Part 6 Sloping Breakwater (Semi Draft)

1. Technical Notes

Reference

1-1. Characteristics of Sloping Breakwater

Sloping breakwater is a structure constructed by piling up rubble and variously shaped concrete blocks. Its primary function is to cause waves to break on slopes, promoting the dispersion of energy and effectively reducing reflected waves. While its construction is simple and straightforward, it requires a significant amount of material as water depth increases, which may lead to economic disadvantages.

1-2. How to Determine a Dimension

(1) Crest Level (Top level of structure)

Sloping breakwaters are often used as primary breakwaters in Vietnam. The crest level of the breakwater is generally set at an appropriate height at least 0.6 times the significant wave height ($H_{1/3}$) above the mean monthly-highest water level in Japan. However, if the crest level is low, hydraulic model tests and other measures should be carried out to estimate damage to the armor stones inside the harbor due to overtopping waves. When the height is about 0.6 times the significant wave height ($H_{1/3}$), the blocks inside the harbor are the same size as or half the blocks outside the harbor in Japan.

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The EurOtop manual also recommends that for low-crested breakwaters, the crest and rear side will be protected similarly to the seaward side.

In addition, the selection of the crest level and the desired level of calmness within the breakwater should be based on the types of ships being accommodated and based on the local requirements.

EurOtop Manual 2nd Edition 3.3.3

Reference:

Examples of wave heights that can safely accommodate ships are presented in TCCS 02 (2017).

(2) Crest Width (Top width of structure)

When using wave-dissipating blocks as armor units, the crest width should be standardized as three or more blocks in a row. Similarly, when using armor stones, it is often the case to have three or more in a row. However, if cap concrete is provided, the crest width of the wave-dissipating blocks can be set to two or more blocks in a row.

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(3) Slope Angle

The slope angle depends on stability calculations, but when using stones as covering material, it is often approximately 1:2.0 on the seaward side and about 1:1.5 on the harbor side. When using wave-dissipating blocks, the slope is frequently set between 1:1.3 to 1:1.5.

(4) Scouring and Suction Measures

In areas at risk of scouring, it is advisable to install scour prevention materials such as gravel mats, asphalt mats, and others at the toe of the slope. Furthermore, in areas at risk of suction of the seabed, laying suction prevention materials like geotextiles on top of the seabed is recommended.

(5) Sedimentation Control

In areas affected by drifting sand, it is desirable to install sand traps inside the sloping breakwater to prevent siltation inside the harbor. Types of sand control measures include constructing walls inside the breakwater using sheet piles or blocks. In other cases, it is also effective to dump stones of various particle sizes in the core of the breakwater or on the slopes facing the harbor side.

(6) Stability at Construction Stage

Generally, sections using rubble stones and wave-dissipating blocks tend to be more economical than those composed entirely of wave-dissipating blocks. However, before the wave-dissipating blocks are installed, the rubble stone is unstable against wave action. Therefore, in areas with severe wave conditions, it is necessary to divide the construction into shorter sections and quickly cover the placed rubble stone with wave-dissipating blocks.

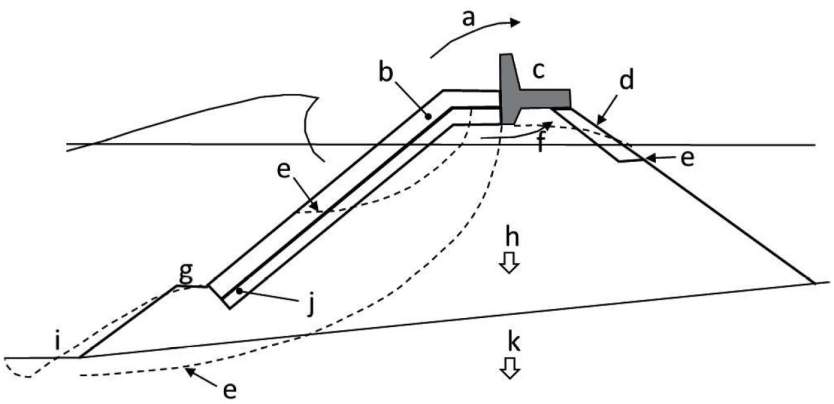
1-3. Performance Verification Items for Sloping Breakwater

Sloping breakwater have problems with overtopping and transmitted waves and are subjected to the following failure modes: scouring and breakages of armor unit; breakages, sliding, and overturning of superstructures; slip failures of front slopes; scouring of mounds below armor units; settlement of core materials; scouring of sandy ground at slope toes; washing out of fine particle components due to internal instability of filtering materials; and ground settlement. Therefore, the performance verification of sloping breakwaters shall be performed to prevent these failure modes.

The performance verification items for sloping breakwaters include the stability of superstructures; the stability of armor units (rubble stones, concrete blocks, and deformed concrete blocks) at sloped sections, the required mass of rubble stones and blocks below the armor units at sloped sections and their internal stability as filtering layers, and the bearing capacity of sloped sections and ground.

OCDI 2020, Part III Chapter 4 3.3.3

TCVN 11820 Part 6: 2023, Hinh 44



Source: TCVN 11820-6-2023, ISO 21650, OCDI 2020

Figure 1.1- Failure Modes of Sloping Breakwater

a. Overtopping
b. Scouring and breakages of armor units
c. Breakages, sliding, and overturning of superstructures
d. Scouring of armor units
e. Slip failures of front slopes
f. Transmitted waves
g. Scouring of mounds below armor units
h. Settlement of core materials
i. Scouring of sandy ground at a slope toe

j. Internal instability of filtering materials

k. Ground settlement

1-4. Performance Verification of Armor Units for Sloped Sections

One of the methods for covering sloped sections is to use rubble stones or deformed concrete blocks as armor units, and another method is to cover sloped surfaces with sand mastic.

The armor units for rubble sections shall have sufficient mass to ensure stability against waves and sufficient thickness to prevent infill from being washed out.

When calculating the required mass of armor units, refer to Chapter 1-5 Stability of Armor Stones and Blocks against Waves. The required mass of armor units shall be appropriately set when constructing armor layers not randomly but by orderly arranging armor units or by laying armor stones. The number of layers is generally set at two when constructing armor layers by randomly arranging armor units.

For the use of sand mastic to cover sloped surfaces, refer to past use cases and the research outcome.

1-5. Stability of Armor Stones and Blocks against Waves

(1) General

Armor units are used on structures like sloping breakwaters to shield the underlying rubble stones. These units must be sufficiently heavy to maintain stability and prevent dispersal. Typically, the necessary mass for these armor units can be determined through hydraulic model testing or by employing relevant appropriate equations.

(2) Basic Equation for Calculation of Required Mass

To determine the necessary mass of rubble stones and concrete blocks that cover the slope of a structure impacted by wave action, one might use Hudson's formula. This formula includes a stability number N_S and is expressed by the equation below.

$$M = \frac{\rho_r H^3}{N_s^3 (S_r - 1)^3} \qquad (1.1)$$

TCVN 11820 Part 2: 2025, Equation (234)

Where:

- M : required mass of rubble stones or concrete blocks (t)
- ρ_r : density of rubble stones or concrete blocks (t/m^3)
- H : wave height used in stability calculation (m)
- N_S : stability number determined primarily by the shape, slope, damage rate of the armor, etc. of the armor units
- S_r : specific gravity of rubble stones or concrete blocks relative to water (ρ_r / ρ_o)
- ρ_o : density of seawater 1.03 (t/m^3)

(3) Stability Number and Nominal Diameter

The stability number directly corresponds to the size (nominal diameter) of the armor required for a certain wave height H. Assume the nominal diameter $D_n = (M/\rho_r)^{1/3}$ and $\Delta = S_r - 1$ and substitute into Equation (1.2), a simple equation

$$H/(\Delta D_n) = N_s \qquad (1.2)$$

TCVN 11820 Part 2: 2025,

is obtained to show that the wave height and the nominal diameter are proportional with

ΔN_S as its proportional constant.

Equation (235)

(4) Wave Height Used for Performance Verification

TCVN 11820 Part 2: 2025, 6.4.6

Hudson's formula, originally derived from experiments with regular waves, encounters challenges when applied to the random nature of actual wave actions. A key issue is determining the appropriate wave height definition to use. For structures composed of rubble stones or concrete blocks, damage typically does not occur from a single maximum-height wave in a random wave train. Instead, damage tends to develop gradually over time as various wave heights continuously impact the armor units. Given this observation and based on historical data, it has become standard practice to use the significant wave height of progressive waves at the location of the slope for the wave height H in Equation (1.1). This choice is made because the significant wave height effectively represents the general scale of a random wave train. It is important to note, however, that if the water depth is less than half the equivalent deepwater wave height, the significant wave height used should be that at a water depth equal to half the equivalent deepwater wave height.

