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VLFS 안전성 확보를 위한 방파제 설계 연구

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Design of Breakwater for the Safety of VLFS

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요 약

초대형부유식 해상구조물의 안전성 확보를 위한 방파제 설계를 수행하여 그 효용성을 검증하고 관련 구조물의 설계에 대한 지침을 제공하였다. 초대형부유식 구조물의 설치 위치에 따라 파랑 하중을 계산하였고, 이 하중에 대한 최적의 직립식 방파제 단면을 통용되고 있는 Goda 식에 의하여 scantling 하였다. 케이슨의 안전성 검증을 위하여 유한요소해석을 수행하였고, 최종적으로 VLFS의 안전성 확보를 위한 하나의 방파제 설계도를 제시하였다.

Keywords:
초대형부유식 해상구조물, 직립식 방파제, 유한요소해석, 방파제 안전성 요소

1. Characteristics of Breakwater Structures

The breakwater is defined as a structure protecting a shore area, harbor, anchorage, or basin from waves. Breakwaters can be used to protect exposed VLFS, otherwise lacking natural protection from adverse affects of mostly waves and currents. Breakwaters for the protection of VLFS can be classified as offshore one that is designed primarily to provide protection from wave action to the structure.[1,2]

In the design of VLFS, the main purpose of using offshore breakwaters is to dissipate or reduce the amount of wave energy reaching to the objects. The effectiveness of a selected breakwaters layout may be checked and confirmed by a physical model test.

1.1. Characteristics of slopping or rubble-mound breakwater

The most typical sloping-type breakwater is one with randomly placed stones. The basic concept of this type breakwater lies in the securing of the structural stability and reduction of wave transmission. This type of the breakwater is simple in construction procedure and has relatively high permeability. However, this is considered to be very conventional.

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1.2. Characteristics of vertical type breakwaters

The original concept of the vertical breakwater was to reflect waves. The basic type of vertical wall breakwater has no rubble-mound foundation. On the other hand, some types are composed of vertical walls and rubble-mound foundations. There are low-mound and high-mound composite breakwaters and the latter has a mound that is higher than the low water level. The low-mound breakwater does not cause wave breaking on the mound, whereas the high-mound breakwater does. Moreover, the high-mound composite-type breakwaters can be unstable due to wave generated impulsive pressure and scouring caused by the breaking waves. For this reason, composite-type breakwaters with a low mound are more practical.

Concrete blocks can be placed in front of the breakwater to reduce both the wave reflection and the breaking-wave force on the vertical wall. This type of construction is termed a composite breakwater covered with wave-energy-dissipating concrete blocks, or the horizontally composite breakwater. The concrete blocks of the rubble-mound breakwater act as the armor for the rubble foundation, whereas the concrete blocks of the horizontally composite breakwater function to reduce the wave force and size of the reflected waves. In general, the horizontally composite breakwaters are considered to be the improved version of the vertical ones.

The rubble-mound foundation of composite breakwaters is vital for stability of the upright section. It must safely carry the vertical parts weight, be stable against wave forces acting against both the upright sections and the foundation itself and must be protected against scouring by currents and wave action.

2. Design Procedure of the Breakwater

2.1. Generals

Design of the optimum wave control structure should be carried out based on the several factors; environmental loads, water depth, utilization plan of the sea area behind the wave control structure. For the VLFS for airport and harbor, fixed type of breakwater as a wave control structure is thought to be more preferable since the motion of the VLFS is extremely important and the site is near to the shore, i.e., the water depth is relatively small, as discussed before. Followings are the design procedures of the vertical breakwater that is supposed to be used for the protection of the VLFS airport in mind.[3,5,8]

Vertical breakwaters reflect the incident waves without dissipating much wave energy. Generally vertical breakwaters are usually built in water of sufficient depth where wave breaking is not a problem. There are so many vertical breakwaters in Japan because of the threat of the breaking waves.[10] Thus for the design of the breakwater, it is recommendable to investigate the breakwaters built in Japan before. Among them, the tsunami breakwater at Ofunato port in Japan was built in 35 m water depth. And the breakwater that is supposed to be built in the deepest water depth is the one in Kamaishi Port and the construction site has the maximum water depth of 63 m.[6,11]

