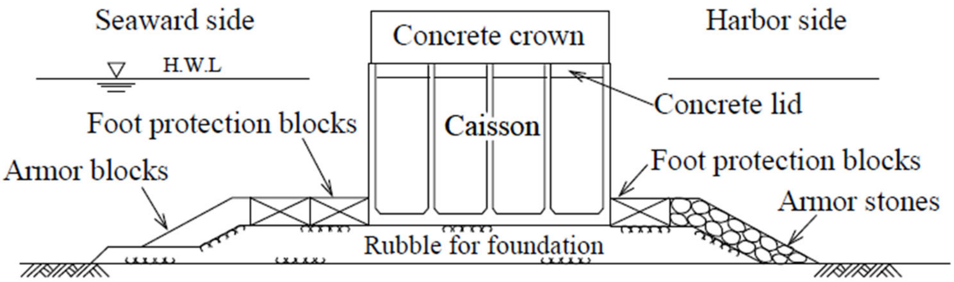
Part 7 Caisson-type Gravity Breakwater (Draft Final)

March 2026
Reference

1. Technical Notes

1-1. Characteristics of Caisson-type Gravity Breakwater

The gravity-type caisson composite breakwater is a structural form of a breakwater that vertical wall sections and rubble mound sections, as shown in Figure 1.1. It has a significant construction track record in the world and can be considered a standard breakwater structure. The characteristics of this structure include reflecting waves using the caisson to reduce wave intrusion into the harbor, enhancing cost-effectiveness, and building a foundation with a rubble mound for load distribution on the caisson. Additionally, Installation in soft ground requires ground improvement methods such as sand replacement or D.M.M. Compared to wave conditions, when the rubble mound level is shallow, it functions similarly to a sloping breakwater, and when deep, it functions similarly to a vertical wall breakwater.



Source: OCDI 2020

Figure 1.1- Gravity-type Caisson Composite Breakwater

1-2. Basic Policy for Performance Verification

The example of the sequence for performance verification of the gravity-type caisson composite breakwater is shown in Figure 1.2.

(1) Stability Verification

Since the caisson composite breakwater maintains stability through the weight of the structure, the verification will be conducted for 1) and 3). The necessity for verification of 2) will be determined based on the wave condition and seismic condition.

1) Conditions related to Wave Motion

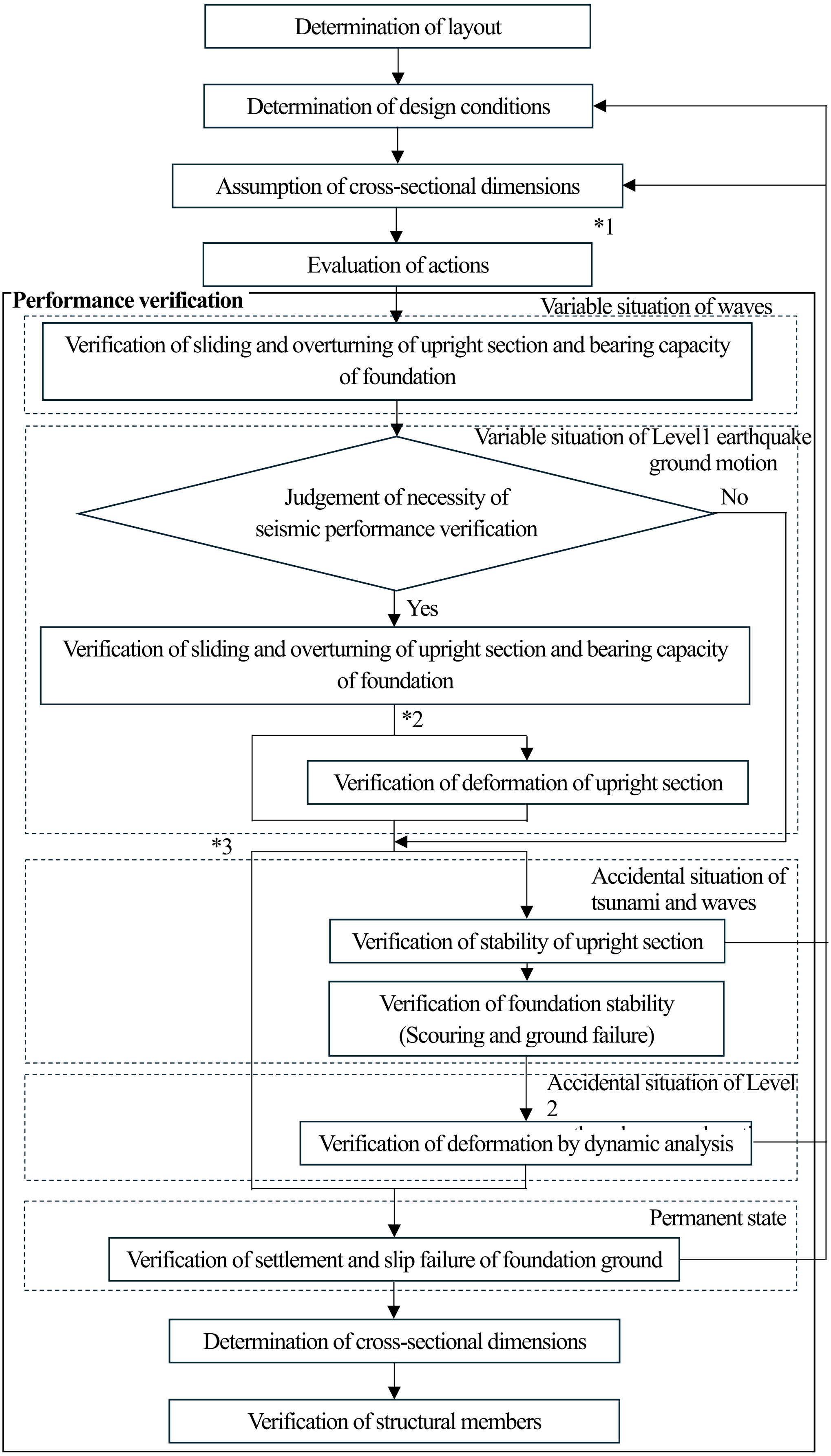
- ✓ Sliding of the breakwater
- ✓ Overturning of the breakwater
- ✓ Bearing capacity of the foundation ground

2) Conditions related to Earthquake Ground Motion

- ✓ Sliding of the breakwater
- ✓ Overturning of the breakwater
- ✓ Bearing capacity of the foundation ground

3) Permanent States

- ✓ Slip failure of the foundation ground
- ✓ Settlement



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*1: The evaluation of the effects of liquefaction and settlement are not shown; therefore, this must be separately considered.

*2: The analysis of deformation due to Level 1 earthquake ground motions may be carried out by dynamic analysis when necessary. For facilities where damage to the objective facilities is assumed to have a serious impact on life, property, and social activity, it is preferable to conduct an examination of deformation by dynamic analysis.
*3: For facilities where damage to the objective facilities is assumed to have a serious impact on life, property, and social activity, it is preferable to conduct a verification for the accidental situations when necessary.

Source: TCVN 11820-6-2023

Figure 1.2- Flow Chart of Caisson-Type Composite Breakwater Design

(2) Performance Verification of Structural Members

The performance verification of structural members is fundamentally based on the performance standards derived from the required performance of the facility. It is essential to establish appropriate verification indicators for the performance of structural members. Verification of members will be conducted regarding the following conditions.

1) Verification of Safety

✓ Verification against cross-sectional failure
✓ Verification against fatigue failure

2) Verification of Serviceability

The performance verification procedure for caissons is shown in Figure 1.3.

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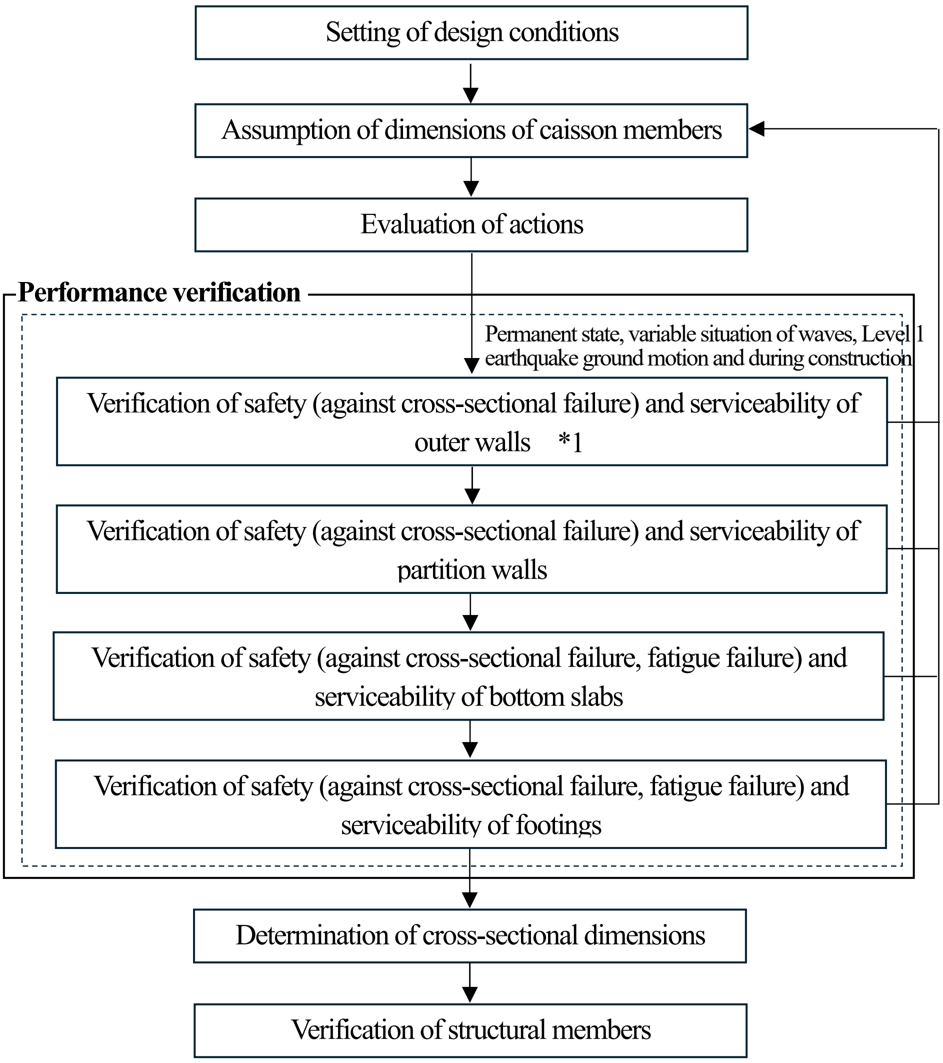


Note:
*1 When the caisson quaywall is affected by waves for a long period, the safety verification of fatigue failure may be added.
*2 For high earthquake-resistance facilities or the facilities to which damage might have a serious impact on human life, property, and social activity, it is preferable to verify the performance under accidental situations, as necessary. Verification of accidental situation associated with waves shall be performed in cases where damage to those facilities might have a serious impact on hazardous material handling facilities located just behind them.

Source: Modified from TCVN 11820-6-2023

Figure 1.3- Example of the Performance Verification Procedure for Caissons

1-3. Design Conditions

(1) Design Water Levels

When calculating wave forces, the design tide level for ports where the effects of storm surges do not need to be considered is typically the mean high-water level (H.W.L) and mean low water level (L.W.L.). For ports where the effects of storm surges need to be considered, the design tide level is set by adding a design deviation to the mean high water and mean low water levels, thus representing the state in which the facility is most unstable.

The tide level used as the premise for wave deformation calculations to determine design waves is generally based on characteristic values of the design tide level; however, depending on the tide level difference, it is often represented by the mean high-water level (H.W.L.) where high waves occur. On the other hand, for calculating design waves related to fatigue limit states, the average tide level (M.S.L.) is often used.

In Japan, the mean high-water level (H.W.L.) and the mean low water level (L.W.L.) are based on the astronomical tide, but in Vietnam, a method is used to calculate the exceedance probability according to the grade of the port facility from the observed value. Therefore, the design water level should be calculated based on the Vietnamese practices according to TCVN11820 Part 2 5.9.2 Design water levels.

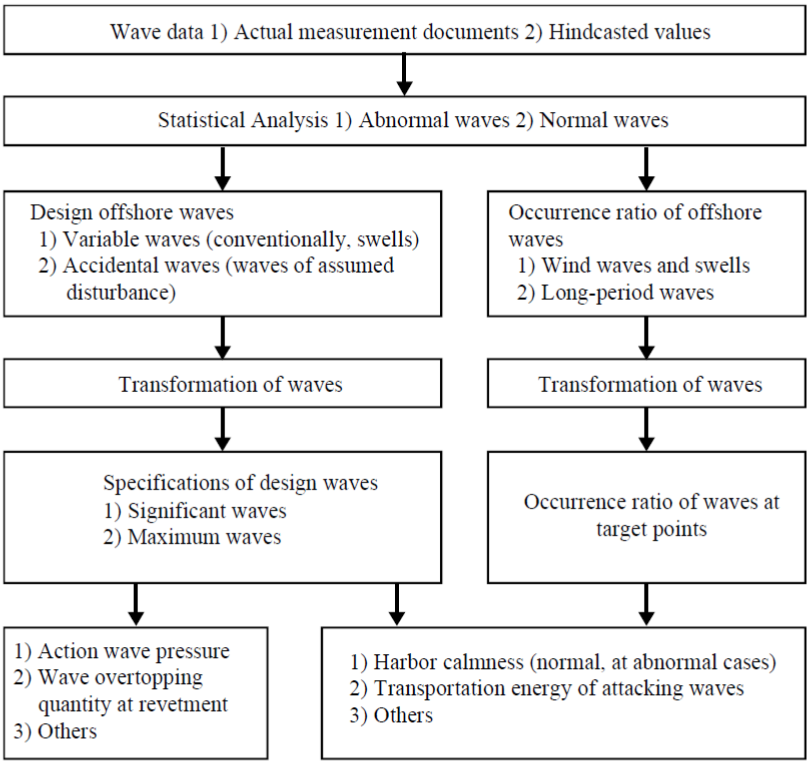
(2) How to determine the Design Waves

The calculation of design wave heights (significant wave height $H_{1/3}$, maximum wave height H_{max}) is performed following the flow shown in Figure 1.4.

For general port facilities with a design service life of 50 years, it is acceptable to consider the 50-year probability wave. However, for waves that act during construction (in cases where the structure is left incomplete for a certain period), it is necessary to appropriately determine the waves based on factors such as the construction period and natural conditions at the site; for convenience, waves with a 10-year probability can be used.

In addition, if reflected waves are to be considered in the calculation of design wave heights, separate investigations of reflected waves will need to be conducted, as mentioned later.

Note: $H_{1/3}$ in TCVN11820 Part 2 is calculated by the zero-up cross method, but the spectral method is sometimes used for wave prediction. The significant wave height H_s estimated by the spectral method may not be exactly the same as $H_{1/3}$.



Source: TCVN 11820-2-2025

Figure 1.4- Setting Procedure of Waves Used in Design

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1) Offshore Waves

The calculation of offshore waves can be approached in two ways, depending on the availability of nearby wave observation data:

i) Actual Measurement Documents of Wave Observation Data

If there are nearby wave observation data such as NOAA data available, extreme waves are selected from the wave observation records based on each factor contributing to wave generation, and statistical processing is performed to represent them as probabilistic wave heights. The statistical processing of probabilistic wave heights is conducted similarly to that of wave prediction. However, if wave deformation between the observation points and the breakwater installation point is anticipated, it is necessary to convert to the equivalent offshore wave height H_0' by considering refraction, diffraction, and shallow water transformations, and then select extreme waves based on each factor contributing to wave generation, conducting statistical processing to calculate probabilistic wave heights.

ii) Hindcasted Values by Wave Prediction

Wave prediction involves using the recorded weather charts that caused high waves at the estimation location, considering the temporal continuity of meteorological elements (e.g., the progression of typhoons or tropical depressions). This can be performed using spectral methods or significant wave methods. The extreme waves obtained through this wave prediction are subjected to statistical processing and represented as probabilistic wave heights. The probability distribution of wave heights can utilize the Gumbel distribution and Weibull distribution.

iii) Wave Direction

Offshore wave parameters should be set for each incoming direction out of 16 compass directions, as a standard practice. However, directions where the wave height is small and are expected to have minimal impact on the structure may be excluded. Additionally, the direction concentration parameter (S_{max}) is calculated from the gradient of the offshore wave profile; if the wave profile gradient is not specified, it is common to use S_{max} =10 for wind waves, S_{max} =25 for initial decaying swells, and S_{max} =75 for swell-type waves.

2) Equivalent Offshore Wave Height

i) Refraction Coefficient (K_r)

When the effects of refraction are considered between the offshore wave prediction point and the breakwater installation point, the refraction coefficient is calculated through numerical computations such as the component wave method, which linearly superimposes the refraction coefficients of regular waves, or energy balance equations. For irregular waves, the refraction coefficient is determined by dividing the wave direction spectrum and frequency spectrum into an appropriate number of component waves, conducting refraction calculations for each component wave, and then applying linear superposition.

Note that the range of applicability for the refraction calculations used in determining the refraction coefficient should have a water depth of at least 0.5 times the offshore wave height.

ii) Diffraction Coefficient (K_d)

If the effects of diffraction are considered from the offshore wave prediction point, the diffraction coefficient is calculated using diffraction diagrams or diffraction calculations. Rough diffraction calculations can be performed using methods that linearly superimpose the analytical solutions of semi-infinite breakwaters. For accurately calculating diffraction in complex harbor shapes, methods such as finite difference methods or Green’s functions are used.

If the length of an island or the width of a bay is more than ten times the wavelength of the incident waves, significant differences arise when estimating wave heights using the energy of waves directly reaching points behind the island or in the harbor without performing diffraction calculations (directional dispersion method). However, if the target point is located immediately behind an island or cape, the influence of diffracted waves is significant, and the directional dispersion method will result in larger errors. Additionally, a secondary breakwater may be affected by the diffraction of the primary breakwater. In such cases, separate diffraction calculations using diffraction diagrams or similar methods are necessary.

When it is essential to consider both the refraction and diffraction of irregular waves simultaneously, methods utilizing energy balance equations considering diffraction and non-stationary weak gradient irregular wave equations can be employed. Finite difference methods for solving the nonlinear equation and the multi-component coupling method can also be utilized.

iii) Reflection Coefficient (K_R)

At the design location, if the influence of reflections from other breakwaters or similar structures is considered, a study of reflected waves is conducted.

iv) Equivalent Offshore Wave Height

The wave height corrected by the refraction coefficient (K_r), diffraction coefficient (K_d), and reflection coefficient (K_R) is referred to as the equivalent offshore wave height (H'_0) and can be calculated using the following Equation (1.1).

$$H'_o = K_r K_d \sqrt{K_R^2 + 1} \cdot H_o \qquad (1.1)$$

Where:

H'_0	:	equivalent offshore wave height (m)
H_0	:	offshore wave height (m)
K_r	:	refraction coefficient
K_d	:	diffraction coefficient
K_R	:	reflection coefficient

3) Design Wave Height

The characteristic value of the design wave height is set considering the effects of shallow water transformation or breaking waves. The shallow water coefficient can be determined according to the TCVN 11820 Part 2: 2025.

At locations where the water depth is generally less than about three times the equivalent offshore wave height, changes in wave height due to breaking waves should be considered, considering the irregularity of the waves. The breaking wave height is determined based on the significant wave height $H_{1/3}$ and the maximum wave height H_{max}, as outlined in the TCVN 11820 Part 2: 2025.

Generally, the wave height in the breaking zone is used as the breaking wave height; however, if the wave height reduction due to breaking is less than 2%, the shallow water coefficient should be applied. While the breaking wave height can also be calculated using simplified formulas from the TCVN 11820 Part 2: 2025. However, since large wave heights may be estimated, it is common to use diagrams of significant wave height and maximum wave height.

Note that at locations where the water depth is about 0.5 times the equivalent offshore wave height or less, the vertical fluctuation of the waves is greater than the energy of the flow. Therefore, when calculating wave forces acting on structures using the Gota formula, it is desirable to use the wave height at a depth of 0.5 times the equivalent offshore wave height.

Since $H_{1/3}$ and H_{max} are taken as the maximum values by direction, $H_{1/3}$ and H_{max} may not be in the same direction.

i) Design Water Depth

The design water depth shall be determined by considering long-term changes in the seabed topography, including the impacts of erosion and sedimentation.

ii) Seabed Slope

The seabed slope is appropriate to use the average seabed slope where the ratio of water depth to equivalent offshore wave height h/H'_0 is in the range of 1.5 to 2.5. If $h/H'_0 \geq 2.5$, the breakwater front slope should be used.

iii) Significant Wave Height

The significant wave height $H_{1/3}$ is determined according to the TCVN 11820 Part 2: 2025.

iv) Maximum Wave Height

When determining wave forces, H_{max} is considered at a depth that is five times the significant wave height away from the front face of a vertical wall, as determined by the TCVN 11820 Part 2: 2025. If the maximum wave height is not influenced by breaking waves, it is calculated as follows:

$$H_{max}=1.8H_{1/3} \tag{1.2}$$

TCVN 11820 Part 2: 2025, Equation (210)

Where:

H_{max} : maximum wave height (m)

$H_{1/3}$: significant wave height at the depth in front of the vertical wall (m)

(3) Ground Conditions

Ground conditions should be established based on the results of soil investigations.

1) Sandy Soil

The shear resistance angle of sandy soil can be calculated using the following formula introduced in TCVN 11820 Part 2: 2025.

$$\varphi=25+3.2\sqrt{100\cdot N/(\sigma'_{vo}+70)} \tag{1.3}$$

TCVN 11820 Part 2: 2025, Equation (65)

Where:

φ : shear resistance angle of sand (°)

N : standard penetration test N value

σ'_{V0} : effective overburden pressure (kN/m²) at the depth where the standard penetration test N value was measured

2) Clayey Soil

The following methods can be used to determine the undrained shear strength c_u of clayey soil. The method for determining strength must be determined comprehensively, taking into account field experience, soil characteristics, the importance of the facility and others.

- ✓ q_u Method
- ✓ Method using strength from triaxial tests considering initial stress and anisotropy
- ✓ Method using strength from direct shear tests
- ✓ Method combining unconfined compressive strength with strength from triaxial compression tests
- ✓ Method derived from in-situ vane shear tests
- ✓ Method derived from electric cone penetration tests

The unit weight should be determined from the results of the soil investigation.

(4) Friction Coefficient

The characteristic value of the static friction coefficient f_k is as follows:

- ✓ Concrete and rubbles: $f_k = 0.6$
- ✓ Friction enhancement mat and rubbles: $f_k = 0.75$
- ✓ Concrete and concrete: $f_k = 0.5$

When selecting materials for a friction enhancement mat at the bottom of caisson, it is crucial to consider factors such as durability, the significance of the facility, ocean conditions, and cost-effectiveness. Additionally, experimental results related to the friction coefficient should be carefully analyzed. For materials like geosynthetics or rubber, a friction coefficient value of 0.75 can generally be used. However, in cold regions, additional considerations are necessary.

(5) Other Structural Parameters

1) Crest Height of Breakwater

The crest height should be estimated from the results of hydraulic model tests. As a guideline, it is set to approximately 0.6 times the significant wave height above the mean high-water level (H.W.L.) and if wave overtopping should be minimized, it may be acceptable to set the height to approximately 1.25 times the significant wave height above the H.W.L or refer to other standards.

2) Superstructure

i) Thickness of the Superstructure

The thickness of the superstructure should be at least 1 m when the significant wave height in front of the breakwater is 2 m or more, and at least 50 cm when the significant wave height is less than 2 m.

ii) Integration of Superstructure and Breakwater Body

To enhance the integration of the caisson and superstructure, methods such as embedding the superstructure into the caisson during pouring, creating roughness on the cover concrete, or inserting reinforcing bars or structural steel can be used.

iii) Construction Joints of Superstructure

Construction joints should be provided for each caisson, with joints spaced between 10 and 20 m.

iv) Parapet

The parapet section should be integrated with the superstructure, and preferred connection methods are to install dowels or reinforcing bars or structural steel at the joints.

3) Cover Concrete for Infill Material

The thickness of the cover concrete should be at least 30 cm to avoid damage, and in rough wave conditions, it should be more than 50 cm. In cases where the wave conditions are severe and the cover concrete is left in place for a long time until the superstructure is constructed, a thickness of more than 1.0 m may be required.

4) Scouring Prevention Works

If scouring is a concern, the prevention works such as placing geotextile sheets and/or rubble stone should be carried out.

5) Specifications of Rubble Mound

i) Top Level

The top level of the rubble mound should be as deep as possible to avoid the effects of impact wave breaking. However, the level should be set at a depth suitable for installation of vertical walls and trimming of the rubble mound. In addition, the thickness of the rubble mound should be at least 1.5 m to effectively distribute the loads transmitted from the vertical wall.

ii) Shoulder Width

The shoulder width of the rubble mound should be sufficient to secure the required support against sliding of the ground and bearing capacity. Additionally, to minimize the effects of impact wave breaking, a width of at least 5 m is desirable for seaward side. The shoulder width on the harbor side may be about two-thirds of that on the seaward side.

iii) Slope of the Slope

The slope of the rubble mound should be determined based on stability calculations; generally, the seaward side can have a slope of about 1:2 to 1:3, while the harbor side can have a slope of about 1:1.5 to 1:2.

iv) Settlement of the Breakwater

The following causes of settlement may exist, and measures may be taken in advance by raising the rubble mound or increasing the height of the superstructure:

- ✓ Consolidation settlement of the existing ground
- ✓ Scouring of the existing ground
- ✓ Lateral flow of the existing ground
- ✓ Penetration of rubble mound or blocks into the existing ground
- ✓ Compression due to the reduction of voids in the rubble mound

6) Foot Protection Blocks

It is recommended to install two or more protection blocks on the seaward side of the upright section and at least one on the harbor side. Some holes in the protection blocks can reduce the uplift force acting on the blocks and significantly improve wave stability.

The required thickness of the protection block can be determined using the following formula:

$$t/H_{1/3} = d_f\,(h'/h)^{-0.787} \qquad (1.4)$$

TCVN 11820 Part 6: 2023, Equation (21)

Where:

t	:	required thickness of the foot protection block (m)
d_f	:	0.18 for the trunk section and 0.21 for a head section of the breakwater
h'	:	water depth at the top of the rubble mound (excluding blocks and stones) (m)
h	:	design water depth (m)

Applicable Range: $h'/h = 0.4 \sim 1.0$

The required thickness of protection block can be calculated, and the specifications for the protective block can be determined as shown in Table 1.1.

Table 1.1- Specifications for Foot Protection Blocks

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Required Thickness, t	Size $l \times b \times t$ (m)	Mass (t/piece)	
		Block with openings	Block without openings
0.8m or less	2.5×1.5×0.8	6.23	6.90
1.0m or less	3.0×2.5×1.0	15.64	17.25
1.2m or less	4.0×2.5×1.2	24.84	27.60
1.4m or less	5.0×2.5×1.4	37.03	40.25
1.6m or less	5.0×2.5×1.6	42.32	46.00
1.8m or less	5.0×2.5×1.8	47.61	51.75
2.0m or less	5.0×2.5×2.0	52.90	57.50
2.2m or less	5.0×2.5×2.2	58.19	63.25

Source: TCVN 11820-6-2023

7) Required Mass and Layer Thickness of Armor Material

The required mass of the armor material is calculated using the methods outlined below. If stone material corresponding to the calculated mass is available, it should be used. Be aware of design tides as dangerous conditions may occur at L.W.L. (low water level). Note that if the water depth above the armor is shallow as it may become unstable due to breaking waves.

i) Basic Equation for Calculation of Required Mass

As the equation for calculation of the required mass of armor stones and blocks in the foundation mound of a composite breakwater, Hudson's formula with the stability number N_S, as shown in the following equation, can be used in the same manner as with armor stones and blocks on sloping breakwater. This partial safety coefficient is the value in cases where the limit value of the damage rate is 1% or the limit value of the degree of damage is 0.3.

$$M = \frac{\rho_r H^3}{N_s^3 (S_r-1)^3} \qquad (1.5)$$

TCVN 11820 Part 2: 2025, Equation (235)

Where:

- M : required mass of stones or concrete blocks (t)
- ρ_r : density of stones or concrete blocks (t/m³)
- S_r : specific gravity of stones or concrete blocks relative to seawater ρ_r / ρ_0
- ρ_0 : density of seawater (1.03 t/m³)
- H : wave height used in stability calculations (m)
- Ns : stability number, a constant determined by the shape, slope, and damage rate of the armor stones

ii) Stability Number for Armor Stones by Extended Tanimoto's Formulas

The stability number Ns may be obtained using the method proposed by Inagaki and Katayama, which is based on the work of Brebner and Donnelly and past damage case of armor stones. However, the following formulas proposed by Tanimoto et al. are based on the current velocity in the vicinity of the foundation mound and allow the incorporation of a variety of conditions. These formulas have been extended by Takahashi et al. so as to include the effects of wave direction.

$$Ns = \max\left[1.8;\ 1.3\frac{1-\kappa}{\kappa^{1/3}}\frac{h'}{H_{1/3}} + 1.8\exp\left[-1.5\frac{(1-\kappa)^2}{\kappa^{1/3}}\frac{h'}{H_{1/3}}\right]\right]$$

$$:\ B_M/L' < 0.25$$

$$\kappa = \kappa_1 (\kappa_2)_B \qquad (1.6)$$

$$\kappa_1 = \frac{4\pi h'/L'}{\sinh(4\pi h'/L')}$$

$$(\kappa_2)_B = \max\left\{\alpha_s \sin^2\beta \cos^2(2\pi l \cos\beta/L');\ \cos^2\beta \sin^2(2\pi l \cos\beta/L')\right\}$$

TCVN 11820 Part 2: 2025, Equation (242)

Where:

- h' : water depth at the crown of the rubble mound foundation excluding the armor layer (m)
- λ : in the case of normal wave incidence, the front berm width of foundation mound B_M (m) in the case of oblique wave

incidence, either B_M or B_M', whichever gives the larger value of $(\kappa_2)_B$

B_M' : width in the normal perpendicular direction of the outer side of the protection block (m)

L' : wavelength corresponding to the design significant wave period at a water depth of h' (m)

α_s : correction coefficient for when the armor layer is horizontal (equal to 0.45)

β : incident wave angle, angle between the line perpendicular to the breakwater face line and the wave direction, no angle correction of 15° is applied

l : horizontal distance from the upright wall to the edge of armor material. Here, $l = B_M$ (m)

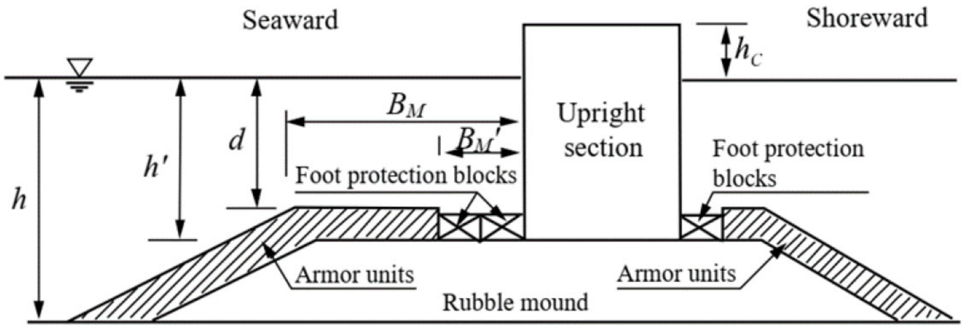
$H_{1/3}$: design significant wave height (m)

The weight of the head of the breakwater is calculated by the formula for κ shown below. However, if the calculated weight is less than 1.5 times that of the trunk of the breakwater, it is advisable to use 1.5 times. Additionally, the required area can generally be approximated to one caisson length of the caisson-type breakwater.