Reference:

TCVN 11820 Part 6: 2023, A.4.2

The design wave height H_D was based on model testing using regular waves. There is no simple method of comparing the results of laboratory tests carried out using regular and random waves. Laboratory studies have shown that the equivalent regular wave height can range between the significant wave height H_S of a random wave train and higher values such as $H_{1/10}$, the mean of the highest one-tenth of wave heights.

Current opinion is that, for non-breaking conditions, $H_{1/10}$ at the site of the structure should be used in Equation (1.1). For conditions where the $H_{1/10}$ waves would break before reaching the breakwater, the wave height used for preliminary design should be H_b (the breaking wave height) or H_S, whichever has the more severe effect.

(5) Parameters Affecting the Stability Number N_S

According to Equation (1.2), the necessary mass of armor stones or concrete blocks depends on several factors including the wave height, the density of the armor units, and the stability number N_S. The N_S value acts as a coefficient that captures the impact of various elements such as the structural characteristics, the properties of the armor units, wave dynamics, and other relevant factors on stability. The primary coefficients that affect the N_S value include:

1) **Characteristics of the structure**
   a) Type of structure; sloping breakwater, breakwater covered with wave-dissipating concrete blocks, and composite breakwater, etc.
   b) Gradient of the armored slope
   c) Position of armor units; breakwater head, breakwater trunk, position relative to still water level, front face and top of slope, back face, and berm, etc.
   d) Crown height and width, and shape of superstructure
   e) Inner layer; permeability coefficient, thickness, and degree of surface roughness
2) **Characteristics of the armor units**
   a) Shape of armor units (shape of armor stones or concrete blocks; for armor stones, their diameter distribution)
   b) Placement of armor units; number of layers, and regular laying or random placement, etc.

c) Strength of armor material

3) Wave characteristics

a) Number of waves acting on armor layers

b) Wave steepness

c) Form of seabed (seabed slope, where about of reef, etc.)

d) Ratio of wave height to water depth as indices of non-breaking or breaking wave condition, breaker type, etc.

e) Wave direction, wave spectrum, and wave group characteristics

4) Extent of damage (damage ratio, deformation level, relative damage level)

Therefore, it's crucial to determine the N_S value accurately for performance validation, based on hydraulic model experiments that reflect specific design conditions. Comparative analysis between experiments using regular waves and random waves revealed that the ratio of regular wave height to significant wave height of random waves, which resulted in a similar damage ratio within a 10% margin of error, ranged from 1.0 to 2.0, depending on the conditions. This indicates that random wave actions tend to cause more damage than regular waves. Consequently, incorporating random waves in experimental setups is recommended for more realistic assessments.

(6) Stability Number N_S and K_D value

In 1959, Hudson introduced what is now known as Hudson's formula, which superseded the earlier Iribarren-Hudson formula. Hudson formulated Equation (1.3) independently, utilizing K_D cotα instead of the stability number N_S.

$$N_s^3 = K_D \cot\alpha \qquad (1.3)$$

TCVN 11820 Part 2: 2025, Equation (237)

Where:

α : angle of the slope from the horizontal line (deg)

K_D : constant determined primarily by the shape of the armor units and the damage ratio

Hudson's formula, established based on extensive model experiments, has been effectively used in practical applications for calculating the mass of armor units on slopes, utilizing the K_D value. Despite its utility, the version of Hudson's formula that employs the stability number N_S from Equation (1.1) has become more prevalent. It is frequently applied not only to the foundation mounds of composite breakwaters but also to armor units in other structures like submerged breakwaters, thus overshadowing the older K_D-based formula. The stability number N_S can be calculated from the K_D value and the slope angle α using Equation (1.3). This calculation process is reliable if the K_D value is well-established, and the slope angle falls within typical design ranges. However, many K_D values used to date have not adequately considered various critical factors, such as the characteristics of the structure and wave dynamics. Consequently, relying solely on this method to determine N_S may not always lead to the most cost-effective designs. For more accurate estimations of the required mass, it is advisable to utilize experimental results that align with specific conditions, or to employ computational formulas and diagrams that incorporate these diverse and relevant coefficients.

(7) Van der Meer's Formula for Armor

In 1987, Van der Meer conducted detailed experiments on the armor stones used in the slopes of high-crown sloping breakwaters. He introduced a new calculation formula for the stability number that takes into account factors such as the slope gradient, wave steepness, wave count, and damage levels. It should be noted that the equations presented here have been modified from Van der Meer's original formulation to simplify the calculations. For instance, the wave height $H_{2\%}$, which represents a 2% exceedance probability, has been substituted with $H_{1/20}$.

TCVN 11820 Part 2: 2025, Equation (238) (239) (240)

$$N_s = \max(N_{spl}, N_{ssr}) \tag{1.4}$$

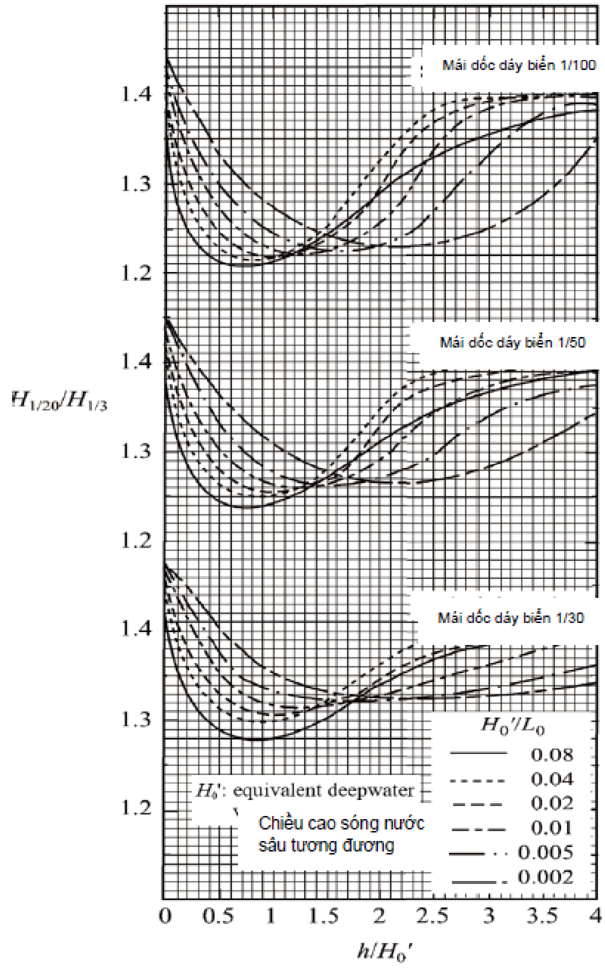
$$N_{spl} = 6.2\, C_H P^{0.18} (S^{0.2}/N^{0.1}) I_r^{-0.5} \tag{1.5}$$

$$N_{ssr} = C_H P^{-0.13} (S^{0.2}/N^{0.1}) (\cot\alpha)^{0.5} I_r^{P} \tag{1.6}$$

Where:

- N_{spl} : stability number for plunging breakers
- N_{ssr} : stability number for surging breaker
- I_r : iribarren number ($\tan \alpha/S_{om}^{0.5}$), also called the surf similarity parameter
- S_{om} : wave steepness ($H_{1/3}/L_0$)
- L_0 : deepwater wavelength ($L_0 = gT_{1/3}^2/2\pi$, g=9.81 m/s^2)
- $T_{1/3}$: significant wave period
- C_H : breaking effect coefficient {$=1.4/(H_{1/20}/H_{1/3})$}, (=1.0 in non-breaking zone)
- $H_{1/3}$: significant wave height
- $H_{1/20}$: highest one-twentieth wave height, see Figure 1.2
- α : angle of slope from the horizontal surface (°)
- D_{n50} : nominal diameter of armor stone ($=(M_{50}/\rho_r)^{1/3}$)
- M_{50} : 50% value of the mass distribution curve of an armor stone namely required mass of an armor stone
- P : permeability index of the inner layer, see Figure 1.3
- S : deformation level ($S = A/D_{n50}^2$), see Table 1.1
- A : erosion area of cross section, see Figure 1.4
- N : number of acting waves

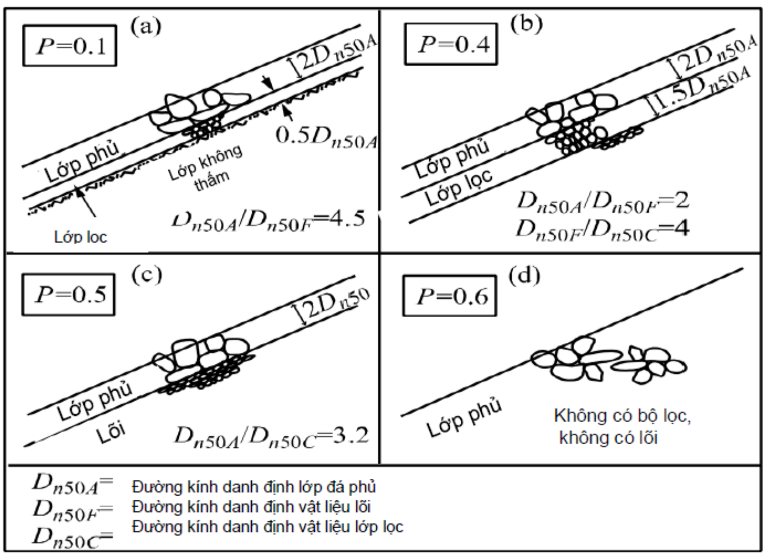
In Figure 1.2, the wave height $H_{1/20}$ is measured at a point located $5H_{1/3}$ away from the breakwater, and H_0' represents the equivalent deepwater wave height. The deformation level S serves as an indicator of the extent of armor stone deformation, essentially a damage ratio. This is calculated by dividing the eroded area A, depicted in Figure 1.4, by the square of the armor stones' nominal diameter D_{n50}. As detailed in Table 1.1, the deformation levels of armor stones are categorized into three stages: initial damage, intermediate damage, and failure. Typically, the deformation level for initial damage is used for N=1,000 waves during standard performance verifications. However, in scenarios where some deformation is acceptable, the deformation level corresponding to intermediate damage might also be considered.