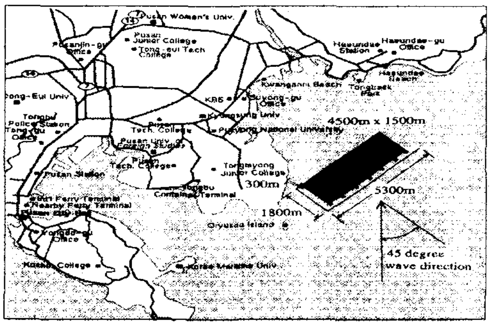

Fig.1 Proposed Mega Float for airport and harbor at Haeundai front sea

For the wave control purpose in Pusan area where the VLFS airport is going to be built and installed, the water depth is about 35 m maximum. Hence the inherent difficulties arising due to the water depth limitation of the vertical breakwater construction seem to be negligible.

Fig.2 shows the idealized typical section of a vertical breakwater

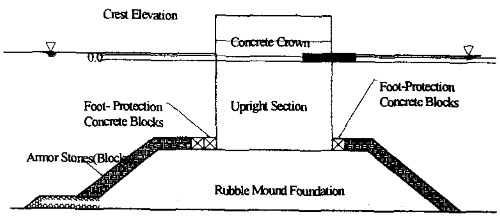

Fig.2 Typical section of vertical breakwater

2.2. Wave Pressure Formulas under Wave Crests

The wave pressure formulas proposed by Japanese designer Goda, for the design of vertical breakwaters is used widely and accepted to be the most reliable one these days.[6,9] This formula assumes the existence of a trapezoidal pressure distribution along a vertical wall as shown in Fig.3. This formula is applicable for the breaking waves as well as non-breaking waves. In Fig.3, where h denotes the water depth in front of the breakwater, d denotes the depth above the armor layer of the rubble foundation, h' the distance from the design water level to the bottom of the upright section, hc the crest elevation of the breakwater above the design water level. For the design of breakwater, the formula by Goda is employed.

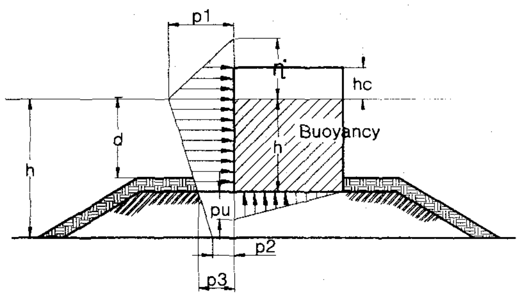

Fig.3 Pressure distributions on the upright section of breakwater

2.2.1. Design wave

The highest wave in the design sea state is used and its value is taken as

H_{max} = 1.8 $H_{1/3}$ seaward of surf zone, whereas within the surf zone the height is taken as the height of random breaking waves H_{max} at the location of a distance $5H_{1/3}$ seaward of the breakwater. $H_{1/3}$ is to be estimated with the random wave-breaking model at the depth of the location of the breakwater. The period of the highest wave is taken as that of significant wave. $T_{max}=T_{1/3}$

2.2.2. Elevation to which the wave pressure is exerted

$$\eta^* = 0.75(1+\cos\beta) H_{max}$$

in which denotes the angle between the direction of wave approach and a line normal to the breakwater.