$\kappa = \kappa_1(\kappa_2)_T$

$(\kappa_2)_T = 0.22$

The stability number N_s of concrete blocks varies depending on the shape and installed method of the blocks, so it is preferable to calculate it through hydraulic model tests with irregular waves. There is a calculation formula based on the reference stability number, which is a characteristic value of the block, proposed by Fujiike et al. as TCVN11820 Part 2 Equation (247) and (248).



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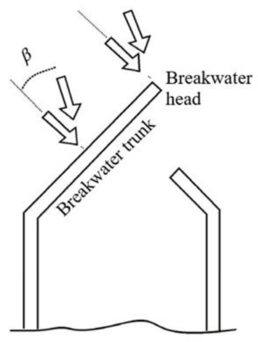
Source: TCVN 11820-2-2025

Figure 1.5- Standard Cross Section of a Composite Breakwater and Notations



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Source: TCVN 11820-2-2025

Figure 1.6- Effects of Shape of Breakwater Alignment and Wave Direction

iii) Stability Number When a Certain Amount of Damage is Permitted

Sudo et al. have carried out stability experiments for the special case such that the mound is low and no wave breaking occurs. They proposed the following equation that gives the stability number N_S* for any given number of waves N and any given damage rate D_N (%).

$$N_s = N_s[D_N/\exp\{0.3(1\text{-}500/N)\}]^{0.25} \qquad (1.7)$$

TCVN 11820 Part 2: 2025, Equation (246)

where N_S is the stability number given by the conventional Tanimoto's formula when N = 500 and the damage rate is 1%. In the performance verification, it is necessary to take N = 1000 considering the progress of damage, while the damage rate 3% to 5% can be allowed for a 2-layer armoring. If N = 1000 and D_N = 5%, then N_S* = 1.44 N_S. This means that the required mass decreases to about 1/3 of that required for N = 500 and D_N = 1%, which leads to N_S* = N_S.

1-4. Wave Force

(1) Wave Pressure on the Front Face according to the Goda's Formulas

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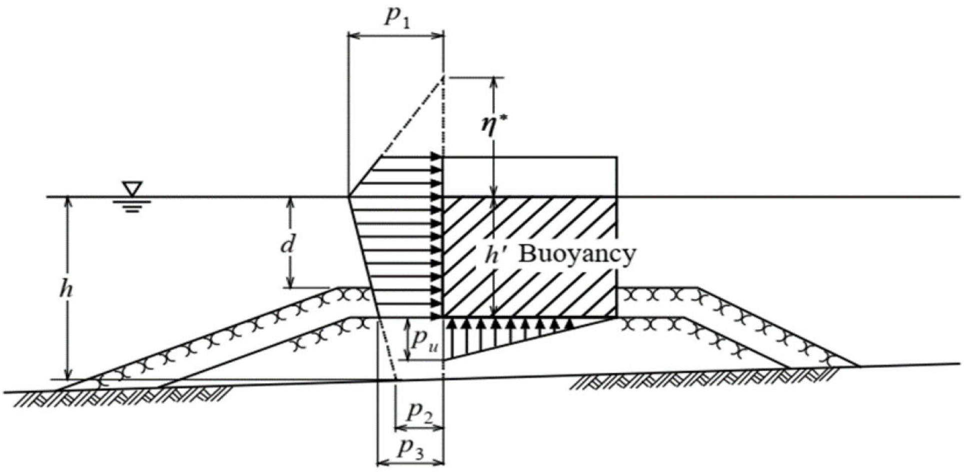

Source: TCVN 11820-2-2025

Figure 1.7- Wave Pressure Distribution Used in Design Calculation

The maximum wave force against a vertical wall and the corresponding uplift pressure are generally calculated using the following Goda formula.

$$\eta^* = 0.75(1+\cos\beta)\,\lambda\, H_D \qquad (1.8)$$

$$p_1 = 0.5(1+\cos\beta)(\alpha_1\lambda_1+\alpha_2\lambda_2\cos^2\beta)\rho_0 g H_D \qquad (1.9)$$

$$p_2 = \frac{p_1}{\cosh(2\pi h/L)} \qquad (1.10)$$

$$p_3 = \alpha_3 p_1 \qquad (1.11)$$

$$p_u = 0.5(1+\cos\beta)\alpha_1\alpha_3\lambda_3\rho_0 g H_D \qquad (1.12)$$

TCVN 11820 Part 2: 2025, Equation (202), (203). (204), (205), (209)

Where:

η^* : height above still water level at which intensity of wave pressure is zero (m)

p_1	:	intensity of wave pressure at still water level (kN/m²)
p_2	:	intensity of wave pressure at sea bottom (kN/m²)
p_3	:	intensity of wave pressure at toe of vertical wall (kN/m²)
p_u	:	uplift pressure acting on the bottom of the vertical wall (kN/m²)
h	:	water depth in front of vertical wall (m)
h_b	:	water depth at a point located five times the significant wave height offshore from the front of vertical wall (m)
h'	:	water depth at toe of vertical wall (m)
d	:	water depth at the crest of either the foot protection works or the mound armoring units of whichever is higher (m)
$\rho_0 g$	:	unit weight of water (kN/m²)
H_D	:	wave height used in the design calculations (m)
L	:	wavelength at water depth h used in calculation (m)
β	:	angle between the most dangerous direction within the range of ±15° from the main wave direction and the line perpendicular to face line of the vertical wall (°)
λ_1, λ_2	:	correction coefficient of wave pressure (standard value is 1.0)
λ_3	:	correction coefficient of uplift pressure (standard value is 1.0)

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{4\pi h/L}{\sinh\ (4\pi h/L)}\right]^2 \qquad (1.13)$$

TCVN 11820 Part 2: 2025, Equation (206), (207), (208)

$$\alpha_2 = \min\left\{\frac{h_b\text{-}d}{3h_b}\left(\frac{H_D}{d}\right)^2;\frac{2d}{H_D}\right\} \qquad (1.14)$$

$$\alpha_3 = 1 - \frac{h'}{h}\left[1\text{-}\frac{1}{\cosh\ (2\pi h/L)}\right] \qquad (1.15)$$

Where:

α_1	:	in the Gōda formula, a parameter representing the component of wave force from standing waves
α_2	:	in the Gōda formula, a parameter representing the impulsive wave force component due to mound height and seabed slope

Takahashi et al. experimentally derived the impulsive breaking wave force coefficient α_I which includes the mound's front shoulder width B_M, within the range of $H_D/h \geq 0.5$.

When the mound is tall and impulsive breaking wave forces act, α_2 in Equation (1.9) is generalized to α^*, and larger values of α_2 and α_I are used:

$$\alpha^* = \max\{\alpha_2;\alpha_I\} \qquad (1.16)$$

For convenience, when $H_D/h < 0.5$, the value of α_{I1} can be obtained by setting $h = 2H_D$ in the calculation.

The impulsive breaking wave force coefficient α_I is represented by the following equation. It is important to set the mound height and mound width carefully to prevent the value of α_I from becoming too large.

TCVN 11820 Part 2: 2025,

$$\alpha_I = \begin{cases} 0; & \alpha_{I1} \leq 0 \\ \alpha_{I0}\alpha_{I1}; & \alpha_{I1} > 0 \end{cases} \qquad (1.17)$$

Where:

α_{I0} : a parameter representing the influence of wave height

α_{I1} : a parameter representing the influence of mound shape

Equation (214)

The parameters α_{I0} and α_{I1} are expressed by the following equations.

TCVN 11820 Part 2: 2025, Equation (215)

$$\alpha_{I0}=\begin{cases}\dfrac{H_D}{d}; & \dfrac{H_D}{d}\leq 2\\ 2.0; & \dfrac{H_D}{d}> 2\end{cases} \tag{1.18}$$

$$\alpha_{I1}=\begin{cases}\dfrac{\cos\delta_2}{\cosh\delta_1}; & \delta_2\leq 0\\ \dfrac{1}{\cosh\delta_1(\cosh\delta_2)^{1/2}}; & \delta_2> 0\end{cases} \tag{1.19}$$

$$\delta_1=\begin{cases}20\delta_{11}; & \delta_{11}\leq 0\\ 15\delta_{11}; & \delta_{11}> 0\end{cases} \tag{1.20}$$

$$\delta_2=\begin{cases}4.9\delta_{22}; & \delta_{22}\leq 0\\ 3\delta_{22}; & \delta_{22}> 0\end{cases} \tag{1.21}$$

$$\delta_{11}=0.93\left(\frac{B_M}{L}\text{-}0.12\right)+0.36\left(\frac{h\text{-}d}{h}\text{-}0.6\right) \tag{1.22}$$

$$\delta_{22}=\text{-}0.36\left(\frac{B_M}{L}\text{-}0.12\right)+0.93\left(\frac{h\text{-}d}{h}\text{-}0.6\right) \tag{1.23}$$

Takahashi et al. have proposed that in general, when the upright wall is sufficiently covered with wave-dissipating concrete blocks, the wave pressure reduction coefficient λ_2 may be taken to be zero, while the values of λ_1 and λ_3 depend primary on the wave height H (the highest wave height). They have thus proposed the following equations:

TCVN 11820 Part 2: 2025, Equation (216)

$$\lambda_1=\begin{Bmatrix}1.0 & (H/h\leq 0.3)\\ 1.2\text{-}2(H/h)/3 & (0.3< H/h\leq 0.6)\\ 0.8 & (H/h>0.6)\end{Bmatrix} \tag{1.24}$$

$$\lambda_3=\lambda_1$$

$$\lambda_2=0$$

In the surf zone, where breakwaters covered with wave-dissipating concrete blocks are generally used, the above equations give $\lambda_1 = \lambda_3$=0.8.

As with the Takahashi formula, the Jensen formula can be applied as a wave pressure formula for impulsive breaking waves, so it may be used according to the verification conditions.

(2) Negative Wave Force of Wave Troughs on Wall Face

When the wave trough hits a wall, a negative wave force acts according to the depth of the trough from the still water surface. Negative wave force is a wave force obtained through appropriate hydraulic model experiments and calculations. This is a force directed toward the sea, and when the water is deep and the wavelength is short, it can be equal in magnitude to the positive wave force.

The negative wave pressure acting on the front of an upright wall at the wave trough can be approximately estimated as shown in Figure 1.8. Especially, it can be assumed that a wave pressure acts toward the sea, with the magnitude of this wave pressure being zero at the still water level and having a constant value of p_n from a depth $0.5H_D$ below the still water level right down to the toe of the wall. Here, p_n is given as follows:

$$p_n = 0.5\rho_0 g H_D \tag{1.25}$$

TCVN 11820 Part 2: 2025, Equation (211)

Where:

p_n	:	intensity of wave pressure in constant region (kN/m²)
$\rho_0 g$	:	unit weight of water (kN/m²)
H_D	:	wave height used in the design calculations (m)

In addition, the negative uplift acting on the bottom of upright wall can be assumed to act as shown in Figure 1.8. Especially, it can be assumed that an uplift acts downwards with its intensity being p_n as given by Equation (1.25) at the front toe, zero at the rear toe, and having a triangular distribution in-between. Incidentally, it is necessary to use the highest wave height as the wave height H_D used in the performance verification.

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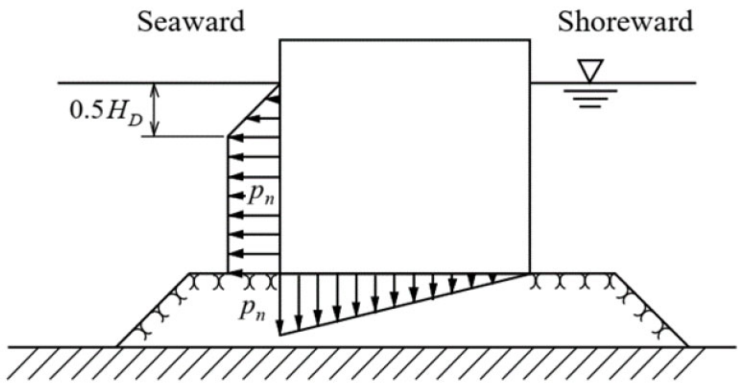


Source: TCVN 11820-2-2025

Figure 1.8- Negative Wave Pressure Distribution

Goda and Kakizaki calculated wave forces based on a fourth order approximate solutions of a finite amplitude standing wave theory and presented calculation diagrams for negative wave pressure. Their calculation results were verified to be in good agreement with experimental results. When the water is deep and standing waves are clearly formed, it is acceptable to use the results of this finite amplitude standing wave theory of higher order approximation.

However, in the case of deep-water breakwaters, it is necessary to note that the negative wave force at the wave trough may be greater than the positive wave force at the wave crest, which may cause the upright wall to shift offshore.

1.5 Performance Verification Method

(1) Stability Verification

For each design condition, performance verification of overall stability related to the breakwater structure is conducted based on static equilibrium equations. The items for performance verification are listed in Table 1.2, which provides a guide for the overall stability check (excluding accidental conditions).

Table 1.2- Performance Verification Items for Stability of the Breakwater Structure

Design State/Situation	Circular Slip Failure	Sliding of Body	Overturning of Body	Bearing Capacity of Foundation
Permanent	○	-	-	-
Wave Motion	-	○	○	○
Earthquake Ground Motion	-	(○)*	(○)*	(○)*

*: Items in ( ) can be omitted as necessary

1) Performance Verification for Overall Stability in Permanent State

In the permanent state, where the primary force is the self-weight of the breakwater structure, performance verification for overall stability typically involves checking for circular slip failure in the foundation ground. The verification of circular slip can be carried out using the following equation (Equation 1.26).

In this equation, the subscripts "*k*" and "*d*" denote characteristic values and design values, respectively. Additionally, the partial factors used in this equation can be found in Table 1.3. For parts where a "-" is indicated in Table 1.3, values in parentheses can be used for convenience when performing the verification.

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

$$R_k = \sum\left[\left\{c'_k . s + \left(w'_k + q_k\right)\cos^2\theta\tan\varphi'_k\right\}\sec\theta\right] \quad (1.26)$$

$$S_k = \sum\left[\left(w_k + q_k\right)\sin\theta\right]$$

TCVN 11820 Part 6: 2023, Equation (7)

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
- c' : undrained shear strength for cohesive soil or apparent adhesion under a drained condition for sandy soil ground (kN/m²)
- s : width of a segment (m)
- w' : effective weight of a segment (kN/m)
- w : total weight of a segment (kN/m)
- q : surcharge acting on a segment (kN/m)
- φ' : apparent shear resistance angle based on effective stress (°)
- θ : angle between the bottom face of a segment and the horizontal plane (°)

Table 1.3- Partial Factors Used for Performance Verification of Circular Slip Failure of Foundation Ground

TCVN 11820 Part 6: 2023, Bang 6

Verification Item	Coefficient of variation of clayey soil for representative layer: CV	Partial Factor γ_R	Partial Factor γ_S	Adjustment factor m
Circular Slip Failure of Foundation (Permanent State)	When no clayey soil exists in the layer through which the arc passes	0.83	1.01	- (1.00)
	Less than 0.10	0.86	1.05	- (1.00)
	0.10 or more and less than 0.15	0.85	1.04	- (1.00)
	0.15 or more and less than 0.25	0.80	1.02	- (1.00)
	0.25 or higher	- (1.00)	- (1.00)	1.30

Source: Modified from TCVN 11820-6-2023

2) Performance Verification for Stability in Variable Situations (Wave Motion)

In the case of variable situations, where the primary force acting on the structure is wave fluctuation, performance verification for stability generally involves checking for sliding and overturning of breakwater body, and the bearing capacity of the foundation ground.

i) Verification for Sliding of Breakwater Body

The stability against sliding due to wave fluctuations can be assessed using the following equation (Equation 1.27). In this equation, subscripts "*k*" and "*d*" denote characteristic values and design values, respectively. The partial factors used in this equation can be referenced from Table 1.4.

In Table 1.4, values marked with a "-" indicate that the values in parentheses can be used for verification purposes for convenience.

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

$$R_k = f_k \cdot (W_k - P_{Bk} - P_{Uk}) \qquad (1.27)$$

$$S_k = P_{Hk}$$

TCVN 11820 Part 6: 2023, Equation (8)

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 : friction coefficient between the bottom face and a foundation
- W : weight of the breakwater body (kN/m)
- P_B : buoyancy (kN/m)
- P_U : uplift force (kN/m)
- P_H : horizontal wave force (kN/m)

Table 1.4- Partial Factors Used Performance Verification of Sliding of Breakwater Bodies

Verification Item	Partial Factor γ_R	Partial Factor γ_S	Adjustment factor m
Sliding of Breakwater Body (Variable Situation of Wave)	0.83	1.08	- (1.00)

Source: TCVN 11820-6-2023

TCVN 11820 Part 6: 2023, Bang 7

The partial factors shown in Table 1.4 were set with reference to the safety levels in past standards. Furthermore, the partial factors above are set under the conditions that the topographies of seabed where breakwaters are installed have gradients of less than 1/30. In cases wherein the topographies of seabed have gradients larger than 1/30, partial factors shall be set with reference to the descriptions in available references.

In cases wherein caissons with footings have rectangular cross sections at both seaward and landward sides, buoyancy P_B can be calculated using the following equation. In this equation, subscript k indicates the characteristic value. For footings with other shapes and hunched sections, buoyancy shall be appropriately set:

$$P_{B_k} = \rho_w \cdot g \cdot \{(wl_k + h) \cdot B_c + 2h_f \cdot B_f\} \quad (1.28)$$

TCVN 11820 Part 6: 2023, Equation (9)

Where:

- ρ_w : unit weight of sea water (kN/m³)
- wl : tidal level (m)
- h : installation depth (m)
- B_c : width of the breakwater body (m)
- h_f : height of footing (m)
- B_f : width of footing (m)

ii) Verification of Overturning of Breakwater Body

The stability of the overturning of breakwater body due to variable waves can be evaluated using the equation (1.29). In this equation, the subscripts "k" and "d" indicate the characteristic values and design values, respectively. The partial factors used in the equation can be selected from the values in Table 1.5. For values marked with "-", it indicates that values in parentheses can be conveniently used for verification.

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

$$R_k = (a_1.W_k - a_2.P_{Bk} - a_3.P_{Uk}) \quad (1.29)$$

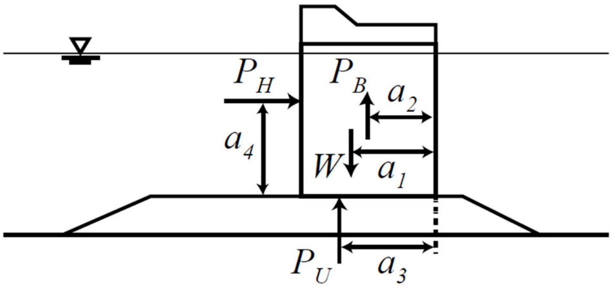
$$S_k = a_4.P_{Hk}$$

TCVN 11820 Part 6: 2023, Equation (10)

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
- W : weight of a breakwater body (kN/m)

P_B : buoyancy (kN/m)
P_U : uplift force (kN/m)
P_H : horizontal wave force (kN/m)
$a_1 \sim a_4$: arm lengths of actions (m) (refer to Figure 1.8)



TCVN 11820 Part 6: 2023, Hinh 31

Source: TCVN 11820-6-2023

Figure 1.9- Arm Lengths Used in Moment Calculation

Table 1.5- Partial Factors Used for Performance Verification of Overturning of Breakwater Bodies

TCVN 11820 Part 6: 2023, Bang 8

Verification Item	Partial Factor γ_R	Partial Factor γ_S	Adjustment factor m
Overturning of Breakwater Body (Variable Situation of Wave)	0.95	1.14	- (1.00)

Source: TCVN 11820-6-2023

iii) Examination of the Bearing Capacity of the Foundation Ground

For evaluating the stability of the foundation ground under the base of the upright section of the breakwater against variable wave loads, the stability can be examined using the following Equation (1.30), which is obtained through the simplified Bishop method. The partial factors in Equation (1.30) can be referenced from Table 1.6. For the parts in Table 1.6 marked with a "-", the values in parentheses can be used for verification as a matter of convenience. In this equation, the subscripts "*k*" and "*d*" denote characteristic values and design values, respectively.

When using Equation (1.30), first determine the auxiliary parameter F_f through iterative calculations to satisfy the condition $R_k = F_f \times S_k$ (note that R_k is included in the equation). Once R_k and S_k are obtained, the stability of the bearing capacity is verified using these values.

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

$$F_f = \frac{R_k(F_f)}{S_k} \quad (1.30)$$

$$R_k = \sum \left[\frac{\{c'_k . s + (w'_k + q_k)\tan\varphi'_k\}\sec\theta}{1 + \tan\theta\tan\varphi'_k / F_f} \right]$$

$$S_k = \sum \{(w_k + q_k)\sin\theta\} + \frac{dP_{Hk}}{r}$$

TCVN 11820 Part 6: 2023, Equation (11)

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
P_H : horizontal wave force (kN/m)
c' : undrained shear strength for cohesive soil ground or apparent adhesion under a drained condition for sandy soil ground (kN/m²)
s : width of a segment (m)
w' : effective weight of a segment (kN/m)
w : total weight of a segment (kN/m)
q : surcharge acting on a segment (kN/m²)
ϕ' : apparent shear resistance angle on the basis of effective stress (°)
θ : angle between the bottom face of a segment and a horizontal plane (°)
F_f : auxiliary parameter representing a ratio of a resistance term to a load term
d : arm length of the horizontal wave force P_H (m)
r : radius of a slip failure circle (m)

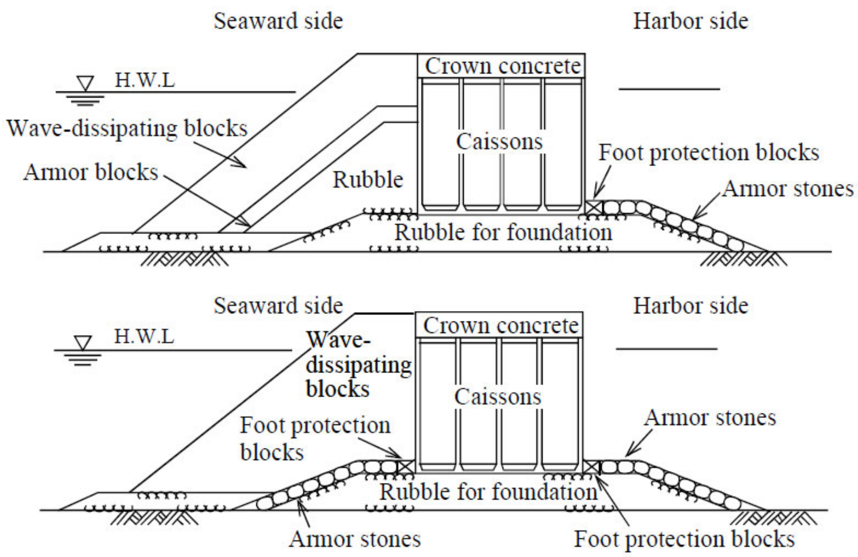
Table 1.6- Partial Factors Used for the Performance Verification of the Bearing Capacity of Breakwater Body

TCVN 11820 Part 6: 2023, Bang 9

Verification Iem	Partial Factor γ_R	Partial Factor γ_S	Adjustment factor m
Bearing Capacity of Foundation (Variable Situation of Wave)	- (1.00)	- (1.00)	1.00

Source: TCVN 11820-6-2023

3) Performance Verification of Wave-Dissipating Block Breakwater



Source: OCDI 2020

Figure 1.10- Examples of Cross Sections of Breakwaters Covered with Wave-Dissipating Blocks

The performance verification of the sliding and overturning failures of breakwaters covered with wave-dissipating blocks with respect to variable waves shall be performed using Equations (1.27) and (1.29). The partial factors in the equation can be selected from the values in Tables 1.7 and 1.8 that the value in parentheses in the column can be used for the performance verification for convenience.

Table 1.7- Partial Factors Used Performance Verification of Sliding of Breakwater Bodies

OCDI 2020 Part III Chapter 4, Table 3.4.1

Verification Item	Partial Factor γ_R	Partial Factor γ_S	Adjustment factor m
Sliding of Breakwater Body (Variable Situation of Wave)	0.79	0.90	- (1.00)

Source: OCDI 2020

Table 1.8- Partial Factors Used for Performance Verification of Overturning of Breakwater Bodies

OCDI 2020 Part III Chapter 4, Table 3.4.2

Verification Item	Partial Factor γ_R	Partial Factor γ_S	Adjustment factor m
Overturning of Breakwater Body (Variable States of Waves)	0.98	0.99	- (1.00)

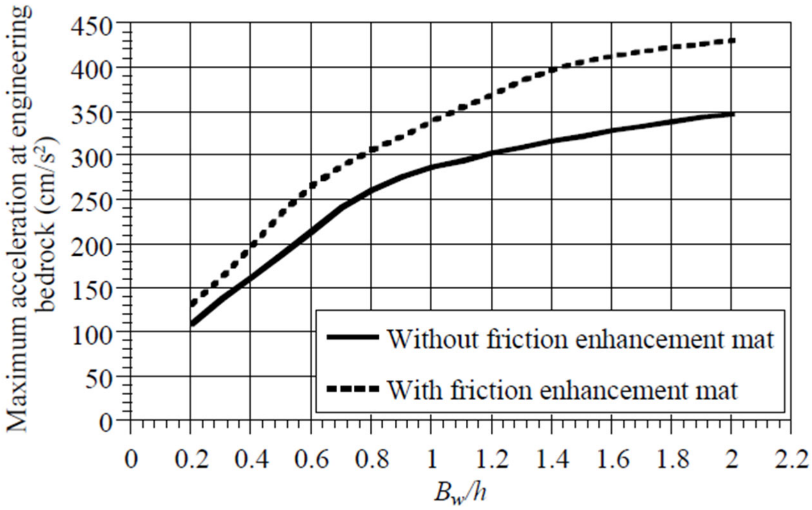
Source: OCDI 2020

4) Performance Verification of Stability of Breakwater Body in Variable Situations (Level 1 Earthquake Ground Motion)

Although the verification of the stability of breakwater bodies against Level 1 earthquake ground motions is often omitted, in cases wherein breakwaters have deep installation depths and small design wave heights, there may be cases wherein earthquake ground motions can be a dominant action. In such cases, the performance verification shall be performed for the earthquake resistance.

The necessity of the performance verification of earthquake resistance in terms of sliding and overturning due to earthquake ground motions can be determined on the basis of the relationship between the cross-sectional dimensions of breakwater bodies and earthquake ground motions under a variable action situation with respect to variable waves.

The necessity of the performance verification of earthquake resistance can be determined using the relationship between the B_w/h ratios of the widths of breakwater bodies B_w, excluding footings to their installation depths h and the maximum engineering acceleration at bed rock (Figure 1.11). The performance verification of earthquake resistance can be omitted when the concerned breakwaters are plotted below the curve in the figure. The figure is established on the basis of 30 cm as an allowable residual deformation of the upright sections of breakwaters subjected to earthquake ground motions. Therefore, when adopting other values for the allowable residual deformation, the specific verification of their appropriateness should be implemented.



TCVN 11820 Part 6: 2023, Hinh 30

Source: TCVN 11820-6-2023

Figure 1.11- Determination the Necessity of Performance Verification of Earthquake Resistance

5) Performance Verification for Stability of Floating Condition

During construction, for the caisson to float and self-stabilize, it must satisfy Equation (1.31).

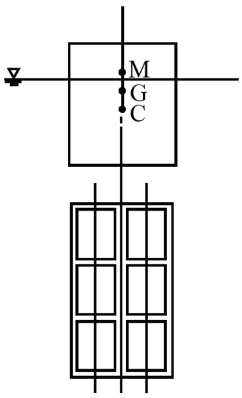
$$\frac{I}{V} - \overline{CG} = \overline{GM} \geq 0 \tag{1.31}$$

TCVN 11820 Part 6: 2023, Equation (B.7)

Where:

- V : displacement volume (m³)
- I : geometrical moment of inertia with respect to long axis at water level (m⁴)
- C : center of buoyancy
- G : center of gravity
- M : metacenter
- $\overline{CG}$: distance between center of gravity and center of buoyancy (m)
- $\overline{GM}$: distance between metacenter and center of gravity (m)

For safety, it is desirable that $\overline{GM}$ (the metacentric height) be at least 5% of the draft.



TCVN 11820 Part 6: 2023, Hinh B.2

Source: TCVN 11820-6-2023

Figure 1.12- Stability of Caisson

Equation (1.31) applies when the cross-section of the caisson is nearly symmetrical, and when the caisson experiences only minor tilts.