TCVN 11820 Part 2: 2025, Hinh 96

Source: TCVN 11820-2-2025

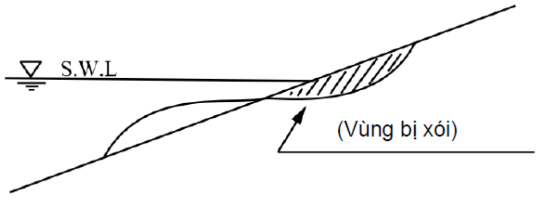
Figure 1.2- Ratio of $H_{1/20}$ to $H_{1/3}$ ($H_{1/20}$ Values are at a Distance $5H_{1/3}$ from the Breakwater)



TCVN 11820 Part 2: 2025, Hinh 97

Source: TCVN 11820-2-2025

Figure 1.3- Permeability Index *P*



Source: TCVN 11820-2-2025

Figure 1.4- Erosion Area *A*

TCVN 11820 Part 2: 2025, Hinh 98

Table 1.1- Deformation Level *S* for Each Failure Stage for a Two-layered Armor

TCVN 11820 Part 2: 2025, Bang 44

Mái dốc	Hư hỏng ban đầu	Hư hỏng trung gian	Phá hoại
1 : 1.5	2	3–5	8
1 : 2	2	4–6	8
1 : 3	2	6–9	12
1 : 4	3	8–12	17
1 : 6	3	8–12	17

Source: TCVN 11820-2-2025

(8) Formulation for Calculating Stability Number for Armor Blocks including Wave Characteristics

Van der Meer has carried out hydraulic model experiments on several kinds of precast concrete blocks and proposed the formulas for calculating the stability number N_S. In addition, other people have also conducted research into establishing calculation formulas for precast concrete blocks. For example, Burcharth and Liu have proposed a calculation formula. However, it should be noted that these are based on the results of experiments for a sloping breakwater with a high crown.

Takahashi et al. showed a performance verification method of the stability against wave action for armor stones of a sloping breakwater using Van der Meer's formula as the verification formula and proposed the performance matrix used for performance verification.

(9) Formulation for Calculating Stability Number for Armor Blocks including Wave Characteristics

Different cross-sectional designs can be seen in the wave-dissipating concrete block sections of breakwaters that are equipped with these blocks. Notably, when the majority of the front face of a vertical wall is adorned with wave-dissipating concrete blocks, the stability tends to be greater than that of traditional armor concrete blocks on a sloping breakwater, due to enhanced permeability. Extensive research has been conducted in Japan regarding the stability of breakwaters fitted with wave-dissipating concrete blocks. Researchers like Tanimoto and colleagues, Kajima and team, and Hanzawa and associates have systematically studied this stability aspect. Furthermore, Takahashi and colleagues have developed a specific equation for assessing the stability of wave-dissipating concrete blocks when they are arranged randomly across the entire front face of an upright wall.

$$N_S = C_H[a(N_0/N^{0.5})^{0.2} + b] \qquad (1.7)$$

TCVN 11820

Where:

N_0	:	degree of damage, a kind of damage rate that represents the extent of damage: it is defined as the number of concrete blocks that have moved within D_n in the direction of breakwater alignment	Part 2: 2025, Equation (241)
D_n	:	nominal diameter of the concrete blocks: $D_n=(M/\rho_r)^{1/3}$, where M is the mass of a concrete block	
N	:	wave number	
C_H	:	breaking effect coefficient; $C_H=1.4/(H_{1/20}/H_{1/3})$, in non-breaking zone $C_H=1.0$	
$a,\ b$	:	coefficients that depend on the shape of the concrete blocks and the slope angle. With deformed shape blocks having a K_D value of 8.3, it may be assumed that a=2.32 and b=1.33, if cotα=4/3, and a=2.32 and b=1.42, if cotα=1.5	

Takahashi et al. have also introduced a technique for estimating the cumulative damage expected over the lifetime of a structure. Moving forward, incorporating such expected damage levels into reliability-based design methods will represent a significant advancement in engineering practices. In areas where wave breaking is not a concern, if the wave count N is 1,000 and the damage degree N_0 is 0.3, the design mass calculated using Takahashi et al.'s method aligns closely with those derived from traditional K_D values. The N_0 value of 0.3 is equivalent to the commonly used damage rate of 1%.

(10) Guidelines for Applying K_D Values

It is recommended that in the design of concrete armoring Hudson's formula should be regarded as no more than a device for comparing the stability of different types of units, and K_D values published from previous hydraulic model testing should be used only as guidance for preliminary selection of armor sizes for full hydraulic model testing. It should be noted that Hudson's formula is not applicable to regular pattern placed armor units.

Values of K_D suggested for preliminary design of the structure trunk are given in Table 1.2 in TCVN 11820 Part 6.

Table 1.2- Proposed K_D Values

TCVN 11820 Part 6: 2023, Bang A.4

Đơn vị lớp phủ	K_D
Dolos	10 đến 12
Stabit	10 đến 12
Tetrapod	6 đến 8
Khối Antifer	6 đến 8
Accropode	10 đến 12
Rakuna IV	6 đến 12
Stoneblock	10 đến 15

Source: TCVN 11820-6-2023

(11) Mass Increase in Breakwater Head

Waves can impact the head of a breakwater from multiple directions, increasing the likelihood of armor units at the top of the slope being dislodged towards the rear rather than the front. As a result, the rubble stones or concrete blocks used at the breakwater head should be heavier than the values suggested by Equation (1.1). Hudson recommended increasing the mass by approximately 10% for rubble stones and 30% for concrete blocks. Nevertheless, given the potential for underestimation, it is advised to use rubble stones or concrete blocks that are at least 1.5 times heavier than the mass specified by Equation (1.1).

Kimura et al. have demonstrated that for breakwaters facing directly perpendicular wave impacts, the stable mass can be achieved by increasing the required mass of the breakwater's main section by 1.5 times. For oblique waves striking at a 45° angle, the necessary mass at the upper side of the breakwater head (relative to the wave direction) remains the same as for a direct 0° impact. However, the lower side can maintain stability with the same mass used in the main body of the breakwater.

Due to the potential for impulsive breaking wave forces at the end of wave-absorbing works, rounding the breakwater head to curve inward towards the port is advisable. This modification is typically done to the extent of one caisson.

(12) K_D Value of Stone Material

Table 1.3 shows the K_D value of armor stones proposed by the Coastal Engineering Research Center, C.E.R.C., of the United States Army Corp of Engineers. This value is proposed for the breakwater trunk, parts other than the breakwater head, in the 1984 Edition of the C.E.R.C.'s Shore Protection Manual. In the table, the values not in parenthesis are based on experiment results by regular waves, and it is considered that those corresponds to 5% or less of the damage rate due to action of random waves. The values in parentheses are estimated values. For example, the value (1.2) for rounded rubble stones which are randomly placed in two-layer under the breaking wave conditions is estimated as the value which is half of 2.4, because the K_D value of two-layer angular rubble stones under the breaking waves condition is 1/2 of the value under the non-breaking wave conditions.

However, in cases where the wave height of regular waves corresponds to the significant wave height, the wave which is close to the maximum wave height of random waves acts continuously under the breaking wave condition in the regular wave experiments. Therefore, the regular wave experiment under the breaking wave condition falls into an extremely severe state in comparison with that under the non-breaking wave conditions. In random waves experiments, as described previously, it is considered that so long as the significant wave height is a standard, K_D has a tendency to increase, conversely, as the breaking wave conditions gets severe. Thus, at least it is not necessary to reduce the value of K_D under the breaking wave conditions.