2.2.3. Wave pressure on the front of a vertical wall

$$p_1 = \frac{1}{2}(1+\cos\beta)(\alpha_1 + \alpha_2 \cos^2\beta)\, w_0 H_{max}$$

$$p_2 = p_1 / \cosh(2\pi h/L)$$

$$p_3 = \alpha_3\, p_1$$

where

$$\alpha_1 = 0.6 + \frac{1}{2}[4\pi h/L/\sinh(4\pi h/L)]^2$$

$\alpha_2 = \min\{h_b - d/3h_b(H_{max}/d)^2,\ 2d/H_{max}\}$

$\alpha_3 = 1 - h'/h[1 - 1/\cosh(2\pi h/L)]$

In which

min {a, b}: smaller of a and b,

h_b: water depth at the location at a distance $5H_{1/3}$ seaward of the breakwater

The above pressure intensities are assumed not to change even if wave overtopping takes place.

2.2.4. Buoyancy and uplift pressure

The buoyancy is to be calculated for the displacement volume of the upright section in still water below the design water level, and the uplift pressure acting on the bottom of the upright section is assumed to have a triangular distribution with the toe pressure p_u with a heel pressure of zero.

$$p_u = \frac{1}{2}(1 + \cos\beta)\alpha_1\alpha_2\alpha_3 w_0 H_{max}$$

2.2.5. Total wave pressure and its moment around the bottom

Total wave pressure and its moment around the bottom of the upright section can be calculated with the above formulas as follows.

$$P = \frac{1}{2}(p_1 + p_3)h' + \frac{1}{2}(p_1 + p_4)h_c^*$$

$$P = \frac{1}{6}(2p_1 + p_3)h'^2 + \frac{1}{2}(p_1 + p_4)h' h_c^* + \frac{1}{6}(p_1 + 2p_4)h_c^{*2}$$

$$P_4 = \{p_1(1 - h_c/\eta^*) : \eta^* > h_c\ ,\ 0:\ \eta^* < h_c\}$$

$$h_c^* = \min\{\eta^*\ ,\ h_c\}$$

The total uplift pressure and its moment around the heel of the upright section are calculated with the equations below.

$$U = \frac{1}{2}p_u B \qquad M_u = \frac{2}{3}U B$$

Where B denotes the width of the bottom of the upright section.

2.3. Design of Upright Section

2.3.1. Stability condition for an upright section

The upright section of a vertical breakwater must be designed to be safe against sliding and overturning. The bearing capacity of the rubble mound foundation and the seabed should be examined to ascertain that they remain below the allowable limit. The safety factors against sliding and overturning of an upright section under wave action are defined by the following equations.

$$\textit{Against sliding}:\quad S.F = \mu(M_g - U)/p$$

$$\textit{Against overturning}:\ S.F = \mu(M_{gt} - M_U)/M_p$$

Where M denotes the mass of the upright section per unit extension in still water, μ the coefficient of friction between the upright section and the rubble mound, and t the horizontal distance between the center of gravity and the heel of the upright section. Generally the safety factors against sliding and overturning are taken as 1.2, and the coefficient of friction between concrete and rubble mound stones is usually taken as 0.6. At sites where the seabed consists of a dense sand layer or soil of good bearing capacity, a simplified method is used. In this method it is assumed that a trapezoidal or triangular distribution of bearing pressure exists beneath the bottom of the upright section, and the largest bearing pressure at the heel is calculated as below.

$$p_e = \{2W_e/3t_e:\ t_e < 1/3B,\quad 2W_e/B(2 - 3t_e/B):\ t_e > 1/3B\}$$

In which

$t_e = M_e/W_e,\ M_e = M_{gt} - M_U - M_p,$

$W_e = M_g - U$

The bearing pressure at the heel is to be kept below the value of 400 to 500 *kPa*, but recent breakwater designs are gradually increasing this limit to 600 *kPa* with advancement of breakwater construction sites into deeper water and with increases in the weight of upright sections.

2.3.2. Design of width of the upright section
2.3.3. Impulsive breaking wave pressure
2.3.4. Design of concrete caisson

2.4. Design of rubble Mound Foundation
2.4.1. Dimensions of rubble mound
2.4.2. Foot protection blocks and armor units
2.4.3. Protection against scouring of the seabed in front of a breakwater

are proceeded as recommended by general breakwater design criteria.

