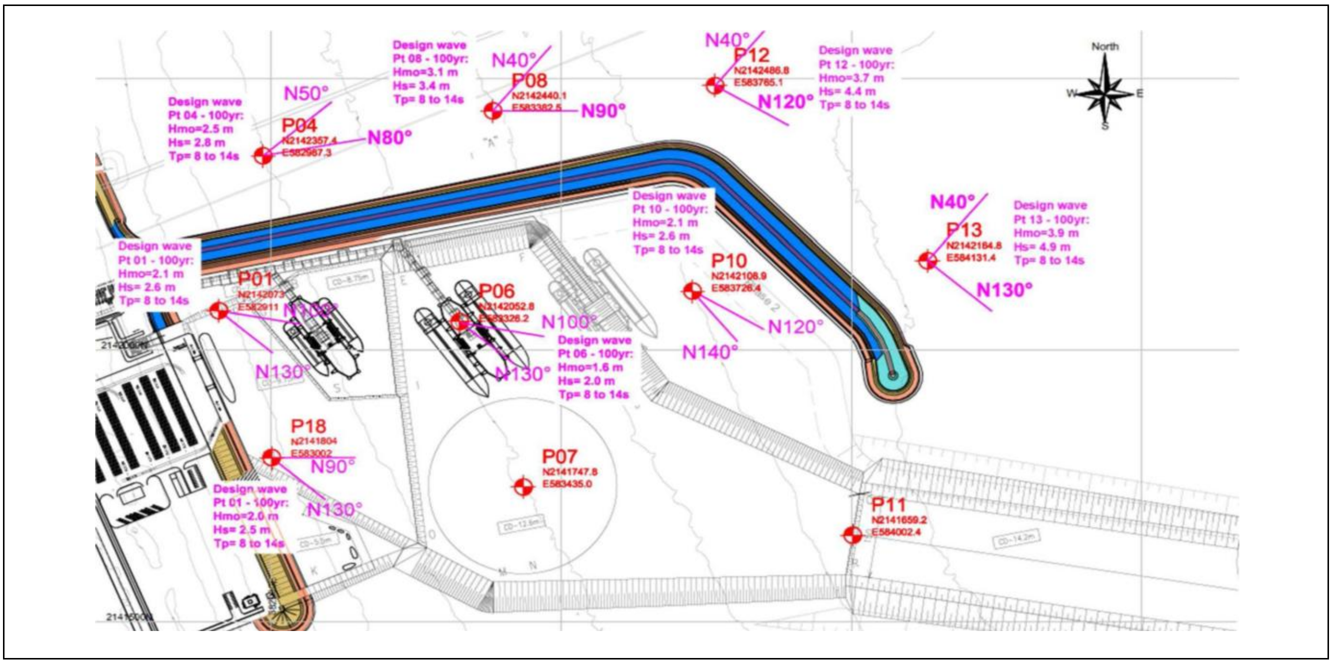


Fig 2: Wave hindcast locations for design and physical model test

Table 1: Wave conditions

Scenarios	Conditions	R (year)	Hm_0 (m)	T_p (s)	Water Level(m)	Considering Factors
Wave 1	1 year storm, high water	1year	1year (1.7~2.3m)	1year (7.1s)	DW1 (CD+3.95)	-Operational wave overtopping
Wave 2	10 years storm, high water	10years	10years (2.1~3.5m)	10years (8.5s)	DW10 (CD+4.70)	-Seaside stability -Operational wave overtopping
Wave 3	10 years storm, high water, influenced of long waves (swell)	10years	10years (2.1~3.5m)	Swell (14.1s)	DW10 (CD+4.70)	-Seaside stability
Wave 4	100 years design condition, high water	100years	100years (2.5~3.9m)	100years (12.0s)	DW100 (CD+5.45)	-Seaside stability -Rear stability (extreme wave overtopping) -Wave overtopping discharge (two-step experiments)
Wave 5	Safety margin (Hm0 of 100-yr +water level CD+6.5)	100years	100years (2.5~3.9m)	100years (12.0s)	CD(+)6.50	- Seaside stability - Crest & Rear stability - Wave overtopping discharge (two-step test)
Wave 6	Toe stability at low water	10-100 years	10-100 years (2.7~3.25m)	100years (12.0s)	MLW (CD+1.00)	Toe stability
Wave 7	Extreme stability	100 years	> 100 years 120%×100 years (3.0~4.7m)	100 years (12.0s)	DW100 (CD+5.45)	-Seaside stability (Might not occur)