Table 1.3- K_D Value of Rubble Stones proposed by C.E.R.C

TCVN 11820 Part 2: 2025, Bang 45

Type of armor	Number of layers	Placement method	K_D		$\cot\alpha$
			Breaking waves	Non-breaking waves	
Rubble stones (rounded)	2 3 or more	Random placement Ditto	(1.2) (1.6)	2.4 (3.2)	1.5-5.0 Ditto
Rubble stones (angular)	2 3 or more	Ditto Ditto	2.0 (2.2)	4.0 (4.5)	Ditto Ditto

( ) shows estimated values

Source: TCVN11820-2-2025, Shore Protection Manual, OCDI 2020

1-6. Performance Verification of Underlayer below Armor Unit

(1) Required Mass of Filter Layers below Armor Units

The required mass of filter layers (rubble stones and blocks) below armor units at sloping breakwaters are preferably approximately 1/10 to 1/15 or more of the mass of armor units.

The mass of the stones (core materials) below the filter layers are preferably approximately 1/20 or more of the mass of filter layers.

The verification of the stability of the mass of the stones (core materials) below the filter layers can be performed with reference to the following equation (ISO 21650):

TCVN 11820 Part 6: 2023, Equation (25)

$$\frac{d_{15,\text{filter}}}{d_{85,\text{core}}} < 4 \text{ to } 5 \qquad (1.8)$$

$$\frac{W_{50,\text{filter}}}{W_{50,\text{core}}} < 15 \text{ to } 20$$

Where:

d	:	particle diameter of stone
W	:	mass of a stone or a concrete block
$d_{15,\text{ filter}}$	:	sieve size for 15% passing by mass
$d_{85,\text{ core}}$	:	sieve size 85% passing by mass
$W_{50,\text{ filter}}$	:	mass of a filter material with a median diameter
$W_{50,\text{core}}$	:	mass of a core material having a median diameter

Furthermore, the verification of the internal stability of filter materials can be performed with reference to the following condition.

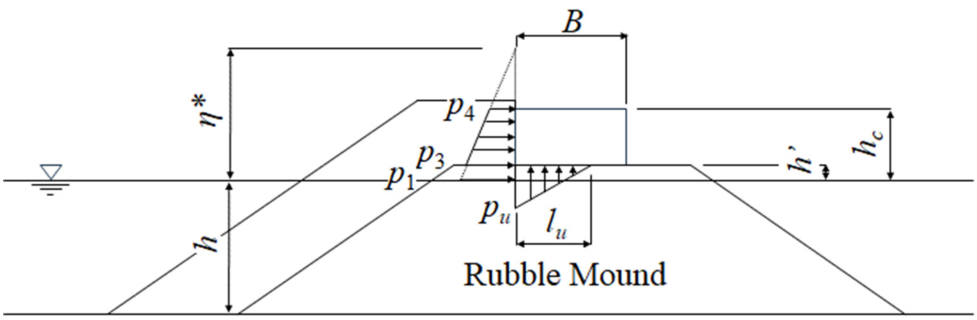
TCVN 11820 Part 6: 2023, Equation (26)

$$\frac{d_{60}}{d_{10}} < 10 \qquad (1.9)$$

1-7. Wave Force on Superstructure

The wave forces acting on a superstructure, which is covered with wave-dissipating blocks and has its bottom level above the design water level, should be calculated using Tanimoto's methods shown below. The uplift force on a superstructure, its bottom level is below the design water level, should be calculated using Goda formula.

Technical Note of PARI No.450: 1983 Fig. 35



Source: Technical Note of PARI

Figure 1.5- Wave Forces Acting on a Superstructure

Technical Note of PARI No.450: 1983 Equation

$$\eta^* = 0.75(1+\cos\beta)\lambda H_D$$

$$p_1 = \frac{1}{2}(1+\cos\beta)\lambda\alpha_1\rho_0 g H_D \qquad (1.10)$$

$$p_3 = p_u = \alpha_3 p_1$$

$$p_4 = \alpha_4 p_1$$

(1) to (6), (11), (12)

$$\lambda = \exp\left[-10\,(h/L)^{1.5}\,(1 - h'/h)^{5}\right]$$

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

$$\alpha_3 = 1 + h'/\eta^* \quad (h' \le 0)$$

$$\alpha_3 = 1 - \frac{h'}{h}\left(1 - \frac{1}{\cosh(2\pi h/L)}\right) \quad (h' > 0)$$

$$\alpha_4 = 1 - h_c^* / \eta^*$$

$$h_c^* = \min(\eta^*, h_c)$$

$$l_u = \min\left\{B,\ 0.2 \times \frac{(\eta^* + h')^2}{|h'|}\right\}$$

Where:

- B : width of superstructure (m)
- H_D : maximum wave height considering transformation due to the breaking of random waves (m)
- h : water depth where the breakwater installed (m)
- h' : water depth at the bottom of superstructure (m)
- h_c : height between the water level and the crown height of superstructure (m)
- l_u : width of uplift force (m)
- L : wavelength at water depth h
- p_u : intensity of uplift pressure acting on the bottom of superstructure (kN/m^2)
- p_1 : intensity of wave pressure at still water level (kN/m^2)
- p_3 : intensity of wave pressure at the bottom of superstructure (kN/m^2)
- p_4 : intensity of wave pressure at the top of superstructure (kN/m^2)
- $\alpha_1 \sim \alpha_4$: parameter
- η^* : height above still water level at which intensity of wave pressure is 0 (m)
- λ : wave pressure correction coefficient

Reference:

K. Tanimoto and R. Ojima: Experimental Study of Wave Forces on a Superstructure of Sloping Breakwaters and on Block Type Composite Breakwaters, Technical Note of PARI No.450, p.32, 1983

1-8. Performance Verification of the Stability of Superstructures

The verification of the stability of superstructures under the variable situation with respect to waves shall be performed for the sliding and overturning of superstructures.

The verification of the stability of superstructures under the variable situation with respect to waves shall be performed using Equations (1.11), (1.12) and (1.13). In these equations, the symbol is the partial factor for each subscript. Furthermore, subscripts *k* and *d* indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in Tables 1.4, 1.5 and 1.6 symbol in a column indicates that the value in parentheses in the column can be used for the performance verification of convenience.

(1) Verification of Sliding

$$m\cdot\frac{S_d}{R_d}\leq 1.0 \quad R_d=\gamma_R R_k \quad S_d=\gamma_S S_k$$

$$R_k=f_k(W_k\text{-}P_{Bk}\text{-}P_{Uk}) \qquad (1.11)$$

$$S_k=P_{Hk}$$

TCVN 11820 Part 6: 2023, Equation (22)

Where:

m : adjustment factor
S_d : value to be used for design of the load term (kN/m)
R_d : value to be used for design of the resistance term (kN/m)
S_k : characteristic value of the load term (kN/m)
R_k : characteristic value of the resistance term (kN/m)
γ_S : partial factor multiplied by load term
γ_R : partial factor multiplied by resistance term
f_k : characteristic value of friction coefficient between a superstructure and rubble stones
W_k : characteristic value of weight of a superstructure (kN/m)
P_{Bk} : characteristic value of buoyancy (kN/m)
P_{Uk} : characteristic value of uplift force (kN/m)
P_{Hk} : characteristic value of horizontal wave force (kN/m)

Table 1.4- Partial Factors Used for the Performance Verification of Sliding of Superstructures

TCVN 11820 Part 6: 2023, Bang 16

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of superstructure (Variable state of waves)	- (1.0)	- (1.0)	1.20

Source: TCVN 11820-6-2023

(2) Verification of Overturning

$$m\cdot\frac{S_d}{R_d}\leq 1.0 \quad R_d=\gamma_R R_k \quad S_d=\gamma_S S_k \qquad (1.12)$$

$$R_k=a_1W_k\text{-}\ a_2P_{Bk}\text{-}\ a_3P_{Uk}\ ,\ S_k=a_4P_{Hk}$$

TCVN 11820 Part 6: 2023, Equation (23)

Where:

m : adjustment factor
S_d : value to be used for design of the load term (kN·m/m)

R_d : value to be used for design of the resistance term (kN·m/m)
S_k : characteristic value of the load term (kN·m/m)
R_k : characteristic value of the resistance term (kN·m/m)
γ_S : partial factor multiplied by load term
γ_R : partial factor multiplied by resistance term
a_1 to a_4 : arm lengths of respective actions (m)
W_k : characteristic value of weight of a superstructure (kN/m)
P_{Bk} : characteristic value of buoyancy (kN/m)
P_{Uk} : characteristic value of uplift force (kN/m)
P_{Hk} : characteristic value of horizontal wave force (kN/m)

Table 1.5- Partial Factors Used for the Performance Verification of Overturning of Superstructures

TCVN 11820 Part 6: 2023, Bang17

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of superstructure (Variable state of waves)	- (1.0)	- (1.0)	1.20

Source: TCVN 11820-6-2023

(3) Verification of Bearing Capacity

Examination of the bearing capacity for eccentric and inclined actions acting on the foundation ground of gravity type structures can be performed by circular slip failure analysis with the simplified Bishop method using the following equation. Partial factors γ_S and γ_R and adjustment factor m shall be appropriate values corresponding to the characteristics of the facilities. It is necessary to appropriately set the strength constant of the ground and others, the forms of the actions, and other factors considering the structural characteristics of the facilities, etc. m is the parameter corresponding to the safety factor considering designing with the traditional safety factor method since γ_S and γ_R are usually set to 1.0, as described later.