When a counter ballast is used for towing, the following equation is used:

- ✓ When using water as a counter ballast:

$$\frac{1}{V'}\left(I'-\sum i\right)-\overline{C'G'}>0 \qquad (1.32)$$

TCVN 11820 Part 6: 2023, Equation (B.8), (B.9)

- ✓ When using sand, stone, or concrete or the like as a counter ballast:

$$\frac{I'}{V'}-\overline{C'G'}>0 \qquad (1.33)$$

Where:

V'	:	displacement volume for caisson with counter ballast (m^3)
I'	:	geometrical moment of inertia with respect to long axis at water level for caisson with counter ballast (m^4)
C'	:	center of buoyancy for caisson with counter ballast
G'	:	center of gravity for caisson with counter ballast
$\overline{C'G'}$	:	distance between center of gravity and center of buoyance for caisson with counter ballast (m)
i	:	geometrical moment of inertia with respect to centerline parallel to axis of rotation of caisson at water level in each chamber (m^4)

(2) Performance Verification of Structural Members

1) Verification of Safety of Structural Members

The verification of safety of the structural members shall determine the verification indices considering the type of structural members, material properties and other factors, and shall compare their responses to their limit values.

i) Verification of cross-sectional failure

Cross-sectional failure shall be verified by confirming that the value obtained by multiplying the ratio of the design force resultant S_d to the design cross-sectional force R_d by the structure factor γ_i is 1.0 or less:

$$\gamma_i \cdot \frac{S_d}{R_d} \leq 1.0 \qquad (1.34)$$

The design force resultant S_d can be obtained by calculating the force resultant S (S is a function of F_d) using the design load F_d and then summarizing values of multiplying S by the structural analysis factor γ_a.

$$S_d = \sum \gamma_a \cdot S(F_d) \qquad (1.35)$$

The design cross-sectional force R_d can be obtained by calculating the resistance R (R is a function of f_d) of the member cross section using the design strength f_d and dividing by the member factor γ_b.

$$R_d = R(f_d)/\gamma_b \qquad (1.36)$$

TCVN 11820 Part 11: 2025, Equation

The design cross-section capacity M_{ud} for the bending moment of reinforced concrete can be calculated using Equation (1.37). (12)

$$M_{ud}=A_s \cdot f_{yd} \cdot d\left(1-\frac{p_w \cdot f_{yd}}{1.7f'_{cd}}\right)/\gamma_b \tag{1.37}$$

TCVN 11820 Part 11: 2025, Equation (16)

Where:

A_s	:	area of tension reinforcement (mm²)
p_w	:	reinforcing bar ratio ($=A_s/(b_w \cdot d)$)
f'_{cd}	:	design compressive strength of concrete $f'_{cd}=f'_{ck}/\gamma_c$
f'_{yd}	:	design yield strength of tensile reinforcement $f_{yd}=f_{yk}/\gamma_s$
d	:	effective height (mm)
γ_b	:	member factor (1.1)
γ_c	:	material factor for steel (1.3)
γ_s	:	material factor for concrete (1.0)

If the design value of the bending moment is denoted as M_d, it can be checked using Equation (1.38).

$$\frac{\gamma_i \cdot M_d}{M_{ud}} \leq 1.0 \tag{1.38}$$

ii) Verification of fatigue failure

Fatigue failure shall be verified by confirming that the value obtained by multiplying the ratio corresponding to the value dividing the design fatigue strength f_{rd} of the design variable stress σ_{rd} by the member factor γ_b by the structure factor γ_i is 1.0 or less:

$$\frac{\gamma_i \cdot \sigma_{rd}}{f_{rd}/\gamma_b} \leq 1.0 \tag{1.39}$$

TCVN 11820 Part 11: 2025 Equation (40)

The design fatigue strength f_{rd} shall be the value obtained by dividing the characteristic value of the material's fatigue strength f_{rk} by the material factor γ_m.

The fatigue failure may also be verified by confirming that the value obtained by multiplying the ratio of the design fluctuating cross-sectional force S_{rd} to the design fatigue resistance R_{rd} by the structure factor γ_i is 1.0 or less:

$$\frac{\gamma_i \cdot S_{rd}}{R_{rd}} \leq 1.0 \tag{1.40}$$

TCVN 11820 Part 11: 2025, Equation (41)

The design fluctuating cross-sectional force S_{rd} shall be the value obtained by multiplying the fluctuating cross-sectional force $S_r(F_{rd})$ obtained by using the design variable action F_{rd} by the structural analysis factor γ_a.

The design fatigue resistance R_{rd} shall be the value obtained by dividing the member's cross-sectional fatigue resistance $R_r(f_{rd})$ obtained by using the material's design fatigue strength f_{rd} by the member factor γ_b.

2) Verification of Serviceability of Structural Members

The verification of serviceability of the structural members shall determine the proper verification indices such as stress, cracks, displacement and deformation while considering the type of structural members, material properties and other factors, and shall compare their responses to their limit values.

The compressive stress and crack width of concrete can be an index for concrete structural members in general port facilities. However, when the response value of the crack width cannot be properly calculated, serviceability may be verified using the stress of a reinforcing bar. When other special functions are required, it is desirable to verify by setting an adequate index referring to the relevant guidelines.

i) Verification of concrete compressive stress in permanent state

Verification of the compressive stress of concrete in a permanent state can be performed using Equation (1.41).

$$\sigma'_c \leq 0.4 f'_{ck} \tag{1.41}$$

TCVN 11820 Part 11: 2025, Equation (42)

Where:

- σ'_c : compressive stress generated in concrete by a permanent action (N/mm²)
- f'_{ck} : characteristic value of compressive strength of concrete (N/mm²)

ii) Verification of crack width

When verifying with the crack width, confirm that the value, which is obtained by multiplying the ratio of the design response value w_d of the crack width generated in the structural member to the design limit value of the crack width w_a by the structure factor γ_i, is 1.0 or less.

$$\gamma_i w_d / w_a \leq 1.0 \tag{1.42}$$

TCVN 11820 Part 11: 2025, Equation (43), (45)

The design response value of crack width can be calculated using Equation (1.43).

$$w = 1.1 \times k_1 \times k_2 \times k_3 \times \{4 \times c + 0.7 \times (c_s - \varphi)\} \times (\sigma_{se}/E_s + \varepsilon'_{csd}) \tag{1.43}$$

Where:

- w : design response value of the crack width (mm)
- k_1 : coefficient expressing the influence of the surface profile of reinforcing bars on crack width (when deformed bars = 1.0)
- k_2 : coefficient expressing the influence of concrete quality on crack width, $k_2 = 15/(f_c' + 20) + 0.7$
- f_c' : compressive strength of concrete (N/mm²). It can normally be the design value of the compressive strength f'_{cd}
- k_3 : coefficient expressing the influence of the number of layers on the tensile bars, $k_3 = 5(n + 2)/(7n + 8)$
- n : number of layers of tension bars
- c : concrete cover (mm)
- c_s : distance between the centers of reinforcing bars (mm)
- φ : diameter of the tension reinforcing bar, nominal diameter of the smallest reinforcing bar (mm)
- E_s : Young's modulus of reinforcing bars (200 kN/mm²)
- ε'_{csd} : value considering the increase in crack width due to concrete shrinkage and creep , on the order of 0.00010
- σ_{se} : stress increment of the reinforcing bars near the surface

The increment of reinforcing bar stress se can be obtained using Equation (1.44) assuming the cross section is in the elastic range.

$$\sigma_{se} = M_d / (A_s \cdot j \cdot d) \tag{1.44}$$

TCVN 11820 Part 11: 2025, Equation (49)

Where:

M_d : design value of the bending moment (N·mm)
A_s : cross-sectional area of reinforcing bars (mm^2)
j : 1-k/3
k : neutral axis ratio (=$\sqrt{2np_w+(np_w)^2}$ - np_w)
n : Young's modulus ratio (=E_s/E_c)
p_w : reinforcing bar ratio (=$A_s/(b_w \cdot d)$)
d : effective height (mm)
b_w width of the member (mm)
A_s cross-sectional area of the reinforcing bars (mm^2)

The limit value of crack width w_a is generally set to the values shown in Table 1.9; however, this table is applicable only when the cover is 100 mm or less. For reinforced concrete members in marine environments, the cover should generally be greater than the values shown in Table 1.10.

Table 1.9- Limit Values of Crack Width w_a

TCVN 11820 Part 11: 2025, Table 3

Environmental classification	Crack width limit value (mm)
Particularly severe corrosion environment	0.0035c
Corrosion environment	0.0040c
Ordinary environment	0.0050c
	c is concrete cover

Source: TCVN 11820-11-2025

Table 1.10- Standard Values for Cover

TCVN 11820 Part 11: 2025, Table 5

Environment classification	Minimum cover thickness, c (mm)	Remarks
Particularly severe corrosion environment	70	Parts in direct contact with seawater Parts washed with seawater Parts exposed to severe sea breezes
Ordinary environment	50	Parts other than the above

Source: TCVN 11820-11-2025

iii) Verification of shear stress

Design shear compressive failure capacity can be calculated using Equation (1.45).

$$V_{dd}=\beta_d \cdot \beta_p \cdot \beta_a \cdot f_{dd} \cdot b_w \cdot d / \gamma_b \tag{1.45}$$

TCVN 11820 Part 11: 2025, Equation (29)

Where:

V_{dd} : design shear compressive failure capacity (N)
f_{dd} : $0.19\sqrt{f'_{cd}}$ (N/mm^2)
β_d : $\sqrt[4]{1000/d}$, set to 1.5 when $\beta_d > 1.5$
β_p : $(1+\sqrt{100p_v})/2$, set to 1.5 when $\beta_p > 1.5$
β_a : $5 / (1+(a/d)^2)$

b_w : width of web (mm)

d : loading point in the case of simple beams; effective depth (mm) at the support of cantilever beams

a : distance from the support frontal surface to the loading point (mm)

p_v : $A_s/(b_w \cdot d)$

A_s : cross-sectional area of reinforcing bars at tension side (mm^2)

f'_{cd} : design compression strength of concrete (N/mm^2)

γ_b : may generally be set to 1.3

3) Partial Factors

The partial factors listed in Table 1.11 can be used for the verification of structural members. This table presents standard values for the partial factors; other methods may be used when appropriate for determining the partial factors.

TCVN 11820 Part 6: 2023, Bang B.1

TCVN 11820 Part 11: 2025, Bang 1

Table 1.11- List of Partial Factors

Partial factor		Cross-sectional failure	Fatigue failure	Other
Material factor γ_m	Concrete	1.3	1.3	1.0
	Reinforcing bars and PC steel members	1.0	1.05	1.0
	Other steel members	1.05	1.05	1.0
Load factor γ_f	Permanent actions	1.0–1.1 (0.9–1.0)	1.0	1.0
	Variable actions			
	Wave force	1.2	1.0	1.0
	Actions other than wave force	1.0–1.2 (0.8–1.0)	1.0	1.0
	Accidental actions	1.0	-	-
	Actions during construction	1.0	-	-
Structural analysis factor γ_a		1.0	1.0	1.0
Member factor γ_b		1.1–1.3	1.0–1.3	1.0
Structure factor γ_i		1.0–1.2	1.0–1.1	1.0

Note 1: The figures in parentheses in the table are applied when a smaller action results in a large risk.

Note 2: The values below may be used for the member factor when examining cross-sectional failure:

- When calculating bending and axial force: 1.1
- When calculating the maximum value of axial compressive force: 1.3
- When calculating shear capacity carried by concrete: 1.3
- When calculating shear capacitycarried by shear reinforcing bars: 1.1

Note 3: Since variations in the fatigue damage accumulated so far in the existing structural members need to be considered in designs for improvement, the member factor is set to an adequate value between 1.0 and 1.3 when examining the fatigue failure.

Note 4: When examining cross-sectional failure, the following values may be used as the structure factor:

		Permanent situation	Variable situation	Accidental situation
Superstructure of piled piers	Slab Beams	1.2 1.1	1.2 1.1	1.0 1.0
Breakwaters		1.0	1.1	1.0
Quay walls (caissons, etc.)		1.0	1.1 (only during earthquakes: 1.0)	1.0
Other (sheet pile superstructures, etc.)		1.0	1.0	1.0

Source: TCVN 11820-6-2023, 11620-11-2025

4) Actions

i) Combination of actions and load factors

The combinations of actions to be considered in performance verification and the standard values of the load factors to be used for multiplying the characteristic values of actions are shown in Table 1.12. Here, the values used for the bottom slab can also be used for footings. The value in the top row in each cell of each table is the load factor to be used in examination of safety (against cross-sectional failure); the value shown in square brackets in the middle row is the load factor to be used in cases where the smaller the action, the larger the design load. These values were determined in consideration of the relationship with external stability and other factors based on reliability analysis. The value shown in parentheses in the bottom row of each cell is the load factor to be used in examination of serviceability. For accidental situations, a load factor of 1.0 may be used.

If the leveling accuracy of a rubble mound is alleviated, a reaction greater than that in case of the normal leveling accuracy of ±5 cm will act on the caisson bottom slab, and in this case, the values shown in Table 1.12 cannot be used.

TCVN 11820 Part 6: 2023, Bang B.3

Table 1.12- Combinations of Actions and Load Factors

Situation	Design situation	Self-weight	Hydrostatic pressure	Internal earth pressure	Bottom slab reaction	Internal water pressure	Uplift	Variable component of bottom slab reaction	Variable component of internal water pressure	Wave force	Dynamic water pressure	Hydrostatic head difference between chambers	Remarks
In service	Permanent situation associated with self-weight	0.9 (1.0)	1.1 (1.0)		1.1 (1.0)								Bottom slab
In service	Permanent situation associated with internal earth pressure			1.1 (1.0)		1.1 (1.0)							Outer wall
In service	Variable situation associated with waves	1.1 [0.9] (1.0)	1.1 [0.9] (1.0)		1.1 [0.9] (1.0)		1.2 [0.8] (1.0)	1.2 [0.8] (1.0)					Bottom slab
In service	Variable situation associated with waves			0.9 (1.0)						1.2 (1.0)			Outer wall
In service	Variable situation associated with waves			1.1 (1.0)		1.1 (1.0)			1.2 (1.0)				Outer wall
In service	Variable situation associated with earthquake ground motion			1.0 (−)		1.0 (−)					1.0 (−)		Outer wall
During construction	Variable situation associated with water pressure while afloat	0.9 (0.5)	1.1 (0.5)										Bottom slab
During construction	Variable situation associated with water pressure while afloat		1.1 (0.5)										Outer wall
During construction	Variable situation associated with water pressure during installation											1.1 (0.5)	Partition wall

Source: TCVN 11820-6-2023

ii) Load factors and combinations of actions of bottom slab

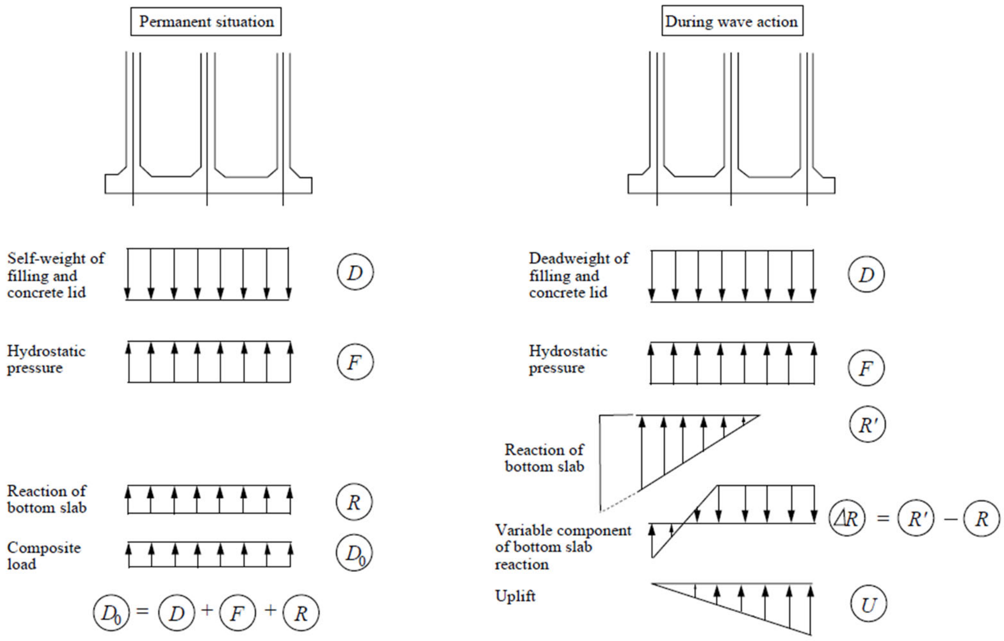
Actions to be considered in verification of the bottom slab of a breakwater caisson during construction can be determined by multiplying the characteristic values of the actions by the load factors shown in Table 1.12. In verification of a caisson in service, the composite load under calm conditions (D_0), the variable component of bottom slab reaction (ΔR) and the uplift (U) shown in Figure 1.10 may be determined using the equations shown in Table 1.14 in accordance with the classification of actions shown in Table 1.13.

Table 1.13- Classifications of Actions During Wave Action (Breakwater)

Classification of action	Action
Permanent Action	Composite load under calm conditions D_0
Variable Action	Variable component of bottom slab reaction (ΔR), Uplift (U)

Source: TCVN 11820-6-2023

TCVN 11820 Part 6: 2023, Bang B.7



Source: TCVN 11820-6-2023

Figure 1.13- Actions on the Bottom Slab (Breakwater)

TCVN 11820 Part 6: 2023, Hinh B.7

Table 1.14- Combinations of Actions and Load Factors of the Bottom Slab

(a) Safety (Section Failure)

Design situation	Direction of ΔR and W				Combination of actions
Permanent situation	—				$0.9D_0+1.1F+1.1R$
Variable situation associated with water pressure while afloat during construction	—				$0.9D_0+1.1F$
Variable situation associated with waves during action of wave crest	ΔR	↑	W	↑	$1.1D_0+1.2\ \Delta R+1.2U$
	ΔR	↓	W	↑	$1.1D_0+0.8\ \Delta R+1.2U$
			W	↓	$0.9D_0+1.2\ \Delta R+0.8U$
Variable situation associated with waves during action of wave trough	ΔR	↑	W	↑	$1.1D_0+1.2\ \Delta R+0.8U$
			W	↓	$0.9D_0+0.8\ \Delta R+1.2U$
	ΔR	↓	W	↑	$1.1D_0+0.8\ \Delta R+0.8U$
			W	↓	$0.9D_0+1.2\ \Delta R+1.2U$

TCVN 11820 Part 6: 2023, Bang B.8

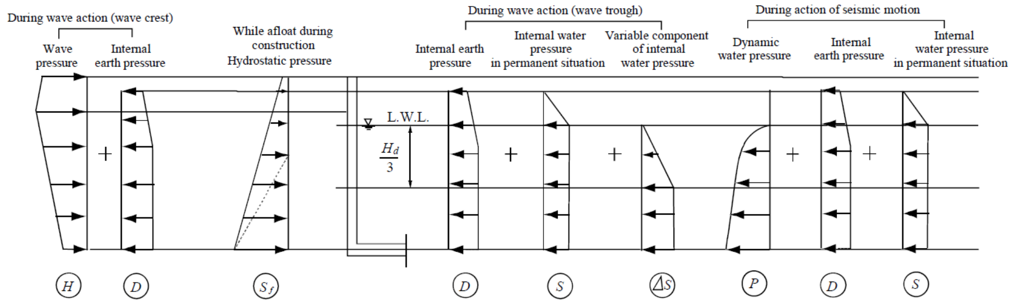
(b) Serviceability

Design situation	Combination of actions
Permanent situation	$1.0D_0+1.0F+1.0R$
Variable situation associated with waves	$1.0D_0+1.0\ \Delta R+1.0U$

Source: TCVN 11820-6-2023

iii) Load factors and combinations of actions of front wall

Actions to be considered in performance verification of the front wall of breakwater caissons are shown in Figure 1.14 and Table 1.15.



*In this figure, H_d stands for design wave height. In verification of the safety (against cross-sectional failure), H_d=H_{max} may be assumed.

Source: TCVN 11820-6-2023

TCVN 11820 Part 6: 2023, Hinh B.4

Figure 1.14- Actions on the Front Wall (Breakwater)

Table 1.15- Combinations of Actions and Load Factors of the Front Wall

TCVN 11820 Part 6: 2023, Bang B.4

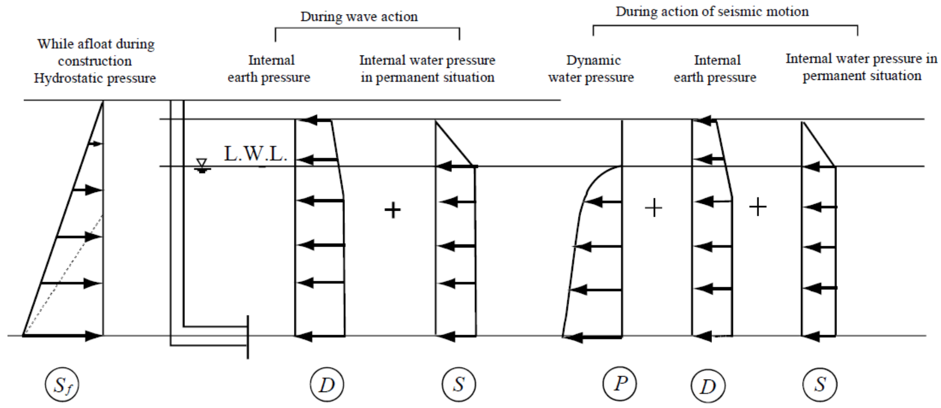
Design situation	Direction of action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with waves during action of wave crest	From outside of caisson	$1.2H-0.9D$	$1.0H-1.0D$
Variable situation associated with water pressure while afloat during construction		$1.1S_f$	$0.5S_f$
Variable situation associated with waves during action of wave trough	From inside of caisson	$1.1D+1.1S+1.2\Delta S$	$1.0D+1.0S+1.0\Delta S$
Variable situation associated with earthquake ground motion		$1.0D+1.0S+1.0P$	Not examined

* For the symbols in the table, see **Fig. 1.8**.

Source: TCVN 11820-6-2023

iv) Load factors and combinations of actions of rear wall

Actions to be considered in performance verification of the rear wall of breakwater caissons are shown in Figure 1.15 and Table 1.16.



TCVN 11820 Part 6: 2023, Hinh B.5

Source: TCVN 11820-6-2023

Figure 1.15- Actions on the Rear Wall (Breakwater)

Table 1.16- Combinations of Actions and Load Factors of the Rear Wall

TCVN 11820 Part 6: 2023, Bang B.5

Design situation	Direction of action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with water pressure while afloat during construction	From outside of caisson	$1.1S_f$	$0.5S_f$
Permanent situation associated with internal earth pressure	From inside of caisson	$1.1D+1.1S$	$1.0D+1.0S$
Variable situation associated with earthquake ground motion		$1.0D+1.0S+1.0P$	Not examined

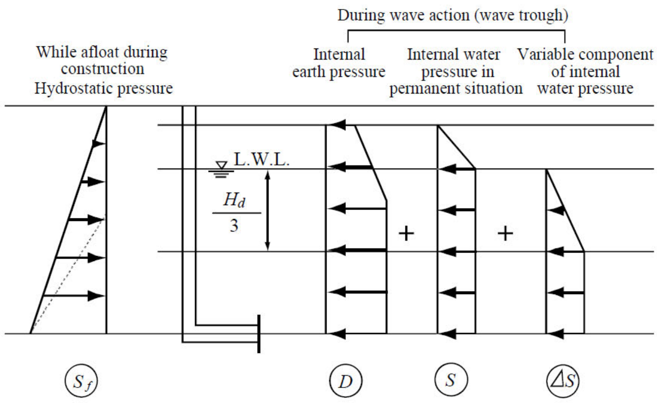
* For the symbols in the table, see **Fig. 1.9**.

Source: TCVN 11820-6-2023

v) Load factors and combinations of actions of side wall

Actions to be considered in performance verification of the side wall of breakwater caissons are shown in Figure 1.16 and Table 1.17.

TCVN 11820 Part 6: 2023, Hinh B.6



*In this figure, H_d stands for design wave height.

Source: TCVN 11820-6-2023

Figure 1.16- Actions on the Side Wall (Breakwater)

Table 1.17- Combinations of Actions and Load Factors of the Side Wall

TCVN 11820 Part 6: 2023, Bang B.6

Design situation	Direction of action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with water pressure while afloat during construction	From outside of caisson	$1.1S_f$	$0.5S_f$
Variable situation associated with waves during action of wave trough	From inside of caisson	$1.1D+1.1S+1.2\ \Delta S$	$1.0D+1.0S+1.0\ \Delta S$

Source: TCVN 11820-6-2023

vi) Load factors and combinations of actions of partition wall

Actions to be considered in performance verification of the partition wall of breakwater caissons are shown in Table 1.18.

Table 1.18- Combinations of Actions and Load Factors of the Partition Wall

TCVN 11820 Part 6: 2023, Bang B.9

Design State	Direction of Action	Safety (Sectional Failure)	Serviceability
Water Pressure during Installation (During construction)	Direction due to difference of water level between chambers	$1.1S_f$	$0.5S_f$
Internal Earth Pressure (Permanent State)	Direction due to pulling out from side wall partition	Maximum load designed for the external wall	Not consider
Self-weight, Wave actions, Earthquake	Direction due to pulling out from bottom plate partition	Maximum load designed for the bottom plate	Not consider

5) Wave Forces

When calculating the wave forces used for the performance verifications of the members, the design waves for each condition are as follows.

Note that for the serviceability verifications and safety verifications (including fatigue failure), the wave direction can be considered to be the most hazardous direction for convenience.

i) Wave conditions for safety verification

✓ Verification of Cross-section Failure

The same waves used for the stability verifications of the breakwater

✓ Verification of Fatigue Failure

The wave height and period are set appropriately according to the frequency of occurrence during the design service life.

ii) Wave conditions for serviceability verification

It is assumed that waves higher than the reference wave will occur approximately 10,000 times during design service life.

6) External Forces during Launching and Floating

For launching and floating in a dry dock, floating dock, or regular slipway (both slideways and trolleys), the external force calculations during launching and floating consider the hydrostatic water pressure based on the draft, with a safety margin. If there is a possibility of higher static water pressure occurring temporarily during launching, a separate analysis is performed.

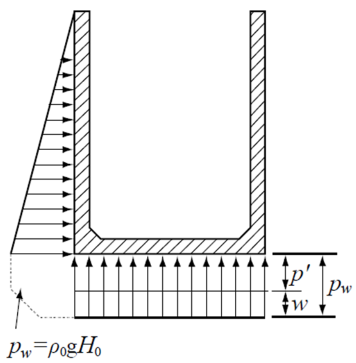
i) Bottom slab

For forces acting on the bottom slab, the value used is the static water pressure at the bottom slab minus the weight of the bottom slab itself (see Figure 1.17).

$$p' = p_w - w = \rho_0 g \cdot H_0 - w \qquad (1.46)$$

Where:

p'	:	force acting on the bottom slab (kN/m²)
p_w	:	hydrostatic pressure acting on bottom slab with an allowance of approximately 1.0m added to the design draft (kN/m²)
w	:	self-weight of the bottom slab (including the weight of any counter ballast materials such as backfill sand, not accounting for buoyancy) (kN/m²)
$\rho_0 g$	:	unit weight of seawater (kN/m³)
H_0	:	water depth with an allowance of approximately 1.0m added to the design draft (m)



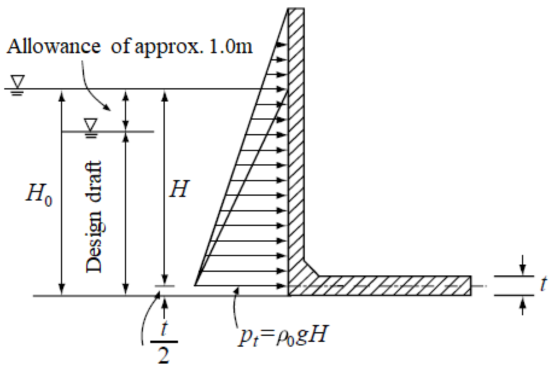
TCVN 11820 Part 6: 2023, Hinh B.8

Source: TCVN 11820-6-2023

Figure 1.17- Actions on Bottom Slab

ii) Outer wall

The water pressure acting on the outer wall can be considered as a triangular load, with the height equal to the hydrostatic water pressure at the centerline of the bottom slab and the base equal to the distance from the top of the structure to the bottom as shown in Figure 1.18. When calculating for the outer wall, it is common to use a calculation table for a beam fixed on three sides and free on one side. In this case, the partial load can be conveniently treated as a triangular load.



Source: OCDI 2020

Figure 1.18- Water Pressure Acting on Outer Walls

OCDI 2020 Part III Chapter 2 Fig. 2.2.10

Where:

$\rho_0 g$	:	unit weight of seawater (kN/m³)
H	:	depth to be considered in calculation of hydrostatic pressure (m) $H = H_0 - t/2$
H_0	:	water depth with an allowance of approximately 1.0m added to the design draft (m)
t	:	thickness of bottom slab (m)

iii) Partition walls

Generally, if the thickness of the partition walls is 20 cm or more, they are sufficiently resistant to bearing loads as columns, and therefore, detailed examination can be omitted.

iv) Other considerations

In cases where the final slope of the slipway is steep during launching, the caisson may be fully submerged, which may necessitate the installation of temporary covers. When using a crane to lift and launch the caisson, the forces acting on the outer wall of the caisson will vary depending on the presence or absence of lifting gear. Therefore, it is necessary to evaluate the loads based on the specific conditions.

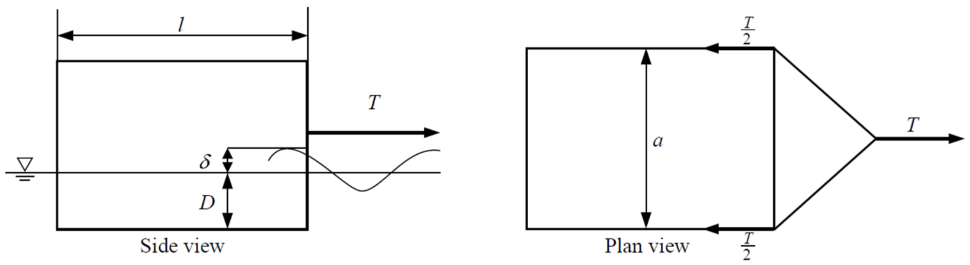
7) External Forces during Towing

i) Wave forces during towing

The static water pressure, dynamic water pressure, and wave forces during towing can be omitted from the analysis.

ii) Towing tension

The towing tension on the caisson during towing can be calculated using Equation (1.47). The schematic towing model is shown in Figure 1.19.