2.2. Experiment Facility and Method

The physical model test was conducted in the wave basin with a 40m length, 27m width, and 1.0m height. The wave generation paddle is 12 meters. The highest wave of 0.25m (H), a regular wave with the maximum return period of 3.0sec (T) and a unidirectional irregular wave can be created in it. During the test, the stability of the main sections of the breakwater was examined with the application of the Froude Number 1/50 as its model scale. As seen in Figure 3, the cross-section for the experiment is a rubble-mound breakwater, and head part and some of the trunk part are composed with cap concrete, and the top of the rest is covered with concrete wave-dissipation blocks. The concrete armour units coating the core layer of the breakwater are Rakuna-IV where 12.0t of the head part and 8.0t of the trunk part are applied. Rakuna-IV has a porosity of 56.5% and K_D values of 10.8 and 7.2 at the head and trunk parts respectively. Two layers of wave-dissipation block were mounted on the cross-section of the breakwater so that the sea water might penetrate between blocks when the sea-level reaches a certain height, and accordingly it was anticipated that the wave overflow rate would create a big difference in terms of the flow in relation to tide level and wave steepness conditions. Consequently, the increase of jet flow may cause block displacement when the flow passing through the blocks increases. Even though

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such different standards as found in shore protection

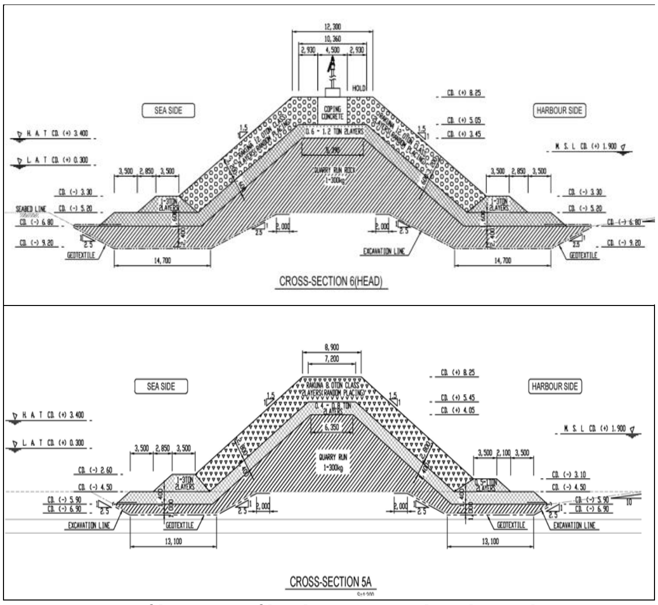


Fig 3: Profile section of head part (top) and trunk part (bottom)

Table2: Damage classification in model breakwaters (BS6349)

No.	Damage	Description
1	Destroyed	Core of breakwater affected
2	Serious	Core of breakwater visible
3	Much	Large gaps in primary layer; 5% of units displaced
4	Moderate	Gaps in primary layer; 3% of Units displaced
5	Little	2% of units displaced
6	Slight	1% of units displaced
7	Hardly	No damage

manuals or harbour-designing criteria were suggested for the determination of stability at the time of block displacement, the damage classification recommended in the rock manual bs-code 6349 [19] was employed for this study, and the examination area was divided into seaside slope, crest slope and rear slope to determine whether there is any instability when there is damage of over 5% or any exposure of the core filter at each section. The following formula (1) was utilized to determine the damage parameter (N_{od}) and Table 2 shows the analysis standard by grade according to the degree of displacement of the block.

$$N_{od} = N_{mov} / B / D_u \tag{1}$$

Here, N_{mov} is the number of blocks displaced by more than 0.5Dn or rocking with a large angle. B is considered the length