TCVN 11820 Part 2: 2025 Equation (A.9a), (A.9b) (A.9c)

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$S_k = \Sigma\{(W_k + q_k)\sin\theta + aP_{Hk}/R\} \tag{1.13}$$

$$R_k = \Sigma \quad c_k s + W'_k + q_k \tan\varphi_k \quad \frac{\sec\theta}{1 + \tan\theta \cdot \tan\varphi_k / \; m/\gamma_R}$$

Where:

Modified from TCVN 11820 Part 4-1: 2020, Equation (B.5)

m : adjustment factor
S_d : value to be used for design of the action term (kN/m)
R_d : value to be used for design of the resistance term (kN/m)
S_k : characteristic value of the action term (kN/m)
R_k : characteristic value of the resistance term (kN/m)
γ_S : partial factor multiplied by action term
γ_R : partial factor multiplied by resistance term
W_k : characteristic value of total weight of a segment, total weight of soil and water (kN/m)
q_k : characteristic value of surcharge acting on a segment (kN/m)

Modified

θ	:	angle of bottom of divided segment to horizontal plane (°)	from TCVN 11820 Part 6: 2023, Equation (11)
a	:	arm length from the center of slip circle in circular slip failure at position of P_H action (m)	
P_{Hk}	:	characteristic value of horizontal action on lumps of soil in slip circle in circular slip failure (kN/m)	
R	:	radius of slip circle in circular slip failure (m)	
c_k	:	characteristic value of undrained shear strength in case of clayey ground, or characteristic value of apparent cohesion in drained condition in case of sandy ground (kN/m^2)	
s	:	width of divided segment (m)	
W'_k	:	characteristic value of effective weight of divided segment per unit length (kN/m) (weight of soil; effective weight in water if submerged)	
φ_k	:	0 in case of clayey ground, or characteristic value of angle of shear resistance in drained condition (°) in case of sandy ground	

Table 1.6- Partial Factors Used for the Performance Verification of Bearing Capacity of Superstructures

TCVN 11820 Part 6: 2023, Bang 9

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Bearing capacity of superstructure (Variable state of waves)	- (1.0)	- (1.0)	1.0

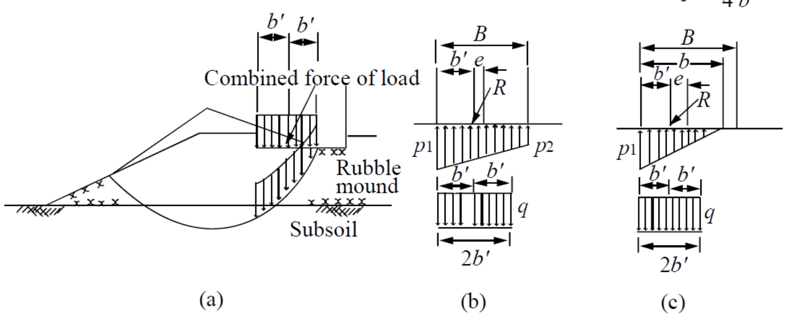
Source: TCVN 11820-6-2023, OCDI 2020

The circular slip failure analysis by the simplified Bishop method is applied under the condition that eccentric and inclined forces act. As shown in Figure. 1.6 (a), the start point of the sliding surface is set symmetrical to one of the foundation edges that is closer to the load acting point. In this case, the vertical action exerting on the bottom of the foundation is converted into uniformly distributed load acting on the width between fore toe of the bottom of the foundation and the start point of the sliding surface as indicated in Figures 1.6 (b) and (c). The horizontal force shall act at the bottom of the foundation.

TCVN 11820 Part 4-1: 2020, Hinh B.4

When subgrade reaction has a trapezoidal distribution; $q=\frac{(p_1+p_2)}{4b'}B$

When subgrade reaction has a triangular distribution; $q=\frac{p_1 b}{4b'}$



Source: TCVN 11820-4-1-2020, OCDI 2020

Figure 1.6- Analysis of Bearing Capacity for Eccentric and Inclined Actions

1-9. Performance Verification of the Circular Slip Failure

The modified Fellenius method assumes that the direction of the resultant force acting on vertical planes between slice segments is parallel to the base of the slice segments. This method is also referred to as the simplified method or Tschebotarioff method. When a circular arc and a slice segment are as shown in Figure 1.7, according to the modified Fellenius method is applicable.

The conventional design, using the safety factor method, is equivalent to the design where both *S* and *R* are 1.0: Factor *m*, that is, equivalent to the safety factor, was set at 1.30 or higher for permanent situations, but in cases where the reliability of the constants used in verification can be considered high, based on actual data for the same ground, and monitoring work is carried out by observing the displacement and stress of the ground during construction, factor m could be set at 1.10 or more for the same situations. In line with these rules, when partial factors *S* and *R* have not been determined, they can be set as 1.0, in accordance with the conventional method, and the adjustment factor m can be set to a value equivalent to the conventional safety factor to verify stability.

$$m\cdot\frac{S_d}{R_d}\leq 1.0 \quad R_d=\gamma_R R_k \quad S_d=\gamma_S S_k$$

$$S_k=\Sigma\left\{\left(W_k+q_k\right)\sin\theta+\frac{1}{R}aP_{Hk}\right\} \qquad (1.14)$$

$$R_k=\Sigma \;\; c_k s+\;\; W'_k+q_k \;\; \cos^2\theta\cdot\tan\varphi_k \;\; \sec\theta$$

TCVN 11820 Part 2: 2025 Equation (A.102) (A.103)

Where:

- m : adjustment factor
- S_d : value to be used for design of the action term (kN/m)
- R_d : value to be used for design of the resistance term (kN/m)
- S_k : characteristic value of the action term (kN/m)
- R_k : characteristic value of the resistance term (kN/m)
- γ_S : partial factor multiplied by action term
- γ_R : partial factor multiplied by resistance term
- W_k : characteristic value of total weight of a segment, total weight of soil and water (kN/m)
- q_k : characteristic value of vertical action from top of slice segment (kN/m)
- θ : angle of bottom of slice segment to horizontal (°)
- a : arm length from the center of slip circle in circular slip failure at position of P_H action (m)
- P_{Hk} : characteristic value of horizontal action on slice segment of soil mass per unit of length in circular slip (kN/m)
- R : radius of circular slip failure (m)
- c_k : characteristic value of undrained shear strength in case of clayey ground, or characteristic value of apparent cohesion in drained condition in case of sandy ground (kN/m^2)
- s : width of slice segment (m)
- W'_k : characteristic value of effective weight of slice segment per unit of length (weight of soil. When submerged, unit weight in water) (kN/m)
- φ_k : characteristic value in case of cohesion soil ground, 0, and in case of sandy ground, characteristic value of angle of

Modified from TCVN 11820 Part 4-1: 2020, Equation (F.1)

Modified from TCVN 11820 Part 6: 2023, Equation (7) shearing resistance in drained condition (º)

Table 1.7- Partial Factors for the Performance Verification of Circular Slip Failure

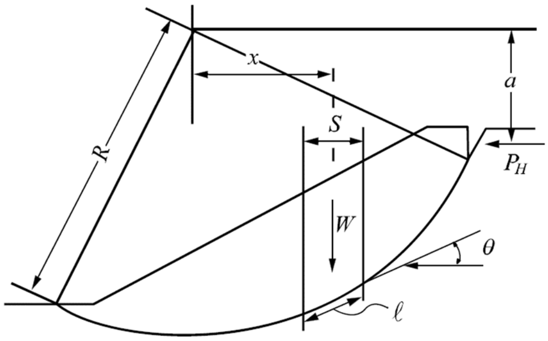
TCVN 11820 Part 6: 2023, Bang 6

Verification object	Coefficient of variation of cohesive soil in the representative soil layer CV	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by action term γ_S	Adjustment factor m
Circular slip failure (Permanent situation)	No cohesive soil	0.83	1.01	(1.0)
	CV <0.10	0.86	1.05	(1.0)
	$0.10 \leq CV < 0.15$	0.85	1.04	(1.0)
	$0.15 \leq CV < 0.25$	0.80	1.02	(1.0)
	$0.25 \leq CV$	(1.0)	(1.0)	1.30

Source: TCVN 11820-6-2023

TCVN 11820 Part 2: 2025 Hinh A.37

TCVN 11820 Part 4-1: 2020, Hinh F.1



Source: TCVN 11820-2-2025, TCVN 11820-4-1-2020, OCDI 2020

Figure 1.7- Circular Slip Failure Analysis Using Modified Fellenius Method

1-10. Structural Details

- ✓ The foundations of sloping breakwaters shall be provided with scouring and washing-out prevention measures as needed.
- ✓ The scour prevention measures include berm of rubble mounds at slope toes or the protection of slope toes with rubble blocks, submerged floor mats, asphalt mats, or synthetic resin mats.
- ✓ The measures to prevent rubble mounds from settlement due to washing out include the installation of submerged floor mats or the laying of canvas sheets.
- ✓ Generally, when constructing superstructures on rubble block and rubble mound breakwaters, the rubble foundations of superstructures shall be blinded with small rubble blocks.
- ✓ The surface finish work of sloping breakwaters shall be implemented in a manner that ensures the adequate interlocking effects of surface armor unit materials with careful attention to the finishing of crown sections.
- ✓ In coastal areas affected by littoral drifts, sloping breakwaters are preferably provided with sediment infiltration prevention work to prevent harbors from possible siltation owing to sand passing through sloping breakwaters together with waves.
- ✓ Sediment infiltration prevention work is normally implemented in a manner that constructs walls with sheet piles or blocks inside breakwaters or dumps stone

materials with a wide particle size distribution inside the sloping breakwaters or on the slopes at a harbor side.