Source: TCVN 11820-6-2023

Figure 1.19- Tension Force during Towing

TCVN 11820 Part 6: 2023, Hinh B.9

$$T=\frac{\rho_0 C_D V^2 A}{2} \tag{1.47}$$

TCVN 11820 Part 6: 2023 Equation (B.11)

Where:

T	:	design value of tensile force during towing (kN); this value may be calculated by assuming that the partial factor to be used for multiplying the action term is 1.0.
C_D	:	drag coefficient
V	:	towing speed (m/s)
A	:	wet surface area on caisson front side (m^2), $A=a\cdot(D+\delta)$
a	:	width of caisson (m)
D	:	draft (m)
δ	:	water level on front side (m)
ρ_0	:	density of sea water (t/m^3)
l	:	length of caisson (m)

Since the caisson has no superstructure like the ones of ships, and towing will not take place in a strong wind, it is enough to consider only the fluid resistance by taking no account of the wind resistance.

Though the drag coefficient varies depending on the shape of the surface perpendicular to the current, the drag coefficient for a rectangular board is used. Towing speed is generally 2 to 3 knots.

iii) Water pressure during towing

In general, the caisson experiences pressure resistance and wave-making resistance. However, since towing is not conducted in high wave conditions, allowing for a draft margin of 1.0m during launching and floating is sufficient. Therefore, pressure and wave-making resistance are not considered.

8) External Forces during Installation

i) Bottom Slab and Outer Wall

The external forces acting during installation are smaller compared to the forces acting during floating and after installation. Therefore, the analysis of the bottom slab and outer wall during installation can be omitted.

ii) Partition Walls

For partition walls, considering construction conditions, a head difference of 1.0m between chambers is considered as the load. The sketch of head difference is shown in Figure 1.20.

When installing the caisson, methods such as using a siphon or pump to fill with water, or using a valve to allow water in, are possible. However, if a head difference of more than 1.0m occurs, partition walls with a thickness of around 20cm may not be sufficient with only single-reinforced steel. Therefore, it is advisable to carefully manage the construction by frequently moving hoses, keeping the difference of hydrostatic head within 1.0m.

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Part IV
Chapter 1
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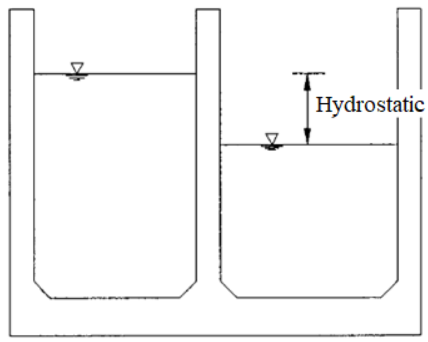


Source: OCDI 2002

Figure 1.20- Head Difference Between Chambers

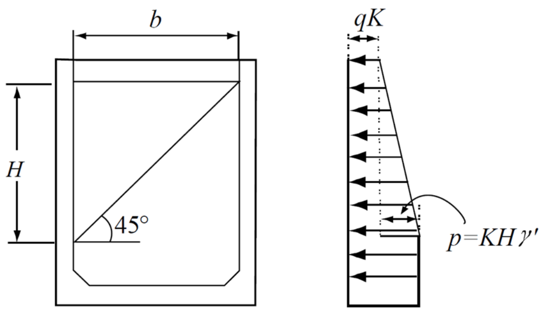
9) External Forces after Completion

i) Bottom Slab

- ✓ For the fixed section surrounded by the outer wall and partition walls, the forces considered are bottom slab reaction, hydrostatic pressure, uplift pressure, the weight of fill material, cover concrete weight, bottom slab weight, and applied loads.
- ✓ The total load distribution often takes an irregular shape, so for design purposes, this distribution can be approximated as equivalent uniform and triangular load distributions.
- ✓ Bottom Slab Reaction: The bottom slab reaction force acting on the retaining structure or wall should be referenced from the stability calculations under each design condition.
- ✓ Hydrostatic Pressure: This is the hydrostatic pressure acting on the caisson's bottom slab at the design water level.
- ✓ Weight of Fill Material: The unit weight of fill material is typically determined through testing of the materials to be used. For standard sand, the typical weight ranges from 19 to 20kN/m^3.
- ✓ Cover Concrete Weight: The weight of cover concrete is considered as its dry weight in the air, where buoyancy is not a factor. The characteristic unit weight values are 22.6kN/m^3 for plain concrete and 24.0kN/m^3 for reinforced concrete.
- ✓ Bottom Slab Weight: The weight of the bottom slab is treated as the dry weight in air, where buoyancy is not considered. The characteristic unit weight for this calculation is 24.0kN/m^3.
- ✓ Applied Loads: Applied loads acting on the bottom slab include the weight of the soil cover on top of the caisson and any imposed loads. However, if cast-in-place concrete exists on top of the caisson and it is assumed that the applied load does not affect the inside of the caisson, then applied loads may be disregarded.

ii) Outer Wall

- ✓ The external forces acting on the outer wall include earth pressure from the fill and internal water pressure.
- ✓ Internal Earth Pressure: It is expected that the earth pressure will decrease at the lower part due to the arching effect of the sand. However, for convenience, the traditional method is used. That is, the pressure increases up to a depth equal to the inner width (b) of the wall, and beyond that depth, it remains constant as Figure 1.21. The coefficient of earth pressure K typically used for standard sand is the static earth pressure coefficient, which is 0.6.



Source: TCVN 11820-6-2023

TCVN 11820 Part 6: 2023, Hinh B.10

Figure 1.21- Internal Earth Pressure Distribution

Where:

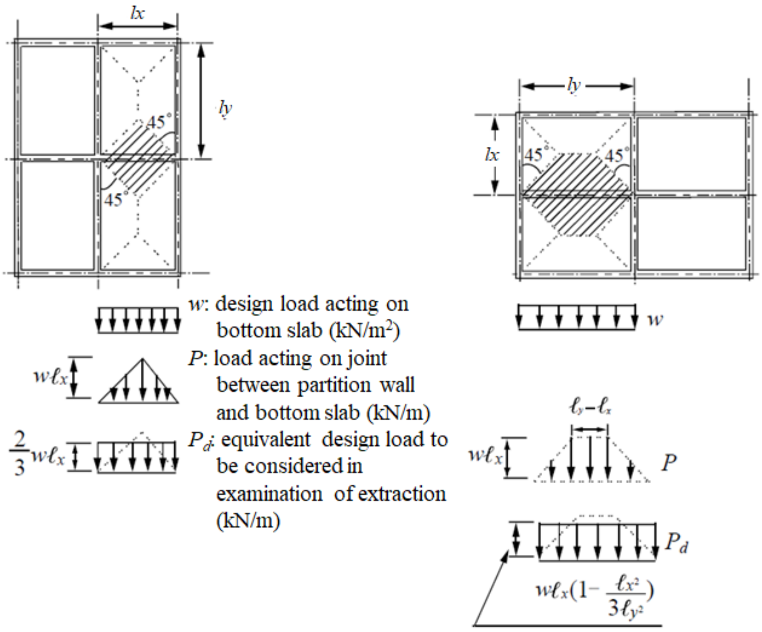
q	:	load transmitted from the top to the backfill (kN/m²)
γ'	:	submerged unit weight of the backfill material (in kN/m³). Typically, the characteristic value of γ' is assumed to be 10.0 kN/m^3
K	:	coefficient of earth pressure for the backfill ($K = 0.6$)
b	:	internal width of the wall (m)

If there is a solid cast-in-place concrete structure on top of the caisson and the applied load does not affect the interior of the caisson, the load is not considered.

- ✓ Internal Water Pressure: The internal water pressure is determined by the head difference between the water level inside the caisson and the Low Water Level (L.W.L).

iii) Partition Walls

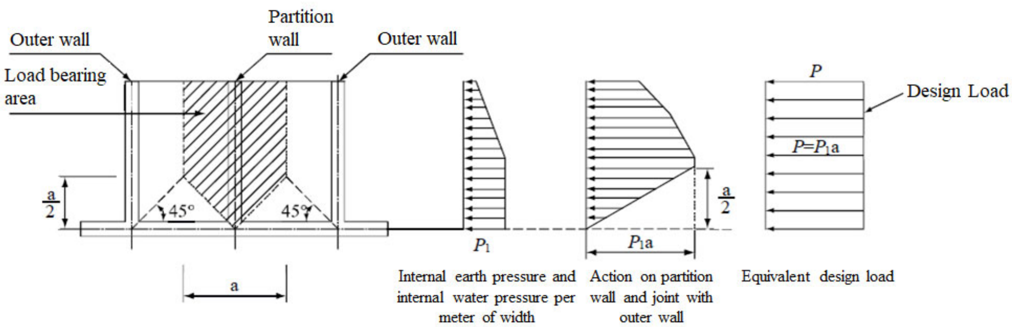
- ✓ To prevent the base slab from separating from the partition walls, the following forces must be considered: the weight of the backfill material acting on the base slab, the applied load, the weight of the base slab itself, the weight of the cover concrete, the reaction force of the base slab, uplift pressure, and static water pressure. These forces are assumed to act on the joint between the partition wall and the base slab as shown in Figure 1.22.
- ✓ For the external walls, in order to prevent them from detaching from the partition walls, the earth pressure of the backfill and the internal water pressure acting on the external walls shall be considered, assuming these forces act on the joint between the external walls and the partition walls as Figure 1.23.



Source: TCVN 11820-6-2023

Figure 1.22- Actions used in Examination of Extrusion Base Slab from Partition Wall

TCVN 11820 Part 6: 2023, Hinh B.15



Source: TCVN 11820-6-2023

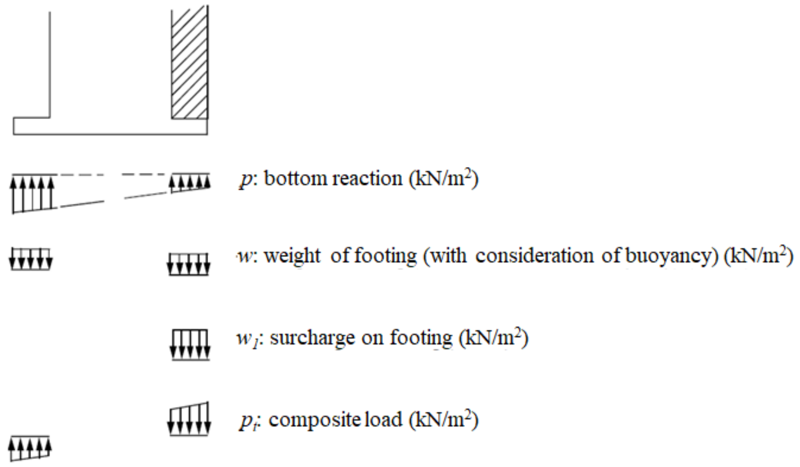
Figure 1.23- Actions Used in Examination of Extrusion of Outer Wall from Partition Wall

TCVN 11820 Part 6: 2023, Hinh B.14

iv) Footing

- ✓ The design loads acting on the footing are based on the load distribution shown in Figure 1.24.
- ✓ Bottom Slab Reaction: The bottom slab reaction force acting on the footing should reference the bottom slab reaction calculated from the stability analysis under each design condition.
- ✓ Footing Weight: The weight of the footing should use its submerged weight, taking buoyancy into account. The characteristic value for the unit weight of the footing in air can be assumed to be 24.0 kN/m³.
- ✓ Applied Loads: The applied loads acting on the footing include the weight of the soil cover on the landward side of the quay wall, as well as any live loads. Buoyancy acting below the design water level should also be considered.

TCVN 11820 Part 6: 2023, Hinh B.13



Source: TCVN 11820-6-2023

Figure 1.24- Actions on Footing

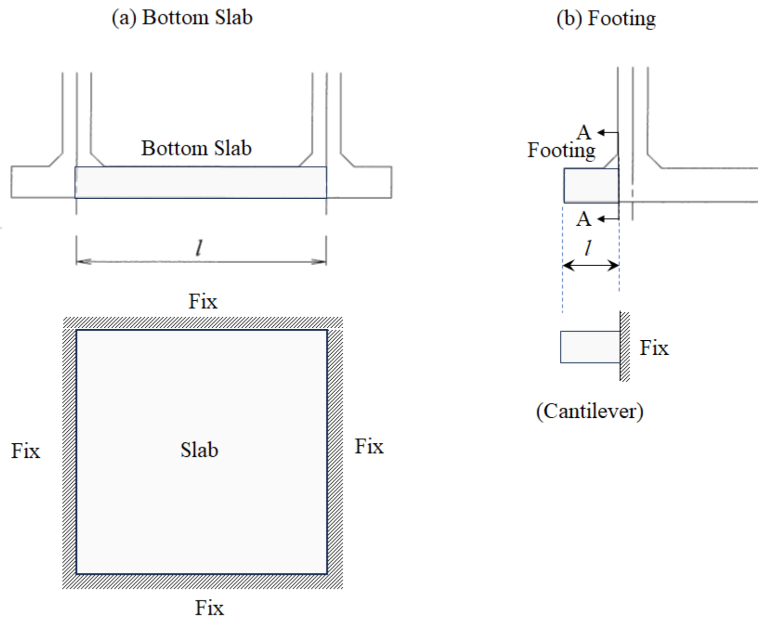
10) Sectional Forces

The sectional forces in members are typically calculated as three-side fixed plates and four-side fixed plates, using the moment calculation tables provided in the TCVN Part 11: 2025 and OCDI 2020. It is common to calculate the bending moments using these tables, but finite element method (FEM) analysis can also be employed.

i) Bottom slab and footing

- ✓ The portion surrounded by external walls and partition walls is treated as a four-side fixed plate model. Footing can be regarded as cantilever slabs.
- ✓ The span used for the calculation of the four-side fixed plate is the distance between the centers as shown in Figure 1.25.

TCVN 11820 Part 11: 2025, Fig. 19



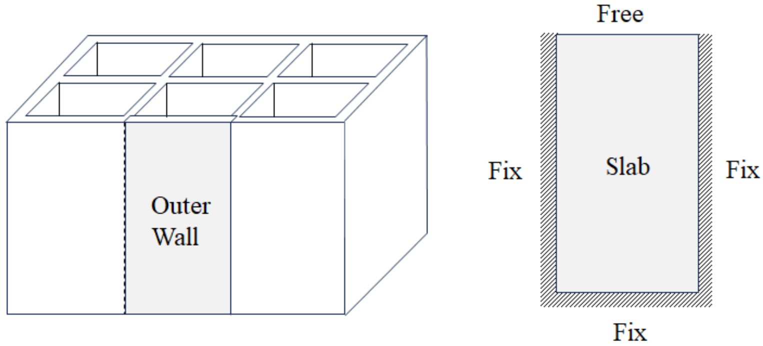
Source: TCVN 11820-11-2025

Figure 1.25- Span Used for Bottom Slab and Footing Design

ii) Outer Walls

- ✓ The design of the outer walls is calculated as a three-side fixed, one-side free plate as Figure 1.26. However, the span used for this calculation is the distance between the centers.

- ✓ For outer walls with an aspect ratio of 1:5 or greater, the values for a 1:5 plate can be used.
- ✓ Correction for Unbalanced Moments: If a significantly large, unbalanced moment occurs at locations considered to be fixed between the outer walls, the moment at the wall's end can be distributed according to the plate's stiffness ratio. Additionally, for the span moment, half of the distributed moment may be added as a correction.
- ✓ For internal supports and areas other than the first span, the effect of unbalanced moment distribution is minimal, so there is no need for specific distribution as shown in Figure 1.27.
- ✓ The distributed moment for the bending moment of the outer wall, shown in Figure 1.27, becomes as described in Equation (1.48) after distribution.



TCVN 11820 Part 11: 2025, Fig. 18

Source: TCVN 11820-11-2025

Figure 1.26- Modeling of Outer Wall

TCVN 11820 Part 11: 2025, Equation (62-67)

$$M'_{BA}=M_{BA}-(M_{BA}-M_{BC})\frac{K_a}{K_a+K_b}$$

$$M'_{BC}=M_{BC}+(M_{BA}-M_{BC})\frac{K_b}{K_a+K_b}$$

$$M'_a=M_a-\frac{1}{2}(M_{BA}-M_{BC})\frac{K_a}{K_a+K_b} \qquad (1.48)$$

$$M'_b=M_b+\frac{1}{2}(M_{BA}-M_{BC})\frac{K_b}{K_a+K_b}$$

$$M'_{AB}=M_{AB}$$
$$M'_{CB}=M_{CB}$$

Where:

M'_{AB}, M'_{BA}, M'_{BC}, M'_{CB}, M'_a, M'_b	:	bending moments after distribution of unbalanced moments
M_{AB}, M_{BA}, M_{BC}, M_{CB}, M_a, M_b	:	bending moments before distribution of unbalanced moments
K_a, K_b	:	relative stiffness of outer walls

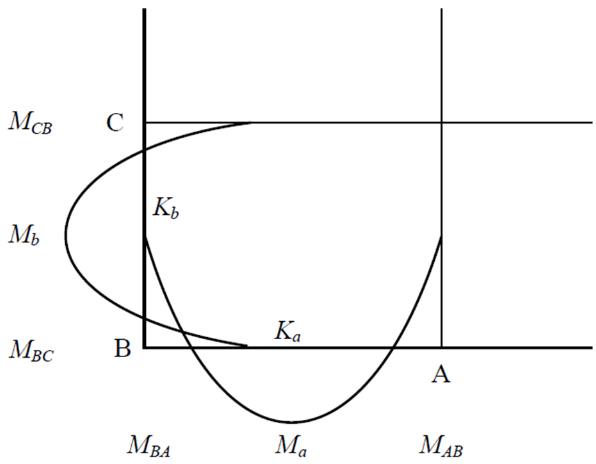
Note: The moments have both plus sign and minus sign.

$K_a = EI_a/l_a$; $K_b = EI_b/l_b$

If $EI_a = EI_b$ then: $K_a = 1/l_a$; $K_b = 1/l_b$

The unbalanced moment between the outer wall and the bottom slab can be used as is without redistribution.

TCVN 11820 Part 11: 2025, Fig. 17



Source: TCVN 11820-11-2025

Figure 1.27- Distribution of Unbalanced Moments

iii) Partition Walls

- ✓ Calculation During Installation: The walls should be calculated as a three-sided fixed one-sided free plate during installation. This implies that the wall is assumed to be fixed at three sides while being free on the one side.
- ✓ Span for Calculation: The span used for calculations should be the center spacing between the walls.
- ✓ Post-Installation Considerations: After installation, it is necessary to examine the potential for the wall to pull away from the outer wall and bottom slab. This is important for ensuring the integrity of the structure over time.
- ✓ Reinforcement Cover: The cover for the main reinforcement bars should generally be at least 5 cm. This is to ensure durability and prevent corrosion.

iv) Footings

- ✓ The footing section should be calculated as a cantilever plate.
- ✓ The section used for calculations related to bending and shear in the footing will be at the front face of the wall as the Figure 1.24 A-A section. However, for examining shear failure due to diagonal tension, the section can be taken at the root part of the front face of the wall. In that case, the calculation of the member height at the front face of the wall will consider the portion that is more gradual than the 1:3 slope of the cantilever section.
- ✓ When $l'/h' < 2$, the shear strength of the deep beam will be assessed. In this case, the section for verification will be at the front face of the wall as the Figure 1.24 A-A section. The reduction of loads indicated on the footing will not be considered. Furthermore, it is mentioned in the same document that "generally, it is sufficient to ensure safety (sectional failure)," but in this case, we will also conduct verifications regarding serviceability.
- ✓ Analyzing the stress at the A-A section when subjected to infill pressure and ground reaction is challenging. However, since the caisson body is surrounded by outer walls, partition walls, and bottom slabs, it behaves as a frame structure. Thus, we will consider the caisson body as a rigid body, assuming that the moment acting on the footing does not affect the main body.

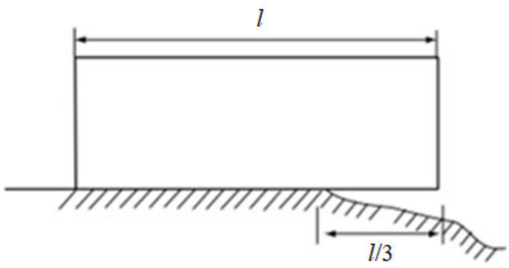
Therefore, since the support conditions for the footing are clear as a cantilever slab, the section used for calculations in the footing part can be taken as the front face of the outer wall.

- ✓ However, if the bottom slab reaction is large and the height of the footing is high, it is necessary to consider reinforcement for the main body.

11) Examination of Actions Due to Uneven Ground Support

In cases where uneven ground support occurs due to differential settlement after installation, it is common to examine the integrity of the caisson itself as a cantilever beam with a length or width of one-third of the caisson as Figure 1.28.

TCVN 11820 Part 6: 2023, Hình B.16



Source: TCVN 11820-6-2023

Figure 1.28- Examination of Actions due to Uneven Ground Support

For the safety verification of members against differential settlement, Equation (1.49) can be used. This equation considers the bending moment acting on the examination section as being resisted solely by the bending strength of the concrete:

$$\frac{\gamma_i \cdot M_d}{M_{ud}} \leq 1.0 \tag{1.49}$$

Where:

γ_i	:	structural coefficient (1.0)
M_d	:	design value of bending moment
M_{ud}	:	design moment capacity

The ultimate bending moment M_{ud} is determined by Equation (1.50):

$$M_{ud} = f_{bk} Z / \gamma_b \tag{1.50}$$

Where:

Z	:	section modulus of the caisson
γ_b	:	member coefficient (1.1)
f_{bk}	:	design bending strength of the concrete, calculated as $f_{bk} = 0.42 f'_{ck}{}^{2/3} / \gamma_c$
γ_c	:	material coefficient (1.3)

If the results of the verification using the above method indicate insufficient strength, it is possible to conduct an examination as a reinforced concrete section.

2. Design Example

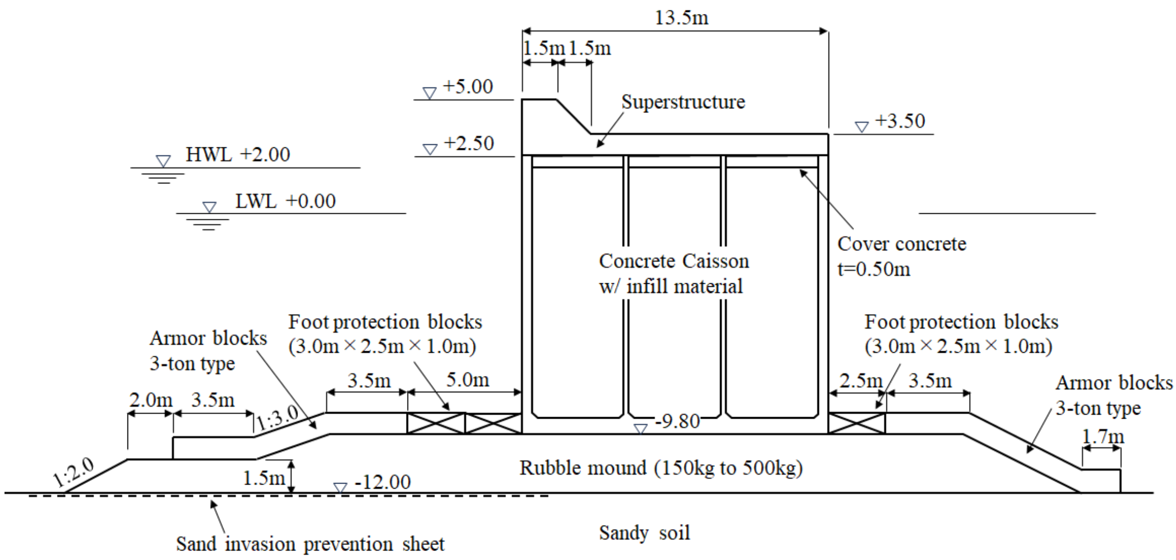


Figure 2.1- Typical Cross Section of Caisson Breakwater

2-1. Design Conditions

In the design examples, the same symbol is used for the partial factor γ and the unit weight γ. Please note that care must be taken to avoid confusion and ensure the correct usage of each.

(1) Tide Level

H.W.L. + 2.00 m
L.W.L. ±0.00 m

(2) Design Wave

	Safety (sectional failure)		Serviceability
Water Level	H.W.L.	L.W.L.	M.S.L
$H_{1/3}$ (m)	4.00	3.80	2.10
$H_D = H_{max}$ (m)	7.20	6.84	3.80
T (sec)	10.0	10.0	6.90

- Fatigue failure

$H_{1/3}$ (m)	$H_D = H_{max}$ (m)	T (sec)	frequency of occurrence (cycle)
0.280	0.5	4.170	366,050,026
1.830	1.5	6.060	7,697,893
1.390	2.5	6.420	767,537
1.940	3.5	6.760	96,475
2.500	4.5	6.980	12,379
3.060	5.5	7.430	1,466
3.610	6.5	7.920	30

(3) Ground conditions

Seabed -12.0 m (Seabed slope: i = 1/100)

Sandy Soil

$\varphi = 30°$

$\gamma_t = 18$ kN/m^3, $\gamma_{sat} = 20$ kN/m^3

$\gamma' = 10$ kN/m^3

-30.00

Sandy Soil

$\varphi = 35°$

$\gamma_t = 18$ kN/m^3, $\gamma_{sat} = 20$ kN/m^3

$\gamma' = 10$ kN/m^3

- Rubble mound

  $\varphi = 40°$

  $\gamma_{sat} = 20$ kN/m^3, $\gamma' = 10$ kN/m^3

(4) Friction Coefficient between Caisson and Rubble Mound

$f = 0.6$

(5) Seismic Coefficient for Verification

The regional seismic coefficient is assumed to be 0.08 at Level 1 earthquake ground motion. The soil condition coefficient is 1.2 for Type C ground, and the importance coefficient is 1.0 for a wharf structure.

Seismic coefficient (k_h) = regional seismic coefficient (k_{h1}) × soil condition coefficient (γ_s) × importance coefficient (γ_i) = 0.08×1.2×1.0 = 0.096

The external forces acting during earthquake motion are smaller compared to the forces acting during wave actions. Therefore, the calculation at earthquake ground motion is omitted in this design example.

(6) Unit Weight

Table 2.1- Unit Weight

Material	Weight per unit volume (kN/m^3)
Reinforced concrete	24.0
Plain concrete	22.6
Infill sand (saturated weight)	20.0
Seawater	10.1

Note: The value for the infill sand should be determined based on the result of the weight per unit volume test.

(7) Partial Factor

1) Variable State of Waves

i) Sliding of breakwater body

$\gamma_R = 0.83$ (resistance term)

$\gamma_S = 1.08$ (load term)

$m = 1.00$ (adjustment factor)

ii) Overturning of breakwater body

$\gamma_R = 0.95$ (resistance term)

γ_S= 1.14 (load term)
m = 1.00 (adjustment factor)

iii) Bearing capacity against eccentric inclined loads
γ_R= 1.00 (resistance term)
γ_S= 1.00 (load term)
m = 1.00 (adjustment factor)

2) Permanent State

i) Circular slip failure (for no cohesive ground)
γ_R= 0.83 (resistance term)
γ_S= 1.01 (load term)
m = 1.00 (adjustment factor)

2-2. Design Specifications

(1) Determination of the Top Level of Caisson

The top level of a caisson in the offshore area will be D.L.+ 2.50 m, as it is generally desirable to make it the mean monthly highest-water level (H.W.L.) + 2.00 m or more to facilitate superstructure.

(2) Thickness of Cover Concrete

The thickness of cover concrete shall be 0.5 m, considering cases where caissons are placed in severe wave conditions.

(3) Shape of Superstructure

At this port, the allowable wave overtopping rate was verified through hydraulic model experiments, and the breakwater crown height was set at +5.0 m, as it exceeds the calculated value of H.W.L. + $0.6H_{1/3} = 2.0 + 0.6 \times 4.0\text{m} = 4.4\text{m}$.

The superstructure concrete is constructed in two stages. In the first stage, concrete is placed up to +3.5 m. In the second stage, additional concrete is placed to reach the final crest level of +5.0 m. The superstructure is shaped in the form of a parapet to enhance the overall stability of the breakwater.

(4) Foot Protection Block

Two blocks shall be placed on the seaward side of the upright section, and one block on the harbor side. Each block shall be provided with perforations, achieving an aperture ratio of approximately 10% (i.e., the ratio of the total area of openings to the surface area of the block), in order to reduce uplift force effectively. The shape of the blocks shall be determined in accordance with Equation (2.1).

$$t = d_f \cdot \left(\frac{h'}{h}\right)^{-0.787} \cdot H_{1/3} \qquad (2.1)$$

Where:

t	:	required thickness of foot protection block (m)
$H_{1/3}$	:	significant wave height (m)
d_f	:	0.18 at the breakwater trunk and 0.21 at the breakwater head
h'	:	depth at the mound crown (excluding blocks) (m)
h	:	design depth (m) (applicable depth: h'/h = 0.4 to 1.0)

The required thickness t is checked at the LWL which is critical for the determination of thickness, as follows:

$$t = d_f(h'/h)^{-0.787} \times H_{1/3} = 0.18 \times (9.80/12.0)^{-0.787} \times 4.0 = 0.844 \text{ (m)}$$

From the above results, the dimensions of the foot protection block are as follows:
Dimension: L(m) × b(m) × t(m)= 3.0 × 2.5 × 1.0
Porous type, W = 15.64 (t/piece)

(5) Armor Block

The required weight of armor block can be estimated by Hudson formula based on stability numbers N_S. Since both γ_{Ns} and γ_H are 1.0, the characteristic value and the design value are the same value.

$$M_d = \frac{\rho_r H_d^3}{N_{Sd}^3 (S_r\text{-}1)^3} \qquad (2.2)$$

Where:

M	:	required mass of rubble stones or concrete blocks (t)
ρ_r	:	density of rubble stones or concrete blocks (2.30 t/m^3)
H	:	wave height used in stability calculation (m) $H_d = \gamma_H \times H_k = 1.0 \times 4.0 = 4.0$ (m)
N_S	:	stability number determined primarily by the shape, slope, damage rate of the armor, etc. of the armor units In this case, N_S^3 is assumed to be 112 according to the catalog specifications
S_r	:	specific gravity of rubble stones or concrete blocks relative to water ($\rho_r / \rho_o = 2.30/1.03 = 2.233$)
ρ_o	:	density of seawater 1.03 (t/m^3)

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

$$= \frac{2.3 \times 4.0^3}{112 \times (2.3/1.03\text{-}1)^3}$$

$$= 0.701 \text{ (tons)}$$

The minimum required weight of armor blocks is 1 ton, but a 3-ton block, which is two ranks higher, is adopted for conservative approach.

(7) Rubble Stone

The minimum required weight of rubble stone is 1/20 of the armor block weight.