3. Configuration of the Designed Breakwater

For the protection of the Mega Float at Haeundae front sea, a breakwater is designed through the steps of sections2 and its configuration is illustrated in Auto Cad drawings, Fig.4,5. The basic dimensions are in international unit, such as kg, sec., m. This breakwater is a modern large rubble-mounded vertical breakwater designed for 35 m water depth installation. Fig.4 illustrates the cross section of the designed breakwater and Fig.5 shows the upright section of the concrete caisson.

The upright sectional dimensions are used basically for the additional structural analysis of the breakwater in the following section. Larger picture of the breakwater is also provided for better view.

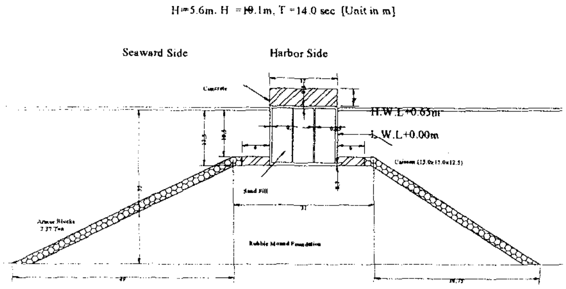

Fig.4 Configuration of the designed breakwater

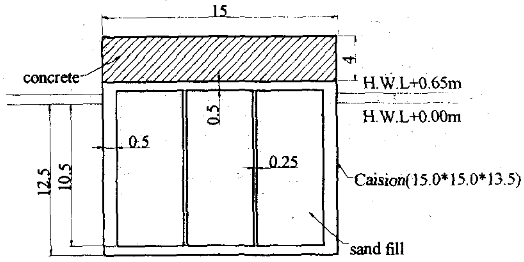

Fig.5 Detail upright section of concrete caisson of breakwater

4. Structural Analysis of the Breakwater

4.1 Finite element analysis of the breakwater

4.1.1. Dimensions of the object structure

The upright section of the breakwater is investigated for the structural safety against the wave loading. The section was designed based on the fundamental failure criteria of sliding and overturning of the upright section and the formula by Goda. The structural analysis is for the confirmation of the designed breakwater structural integrity.

Principal dimensions of the upright section of the breakwater are given in Table 1, and the profile of the section with wave loads is shown in Fig6.

4.1.2. Wave pressure and structural modeling

External loads to the structure are given as wave pressure forms calculate and the maximum wave pressure is 7.787 t/m^2 at the mean water level, and 4.781 t/m^2 at the top of the structure and 6.883 t/m^2 at the bottom of the structure. The loads are distributed through the front wall of the structure as shown in Fig.6

Items of structure	Dimension
Total height	17.5m
Breadth	15.0m
Water depth	12.5m
Modulus of elasticity of steel	0.204×10^8 t/m^2
Modulus of elasticity of concrete	0.232×10^7 t/m^2

Table 1 Principal dimensions of the upright section of the breakwater

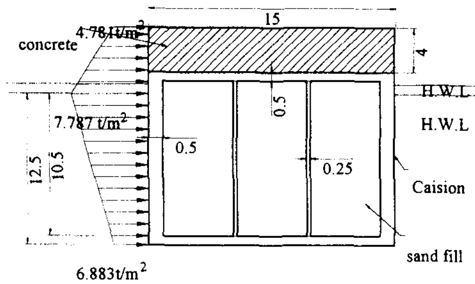

Fig.6 Profile of the upright section with wave loads

For the structural analysis, FEMAP with mTAP*STRESS finite element program are used.[4,7] For this kind of continuous breakwater type structural analysis 2-d plane strain element of unit thickness is employed. To simulate the reinforced steel in the concrete caisson, truss element is used and the initial steel quantity is taken as 5 % of the concrete sectional area. Modules of elasticity of steel is taken as 0.204 x 10^8 t/m^2 and that of concrete is taken as 0.232 x 10^7 t/m^2.

Boundary condition of the model is set such that the vertical movements of the structure bottom are fixed and horizontal movement at the right corner of bottom is restrained.