- ✓ It shall be noted that sloping breakwaters are susceptible to wave actions that scatter stones.
- ✓ For the mixture of materials to be used when covering sloping breakwaters by sand mastic method, refer to OCDI 2020 Part II, Chapter 11, 4 Asphalt Materials.
- ✓ When constructing sloping breakwaters on soft ground, the settlement and subduction of breakwater bodies generally cause the quantities of rubble stones or blocks required in actual construction to be significantly larger than those based on the cross sections obtained by performance verification. Even in cases of favorable ground conditions, additional quantities of stones are preferably procured in actual construction in anticipation of the scattering and consolidation of stones due to waves.

2. Design Example

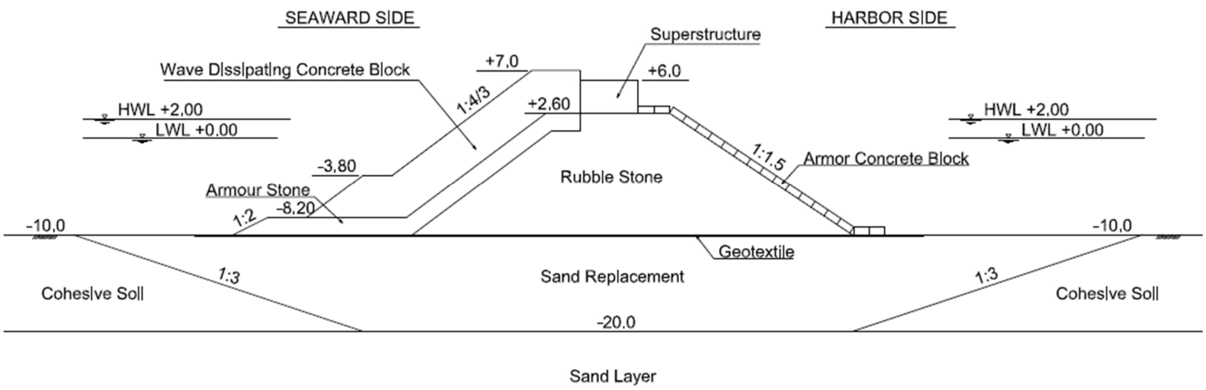


Figure 2.1- Sloping Breakwater

2-1. Design Conditions

(1) Design Wave (Wave estimation method is introduced in Part 7 Caisson Breakwater)

$H_{1/3} = 5.9$ (m)

$H_D = H_{max} = 1.8 \times 5.9 = 10.6$ (m)

$T = 10.0$ (sec)

(2) Tide Level

H.W.L. +2.0 (m)

L.W.L. ± 0.0 (m)

(3) Design Water Depth where the Breakwater Installed

-10.0 (m)

(4) Soil Condition

-10.0 (m)~-20.0 (m):

Cohesive soil

$c_k = 15$ (kN/m^2), CV = 0.25, $\gamma = 16.0$ (kN/m^3), $\gamma' = 6.0$ (kN/m^3)

-20.0 (m)~ -40.0 (m):

Sand layer

$\varphi = 35$ (degree), $\gamma_t = 18.0$ (kN/m^3), $\gamma' = 10.0$ (kN/m^3)

(5) Friction coefficient between concrete and rubble stone

$f = 0.6$

(6) Unit weight

Non-reinforced concrete $\gamma_c = 22.6$ (kN/m^3, in air)

Rubble stone $\gamma_t = 18.0$ (kN/m^3, in air), $\gamma' = 10.0$ (kN/m^3, in water)

(7) Partial factor

1) Variable state of waves

i) Sliding of superstructure

$\gamma_R = 1.00$ (resistance term)

$\gamma_S = 1.00$ (load term)

$m = 1.20$ (adjustment factor)

ii) Overturning of superstructure

$\gamma_R = 1.00$ (resistance term)

$\gamma_S = 1.00$ (load term)

$m = 1.20$ (adjustment factor)

iii) Bearing capacity against eccentric inclined loads

$\gamma_R = 1.00$ (resistance term)

$\gamma_S = 1.00$ (load term)

$m = 1.00$ (adjustment factor)

2) Permanent state

i) Circular slip failure (for cohesion ground, CV ≥ 0.25)

$\gamma_R = 1.00$ (resistance term)

$\gamma_S = 1.00$ (load term)

$m = 1.30$ (adjustment factor)

2-2. Determination of Each Dimension

(1) Crest Height of Breakwater

H.W.L.$+0.6H_{1/3}$

$= +2.0+0.6\times5.9 = +5.54 \rightarrow +6.0$ (m)

Note: In order to prevent the wave-dissipating blocks from falling behind the breakwater, the height of the top of the superstructure shall be equal to or higher than the height of the center of gravity of the wave-dissipating blocks.

(2) Top Elevation of Rubble Stone

Considering the land-based construction, the top elevation of rubble stone was assumed to be +2.6 (m), 0.6 (m) above H.W.L.

(3) Crest Width of Rubble Stone

Considering the working width of the heavy equipment such as crane or backhoe for installation, the crest width of rubble stone shall be sufficiently wide.

(4) Required Mass of Wave-Dissipating Block

According to the results of hydraulic model test, wave-dissipating blocks with a K_D value of 8.3 and a slope of 1:4/3 are verified. Calculate the required mass using the Hudson formula based on stability numbers.

Since both γ_{Ns} and γ_H are 1.0, the characteristic value and the design value are the same value.

$$N_{Sd}^3 = N_{Sk}^3 = K_D \cot\alpha = 8.3 \times 4/3 = 11.07$$

$$H_d = H_k = 5.9 \text{ (m)}$$

$$M_d = \frac{\rho_r H_d^3}{N_{Sd}^3 (S_r - 1)^3}$$

$$= \frac{2.3 \times 5.9^3}{11.07 \times (2.3/1.03 - 1)^3}$$

$$= 22.8 \text{ (tons)}$$

Therefore, 25 (tons) wave-dissipating blocks (actual mass 23.0 tons) is used.

(Reference)

Required mass of wave dissipation block when using the stability constant according to the equation of Takahashi, Hanzawa

Takahashi and Hanzawa proposed the Equation (1.7) for wave-dissipating blocks in a fully layered cross-section:

$$N_S = C_H[a(N_0/N^{0.5})^{0.2} + b] \qquad \text{Copied (1.7)}$$

Where:

- N_0 : degree of damage, a kind of damage rate that represents the extent of damage: it is defined as the number of concrete blocks that have moved within D_n in the direction of breakwater alignment
- D_n : nominal diameter of the concrete blocks: $D_n = (M/\rho_r)^{1/3}$, where M is the mass of a concrete block
- N : wave number
- C_H : breaking effect coefficient; $C_H = 1.4/(H_{1/20}/H_{1/3})$, in non-breaking zone $C_H = 1.0$
- a, b : coefficients that depend on the shape of the concrete blocks and the slope angle. With deformed shape blocks having a K_D value of 8.3, it may be assumed that a=2.32 and b=1.33, if $\cot\alpha$=4/3, and a=2.32 and b=1.42, if $\cot\alpha$=1.5

Assuming that the seabed slope is 1/100 and H'_0 = 6.0 (m) (breaking zone),

$$h/H_0' = 12.0/6.0 = 2.0$$

$$H_0'/L_0 = 6.0/156 = 0.038$$

Therefore, from Figure 2.2, $(H_{1/20}/H_{1/3})$ = 1.32

Calculate the stable number N_S from Equation (2.1) with N_0 = 0.3 and N = 1,000.