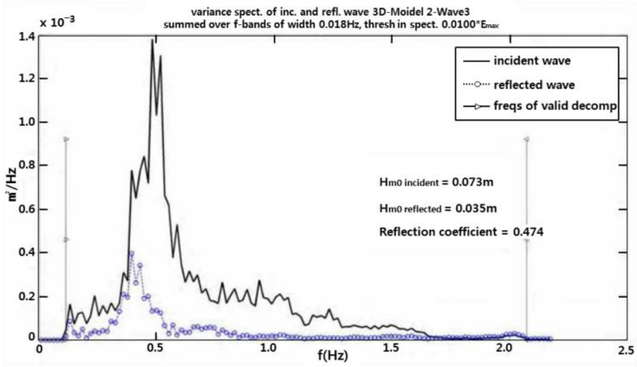


Fig 4: Spectrum of wave 3(H_{m0}:3.50m, T_p:14.70sec)

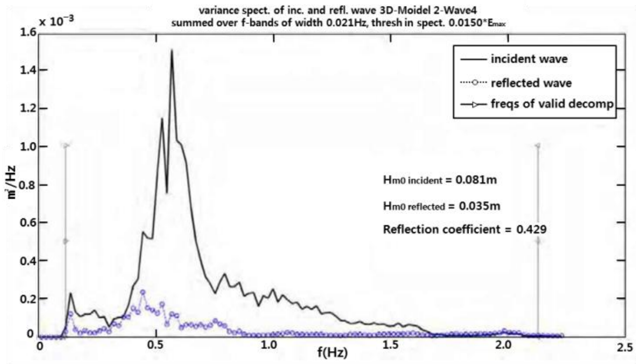


Fig 5: Spectrum of wave 4(H_{m0}:4.05m, T_p:12.40sec)

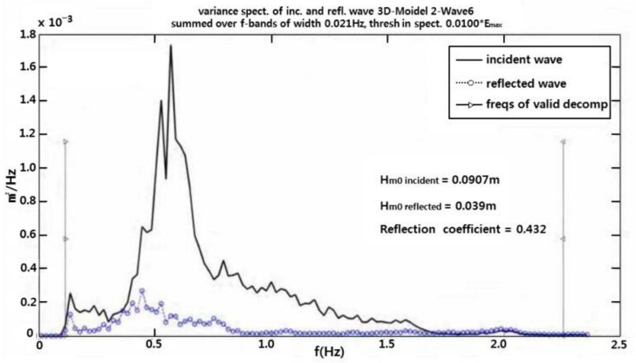


Fig 6: Spectrum of wave 6(H_{m0}:4.55m, T_p:12.40sec)

along the crest if the breakwater (considered trunk length B=5.8m, 1.3m at the boundary with the untested root part). Dn is the Nominal diameter of the model block (Dn=0.03 and 0.034m for 8.0t and 12.0t Rakuna units, respectably). When the damage classification exceeded the level much, it was concluded that there was instability. On the other hand, the total seawater passing over the top to the rear of the breakwater at the time of a wave attack due to the change in the tide level can be classified into wave overflow rate penetrating between concrete blocks and overtopping rate passing

Table 3: Wave overtopping classification (EurOtop,2007)

No.	Wave overtopping discharge	Description	Wave condition in the test	Remarks
1	q<0.1 ℓ/s/m	Insignificant with respect to strength of crest and rear structure	-	-
2	q=1.0 ℓ/s/m	On crest and inner slopes grass and/ or clay may start to erode	Wave1	-
3	q=10 ℓ/s/m	Significant overtopping for dikes and embankments. some overtopping for rubble mound breakwater	Wave2	Wave3 (50 ℓ/s/m)
4	q=100 ℓ/s/m	Crest and inner slopes of dikes have to be protected by asphalt or concrete; for rubble mound breakwaters transmitted waves may be generated.	Wave4	

the crest of the block. In this experiment not the flow penetrating between blocks, but the wave completely flowing over the crest was regarded as overtopping according to EurOtop2007(table 3) that suggests the criterion of allowable overtopping rate [20]. The above Figures 4~6 show the wave spectra applied to the experiment. Among the scenarios, Waves 3, 4 and 6 are shown representatively, and experiments were conducted based on the measured values. The measured values were found to be similar to each other in comparison with the target value.

Table 4: Test scene of bend part under wave 4 condition with NE dir.