M_d= 1/20 × 3,000 = 150 (kg) →150 to 500 (kg)

2-3. Assumptions of Caisson Model and Design Cross-sections

Assume the design cross-section of caisson as shown in Figure 2.2 and 2.3.

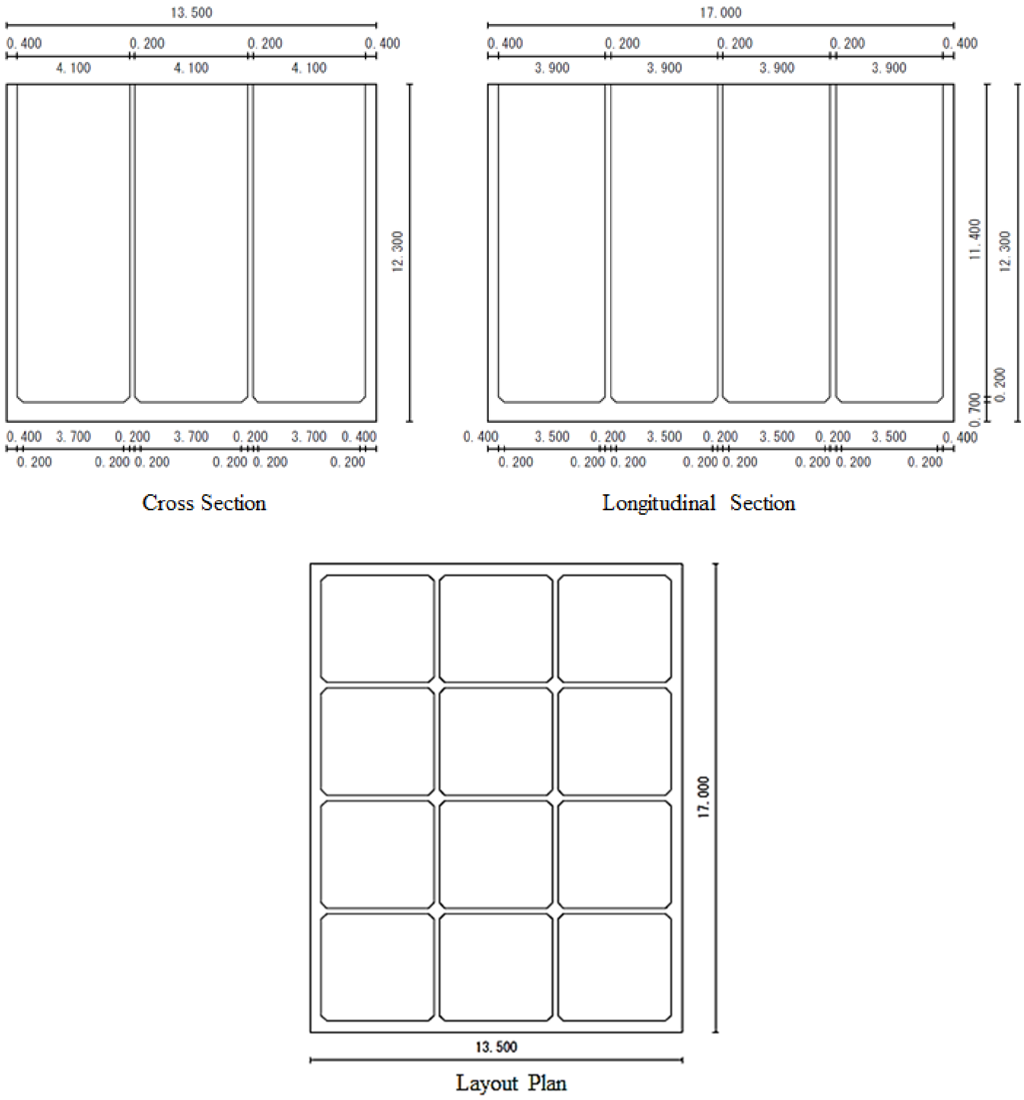


Figure 2.2- Assumed Caisson Model

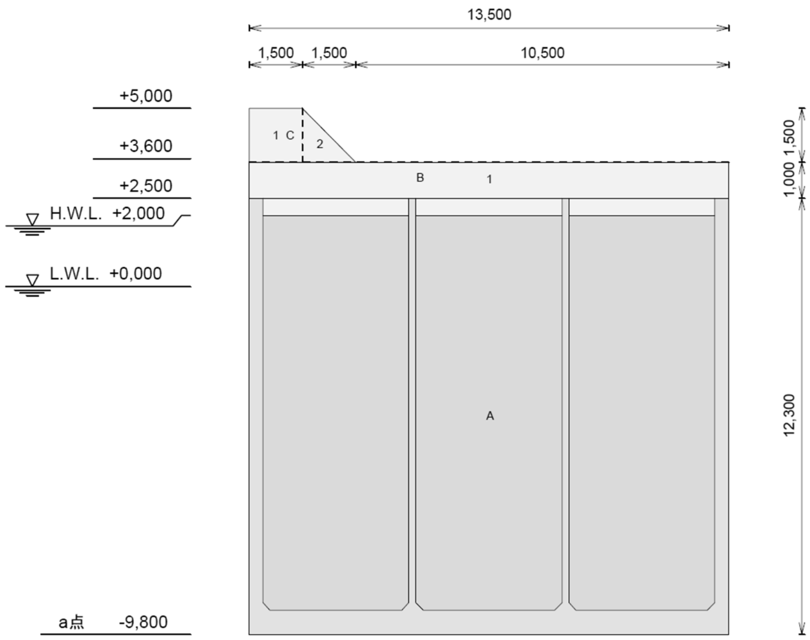


Figure 2.3- Assumed Design Cross-section

2-4. Characteristic Value of Design Load

(1) Self Weight and Moment

1) Superstructure

The weights and moments of the superstructure are presented in Table 2.2.

Table 2.2- Characteristic Values of Superstructure Weight and Moment

Name	Dimensions (m)	Nos.	V (m^3) W (kN)	x (m) y (m)	$W{\cdot}x$ (kN·m) $W{\cdot}y$ (kN·m)
Superstructure 1B	13.500 × 1.000 × 17.000	1	229.500 5,186.700	6.750 12.800	35,010.225 66,389.760
Superstructure 2	1.500 × 1.500 × 17.000 ×1/2	1	19.125 432.225	2.000 13.800	864.450 5,964.705
Superstructure 1C	1.500 × 1.500 × 17.000	1	38.250 864.450	0.750 14.050	648.338 12,145.523
Total			286.875 6,483.375	5.633 13.033	36,523.013 84,499.988
Weight per meter W(kN/m) (L=17.000m)			381.375		

Note: γ=22.6 (kN/m^3)

2) Caisson

The weights and moments of the caisson are presented in Table 2.3.

Table 2.3- Characteristic Values of Caisson Weight and Moment

Name	Dimensions (m)	Nos.	V (m^3) W (kN)	x (m) y (m)	$W{\cdot}x$ (kN·m) $W{\cdot}y$ (kN·m)
Bottom plate	13.500 × 17.000 × 0.700	1	160.650 3,855.600	6.750 0.350	26,025.300 1,349.460
Side Wall Longitudinal	0.400 × 17.000 × 11.600	2	157.760 3,786.240	6.750 6.500	25,557.120 24,610.560
Side Wall Transverse	12.700 × 0.400 × 11.600	2	117.856 2,828.544	6.750 6.500	19,092.672 18,385.536
Partition Longitudinal	0.200 × 16.200 × 11.600	2	75.168 1,804.032	6.750 6.500	12,177.216 11,726.208
Partition Transverse	12.300 × 0.200 × 11.600	3	85.608 2,054.592	6.750 6.500	13,868.496 13,354.848
Vertical haunch	0.200 × 0.200 × 11.600 ×1/2	48	11.136 267.264	6.750 6.500	1,804.032 1,737.216
Horizontal haunch Longitudinal	0.200 × 14.000 × 0.200 ×1/2	6	1.680 40.320	6.750 0.767	272.160 30.925
Horizontal haunch Transverse	11.100 × 0.200 × 0.200 ×1/2	8	1.776 42.624	6.750 0.767	287.712 32.693
Corner haunch	0.200 × 0.200 × 0.200 ×1/3	48	0.128 3.072	6.750 0.775	20.736 2.381
Total			611.762 14,682.288	6.750 4.851	99,105.444 71,229.827
Weight per meter W(kN/m) (L=17.000m)			863.664		

Note: γ=24.0 (kN/m^3)

3) Cover Concrete

The weights and moments of the cover concrete are presented in Table 2.4.

Table 2.4- Characteristic Values of Cover Weight and Moment

Name	Dimensions (m)	Nos.	V (m^3) W (kN)	x (m) y (m)	$W{\cdot}x$ (kN·m) $W{\cdot}y$ (kN·m)
Cover concrete	4.100 × 3.900 × 0.500	12	95.940 2,168.244	6.750 12.050	14,635.647 26,127.340
Vertical haunch	0.200 × 0.200 × 0.500 ×1/2	48	-0.480 -10.848	6.750 12.050	-73.224 -130.718
Total			95.460 2,157.396	6.750 12.050	14,562.423 25,996.622
Weight per meter W(kN/m) (L=17.000m)			126.906		

Note: γ=22.6 (kN/m^3)

4) Infill Material

The weights and moments of the infill material are presented in Table 2.5.

Table 2.5- Characteristic Values of Infill Weight and Moment

Name	Dimensions (m)	Nos.	V (m^3) W (kN)	x (m) y (m)	$W{\cdot}x$ (kN·m) $W{\cdot}y$ (kN·m)
Infill material	4.100 × 3.900 × 11.100	12	2,129.868 42,597.360	6.750 6.250	287,532.180 266,233.500
Vertical haunch	0.200 × 0.200 × 11.100 ×1/2	48	-10.656 -213.120	6.750 6.250	-1,438.560 -1,332.000
Horizontal haunch Transverse	3.700 × 0.200 × 0.200 ×1/2	24	-1.776 -35.520	6.750 0.767	-239.760 -27.244
Horizontal haunch Longitudinal	0.200 × 3.500 × 0.200 ×1/2	24	-1.680 -33.600	6.750 0.767	-226.800 -25.771
Corner haunch	0.200 × 0.200 × 0.200 ×1/3	48	-0.128 -2.560	6.750 0.775	-17.280 -1.984
Total			2,115.628 42,312.560	6.750 6.259	285,609.780 264,846.501
Weight per meter W(kN/m) (L=17.000m)			2,488.974		

Note: γ_{sat}=20.0 (kN/m^3)

(2) Characteristic Value of Design Load

1) Breakwater Body Weight and Moment

The characteristic values of the breakwater body weight and moment per 1.0 meter are shown in Table 2.6.

Table 2.6- Characteristic Values of Breakwater Body Weight and Moment

No	Name	W (kN/m)	x (m)	$W{\cdot}x$ (kN·m/m)
A	Caisson	863.664	6.750	5,829.732
A	Cover concrete	126.906	6.750	856.616
A	Infill material	2,488.974	6.750	16,800.575
B,C	Superstructure	381.375	7.867	3,000.125
Point a (-9.800m)		3,860.919		26,487.048

2) Buoyancy and Moment

The characteristic value of buoyancy per meter is shown below:

H.W.L.

Name	Dimensions (m)	Nos	P_B (kN/m)	x (m)	$P_B{\cdot}x$ (kN·m/m)
Caisson	13.500 × 11.800 × 10.100	1	1,608.930	6.750	10,860.278
Total			1,608.930	6.750	10,860.278

L.W.L.

Name	Dimensions (m)	Nos	P_B (kN/m)	x (m)	$P_B{\cdot}x$ (kN·m/m)
Caisson	13.500 × 9.800 × 10.100	1	1,336.230	6.750	9,019.553
Total			1,336.230	6.750	9,019.553

(3) Wave Force and Moment

Wave force can be calculated using Goda's formula. A calculation example at the time of H.W.L. and wave crest is shown below:

The design wave height H_D and the period in Goda's formula are the wave height and period of the highest wave, respectively. The design wave height is given by:

$$H_D = 1.8 \times H_{1/3} = 1.8 \times 4.0 = 7.20 \text{ (m)}$$

The period of the design wave is 10.0 s, the water depth h is 14.00 m, and the wavelength L is 106.140 m.

$h_b = 14.00 + 5 \times 4.0 \times 1/100 = 14.20$ (m)

$d = 10.80$ (m) (top level of foot protection block)

1) Impulsive Breaking Wave Force Coefficient

Where the rubble mound is high and waves are breaking against the mound, α_2 is generalized to α^*, and either α_2 or α_I, whichever is larger, should be used.

$$\alpha^* = \max \{\alpha_2, \alpha_I\}$$

Where B_M is the mound width and L is the wavelength at the depth where the breakwater is installed.

$B_M = 8.50$ (m)

$B_M/L = 8.50/106.140 = 0.080$

$(h - d)/h = (14.00 - 10.80)/14.00 = 0.229$

$\delta_{11} = 0.93 \times (0.080 - 0.12) + 0.36 \times (0.229 - 0.6) = -0.1708$

$\delta_{22} = -0.36 \times (0.080 - 0.12) + 0.93 \times (0.229 - 0.6) = -0.3306$

$\delta_1 = 20 \times (-0.1708) = -3.4160$

$\delta_2 = 4.9 \times (-0.3306) = -1.6199$

$\alpha_{I1} = \cos(-1.6199) / \cosh(-3.4160) = -0.0032$

$\alpha_{I0} = 10.60 / 10.40 = 1.019$

$\alpha_{I1} \leq 0$ gives $\alpha_I = 0$

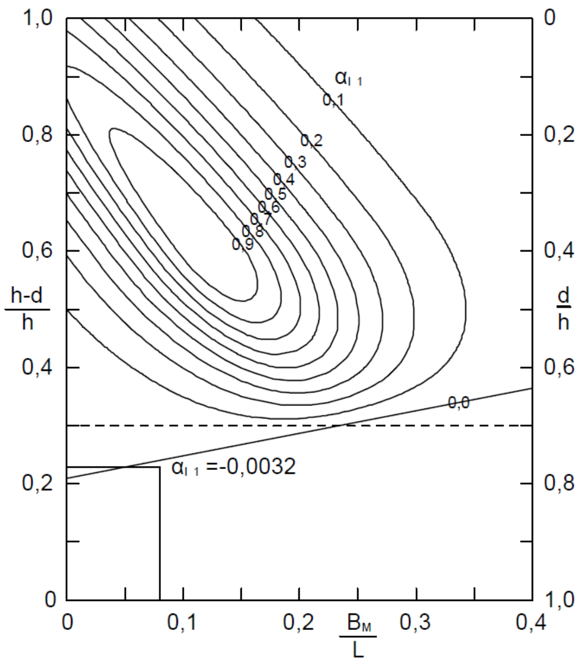


Figure 2.4- Impulsive Breaking Wave Force Coefficient

2) Wave Force by Goda's Formula

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

$$= 0.6 + \frac{1}{2}\left[\frac{4\pi \times 14.000/106.140}{\sinh(4\pi \times 14.000/106.140)}\right]^2 = 0.815$$

$$\alpha_2 = \min\left\{\frac{h_b - d}{3h_b}\left(\frac{H_D}{d}\right)^2, \frac{2d}{H_D}\right\}$$

$$= \min\left\{\frac{14.200-10.800}{3\times 14.200}\left(\frac{7.20}{10.800}\right)^2, \frac{2\times 10.800}{7.200}\right\}$$

$$= \min\{0.035, 3.00\} = 0.035$$

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

$$= 1 - \frac{11.80}{14.00}\left[1 - \frac{1}{\cosh(2\pi \times 14.00/106.140)}\right] = 0.775$$

$\alpha^* = \max\{\alpha_2, \alpha_I\} = \alpha_2 = 0.035$

$\eta^* = 0.75(1+\cos\beta)\lambda_1 H_D$

$= 0.75 \times (1+\cos 0) \times 0.8 \times 7.20 = 10.800$ (m)

$p_1 = 0.5(1+\cos\beta)(\alpha_1\lambda_1 + \alpha_2\lambda_2\cos^2\beta)w_o H_D$

$= 0.5 \times (1+\cos 0) \times (0.815 \times 1.00 + 0.035 \times 1.000 \times \cos^2 0) \times 1.03 \times 9.81 \times 7.20 = 61.838$ (kN/m^2)

$p_2 = p_1/\cosh(2\pi h/L) = 61.838/\cosh(2 \times \pi \times 14.00/106.140) = 45.351$ (kN/m^2)

$p_3 = \alpha_3 p_1 = 0.775 \times 61.838 = 47.924$ (kN/m^2)

$p_u = 0.5(1+\cos\beta)\,\alpha_1\alpha_3\lambda_3 w_o H_D = 0.5 \times (1+\cos 0) \times 0.815 \times 0.775 \times 1.00 \times 1.03 \times 9.81 \times 7.20 = 45.951$ (kN/m^2)

Figure 2.5 shows the wave pressure distribution calculated from Goda's formula. The wave force and moment are given in Table 2.4.

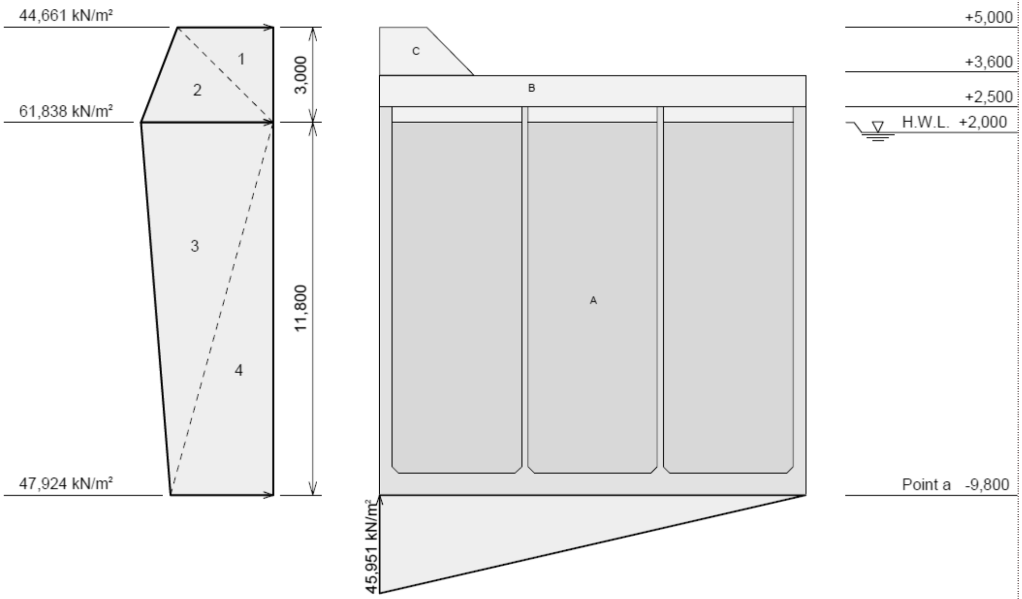


Figure 2.5- Wave Pressure Distribution Chart (Wave Crest)

Table 2.7- Wave Force and Moment (Wave Crest)

Wave force and moment

No	Calculation formula	P_H (kN/m)	y (m)	$P_H y$ (kN·m/m)
1	1/2 × 44.661 × 3.000	66.992	13.800	924.4908
2	1/2 × 61.838 × 3.000	92.757	12.800	1,187.2905
3	1/2 × 61.838 × 11.800	364.844	7.867	2,870.2288
4	1/2 × 47.924 × 11.800	282.752	3.933	1,112.0641
Point a (-9.800m)		807.345		6,094.0720

Uplift force and moment

	Calculation formula	P_U (kN/m)	x (m)	$P_U x$ (kN·m/m)
Point a (-9.800m)	1/2 × 45.951 × 13.500	310.169	9.000	2,791.521

3) Negative Wave Force

The wave forces acting on the wave trough are shown below:

$p_n = 0.5 w_o H_D$

$= 0.5 \times 1.03 \times 9.81 \times 7.20 = 36.375$ (kN/m^2)

$p_u = p_n = 0.5 w_o H_D$

$= 0.5 \times 1.03 \times 9.81 \times 7.20 = 36.375$ (kN/m^2)

Wave force and moment

No	Calculation formula	P_H (kN/m)	y (m)	$P_H y$ (kN·m/m)
1	1/2 × 36.375 × 3.600	65.475	9.400	615.465
2	36.375 × 8.200	298.275	4.100	1,222.928
Point a (-9.800m)		363.750		1,838.393

Uplift force and moment

	Calculation formula	P_U (kN/m)	x (m)	$P_U x$ (kN·m/m)
Point a (-9.800m)	1/2 × 36.375 × 13.500	245.531	4.500	1,104.890

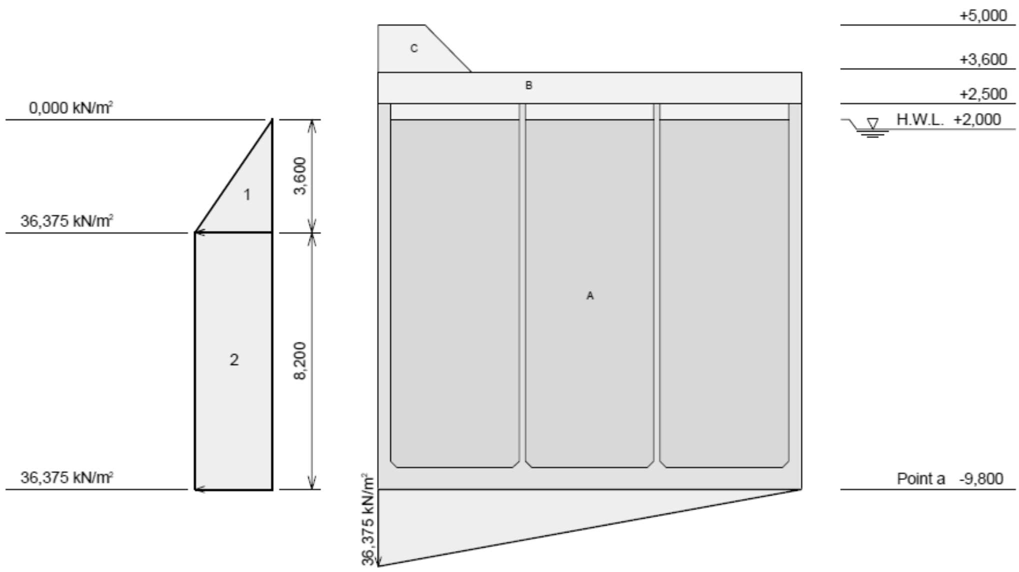


Figure 2.6- Wave Pressure Distribution Chart (Wave Trough)

2-5. Verification of Stability

A calculation example at the time of H.W.L. and wave crest is shown below:

(1) Verification of Sliding/Overturning of Caisson

Point a (-9.800m)

	V (kN/m)	H (kN/m)	M_V (kN·m/m)	M_H (kN·m/m)
Wave pressure	———	807.345	———	6,094.072
Uplift force	-310.169	———	-2,791.521	———
Wall weight	3,860.919	———	26,487.048	———
Buoyancy	-1,608.930	———	-10,860.278	———
Total	1,941.820	807.345	12,835.249	6,094.072

1) Verification of Sliding Stability

Resistance term $R_d = \gamma_R \cdot f \cdot V = 0.83 \times 0.60 \times 1{,}941.820 = 967.026$ (kN/m)

Load term $S_d = \gamma_S \cdot H = 1.08 \times 807.345 = 871.933$ (kN/m)

$m \cdot S_d / R_d = 1.00 \times 871.933 / 967.026 = 0.902 \leqq 1.00$ O.K

2) Verification of Overturning Stability

Resistance term $R_d = \gamma_R \cdot M_V = 0.95 \times 12{,}835.249 = 12{,}193.487$ (kN·m/m)

Load term $S_d = \gamma_S \cdot M_H = 1.14 \times 6{,}094.072 = 6.947.242$ (kN·m/m)

$m \cdot S_d / R_d = 1.00 \times 6{,}947.242 / 12{,}193.487 = 0.570 \leqq 1.00$ O.K

(2) Verification of Stability against Bearing Capacity of Foundation Ground

Verification for failure of the bearing capacity of the foundation ground is conducted by Bishop's method.

When subgrade reaction has a trapezoidal distribution	$q = -\dfrac{(p_1+p_2)}{4\,b'}B$
When subgrade reaction has a triangular distribution	$q = -\dfrac{p_1 b}{4\,b'}$

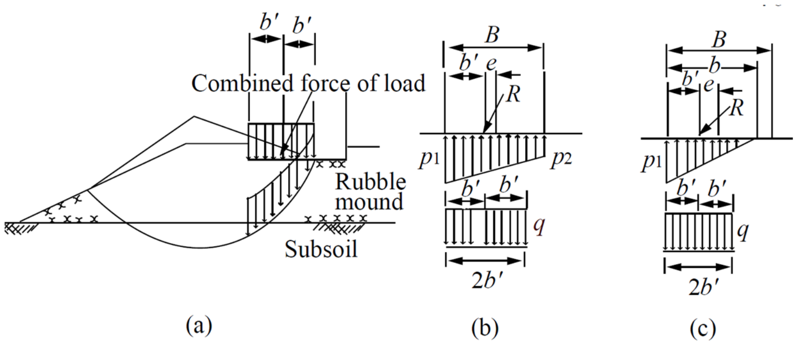


Figure 2.7- Analysis of Bearing Capacity for Eccentric and Inclined Actions

1) Strength Parameter in Bishop's Method

According to a report by the Port and Airport Research Institute in Japan, using shear strength parameters obtained from triaxial compression tests for mound materials and foundation soils in circular slip surface analysis based on Bishop's method yields highly accurate results.

In cases where such tests are not conducted, typical values used for the strength parameters of mound materials are cohesion c=20.0 kN/m^2 and internal friction angle φ= 35º. For foundation soils, a standard value of φ=40º is used for sandy soils with an N-value less than 10, and φ=45º for soils with an N-value of 10 or greater. When the foundation soil is cohesive, it is standard practice to determine the strength parameters based on the shear characteristics of the soil.

Table 2.8- Characteristic Values of Ground Conditions for Verification of Bearing Capacity (for Bishop's method)

	Saturated weight γ_{sat} (kN/m^3)	Weight in water γ' (kN/m^3)	Shear resistance angle φ'_k (°)	Cohesion	
				c'_k (kN/m^2)	Primary coefficient of cohesion
Foundation Rubble mound	20.00	10.00	35.00 (tan φ'_k=0.700)	20.00	0.00
Sand replacement	20.00	10.00	40.00 (tan φ'_k=0.839)	0.00	0.00

2) Distribution Load of Vertical Subgrade Reaction of Caisson Bottom Slab

The shape of distribution load of vertical subgrade reaction of caisson bottom slab

$$x=\frac{\Sigma M}{\Sigma V}=\frac{\Sigma M_V\text{-}\Sigma M_H}{\Sigma V}=\frac{12{,}835.249 - 6{,}094.072}{1{,}941.820}= 3.472 \text{ (m)}$$

$e = B/2 - x = 13.500/2 - 3.472 = 3.278$ (m)
$e > B/6 = 13.500 / 6 = 2.250$ (m)

Therefore, the distribution is triangular, and

$$p_1=\frac{2\Sigma V}{3(B/2\text{-}e)}=\frac{2\times 1{,}941.820}{3\times(13.500/2\text{-}3.278)}= 372.854 \text{ (kN/m}^2\text{)}$$

Width of distribution $b'=3(B/2-e)=3\times(13.500/2-3.278)=10.416$ (m)

Calculation of width of load

$$2b'=\frac{2\Sigma M}{\Sigma V}=\frac{2(\Sigma V\cdot x-\Sigma H\cdot y)}{\Sigma V}=\frac{2\times(12{,}835.249-6{,}094.072)}{1{,}941.820}=6.944\ \text{(m)}$$

Calculation of average value of uniformly-distributed load

$$q=\frac{\Sigma V}{2b'}=\frac{1{,}941.820}{6.944}=279.641\ (\text{kN/m}^2)$$

Horizontal force

H=807.345 (kN/m)

3) Verification Results

In verification of bearing capacity of the foundation ground, the adjustment factor m is considered. Figure 2.8 shows the verification results for the case of variable situation associated with wave motion, which is the most dangerous condition.

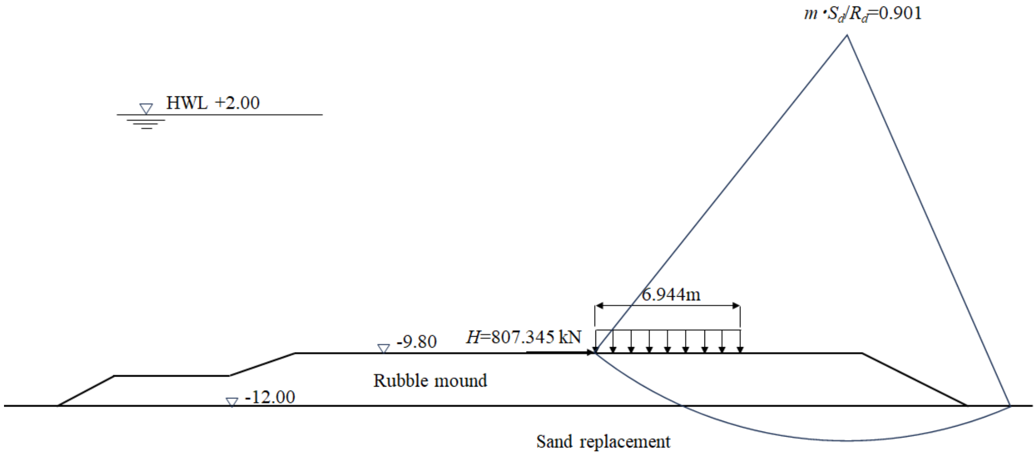


Figure 2.8- Verification of Bearing Capacity by Bishop's Method (Variable Situation: Wave Motion)

(3) Verification of Circular Slip Failure of Foundation Ground

Verification is performed for circular slip failure of the foundation ground in the permanent state.