$$C_H = 1.4/1.32 = 1.06$$

$$N_S = 1.06 \times [2.32 \times (0.3/1{,}000^{0.5})^{0.2} + 1.33] = 2.38$$

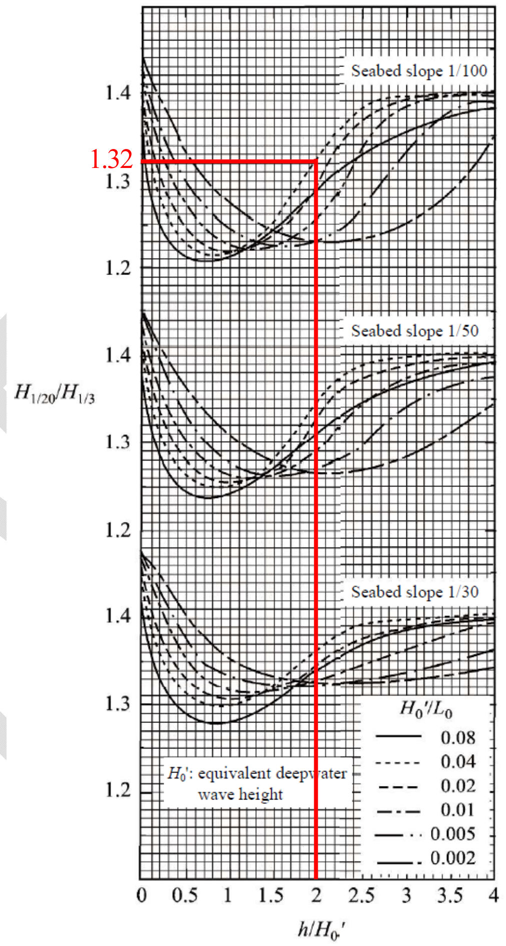
The required mass is calculated by the Hudson equation using a stable number. Since both γ_{Ns} and γ_H are 1.0, the characteristic value and the design value are the same.

$$M_d = \frac{\rho_r H_d^3}{N_{Sd}^3 (S_r - 1)^3}$$

$$= 1.0 \times \frac{2.3 \times 5.9^3}{2.38^3 \times (2.3/1.03-1)^3}$$

$$= 18.7 \text{ (tons)}$$

Therefore, according to the formula by Takahashi and Hirasawa et al., the wave-dissipating block in this design example can be of the 25.0 (tons) type (with an actual weight of 23.0 tons), which results from the large wave-breaking effect coefficient C_H in this calculation example.



Source: TCVN 11820-2-2025

Figure 2.2- Ratio of $H_{1/20}$ to $H_{1/3}$ ($H_{1/20}$ values: 5$H_{1/3}$ distance from the breakwater)

(5) Armor Stone Mass

The mass is 1/10~1/15 of the wave dissipation block mass.

M_d = (1/10~1/15) ×23.0 = 2.3~1.5 (ton/piece)

$D_n = (M/\rho_r)^{1/3} = (1.9/2.6)^{1/3}$ = 0.9 (m/layer) →1.8 (m): 2 Layers

(6) Required Mass of Blocks on the Harbor Side

The required mass of blocks on the harbor side is usually set to be equal to or half the required mass of blocks on the seaward side. The wave height of the harbor side shall be accurately calculated using hydraulic model experiments to determine the overtopping transmission and the effects of diffraction.

However, this design example indicates the required mass of blocks on the harbor side calculated through hydraulic model experiments.

$$M_d = 2.9 \text{ (ton)}$$

Therefore, a 3.0-ton type armor concrete block is used.

2-3. Assumptions of Design Cross-sections

Assume the design cross-section as shown in Figure 2.3.

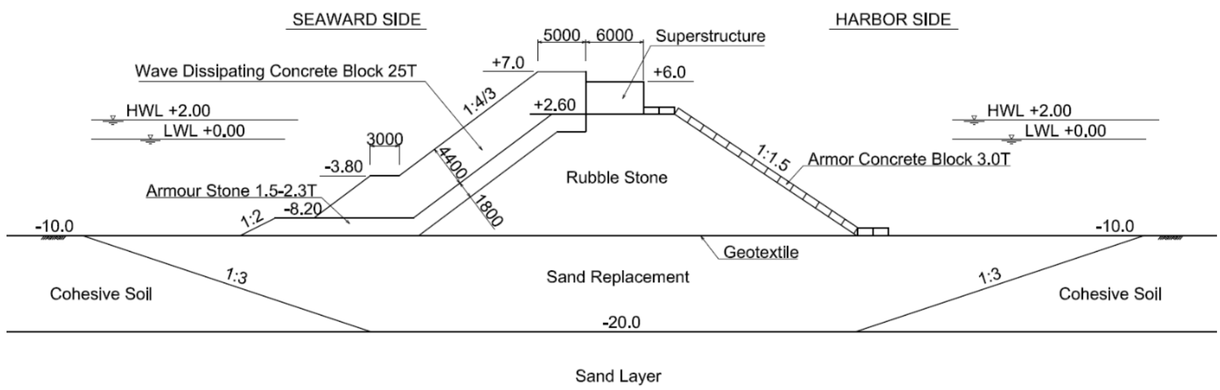


Figure 2.3- Assumed Design Cross-section

2-4. Stability of Superstructure

(1) Weight and Resistance Moment

$$W_k = 6.0\times3.4\times22.6 = 461.04 \text{ (kN/m)}$$

$$M_{Wk} = 461.04\times6.0/2 = 1{,}383.12 \text{ (kN·m/m)}$$

(2) Wave Force and Overturning Moment

1) Wave Pressure

$$\lambda = \exp[-10(h/L)^{1.5}(1-h'/h)^5]$$

$$= \exp[-10\times(12.0/99.7)^{1.5}\times\{1-(-0.6/12.0)\}^5] = 0.59$$

$$L = \frac{gT^2}{2\pi}\tanh\frac{2\pi h}{L}$$

$$= \frac{9.81\times10.0^2}{2\pi}\tanh\frac{2\pi\times12.0}{L} = 99.7 \text{ (m)}$$

$\eta^* = 0.75(\ 1+\cos\beta\ )\ \lambda\ H_D$

$= 0.75\times(1+\cos 0°)\ \times 0.59\times 10.6\ = 9.38\ (m)$

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

$$= 0.6 + \frac{1}{2}\left[\frac{4\ \pi\times 12.0/99.7}{\sinh\ (4\pi\times 12.0/99.7)}\right]^2 = 0.845$$

$\alpha_3 = 1\ +\ h'/\ \eta^*\ (h' \leq 0)$

$= 1\ +\ (-0.6)/9.38 = 0.936$

$h_c^* = \min\ (\ \eta^*,\ h_c\ )$

$= \min\ (9.38,\ 4.0)$

$= 4.0\ (m)$

$\alpha_4 = 1\ -\ h_c^*\ /\ \eta^*$

$= 1\ -\ 4.0/9.38 = 0.573$

$p_1 = {}^1\!/_2\ (\ 1+\cos\beta\ )\ \lambda\ \alpha_1\ \rho_0\ g\ H_D$

$= {}^1\!/_2\ (\ 1+\cos 0°)\times 0.59\times 0.845\times 1.03\times 9.81\times 10.6 = 53.40\ (kN/m^2)$

$p_3 = p_u = \alpha_3\ p_1$

$= 0.936\times 53.40 = 49.98\ (kN/m^2)$

$p_4 = \alpha_4\ p_1$

$= 0.573\times 53.40 = 30.60\ (kN/m^2)$

$$l_u = \min\ \ \ B,\ 0.2\ \times\frac{(\ \eta^*+h')^2}{|h'|}$$

$$= \min\ \ \ 6.0,\ 0.2\ \times\frac{(\ 9.38+(-0.6))^2}{|-0.6\ |}$$

$= \min\{\ 6.0,\ 25.70\ \ \}\ = 6.0\ (m)$

2) **Wave Pressure Distribution**

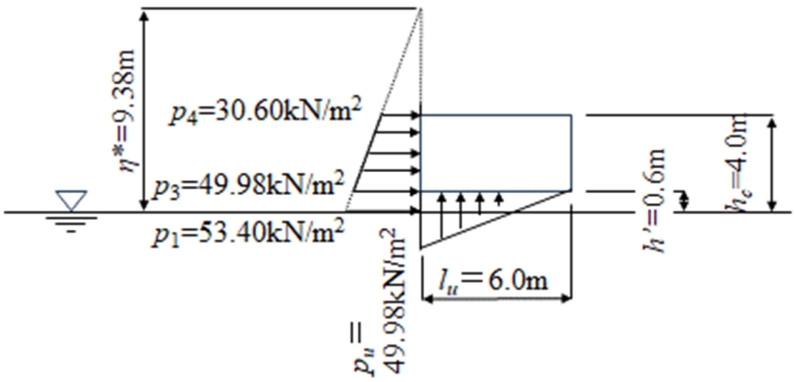


Figure 2.4- Wave Pressure Distribution

3) **Wave Force and Overturning Moment**

P_{Hk}= 1/2×(30.60+49.98)×(4.0-0.6) = 136.99 (kN/m)

$$M_{Pk}=\frac{(4.0\text{-}0.6)^2}{6}\times(2\times30.60+49.98) = 214.21 \text{ (kN·m/m)}$$