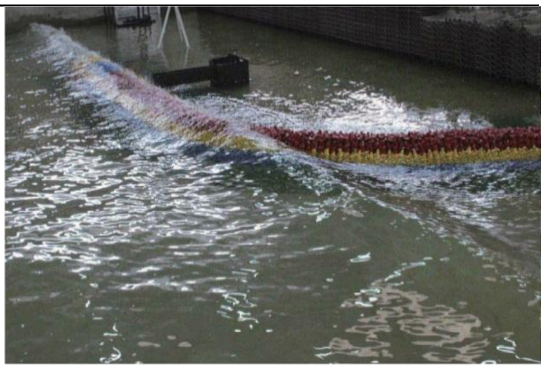
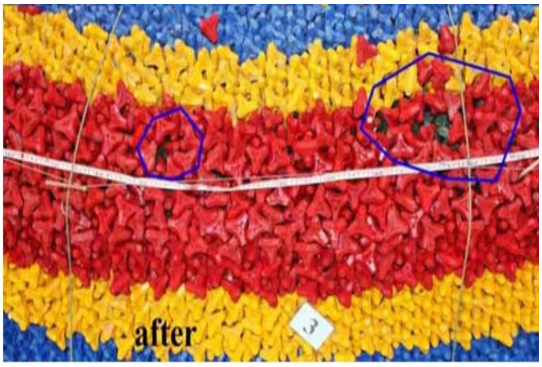
Condition	Result of before and after wave generation	Under attack of wave 4
Original (planned) design Condition		
		

Table 5: Test scene of round head part and major trunk part under wave 4 condition with NE dir. (original design)

Condition	Result of before and after wave generation	Under attack of wave 4 and result
Original (planned) design condition		
		

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3. Experiment results

As seen in Figure 1 and Table 1, the physical model test was carried out in the order of Bend part (NE dir.), Roundhead and Trunk part (NE, SE dir.), Root part (NE, SE dir.) to match each scenario. The outcomes from the tests at the Roundhead part, Trunk part and Bend part are presented in this paper in order to gain an understanding of the damage inflicted on blocks caused by overtopping. The test scenes and the results under the Wave 4 attack, which is the major external force for stability of breakwater, are shown in Tables 4 to 7.

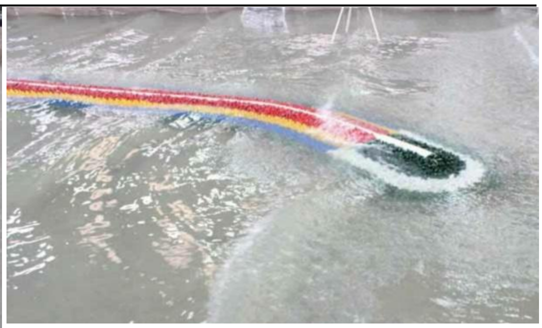
3.1. Analysis of Stability on Cross Section of Breakwater

For the stability test, the wave and tide level conditions were applied in each scenario. The damage parameter (N_{od}) based on the result from the creation of a stationary wave to the impact of 1000 waves became the criterion to determine

Table 6: Test scene of round head part and major trunk part under wave 4 condition with NE dir. (revised design)

Condition	Result of before and after wave generation	Under attack of wave 4 and result
Revised design condition		
		

Table 7: Test scene of round head part and major trunk part under wave 4 condition with SE dir. (revised design)

Condition	Result of before and after wave generation	Under attack of wave 4 and result
Revised design condition		
		