$\sum M=W\cdot x-P_B\cdot x=26{,}487.048-9{,}019.553=17{,}467.495$ (kN·m/m)

$\sum V=W-P_B=3{,}860.919-1{,}336.230=2{,}524.689$ (kN/m)

$$x=\frac{\Sigma M}{\Sigma V}=\frac{17{,}467.495}{2{,}524.689}=6.919\ \text{(m)}$$

$e=B/2-x=13.500/2-6.919=-0.169$ (m)

$e<B/6=13.500/6=2.250$ (m)

Therefore, the distribution is trapezoidal, and

$$p_2 = \left(1 - \frac{6e}{B}\right)\frac{\Sigma V}{B} = \left(1 - \frac{6\times0.169}{13.500}\right)\times\frac{2{,}524.689}{13.500} = 172.967\ (\text{kN/m}^2)$$

$$p_1 = \left(1 + \frac{6e}{B}\right)\frac{\Sigma V}{B} = \left(1 + \frac{6\times0.169}{13.500}\right)\times\frac{2{,}524.689}{13.500} = 201.061\ (\text{kN/m}^2)$$

Table 2.9- Characteristic Values of Ground Conditions for Verification of Circular Slip Failure of Foundation Ground

	Saturated weight γ (kN/m³)	Wet weight γ_t (kN/m³)	Weight in water γ' (kN/m³)	Shear resistance angle φ'_k(°)	Cohesion c'_k (kN/m²)	Cohesion Primary coefficient of cohesion
Foundation rubble	20.00	18.00	10.00	40.00	0.00	0.00
Sandy soil	20.00	18.00	10.00	30.00	0.00	0.00
Sandy soil	20.00	18.00	10.00	35.00	0.00	0.00

Table 2.10- Verification of Circular Slip Failure of Foundation Ground

State	Resistance term R_d (kN.m)	Load term S_d (kN.m)	Adjustment factor m	Load-resistance ratio $m{\cdot}S_d/R_d$
Permanent state	41,990.8	30,380.6	1.0	0.724

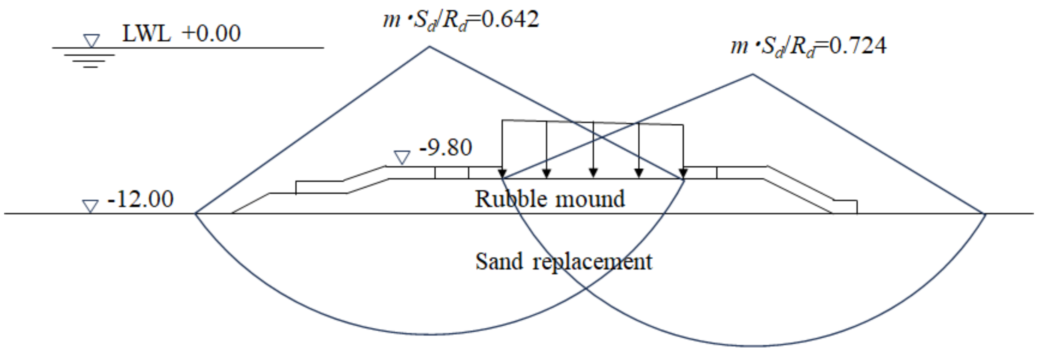


Figure 2.9- Verification of Circular Slip Failure Stability of Foundation Ground (Permanent State)

(4) Verification of Stability under Floating Condition

1) Weight and Moment

Table 2.11- Characteristic Values of Caisson Weight and Moment

Name	Dimensions (m)	Nos.	V (m³) W (kN)	x (m) y (m)	$W{\cdot}x$ (kN·m) $W{\cdot}y$ (kN·m)
Base slab	13.500 × 17.000 × 0.700	1	160.650 3,855.600	6.750 0.350	26,025.300 1,349.460
Side wall Longitudinal	0.400 × 17.000 × 11.600	2	157.760 3,786.240	6.750 6.500	25,557.120 24,610.560
Side wall Transverse	12.700 × 0.400 × 11.600	2	117.856 2,828.544	6.750 6.500	19,092.672 18,385.560
Partition wall Longitudinal	0.200 × 16.200 × 11.600	2	75.168 1,804.032	6.750 6.500	12,177.216 11,726.208
Partition wall	12.300 × 0.200 × 11.600	3	85.608	6.750	13,868.496

Transverse			2,054.592	6.500	13,354.848
Vertical haunch	0.200 × 0.200 × 11.600 ×1/2	48	11.136 267.264	6.750 6.500	1,804.032 1,737.216
Horizontal haunch Longitudinal	0.200 × 14.000 × 0.200 ×1/2	6	1.680 40.320	6.750 0.767	272.160 30.925
Horizontal haunch Transverse	11.100 × 0.200 × 0.200 ×1/2	8	1.776 42.624	6.750 0.767	287.712 32.693
Corner haunch	0.200 × 0.200 × 0.200 ×1/3	48	0.128 3.072	6.750 0.767	20.736 2.381
Total			611.762 14,682.288	6.750 4.851	99,105.444 71,229.827

2) Total Weight, Center of Gravity and Moment

Table 2.12- Characteristic Values of Total Weight and Moment

	W (kN)	x (m)	y (m)	$W{\cdot}x$ (kN·m)	$W{\cdot}y$ (kN·m)
Caisson	14,682.288	6.750	4.851	99,105.444	71,229.827
Total	14,682.288	6.750	4.851	99,105.444	71,229.827

3) Draft Calculation

$$d = \frac{W}{B \times L \times \gamma_w} = \frac{14{,}682.288}{13.5 \times 17.000 \times 10.1} = 6.334 \text{ (m)}$$

Where:

d : draft depth (m)
W : total weight of caisson (kN)
γ_w : unit weight of seawater (kN/m^3)
B : width of caisson body (m)
L : length of caisson body (m)

The freeboard height (f) above water level:

$f = H - d = 12.300 - 6.334 = 5.966 \geq 1.00$ (m) O.K

4) Stability Verification under Floating Condition

Table 2.13- Characteristic Values of Buoyancy and Moment

Name	Dimensions (m)	Nos.	V (m^3) W (kN)	x (m) y (m)	$W{\cdot}x$ (kN·m) $W{\cdot}y$ (kN·m)
Caisson	13.500 × 17.000 × 6.334	1	1,453.653 14,681.895	6.750 3.167	99,102.791 46,497.561
Total			1,453.653 14,681.895	6.750 3.167	99,102.791 46,497.561

$$\frac{I'}{V'} - \overline{C'G'} > 0$$

Where:

V : displacement volume (m^3)
I : geometrical moment of interia with respect to long axis at water level (m^4)
C : center of buoyancy
G : center of gravity
V', I', C', G' : corresponding values or positions during the application of the counter ballast

$I = L \times B^3/12 = 17.000 \times 13.500^3/12 = 3{,}485.531\ (m^4)$

$\overline{GM} = I/V - \overline{CG} = 3{,}485.531/1{,}453.653 - (4.851 - 3.167) = 0.714\ (m) \geq 0$

For safety, it is desirable that GM (the metacentric height) be at least 5% of the draft.

$\overline{GM} = 0.714\ (m) \geq 0.05d = 0.317\ (m)$ O.K

2-6. Summary of Load-Resistance Ratio

Table 2.14 shows the load-resistance ratios obtained in a stability verification considering partial factors based on the study presented above.

Table 2.14- Summary of Load-Resistance Ratio

State/Situation		Unit	Resistance term	Load term	Adjustment factor	Load-resistance ratio
Permanent state	Circular slip failure of foundation ground	kN･m	41,990.8	30,380.6	1.0	0.724 ≦ 1.0
Variable situation: Wave motion	Sliding of wall body	kN/m	967.026	871.933	1.0	0.902 ≦ 1.0
	Overturning of wall body	kN･m/m	12,193.487	6.947.242	1.0	0.570 ≦ 1.0
	Bearing capacity failure of foundation ground	kN･m	32,071.800	28,896.700	1.0	0.901 ≦ 1.0

2-7. Other Safety Verification

The safety verification of uneven settlement is omitted in this example. For verification method, refer to Part 4 Caisson-type Gravity Quaywall.

The results of the stability verification such as subgrade reaction forces and wave forces for other cases are shown in Tables 2-15 to 2-17.

Table 2.15- Summary of Stability Verification (Permanent State)

Permanent state (calm condition)			H.W.L.	L.W.L.		
Tide level			2.000	0.000		
Breakwater body weight	W	kN/m	3,860.919	3,860.919		
Breakwater body moment	M_W	kN·m/m	26,487.048	26,487.048		
Buoyancy	W_f	kN/m	-1,608.930	-1,336.230		
Buoyancy moment	M_{Wf}	kN·m/m	-10,860.278	-9,019,553		
Total vertical force	$\sum V$	kN/m	2,251.989	2,524.689		
Total horizontal force	$\sum H$	kN/m	0.000	0.000		
Total vertical force moment	$\sum M_V$	kN·m/m	15,626.770	17,467.495		
Total horizontal force moment	$\sum M_H$	kN·m/m	0.000	0.000		
Eccentricity	e	m	-0.189	-0.169		
Subgrade reaction force (Seaside)	q_1	kN/m²	180.827	201.061		

Subgrade reaction force (Landside)	q_2	kN/m^2	152.802	172.968		
Width of subgrade reaction force	b	m	13.500	13.500		

Table 2.16- Summary of Stability Verification (Variable situation, Wave Crest)

Variable situation (wave crest)			Safety (Sectional failure)		Serviceability	
			H.W.L.	L.W.L.	H.W.L.	L.W.L.
Tide level	-	m	2.000	0.000	2.000	0.000
Design wave height	H_D	m	7.200	6.840	3.800	3.800
Significant wave height	$H_{1/3}$	m	4.000	3.800	2.100	2.100
Wave period	T	sec	10.000	10.000	6.900	6.900
Wavelength	L	m	106.140	99.727	65.022	62.230
Water depth	h	m	14.000	12.000	14.000	12.000
Water depth	h'	m	11.800	9.800	11.800	9.800
Water depth	h_b	m	14.200	12.190	14.105	12.105
Water depth	d	m	10.800	8.800	10.800	8.800
Coefficient	λ_1	-	1.000	1.000	1.000	1.000
Coefficient	λ_2	-	1.000	1.000	1.000	1.000
Coefficient	λ_3	-	1.000	1.000	1.000	1.000
Wave pressure height	η^*	m	10.800	10.260	5.700	5.700
Coefficient	α_1	-	0.815	0.846	0.666	0.694
Coefficient	α_2	-	0.035	0.056	0.010	0.017
Coefficient	α_3	-	0.775	0.812	0.566	0.630
Wave pressure	p_1	kN/m^2	61.838	62.340	25.956	27.300
Wave pressure	p_2	kN/m^2	45.351	47.966	12.579	14.932
Wave pressure	p_3	kN/m^2	47.924	50.620	14.691	17.199
Uplift pressure	p_U	kN/m^2	45.951	47.478	14.474	16.788
Wave force	P_h	kN/m	807.345	789.254	297.194	294.678
Wave force moment	M_{Ph}	kN·m/m	6,094.072	5,642.408	2,298.521	2,041.962
Uplift force	P_U	kN/m	-310.169	-320.477	-97.700	-113.319
Uplift force moment	M_{PU}	kN·m/m	-2,791.521	-2,884.293	-879.300	-1,019.871
Breakwater body weight	W	kN/m	3,860.919	3,860.919	3,860.919	3,860.919
Breakwater body moment	M_W	kN·m/m	26,487.048	26,487.048	26,487.048	26,487.048
Buoyancy	W_f	kN/m	-1,608.930	-1,336.230	-1,608.930	-1,336.230
Buoyancy moment	M_{Wf}	kN·m/m	-10,860.278	-9,019,553	-10,860.278	-9,019,553
Total vertical force	$\sum V$	kN/m	1,941.820	2,204.212	2,154.289	2,411.370
Total horizontal force	$\sum H$	kN/m	807.345	789.254	297.194	294.678
Total vertical force moment	$\sum M_V$	kN·m/m	12,835.249	14,583.202	14,747.470	16,447.624
Total horizontal force moment	$\sum M_H$	kN·m/m	6,094.072	5,642.408	2,298.521	2,041.962
Eccentricity	e	m	3.278	2.694	0.971	0.776

Subgrade reaction force (Seaside)	q_1	kN/m^2	0.000	0.000	90.711	117.016
Subgrade reaction force (Landside)	q_2	kN/m^2	372.854	362.297	228.444	240.225
Width of subgrade reaction force	b	m	10.416	12.168	13.500	13.500

Table 2.17- Summary of Stability Verification (Variable situation, Wave Trough)

Variable situation (wave crest)			Safety (Sectional failure)		Serviceability	
			H.W.L.	L.W.L.	H.W.L.	L.W.L.
Tide level	-	m	2.000	0.000	2.000	0.000
Design wave height	H_D	m	7.200	6.840	3.800	3.800
Wave pressure	p_n	kN/m^2	36.375	34.557	19.198	19.198
Uplift pressure	p_U	kN/m^2	36.375	34.557	19.198	19.198
Wave force	P_h	kN/m	363.750	279.566	208.298	169.902
Wave force moment	M_{Ph}	kN·m/m	1,838.393	1,147.684	1,132.898	754.698
Uplift force	P_U	kN/m	245.531	233.260	129.587	129.587
Uplift force moment	M_{PU}	kN·m/m	1,104.890	1,049.670	583.142	583.142
Breakwater body weight	W	kN/m	3,860.919	3,860.919	3,860.919	3,860.919
Breakwater body moment	M_W	kN·m/m	25,635.361	25,635.361	25,635.361	25,635.361
Buoyancy	W_f	kN/m	-1,608.930	-1,336.230	-1,608.930	-1,336.230
Buoyancy moment	M_{Wf}	kN·m/m	-10,860.278	-9,019,553	-10,860.278	-9,019,553
Total vertical force	$\sum V$	kN/m	2,497.520	2,757.949	2,381.576	2,654.276
Total horizontal force	$\sum H$	kN/m	363.750	279.566	208.298	169.902
Total vertical force moment	$\sum M_V$	kN·m/m	15,879.973	17,665.478	15,358.225	17,198.950
Total horizontal force moment	$\sum M_H$	kN·m/m	1,838.393	1,147.684	1,132.898	754.698
Eccentricity	e	m	1.128	0.761	0.777	0.555
Subgrade reaction force (Seaside)	q_1	kN/m^2	277.749	273.389	237.335	245.111
Subgrade reaction force (Landside)	q_2	kN/m^2	92.255	135.197	115.492	148.116
Width of subgrade reaction force	b	m	13.500	13.500	13.500	13.500

2-8. Verification of Structural Members

(1) Design Conditions

1) Material Conditions

The materials that can be procured vary depending on the construction conditions. Therefore, designers should verify the applicable materials.

i) Concrete

- Design characteristic strength f'_{ck} = 30 N/mm²
- Design compressive strength $f'_{cd} = f'_{ck}/\gamma_c$ = 30/1.3 = 23.1 N/mm²
- Young's modulus E_c = 28 kN/mm²

ii) Reinforcement (SD345)

- Design tensile yield strength f_{yd} = 345 N/mm²
- Young's modulus E_s = 200 kN/mm²

2) Load Case

Table 2.18 shows the load cases used for verification. In this casebook, only the shaded items are introduced. The remaining cases are referenced in Part 4 Caisson-type Gravity Quaywall. As there are no reported cases of damage to breakwater sidewalls caused by seismic ground motion and wave forces are considered to exceed seismic forces, the seismic action is not considered in this verification.

It should be noted that both H.W.L. and L.W.L. shall be calculated in each case.

Table 2.18- Load Cases Used for Verification (H.W.L. and L.W.L.)

Member	Permanent state	Variable situation associated with wave motion		During construction	
	Calm condition	Wave crest	Wave trough	Floating	Installation
Bottom slab	○	○	○	○	-
Front wall	○	○	○	○	-
Side wall	○	-	○	○	-
Rear wall	○	-	-	○	-
Partition wall	-	-	-	-	○

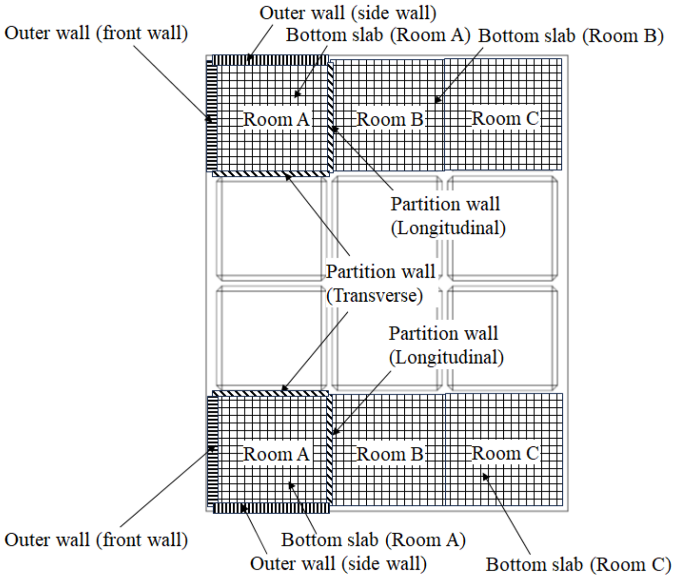


Figure 2.10- Target Members for Structural Verification

(2) Design Load

1) Bottom Slab

i) Floating condition

The design load is the hydrostatic pressure at the bottom of the caisson minus the weight of the bottom slab.

$S_f = (6.334 + 1.000) \times 10.10 = 74.07$ (kN/m^2)
$D_f = -0.700 \times 24.00 = -16.80$ (kN/m^2)

- Safety (against cross-sectional failure)
  $P = 1.1 \times S_f + 0.9 \times D_f = 66.36$ (kN/m^2)
- Serviceability
  $P = 0.5 \times S_f + 0.5 \times D_f = 28.64$ (kN/m^2)

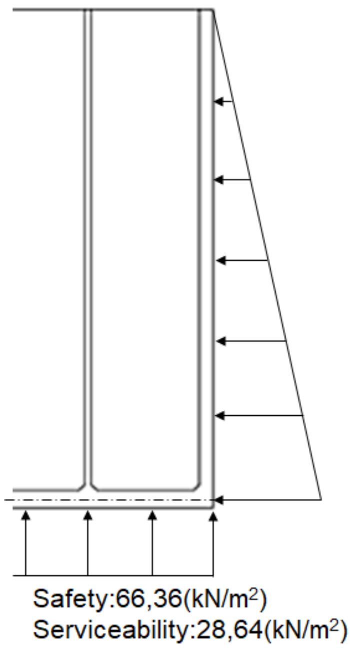


Figure 2.11- Design Load at Floating Condition

ii) Permanent state (Safety verification, H.W.L.)

✓ Load calculation under permanent state

- Self-weight of each chamber: D

D = Cover concrete weight + Self-weight of Infill sand + Self-weight of Bottom slab
= 0.500 × 22.60 + 11.100 × 20.00 + 0.700 × 24.00 = 250.10 (kN/m^2)

- Water pressure: F

F = (H.W.L - Installed depth) × γ_w
= (2.000 - (-9.800)) × 10.10 = 119.18 (kN/m^2)

- Bottom reaction force: R

Seaside: R =180.83 (kN/m^2), Landside: R = 152.80 (kN/m^2) , Width: B = 13.5 (m)

✓ Design load under permanent state

The combined load in the permanent state is distributed as shown in Figure 2.10.

Seaside: $W = 0.9D + 1.1F + 1.1R$ = - 0.9 × 250.10 + 1.1 × 119.18 +1.1 × 180.83
= 104.92 (kN/m^2)

Landside: $W = 0.9D + 1.1F + 1.1R$ = - 0.9 × 250.10 + 1.1 × 119.18 +1.1 × 152.80
= 74.09 (kN/m^2)

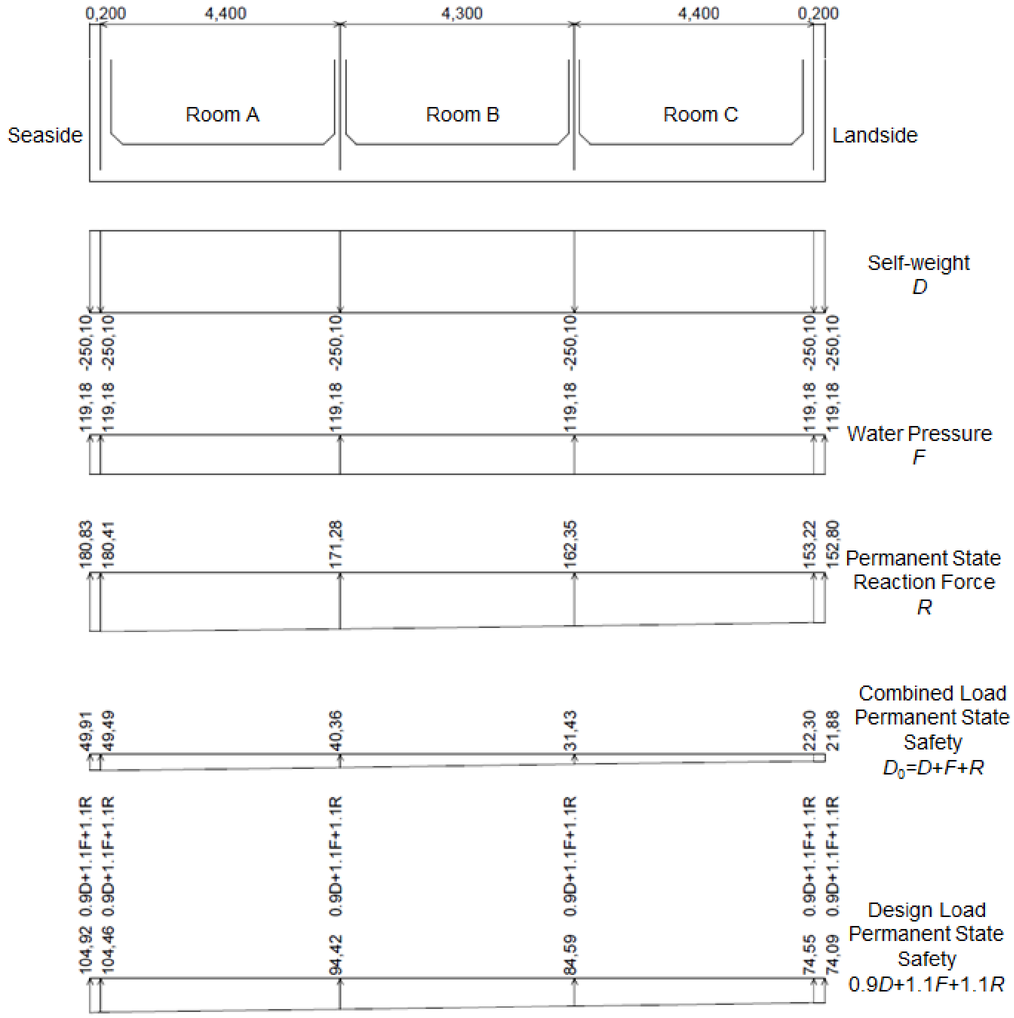


Figure 2.12- Design Load in Permanent States (Safety, H.W.L.)

iii) Permanent state (Safety verification, L.W.L.)

✓ Load calculation under permanent state

- Self-weight of each chamber: D

D = Cover concrete weight + Self-weight of Infill sand + Self-weight of Bottom slab
= 0.500 × 22.60 + 11.100 × 20.00 + 0.700 × 24.00 = 250.10 (kN/m^2)

- Water pressure: F

F = (L.W.L - Installed depth) × γ_w
= (0.000 - (-9.800)) × 10.10 = 98.98 (kN/m^2)

- Bottom reaction force: R

Seaside: R =201.06 (kN/m^2), Landside: R = 172.97 (kN/m^2) , Width: B = 13.5 (m)

✓ Design load under permanent state

The combined load in the permanent state is distributed as shown in Figure 2.11.

Seaside: $W = 0.9D + 1.1F + 1.1R$ = - 0.9 × 250.10 + 1.1 × 98.98 +1.1 × 201.06
= 104.95 (kN/m^2)

Landside: $W = 0.9D + 1.1F + 1.1R$ = - 0.9 × 250.10 + 1.1 × 98.98 +1.1 × 172.97
= 74.06 (kN/m^2)

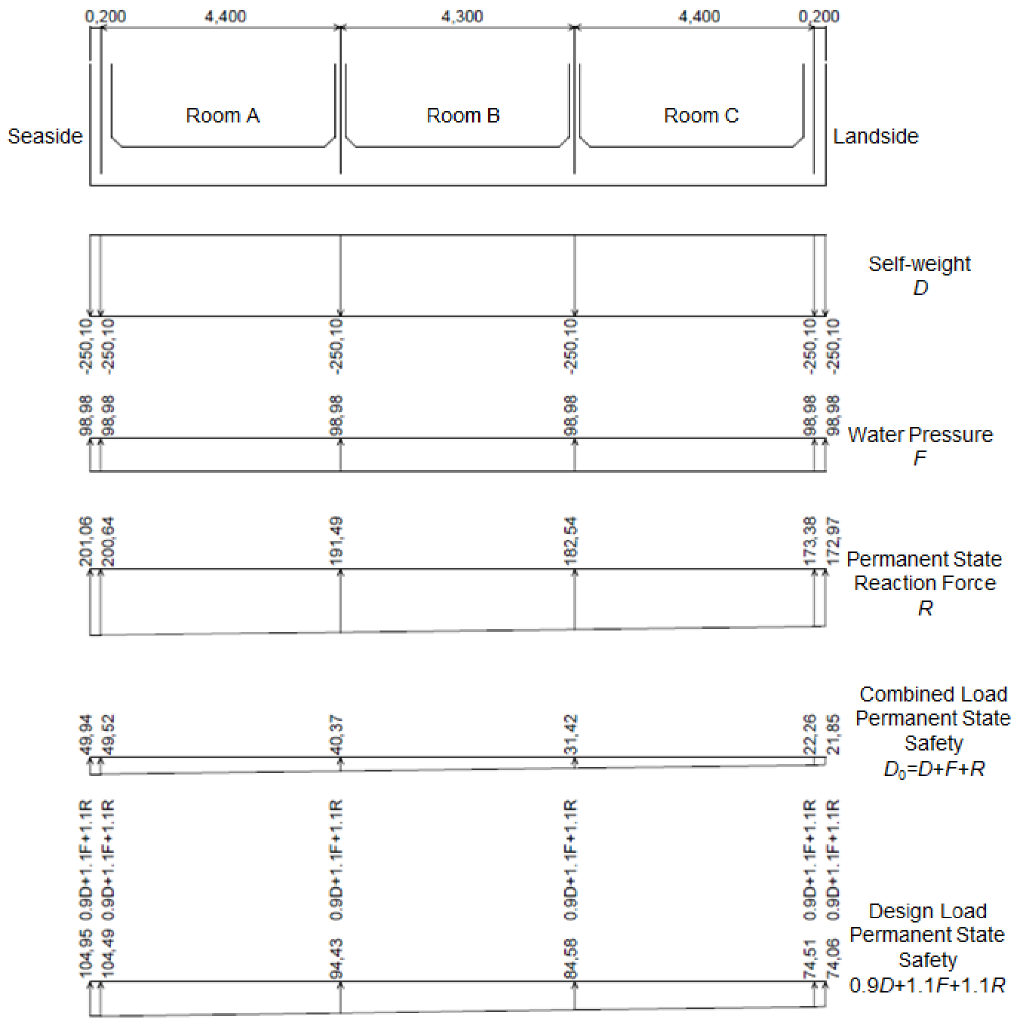


Figure 2.13- Design Load in Permanent States (Safety, L.W.L.)

iv) Permanent state (Serviceability verification, H.W.L.)

✓ Load calculation under permanent state

- Self-weight of each chamber: D

D = Cover concrete weight + Self-weight of Infill sand + Self-weight of Bottom slab
$= 0.500 \times 22.60 + 11.100 \times 20.00 + 0.700 \times 24.00 = 250.10$ (kN/m^2)

- Water pressure: F

F = (H.W.L - Installed depth) $\times \gamma_w$
$= (2.000 - (-9.800)) \times 10.10 = 119.18$ (kN/m^2)

- Bottom reaction force: R

Seaside: R =180.83 (kN/m^2), Landside: R = 152.80 (kN/m^2) , Width: B = 13.5 (m)

✓ Design load under permanent state

The combined load in the permanent state is distributed as shown in Figure 2.12.

Seaside: $W = 1.0D + 1.0F + 1.0R = -1.0 \times 250.10 + 1.0 \times 119.18 + 1.0 \times 180.83$
$= 49.91$ (kN/m^2)

Landside: $W = 1.0D + 1.0F + 1.0R = -1.0 \times 250.10 + 1.0 \times 119.18 + 1.0 \times 152.80$
$= 21.88$ (kN/m^2)

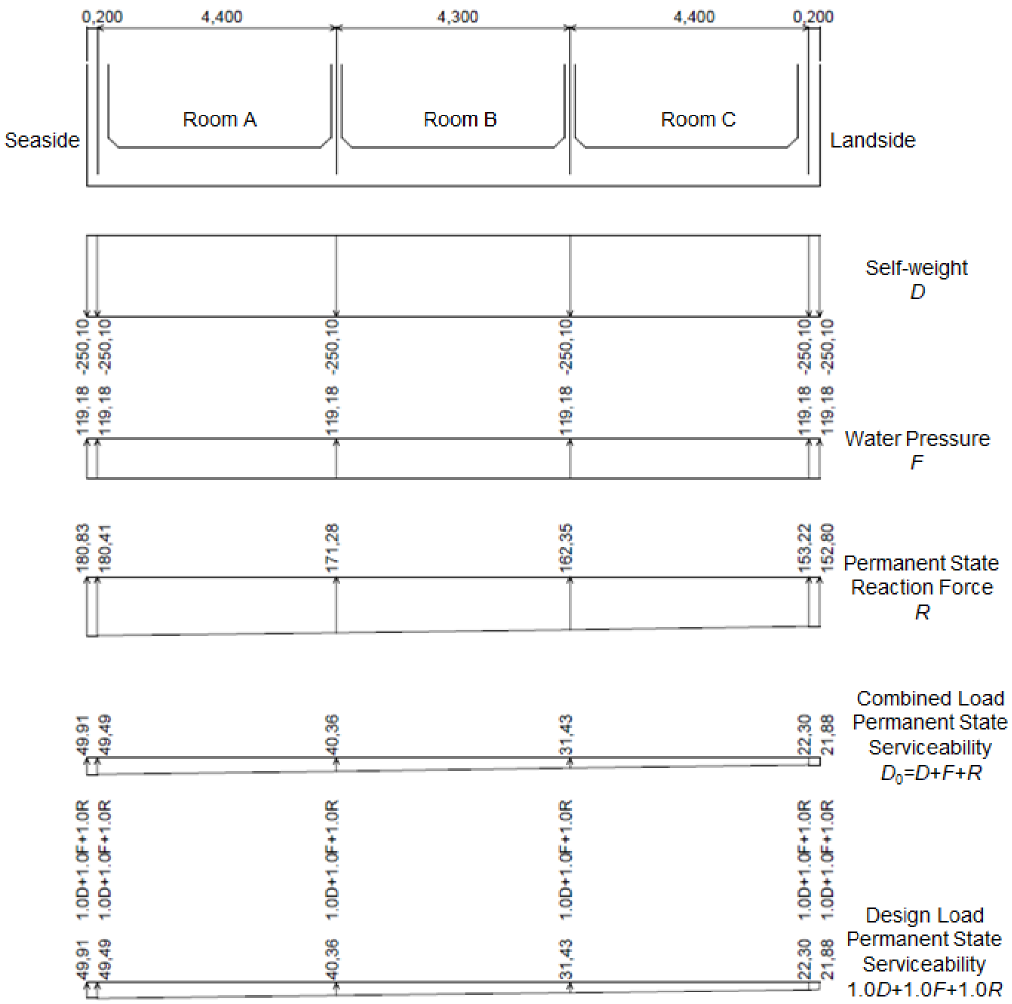


Figure 2.14- Design Load in Permanent States (Serviceability, H.W.L.)

v) Permanent state (Serviceability verification, L.W.L.)