4.2. FE analysis results and discussions

Structural analysis results of the upright section are shown in Fig.7 through Fig.10. Global deformation trends are looked like same as the typical frame structures deformations as shown in Fig.7. The maximum horizontal displacement is to be 0.0297 m. For the detailed stress distributions of the hot spots, enlarged picture of stress distributions of the front wall is illustrated in Fig.8.

The safety of the concrete of the upright section, in viewpoint of compressive stress, is far enough, since the maximum compressive stress occurred is 52.834 t/m^2. The maximum allowable compressive stress of the concrete for ocean usage is 960.0 t/m^2.

However, for the shear stress case, it is not enough. Fig.9, Fig.10 show the maximum shear stress distributions of whole structure, front wall part, respectively, with reinforced steel usage of 20% area of the concrete section area. The maximum shear stress at the front wall hot spot shows the value of 231 t/m^2 and this value is a little bit above the allowable shear stress of 178 t/m^2. Maximum shear stress in the back wall hot spot is ranged from 174 t/m^2 to 189 t/m^2. This is marginal and in viewpoint of shear stress, back wall part of the structure seems to be fine. However, for the shear stress of the front wall case, it is not enough.[12]

The remedy for the maximum shear stress problem can be the reinforcement of the front wall by using the high strength concrete or by the thickening of the section width. Since the maximum shear stress value is not far above the allowable shear stress, this problem is not very significant. In the detail design stage, this is surely overcome and fine results can be accomplished.

As pointed out earlier the maximum horizontal displacement is 0.0297 m, and comparison with the similar structures displacement of 0.0334 m gives the sound prospective.

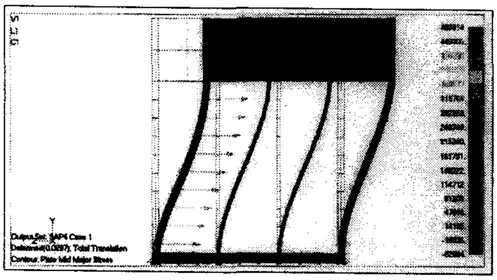

Fig.7 FE model of the upright section of the breakwater with loads

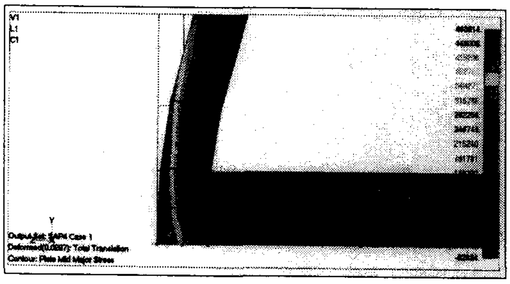

Fig.8 Hot spot stress distributions in the front wall foot

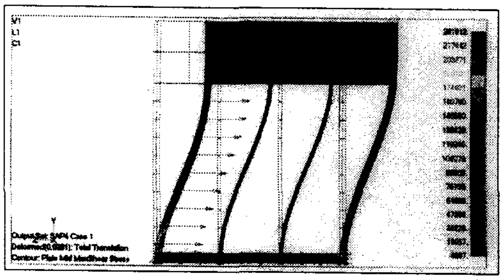

Fig.9 Shear stress distributions in the concrete caisson of breakwater

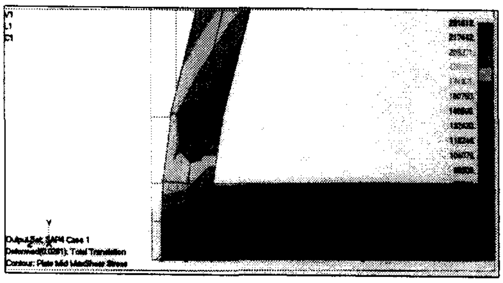

Fig.10 Maximum shear stress distribution in front wall of breakwater

In conclusion, structural analysis results of the upright section of the breakwater carried out here provide the confidence that the basic structural design of the breakwater system is reasonable.

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