stability. During the test, from the attack of Wave 2 (10yrs.) the displacement took place independently at the Root Part, Bend Part and Round Part. At the Roundhead part and Major Trunk part, there was a large amount of overtopping discharge in the scenario of NE Wave 4 (100 yrs.). A complex flow occurred when the sea-water passed between the blocks mounted on the crest and the strong flow caused the displacement of blocks mostly mounted on the crest. Most displacements of blocks occurred at the straight sections of the Trunk part along with local exposure of the core filter. The cause of the intensive displacement of blocks on the crest part compared to the slope is that interlocking is weaker than the slope part of the block installed. When the overtopping happens, the moment force at the rear side of the shoulder part due to sea-water is increased. Though the displacement rate of the block was only 0.4%, it was determined to be unstable in Wave 4 (100 yrs.) conditions since it is regarded as Serious when the standards of Table 2 were applied. A plan to extend cap concrete from the major trunk part to the bend part in order to control the jet flow passing between blocks mounted on the crest was reviewed as a counter-measure against this. As seen in Table 4, the penetrating flow velocity was controlled during the Wave 4 (100 yrs.) scenario where the revised cross-section and the decrease of block displacement caused by overtopping led to no filter stone exposure which would occur due to the displacement. After repeating the test three times, the displacement rate of blocks was found to be 0.07%, 0.08%, and 0.06% respectively, and accordingly the stability was considered to be secured since the damage levels were very little. In the case of Wave 5 with the tide level 1.25 meters higher than Wave 4 (100 yrs.), the overtopping rate in the test duration significantly increased compared with that in the conditions applied in Wave 4. This was thought to be a phenomenon whereby the overflow depth deepened as a result of the relatively lowered freeboard (R_C) and the strong wave run-up and overflowing of the breakwater crest increased the rotation component to cause the displacement. The increased wave condition caused local exposure of the core. In the Wave 7 scenario where the wave is 1.2 times as high as that in Wave 4, the overtopping rate was more than that in Wave 4 and there was some exposure of the core due to block displacement as in Wave 5. However, since the two cases were for the only analysis of phenomena by scenario, the test result was not reflected in the design and construction. By applying these test results to understand what the difference in the number of blocks displaced would be when the crown concrete was extended and when not extended and to appreciate the correlation between ratio of wave height and water depth (H_{m0}/D) and wave steepness (H_{m0}/L) the range of stability by parameter and block displacement could be secured. As seen in Figure 7, when there were about 15 displaced blocks, the core was exposed, and with more external force than that in Wave 4 on the cross section of an original trunk part design there was local exposure. When the crown wall was extended, there was stability in Wave 4, while in Wave 5 and 7 the overtopping caused the exposure of the core part. According to the analysis of stability by changes in wave steepness (Figure 8), it was found that the steeper the wave, the more block displacement occurred. In particular, there was a drastic change in the trunk part. As seen in Figure 9, the analysis of damage parameter and relative wave revealed that there was the displacement of the biggest block in Wave 5 (H_{m0}/D: 0.33~0.36) with relatively low R_C. In spite of the relatively low H_{m0}/D, there was a high damage rate, the reason for which was that the increase in tide level caused the decrease of R_C resulting in the increase of overtopping. The examination of the round head & trunk part for SE wave in the circumstances of a cross section at the extended crown wall revealed that there was stability in Waves 4 and 7. However, in Wave 5 with a higher tide level, overtopping caused intensive displacement of blocks on the sides of the trunk part and crest part and rear part.

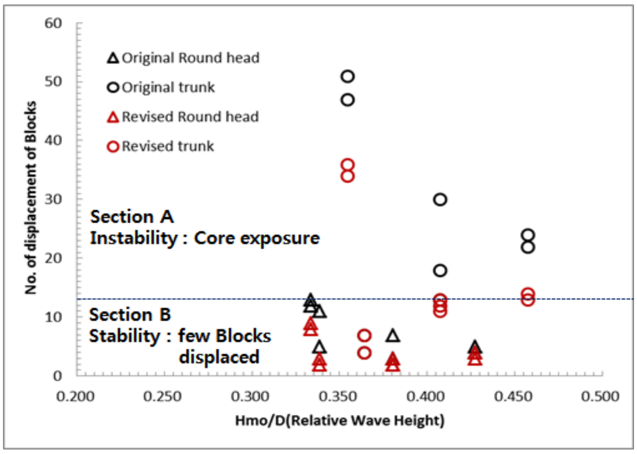


Fig 7: Relationship between relative wave height and number of displacement blocks