✓ Load calculation under permanent state

- Self-weight of each chamber: D

D = Cover concrete weight + Self-weight of Infill sand + Self-weight of Bottom slab
$= 0.500 \times 22.60 + 11.100 \times 20.00 + 0.700 \times 24.00 = 250.10$ (kN/m^2)

- Water pressure: F

F = (L.W.L - Installed depth) × γ_w
$= (0.000 - (-9.800)) \times 10.10 = 98.98$ (kN/m^2)

- Bottom reaction force: R

Seaside: R =201.06 (kN/m^2), Landside: R = 172.97 (kN/m^2) , Width: B = 13.5 (m)

✓ Design load under permanent state

The combined load in the permanent state is distributed as shown in Figure 2.13.

Seaside: $W = 1.0D + 1.0F + 1.0R = -1.0 \times 250.10 + 1.0 \times 98.98 + 1.0 \times 201.06$
$= 49.94$ (kN/m^2)

Landside: $W = 1.0D + 1.0F + 1.0R = -1.0 \times 250.10 + 1.0 \times 98.98 + 1.0 \times 172.97$
$= 21.85$ (kN/m^2)

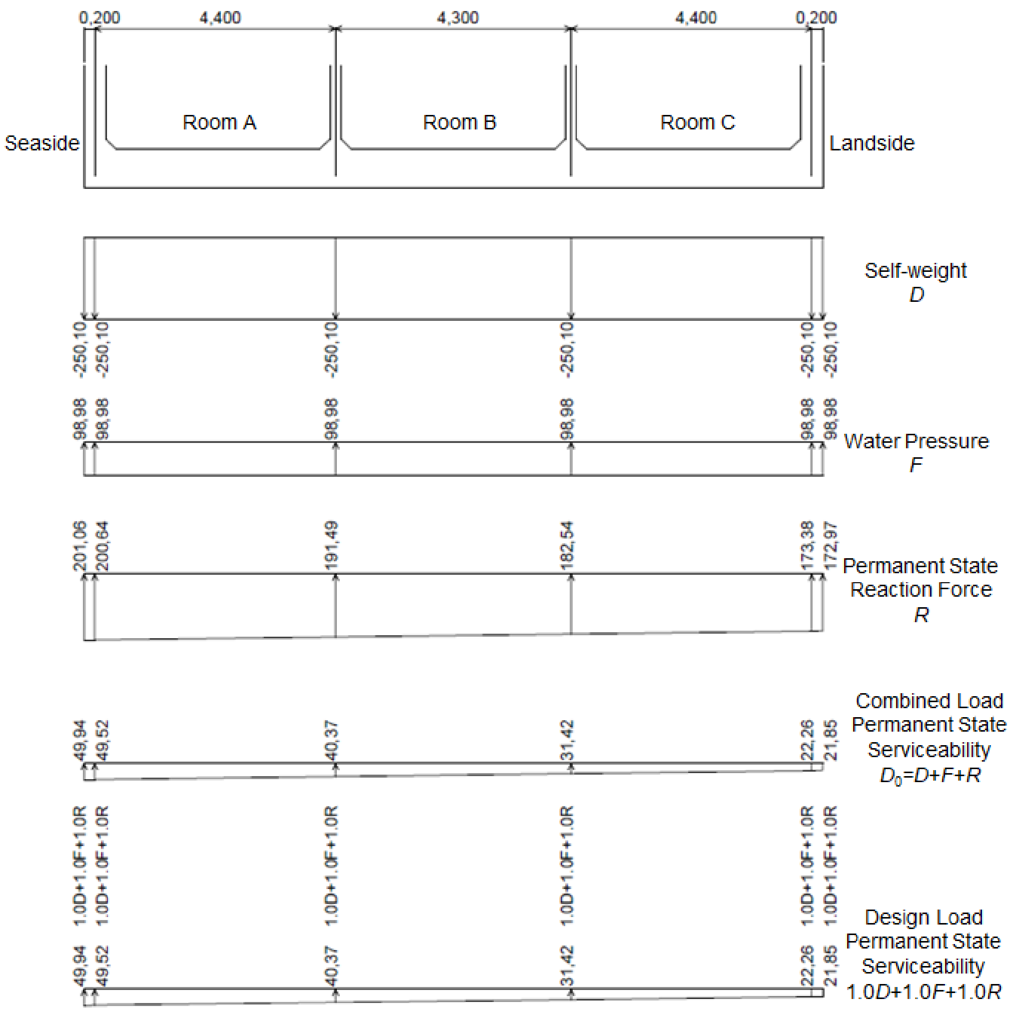


Figure 2.15- Design Load in Permanent States (Serviceability, L.W.L.)

vi) Variable situation (Wave motion, Safety verification, H.W.L., Wave crest)

- Bottom reaction force at wave motion: R'

  Seaside: R'=0.000 (kN/m^2), Landside: R' = 372.85 (kN/m^2) , Width: B = 10.416 (m)

- Variable component of bottom reaction force: ΔR

  Irregular shapes are transformed into uniform and triangular loads.

  $1/2 \times (-180.410 - 174.430) \times 2.884 \qquad = -511.68$ (kN/m^2)

  $1/2 \times (-174.430 - 117.010) \times 1.516 \qquad = -220.91$ (kN/m^2)

  $\sum A \qquad = -732.59$ (kN/m^2)

  Equivakent loads

  $P1 \qquad = -180.41$ (kN/m^2)

  $P2 = (2 \cdot \sum A / L) - \mathrm{P1}$

  $\quad = (2 \times -732.59 / 4.400) - (-180.41) \qquad = -152.59$ (kN/m^2)

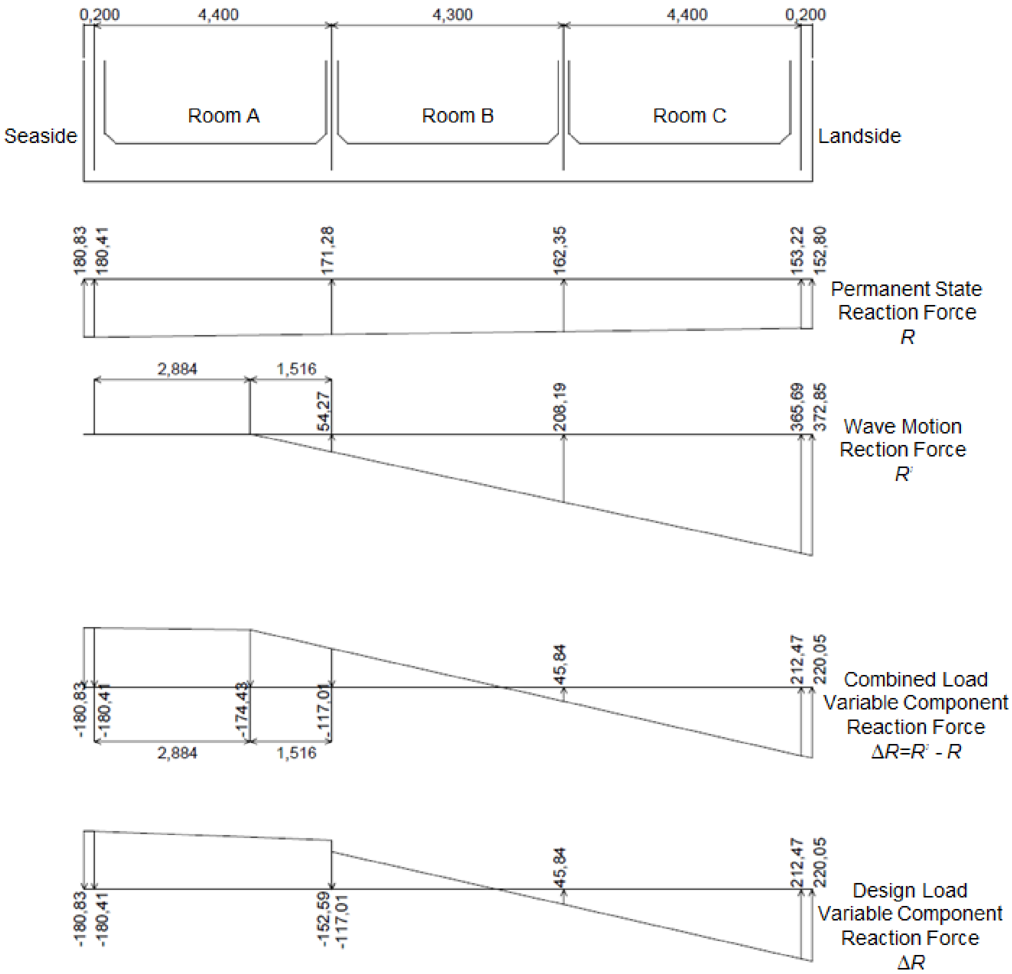


Figure 2.16- Bottom Reaction Force at Wave Motion (Safety, H.W.L.)

✓ Design load under variable situations associated with wave motion

The combined load in the variable situation is distributed as shown in Figure 2.17.

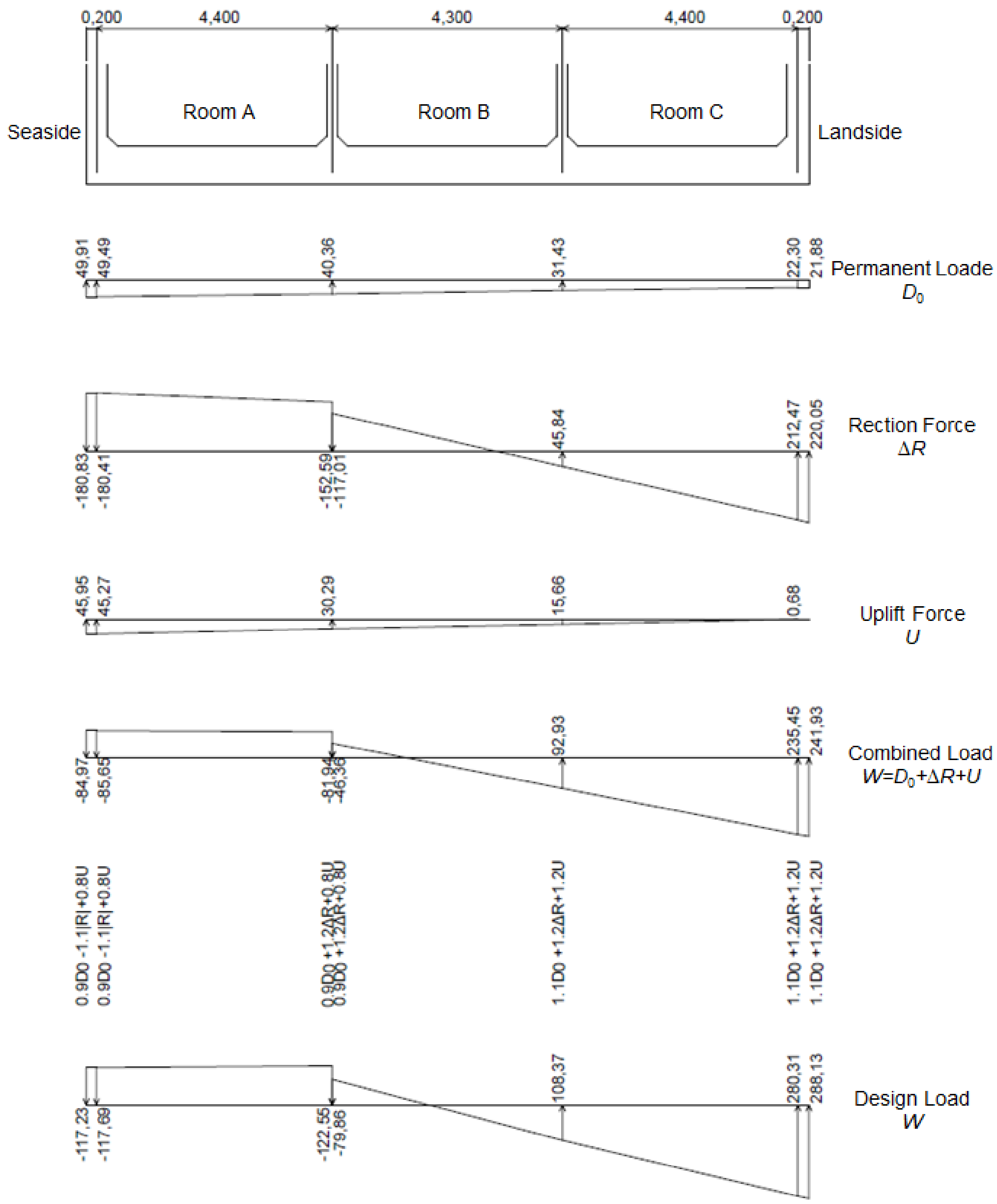


Figure 2.17- Design Load in Variable Situation (Safety, H.W.L.)

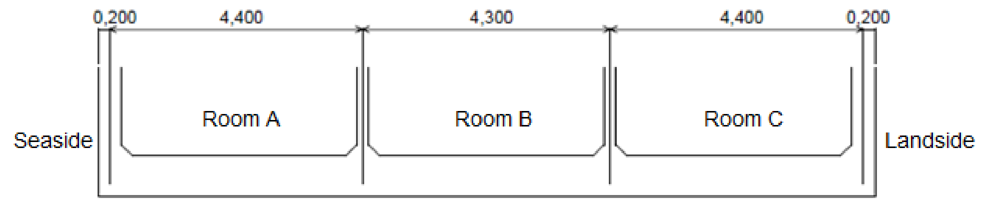
vii) Summary of design load

Similarly, the design loads for the following cases are calculated, but the calculation process is omitted.

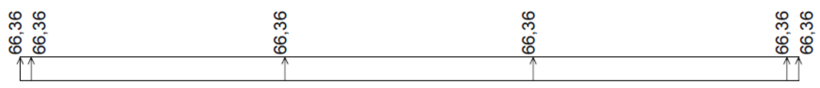
- Safety, L.W.L., Wave crest
- Safety, H.W.L., Wave trough
- Safety, L.W.L., Wave trough
- Serviceability, H.W.L., Wave crest
- Serviceability, L.W.L., Wave crest
- Serviceability, H.W.L., Wave trough
- Serviceability, L.W.L., Wave trough

The summary of design load is shown in Figure 2.18, 2.19 and 2.20.

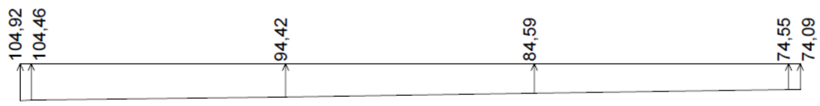
✓ Design load for safety verification



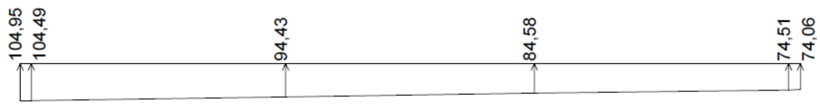
Floating condition



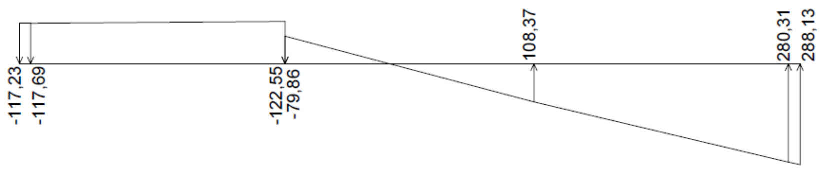
Permanent state: H.W.L



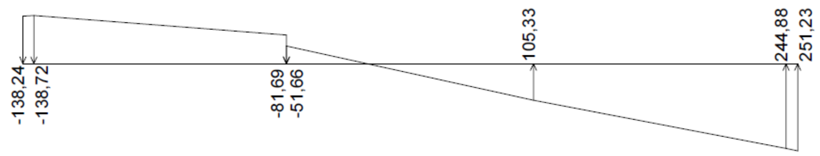
Permanent state: L.W.L



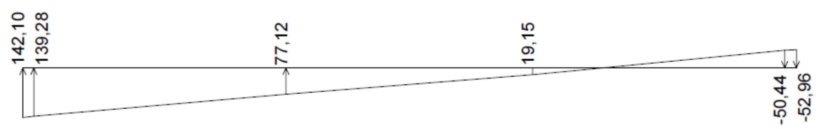
Variable situation: H.W.L, Wave crest



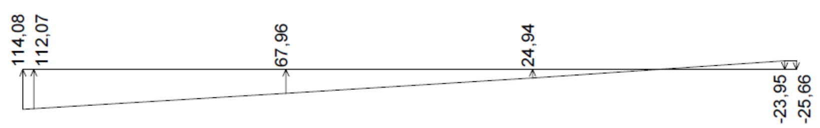
Variable situation: L.W.L, Wave crest



Variable situation: H.W.L, Wave trough



Variable situation: L.W.L, Wave trough



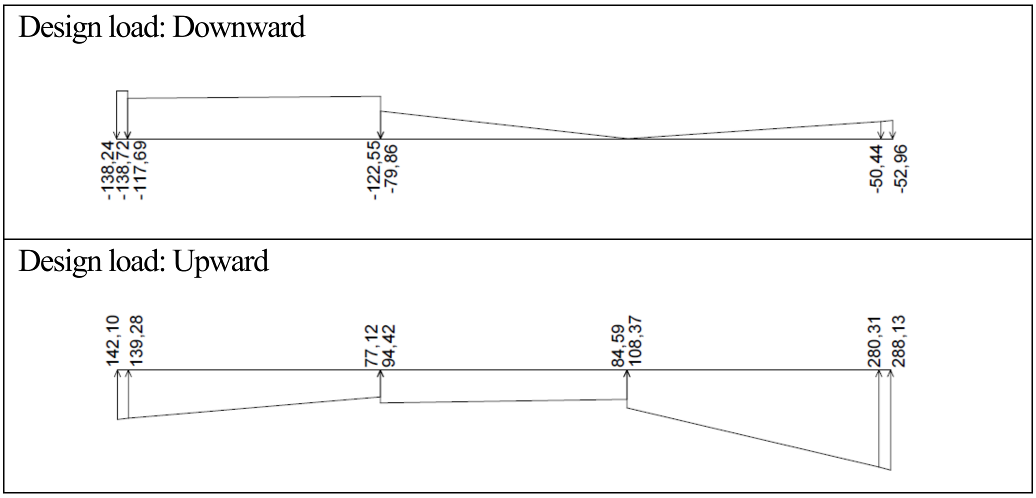
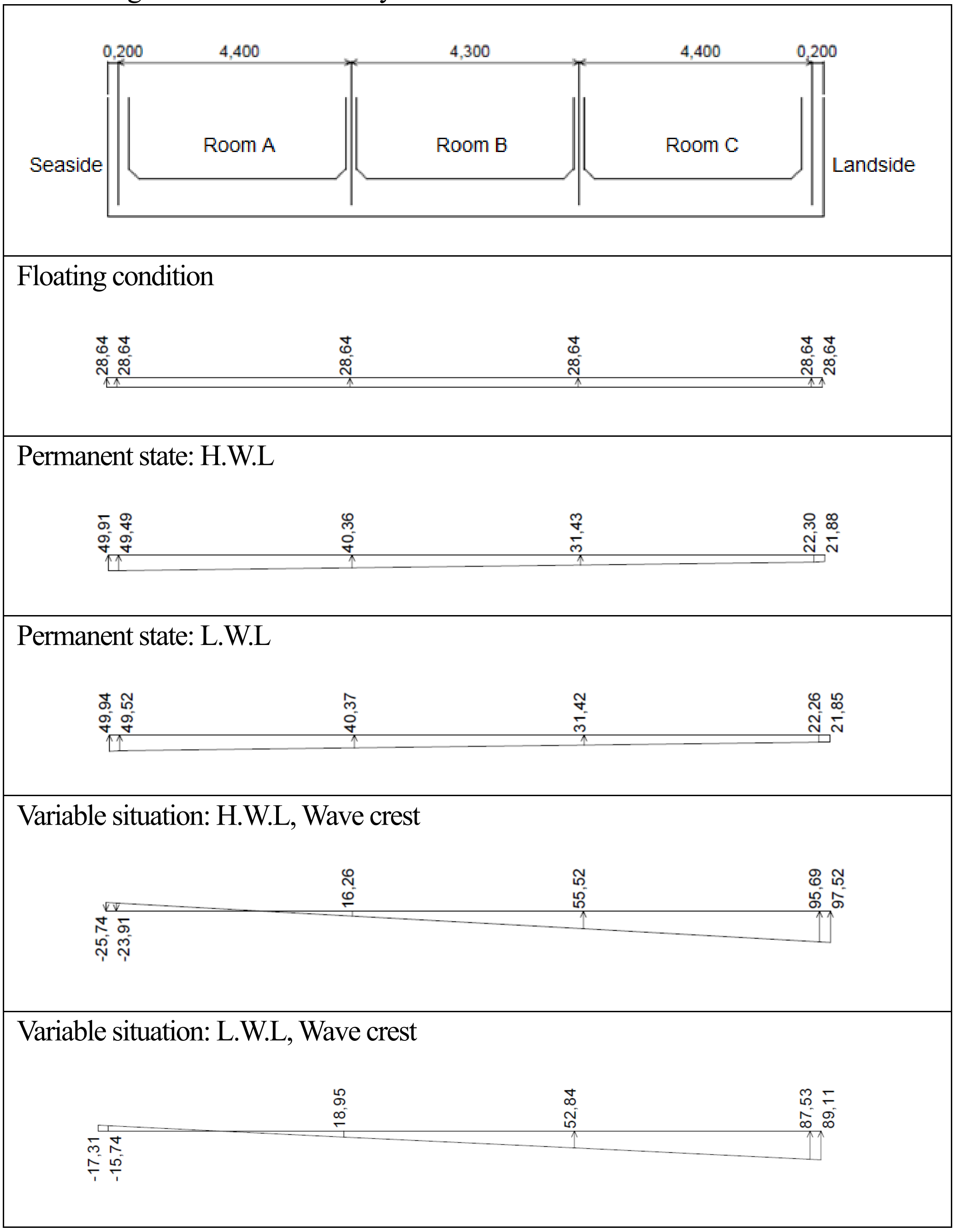


Figure 2.18- Summary of Design Load for Bottom Slab (Safety)

✓ Design load for serviceability verification



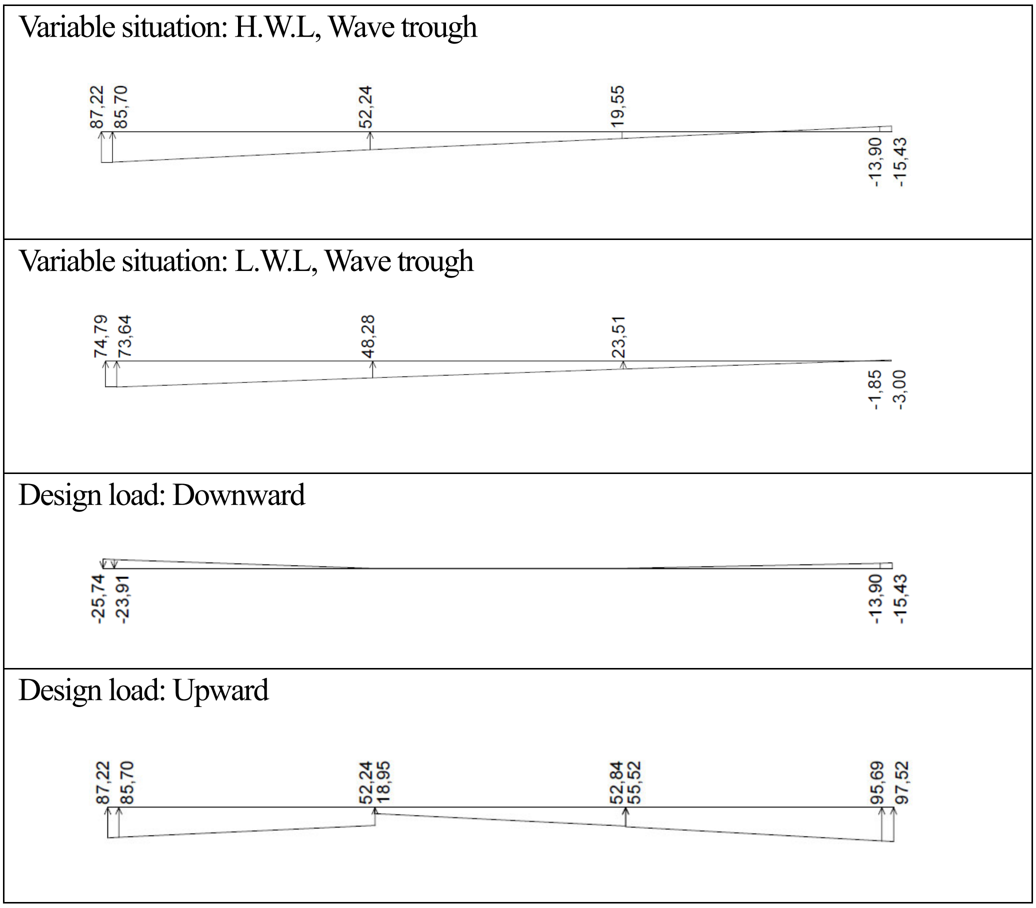


Figure 2.19- Summary of Design Load for Bottom Slab (Serviceability)

✓ Design load for serviceability verification (Compression)

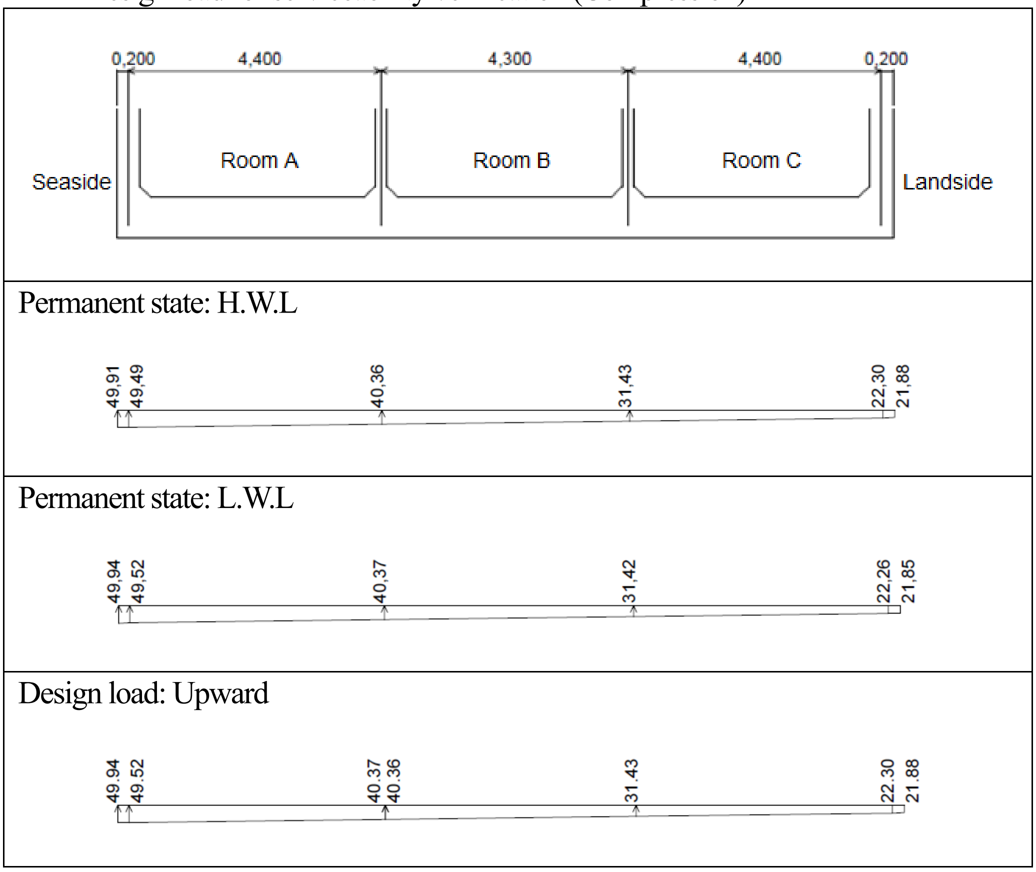


Figure 2.20- Summary of Design Load for Bottom Slab (Compression)

2) Outer Wall (Front Wall)

i) Floating condition

The design load is the hydrostatic pressure of draft + 1.0 m.

$S_f = (6.334 + 1.000 – 0.700 / 2) \times 10.10 = 70.54$ (kN/m^2)

- Safety (against cross-sectional failure)

$P = 1.1 \times S_f = 77.59$ (kN/m^2)

- Serviceability

$P = 0.5 \times S_f = 35.27$ (kN/m^2)

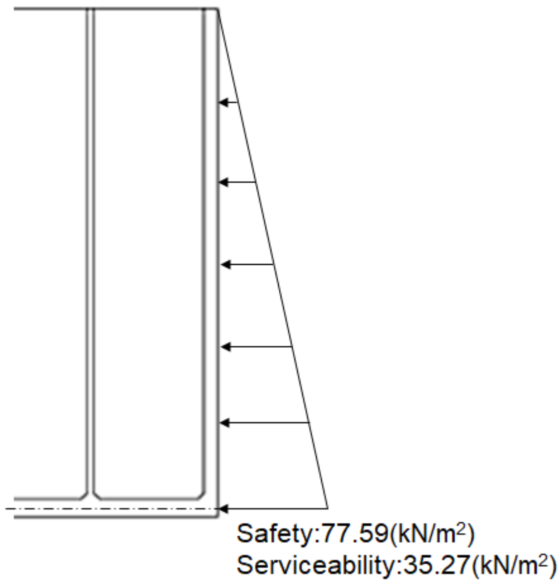


Figure 2.21- Design Load at Floating Condition

ii) After completion (Permanent state)

After completion, the internal earth pressure will increase to a height equal to the inside dimension of the wall and will not increase any deeper. The earth pressure coefficient of the filling sand is $K = 0.6$, and the unit volume weight is $\gamma' = 10.0$ (kN/m^3). The internal water pressure will take into account the water level difference from the bottom of the cover concrete to the LWL.