4) **Lifting Pressure and Overturning Moment**

P_{Uk}= 1/2×49.98×6.0 = 149.94 (kN/m)

M_{Uk}= 149.94×(2/3×6.0) = 599.76 (kN·m/m)

2-5.Stability Verification of Superstructure

(1) Performance Verification of Sliding

$$m\cdot\frac{S_d}{R_d}\le 1.0 \quad R_d=\gamma_R R_k \quad S_d=\gamma_S S_k$$

$$R_k=f_k(W_k\text{-}P_{Bk}\text{-}P_{Uk})$$

$$S_k=P_{Hk}$$

$$m\cdot\frac{S_d}{R_d}=m\frac{\gamma_S\times P_{Hk}}{\gamma_R\times(f_k\times(W_k\text{-}P_{Uk}))}$$

$$=1.20\times\frac{1.0\times136.99}{1.0\times(0.60\times(461.04\text{-}149.94))}=0.88\le 1.0$$

(2) Performance Verification of Overturning

$$m\cdot\frac{S_d}{R_d}\le 1.0 \quad R_d=\gamma_R R_k \quad S_d=\gamma_S S_k$$

$$R_k=a_1W_k\text{-}\ a_2P_{B_k}\text{-}\ a_3P_{U_k}$$

$$S_k = a_4 P_{H_k}$$

$$m \cdot \frac{S_d}{R_d} = \frac{\gamma_s \times M_{Pk}}{\gamma_R \times (M_{Wk} - M_{Uk})}$$

$$= 1.20 \times \frac{1.0 \times 214.21}{1.0 \times (1{,}383.12 - 599.76)} = 0.33 \leq 1.0$$

(3) Performance Verification of Bearing Capacity (Bishop's method)

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$S_d = \gamma_s S_k = \gamma_s \Sigma \quad W_k + q_k \ \sin\theta + aP_{Hk} \ /R$$

$$S_k = \gamma_R R_k = \gamma_R \Sigma \left[\left\{ c_k s + \left(W_k' + q_k \right) \tan\varphi_k \right\} \frac{\sec\theta}{1 + \tan\theta \cdot \tan\varphi_k / \left(m/\gamma_R \right)} \right]$$

1) Load Acting Position from the Toe

$$M_k = M_{Wk} - M_{Uk} - M_{Pk}$$

$$= 1{,}383.12 - 599.76 - 214.21 = 569.15 \ (\text{kN·m/m})$$

$$V_k = W_k - P_{Uk}$$
$$= 461.04 - 149.94 = 311.1 \ (\text{kN/m})$$

$$b' = M_k / V_k$$
$$= 569.15/311.1 = 1.83 \ (\text{m}) \ < B/3 = 6.0/3 = 2.0 \ (\text{m})$$

2) Distribution Load

$$p_1 = \frac{2}{3} \cdot \frac{V}{b'} = \frac{2}{3} \times \frac{311.1}{1.83} = 113.33 \ (\text{kN/m}^2)$$

$$b = 3b' = 3 \times 1.83 = 5.49\text{m}$$

The equivalent distributed load is,

$$q = \frac{p_1 b}{4b'} = \frac{113.33 \times 5.49}{4 \times 1.83} = 85.00 \ (\text{kN/m}^2)$$

3) Load Acting Width

$$B = 2b' = 2 \times 1.83 = 3.66 \ (\text{m})$$

4) Horizontal Load

The horizontal force H (= P_{Hk} = 136.99 kN/m) acts on the bottom of the foundation.

5) Soil Parameter

The strength of the stone materials used in the Bishop method is assumed to be as follows:

Stone $\varphi = 35°$, $c = 20$ (kN/m^2)

The results of the analysis are shown in Figure 2.5.

$$m \cdot \frac{S_d}{R_d} = 1.0 \times 30{,}141.1/53{,}474.7 = 0.56 \leq 1.0$$

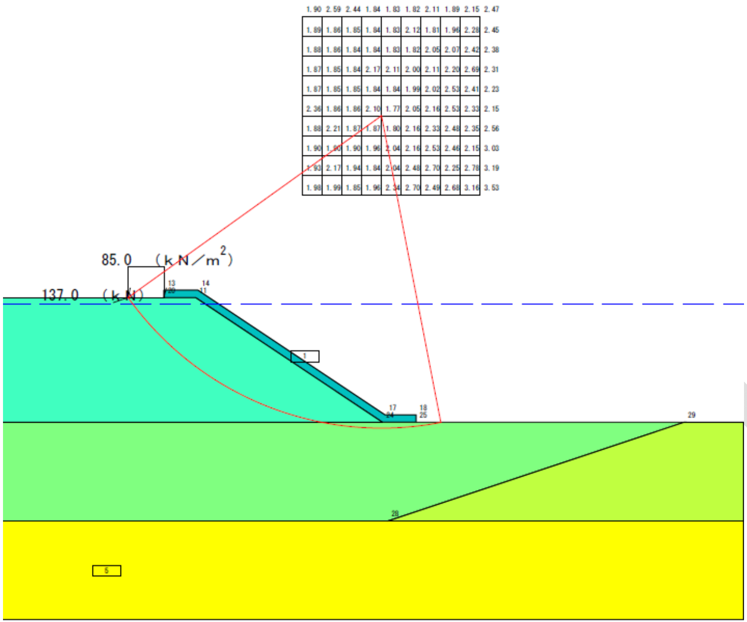


Figure 2.5- Performance Verification of Bearing Capacity

(4) Performance Verification of the Circular Slip Failure

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$S_d = \gamma_s S_k = \gamma_s \Sigma \quad W_k + q_k \quad \sin\theta \; + \; aP_{Hk} \;/R$$

$$S_k = \gamma_R R_k = \gamma_R \Sigma \quad c_k s + \quad W'_k + q_k \quad \cos^2\theta \cdot \tan\varphi_k \quad \sec\theta$$

The modified Fellenious method is applied to verify the circular slip failure. The partial factor for the strength of cohesive soil should be based on the coefficient of variation (CV). Given that 0.25 of the CV is applied, the partial factors for both action and resistance are set at 1.0.

The unit weight of the wave-dissipating block for circular slip failure analysis can be referred to the values in the following table.

Unit weight of wave-dissipating block for circular slip failure:

Porosity of wave-dissipating block: 50.0 (%) (Specified by applied block)

Unit weight of non-reinforced concrete: 22.6 (kN/m^3)

Unit weight of seawater: 10.1 (kN/m^3)

Above water level	22.6 × (1- 0.50) = 11.3 (kN/m^3)
Below water level (Saturated)	22.6 × (1- 0.50) + 10.1×0.50= 16.35 (kN/m^3)

Below water level (Submerged)	16.35-10.1= 6.25 (kN/m^3)

The results of the analysis are shown in Figure 2.6.

$$m \cdot \frac{S_d}{R_d} = 1.3 \times 75{,}864.7/98{,}875.0 = 0.99 \leq 1.0$$

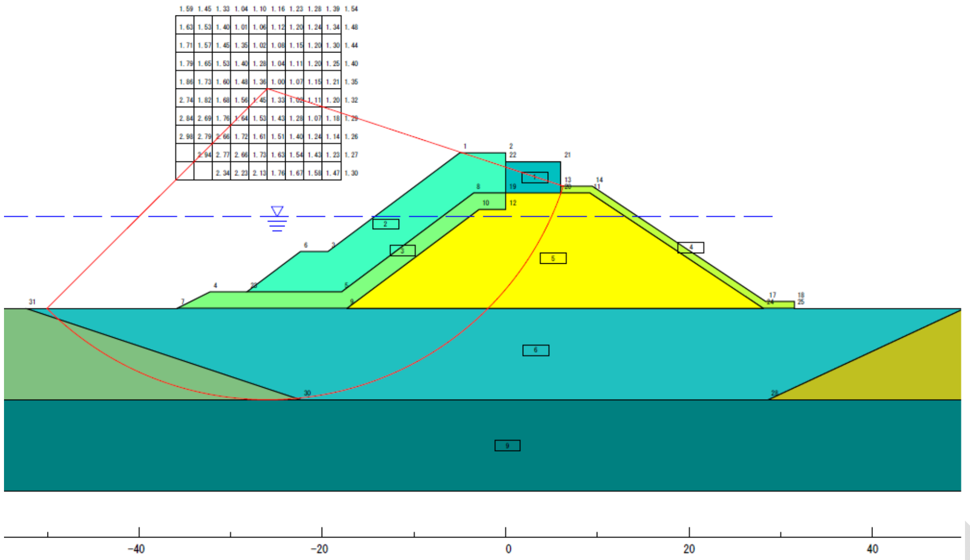


Figure 2.6- Performance Verification of Circular Slip Failure

- End –