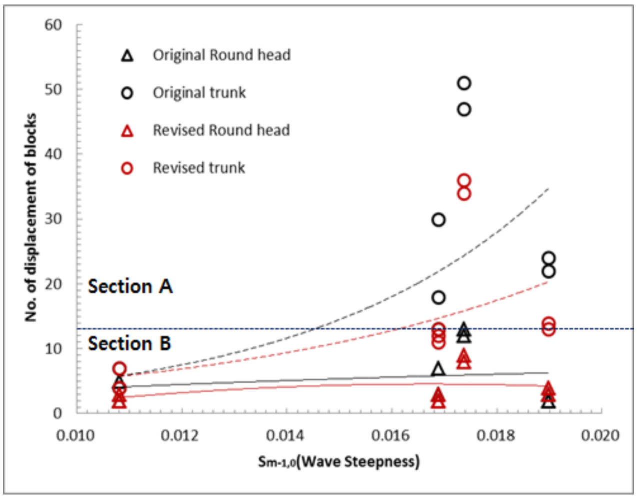


Fig 8: Relationship between Wave steepness and number of displacement blocks

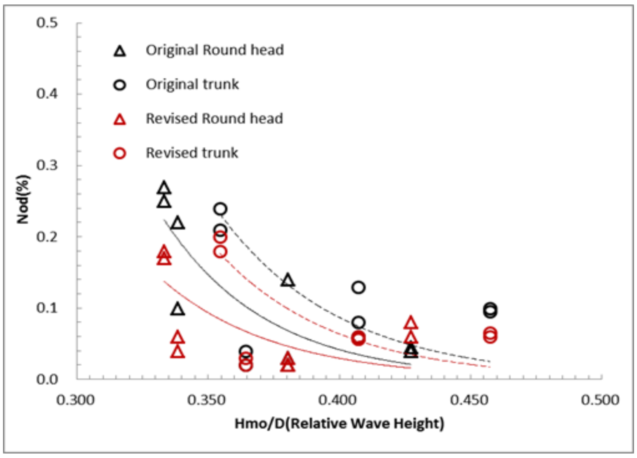


Fig 9: Nod values about extension of crown wall (Round head & trunk part)

The test showed that the movement of blocks decreased remarkably when the penetrating flow was controlled by the installation of cap concrete on the top; on the other hand, it was confirmed that the increase of the overtopping rate due to higher water-level and higher wave height in the same circumstances of a cross section accelerated the displacement of the armor unit and at the same time was found to be the main reason for the worsened stability of the rubble mound breakwater.

3.2. Analysis of Structure Stability & Overtopping Phenomenon

As mentioned in the previous chapter, EurOtop2007 was employed as the standard of the overtopping rate and, among various cross-section plans, $A_C \geq R_C = 0$ was chosen for this test. With the application of EurOtop2007 suggesting the kinds of rear-side facilities and the allowable overtopping conditions to their degrees of damage as seen in Table 2, the outcomes from the test were compared and analysed. An overtopping rate of 74.6 ℓ/s/m took place on the round head part under condition of a NE dir. wave with Wave 4 which is the design criterion, while there was a 65.9 ℓ /s/m rate under condition of SE wave. The overtopping by extended crown wall increased a little owing to the influence of reflected waves, while the analysis for each scenario revealed that the increase of the overtopping rate was no more than 5% in the conditions where the overflow was not regarded as part of the overtopping rate and no crown wall was mounted. This explains why the installation of the crown wall only had a direct effect on the stability of blocks but little influence on the decrease in the overtopping rate. It was found that there took place the maximum overtopping of 74.6 ℓ/s/m, 38.5 ℓ/s/m, 54.1 ℓ/s/m and 0 ℓ/s/m at round head, major trunk part, minor trunk part and root corner part respectively in all test plans, which means that those rates don't exceed the allowable overtopping rate. However, in Wave 5 with the tide level increase of 1.25 meters, it was 1.5 ~ 2.0 times more than the maximum allowable overtopping rate of 100 ℓ/s/m, which phenomenon took place mostly at the round head ~ major trunk part whose H_{m0}/D is relatively low. There was a sharp change to the overtopping rate due to the tide level rise but it was determined that the occurrence probability of the tide level set up for Wave 5 was relatively low, and the outcome was not reflected in the design and construction. The above-mentioned overtopping rates can be considered to be an average overtopping rate of dimensionless value, leading to the following formula.

$$q / (g H_{m0}^3)^{0.5} \qquad (2)$$

Here, q is the mean overtopping discharge per meter structure width (m^3/sec/m), g is acceleration due to gravity (m/s^2), H_{m0} is the estimate of significant wave height from spectral analysis (m) In figure 10 is shown the relative freeboard (R_C/H_{m0}) of the dimensionless value. In Figure 10, the conditions of overtopping occurrence and exposure of the core part are distinguished - the analysis revealed the section A with the condition of $R_C/H_{m0} < 0.2$ was unstable due to the exposure of the core part. These outcomes are based on changes to cross sections on the round head and major trunk part and overtopping rate by scenario. Next, the correlation between overtopping by the changes in parameters employed for this test and the dimensionless value was investigated to conduct an analysis on the relationship between stability and overtopping occurrence by tide level and waves acting on a breakwater. Figures 11 and 12 show the comparative results of the relative overtopping rate (Q) and relative freeboard (R) in the conditions of the cross section from round head to bend part, incident wave direction and wave. Those relative overtopping rates (Q) and relative freeboard (R) can be described as in the following formula.

$$Q = \frac{q}{\sqrt{g \cdot H_{mo}^3}} \times \sqrt{\frac{S_{m-1,0}}{tan\alpha}} \times \frac{1}{\gamma_b}. \qquad (3)$$

Here, $q/\sqrt{g \cdot H_{m0}^3}$ is dimensionless wave overtopping [-], $S_{m-1,0}$ is wave steepness (H_{m0}/L_0), tanα is slope steepness (1/n), $\gamma_b$is influence factor for a berm [-].

$$R = \frac{R_c}{H_{mo}} \times \sqrt{\frac{S_{m-1,0}}{tan\alpha}} \times \frac{1}{\gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v} \qquad (4)$$

Here, R_C/H_{m0} is relative freeboard, $\gamma_b$is influence factor for roughness elements on a slope [-], $\gamma_\beta$is influence factor for oblique wave attack [-], γ_v is correlation factor for a vertical wall on the slope [-]. The analysis found that Sections A and B could be distinguished on the point of 6.0E-0.4 on the y-axis, when Section A is an area deemed "Serious" because of the core exposure caused by overtopping with the consideration of Table 2. There is local displacement of blocks at Section B, while there is no core exposure and also N_{od} (%) is the area within 5%. Accordingly, Section A may be thought of as the area with stability unsecured. The following Figure 13 shows the relation of the breaker parameter ($\xi_{m-1,0}$) to formula (2) of the dimensionless overtopping rate that has been converted to a natural log value, where $\xi_{m-1,0}$ is expressed in the following formula.

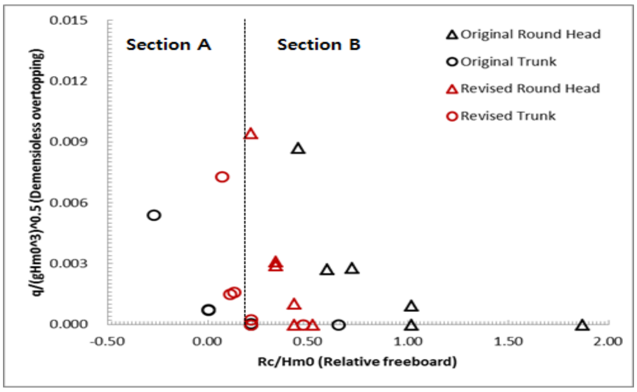


Fig 10: Relationship among wave overtopping, freeboard and wave height

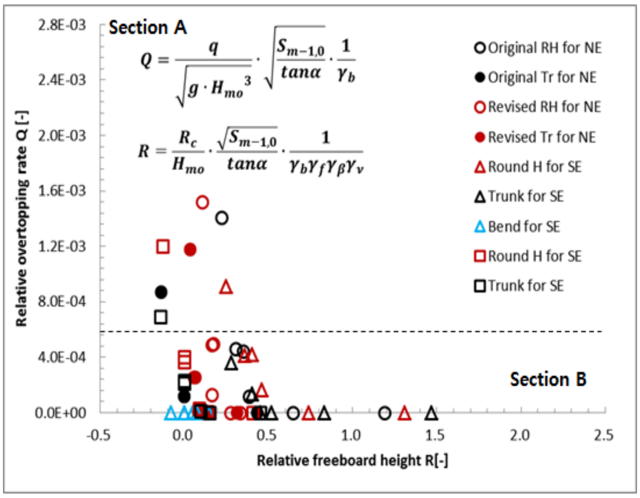


Fig 11: Analysis on Stability of breakwater depending on the Q & R

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$$\xi_{m-1,0} = \frac{tan\alpha}{\sqrt{S_{m-1,0}}} \tag{5}$$

The analysis enabled the division of Sections A and B on around 0.0008 on the y-axis, and Section A and B can be determined as an Unstable Area with core exposure and Stable Area with hardly ~ little N_{od} (%) parameter.

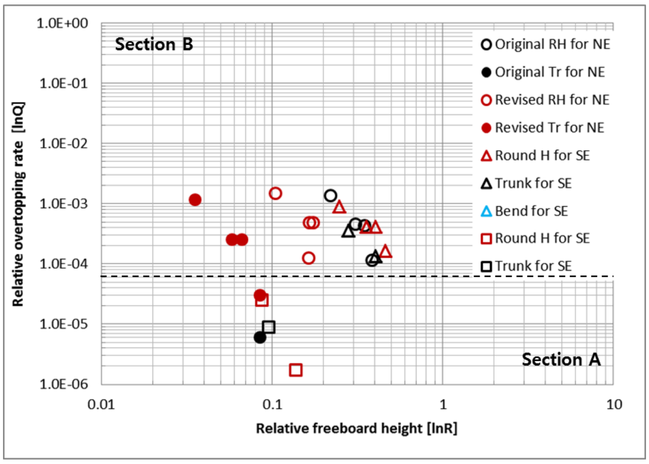

Fig 12: Analysis on Stability of breakwater depending on the Q & R

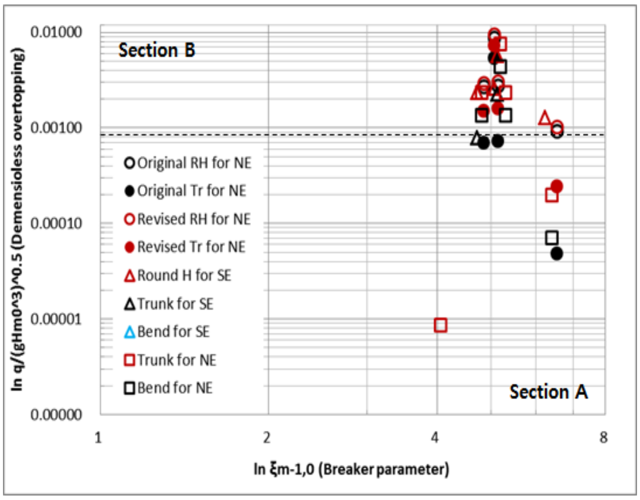

Fig 13: Analysis on Stability of breakwater depending on the wave overtopping discharge and breaker parameter

4. Conclusion

The construction of a breakwater was planned in order to protect marine loading and unloading facilities dealing with products from a petrochemical refinery plant and to secure the tranquility of the harbor to promote safe cargo working conditions on the sea. Accordingly, the appropriateness of the design and construction was reviewed by the application of a physical model test. The examination of structure stability, which was the most basic element of the test, was directly related to the determination of the design section. In particular, since the crest of all sections except for the head section was coated with only blocks, even low waves flowed to the rear of the breakwater and the waves of 10~100yrs. return period caused a strong flow from the pores of blocks mounted on crests to make the blocks become displaced, leading to the exposure of the core filter. In order to secure the stability by controlling the flow velocity at the crest, the crown wall on the trunk part was extended by 150 meters. With the application of an extended cross section, there was stability under conditions in Wave 4 (100yrs.) as a design criterion. However, the increase of the tide level (Wave 5) and wave (Wave 7) increased the overtopping rate, which caused an increase in the block displacement rate (N_{od}) and the exposure of the core part, leading to no stability being secured. Nevertheless, since any change in external forces more than the 100yrs. the return period is very low probabilistically, the test results derived from Wave 5 and 7 were not reflected in the design and construction. The factors most affecting the stability of breakwater revealed by the test were the increase in the tide level and wave, and as seen in Figures 7~13, a specific value was employed for distinguishing between the Stable Area (Section B) and Unstable Area (Section A). The comparative results from the dimensionless value (Figure 10~13) can be utilized as basic data for designing the cross-section condition applied in this test. Construction was carried out reflecting the results of the above study. The construction period was about three years, and various mounting methods of the concrete armor blocks were discussed to improve the stability. Finally, a marine structure as shown in Figure 14, was completed. Three years have passed since the construction, but it has been confirmed that there has been no damage in the stability of the breakwater and unloading work has proceeded smoothly. As sea levels continue to rise, rising water-levels and increasing waves and return periods will have a direct effect on the stability of structures and the increase in the overtopping rate, and thus it is imperative to devise and implement appropriate countermeasures against rising water-levels.

Fig 14: Analysis on Stability of breakwater depending on the wave overtopping discharge and breaker parameter

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