✓ Load calculation

- Internal earth pressure: D

P_1 = (Surcharge Load + Equipment Load + Cover concrete weight) × K

$P_2 = P_1$ + Inside dimension × Unit weight of infill sand × K

K : earth pressure coefficient at rest of the filling sand

$P_1 = (0.00 + 0.00 + 0.500 \times 22.60) \times 0.60 = 6.78$ (kN/m^2)

$P_2 = 6.78 + (4.100 \times 10.00) \times 0.60 = 31.38$ (kN/m^2)

$D = 1/2 \times (6.78 + 31.38) \times 4.100 + 31.38 \times 7.350 = 308.87$ (kN/m)

- Internal water pressure: S

The internal water pressure is estimated based on the head difference between the outside and inside of the caisson.

$P = 2.000 \times 10.10 = 20.20$ (kN/m^2)

$S = 1/2 \times 20.20 \times 2.000 + 20.20 \times 9.450 = 211.09$ (kN/m)

✓ Design load

The design load is calculated by converting the total load to a uniformly distributed load and a triangular distributed load so that the load area is equal to the total load.

- Safety (against cross-sectional failure)

$\sum P = 1.1 \times D + 1.1 \times S = 571.96$ (kN/m)

Trapezoidal load (Bottom)
$P_1 = 31.38 \times 1.1 + 20.20 \times 1.1 = 56.74$ (kN/m^2)

Trapezoidal load (Top)

$P_2 = (2 \times \sum P / \mathrm{L}) - P_1$
$= 2 \times 571.96 / 11.450 - 56.74 = 43.17$ (kN/m^2)

- Serviceability

$\sum P = 1.0 \times D + 1.0 \times S = 519.96$ (kN/m)

Trapezoidal load (Bottom)

$P_1 = 31.38 \times 1.0 + 20.20 \times 1.0 = 51.58$ (kN/m^2)

Trapezoidal load (Top)

$P_2 = (2 \times \sum P / \mathrm{L}) - P_1$
$= 2 \times 519.96 / 11.450 - 51.58 = 39.24$ (kN/m^2)

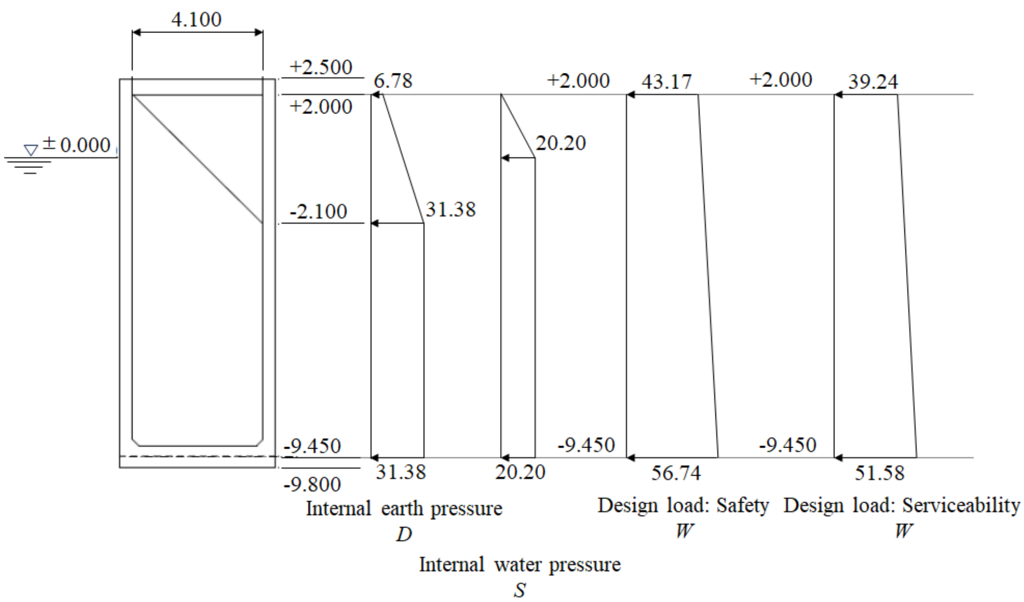


Figure 2.22- Design Load after Completion (Permanent State, Front wall)

iii) After completion (Variable situation of wave trough)

✓ Load calculation

- Internal earth pressure: *D*

$P_1 =$ (Surcharge Load + Equipment Load + Cover concrete weight) $\times K$
$P_2 = P_1 +$ Inside dimension $\times$ Unit weight of infill sand $\times K$
K : earth pressure coefficient at rest of the filling sand
$P_1 = (0.00 + 0.00 + 0.500 \times 22.60) \times 0.60 = 6.78$ (kN/m^2)

$P_2 = 6.78 + (4.100 \times 10.00) \times 0.60 = 31.38$ (kN/m^2)
$D = 1/2 \times (6.78 + 31.38) \times 4.100 + 31.38 \times 7.350 = 308.87$ (kN/m)

- Internal water pressure: S

The internal water pressure is estimated based on the head difference between the outside and inside of the caisson.
$P = 2.000 \times 10.10 = 20.20$ (kN/m^2)
$S = 1/2 \times 20.20 \times 2.000 + 20.20 \times 9.450 = 211.09$ (kN/m)

- Internal water pressure at wave trough (Safety): S'

The internal water pressure at wave trough is estimated based on the water level difference between the front of the wall (tide level $- H_{max}/3$) and the water level inside the caisson.
$P = (6.840 /3 + 2.000) \times 10.10 = 43.23$ (kN/m^2)
$S' = 1/2 \times 43.23 \times 4.280 + 43.23 \times 7.170 = 402.47$ (kN/m)

- Internal water pressure at wave trough (Serviceability): S'

$P = (3.800 /3 + 2.000) \times 10.10 = 33.00$ (kN/m^2)
$S' = 1/2 \times 33.00 \times 3.267 + 33.00 \times 8.183 = 323.94$ (kN/m)

- Variable component of internal water pressure (Safety): ΔS

$P = 43.23 – 20.20 = 23.03$ (kN/m^2)
$\Delta S = 402.47 – 211.09 = 191.38$ (kN/m)

- Variable component of internal water pressure (Serviceability): ΔS

$P = 33.00 – 20.20 = 12.80$ (kN/m^2)
$\Delta S = 323.94 – 211.09 = 112.85$ (kN/m)

✓ Design load

The design load is calculated by converting the total load to a uniformly distributed load and a triangular distributed load so that the load area is equal to the total load.

- Safety (against cross-sectional failure)

$\sum P = 1.1 \times D + 1.1 \times S + 1.2 \times \Delta S = 801.61$ (kN/m)

Trapezoidal load (Bottom)

$P_1 = 31.38 \times 1.1 + 20.20 \times 1.1 + 23.03 \times 1.2 = 84.37$ (kN/m^2)

Trapezoidal load (Top)

$P_2 = (2 \times \sum P / \mathrm{L}) - P_1$
$= 2 \times 801.61 / 11.450 – 84.37 = 55.65$ (kN/m^2)

- Serviceability

$\sum P = 1.0 \times D + 1.0 \times S + 1.0 \times \Delta S = 632.81$ (kN/m)

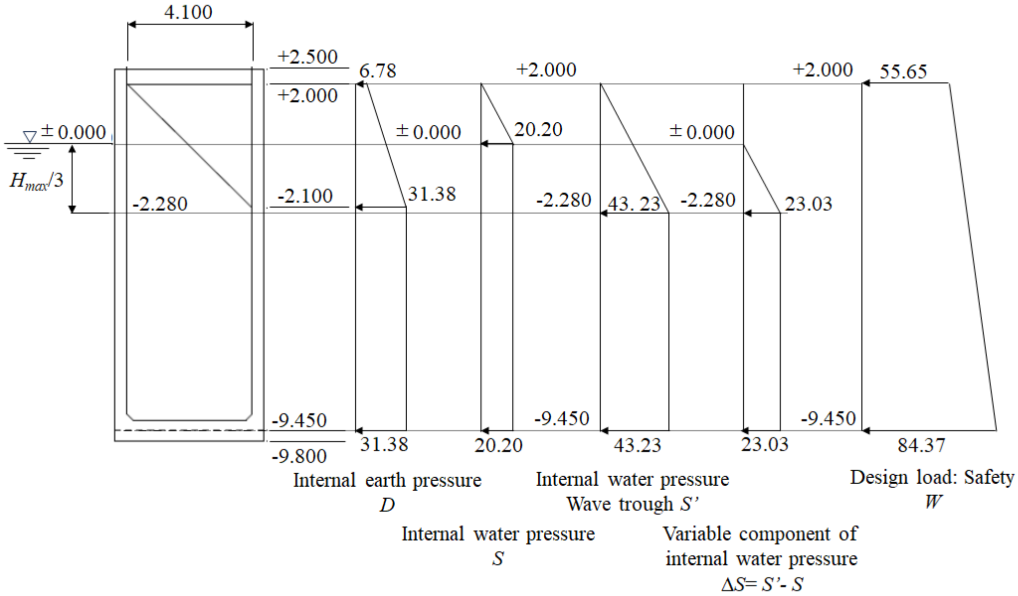
Trapezoidal load (Bottom)

$P_1 = 31.38 \times 1.0 + 20.20 \times 1.0 + 12.80 \times 1.0 = 64.38$ (kN/m^2)

Trapezoidal load (Top)

$$P_2 = (2 \times \sum P / L) - P_1$$
$$= 2 \times 632.81 / 11.450 - 64.38 = 46.15 \ (kN/m^2)$$

- Safety (against cross-sectional failure)



- Serviceability

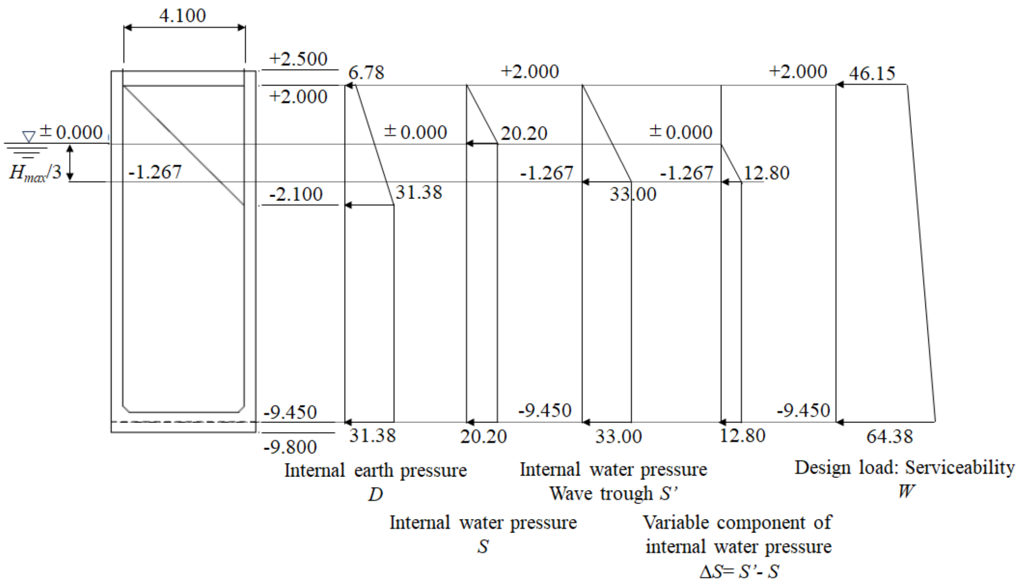


Figure 2.23- Design Load after Completion (Wave Trough, Front wall)

iv) After completion (Variable situation of wave crest, H.W.L)

✓ Load calculation

- Internal earth pressure: *D*

P_1 = (Surcharge Load + Equipment Load + Cover concrete weight) × K

$P_2 = P_1$ + Inside dimension × Unit weight of infill sand × K

K : earth pressure coefficient at rest of the filling sand

$P_1 = (0.00 + 0.00 + 0.500 \times 22.60) \times 0.60 = 6.78 \ (kN/m^2)$

$P_2 = 6.78 + (4.100 \times 10.00) \times 0.60 = 31.38 \ (kN/m^2)$

$D = 1/2 \times (6.78 + 31.38) \times 4.100 + 31.38 \times 7.350 = 308.87$ (kN/m)

- Wave force (Safety): H

$H_1 = 1/2 \times (61.84 + 48.34) \times 11.450 = 630.78$ (kN/m)
$H = 630.78$ (kN/m)

- Wave force (Serviceability): H

$H_1 = 1/2 \times (25.96 + 15.03) \times 11.450 = 234.67$ (kN/m)
$H = 234.67$ (kN/m)

✓ Design load

The design load is calculated by converting the total load to a uniformly distributed load and a triangular distributed load so that the load area is equal to the total load.

- Safety (against cross-sectional failure)

$\sum P = 1.2 \times H - 0.9 \times D = 478.95$ (kN/m)

Trapezoidal load (Top)

$P_1 = 1.2 \times 61.84 - 0.9 \times 6.78 = 68.10$ (kN/m^2)

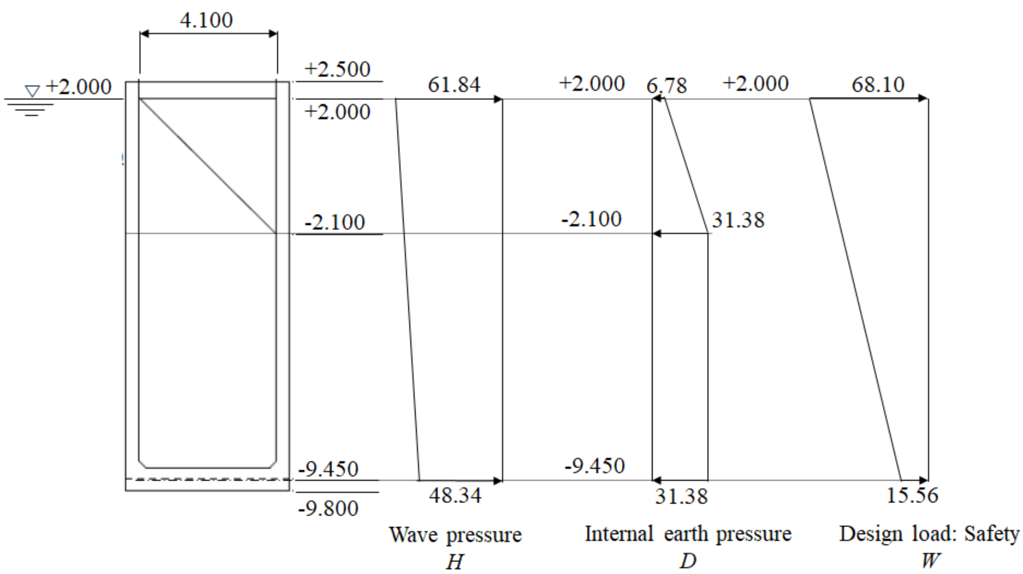
Trapezoidal load (Bottom)

$P_2 = (2 \times \sum P / \mathrm{L}) - P_1$
$= 2 \times 478.95 / 11.450 - 68.10 = 15.56$ (kN/m^2)

- Serviceability

$\sum P = 1.0 \times \mathrm{H} - 1.0 \times D =$ -74.20 (kN/m)
Since the total load is negative, the force from the seaside can be neglected.

- Safety (against cross-sectional failure)



- Serviceability: Not applicable

Figure 2.24- Design Load after Completion (Wave Crest, H.W.L, Front wall)

v) After completion (Variable situation of wave crest, L.W.L)

✓ Load calculation

- Internal earth pressure: D

P_1 = (Surcharge Load + Equipment Load + Cover concrete weight) × K
$P_2 = P_1$ + Inside dimension × Unit weight of infill sand × K
K : earth pressure coefficient at rest of the filling sand
$P_1 = (0.00 + 0.00 + 0.500 \times 22.60) \times 0.60 = 6.78\ (\mathrm{kN/m^2})$
$P_2 = 6.78 + (4.100 \times 10.00) \times 0.60 = 31.38\ (\mathrm{kN/m^2})$
$D = 1/2 \times (6.78 + 31.38) \times 4.100 + 31.38 \times 7.350 = 308.87\ (\mathrm{kN/m})$

- Wave force (Safety): H

$H_1 = 1/2 \times (50.19 + 62.34) \times 2.000 = 112.53\ (\mathrm{kN/m})$
$H_2 = 1/2 \times (62.34 + 51.04) \times 9.450 = 535.72\ (\mathrm{kN/m})$
$H = 648.25\ (\mathrm{kN/m})$

- Wave force (Serviceability): H

$H_1 = 1/2 \times (17.72 + 27.30) \times 2.000 = 45.02\ (\mathrm{kN/m})$
$H_2 = 1/2 \times (27.30 + 17.56) \times 9.450 = 211.96\ (\mathrm{kN/m})$
$H = 256.98\ (\mathrm{kN/m})$

✓ Design load

The design load is calculated by converting the total load to a uniformly distributed load and a triangular distributed load so that the load area is equal to the total load.

- Safety (against cross-sectional failure)

$\sum P = 1.2 \times H - 0.9 \times D = 499.92\ (\mathrm{kN/m})$

Trapezoidal load (Top)

$P_1 = 1.2 \times 50.19 - 0.9 \times 6.78 = 54.13\ (\mathrm{kN/m^2})$

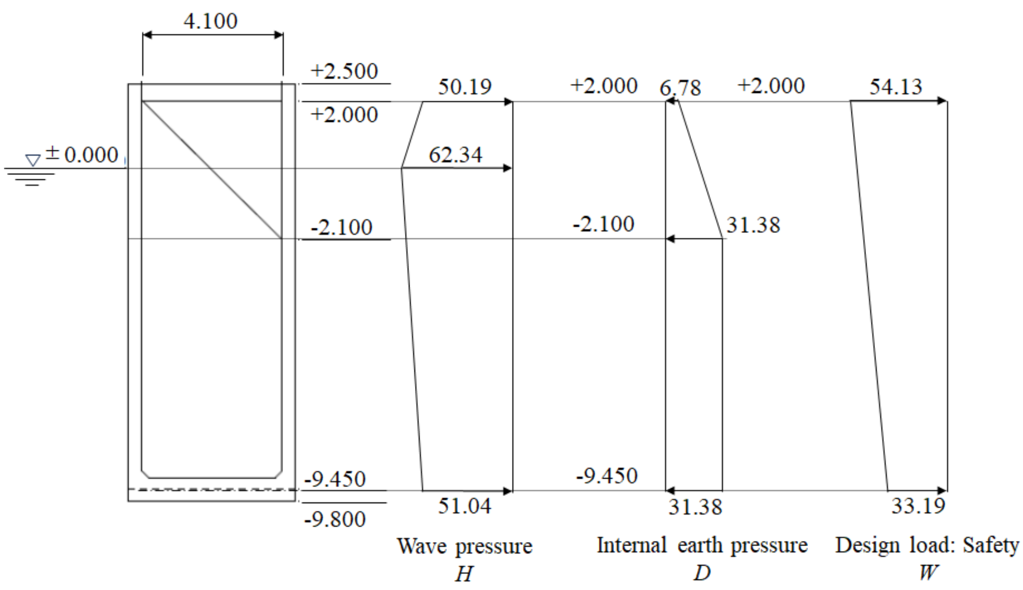
Trapezoidal load (Bottom)

$P_2 = (2 \times \sum P / \mathrm{L}) - P_1$
$= 2 \times 499.92 / 11.450 - 54.13 = 33.19\ (\mathrm{kN/m^2})$

- Serviceability

$\sum P = 1.0 \times \mathrm{H} - 1.0 \times D = -51.89\ (\mathrm{kN/m})$
Since the total load is negative, the force from the seaside can be neglected.

- Safety (against cross-sectional failure)



- Serviceability: Not applicable

Figure 2.25- Design Load after Completion (Wave Crest, L.W.L, Front wall)

(3) Sectional Force for Safety Verification

1) Numerical Tables for Slabs

Each numerical table of slabs supported on three sides and free on one side is based on a grid of points represented by x, y coordinates, in which the y-direction and x-direction correspond to the direction of the three side and the direction perpendicular to the free side, respectively. The side in the y-direction is divided into four equal parts, and in the x-direction is divided into six or eight equal parts in Figure 2.26 and 2.27.

Each numerical table of slabs supported on four sides is based on a grid of points represented by x, y coordinates, in which the x- and y-directions are determined as shown in Figure 2.28.

The bending moment at each grid point is determined by using the Equation (2.1) and (2.2).

When $\lambda \leq 1$

$$M_X = X \cdot q \cdot l_X^2$$
$$M_Y = Y \cdot q \cdot l_X^2 \tag{2.1}$$

When $\lambda > 1$

$$M_X = X \cdot q \cdot l_Y^2$$
$$M_Y = Y \cdot q \cdot l_Y^2 \tag{2.2}$$

Where:

λ	:	ratio of side lengths $\lambda = l_X / l_Y$
M_X, M_Y	:	bending moment in the *x*- or *y*-direction at a given point (kN·m/m)

X, Y	:	bending moment coefficient in the *x*- or *y*-direction at a given point
l_X, l_Y	:	length in the *x*- or *y*-direction (m)
q	:	load intensity for a uniformly distributed load or maximum load intensity for a triangularly distributed load (kN/m^2)

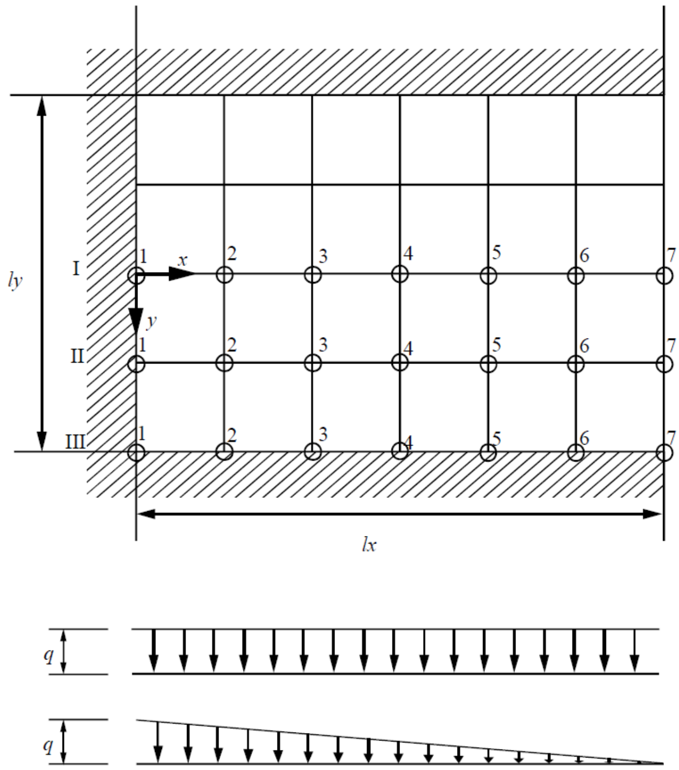


Figure 2.26- Slab Supported on Three Sides and Free on One Side (6 Equal Parts)

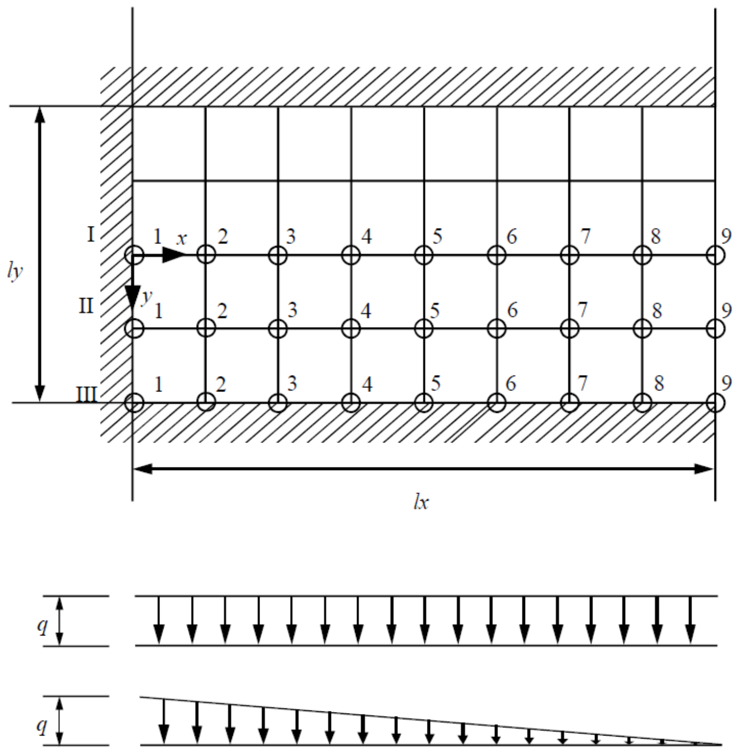


Figure 2.27- Slab Supported on Three Sides and Free on One Side (8 Equal Parts)

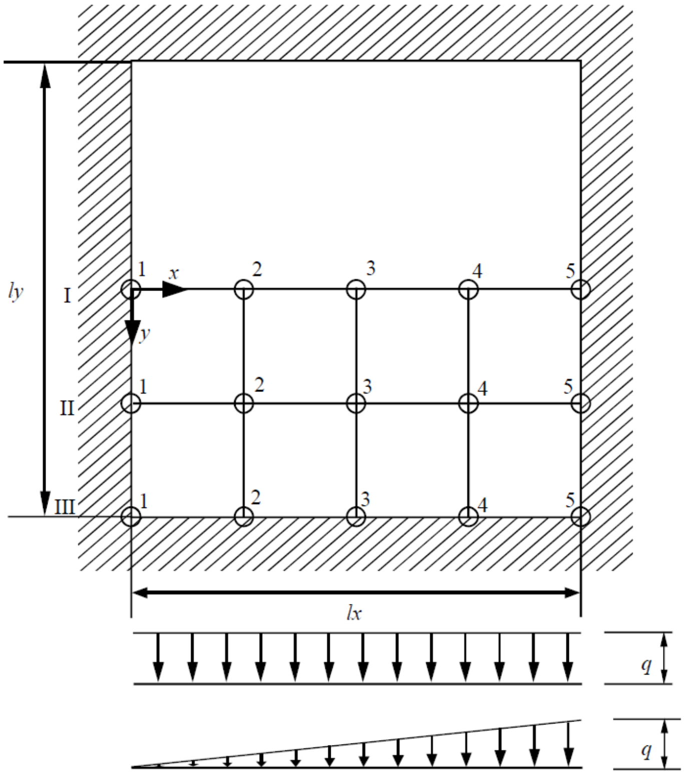


Figure 2.28- Slab Supported on Four Sides (4 Equal Parts)

2) Sectional Force of Bottom Slab

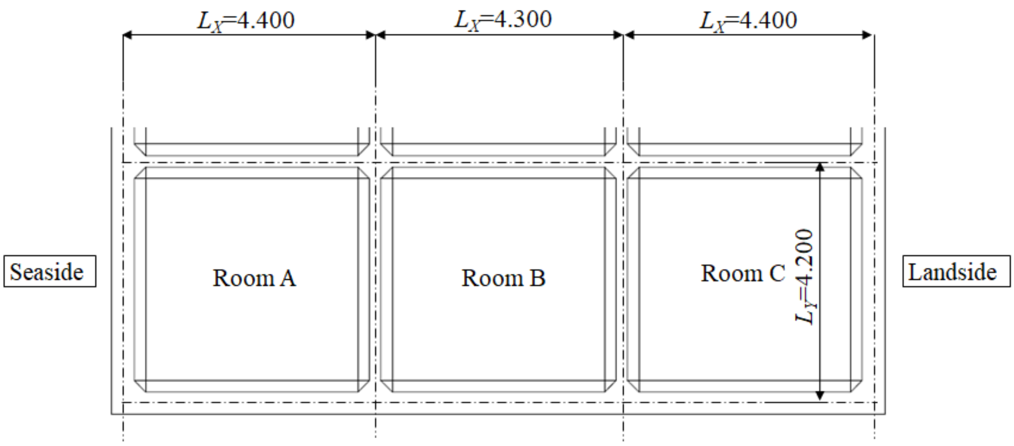


Figure 2.29- Design Span of Bottom Slab

The sectional force of bottom slab is estimated by the model of slabs supported on four sides according to constraint conditions and making calculations based on numerical tables. A positive bending moment is defined as causing tension on the upper side, while a negative bending moment causes tension on the lower side.

i) Room A: Variable situation, H.W.L, Wave trough (upward load)

$P_1 = 77.12$ (kN/m^2)

$P_2 = 139.28$ (kN/m^2)

$L_X = 4.400$ (m)

$L_Y = 4.200$ (m)

$\lambda = 4.400/4.200 = 1.05$, The coefficient table for $\lambda = 1.00$ is applied.

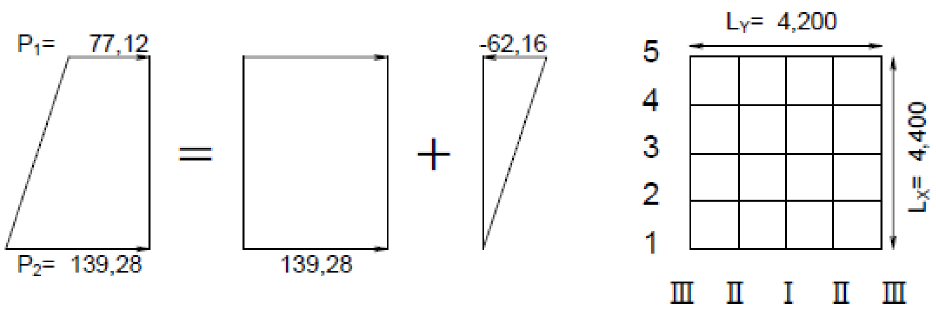


Figure 2.30- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$P = 139.28$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = 139.28 \times 4.200^2 \times X = 2{,}456.90 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = 139.28 \times 4.200^2 \times Y = 2{,}456.90 \times Y$ (kN·m)

Table 2.19- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0513	-126.04	-0.0086	-21.13
	4	0.0096	23.59	0.0116	28.50
	3	0.0206	50.61	0.0206	50.61
	2	0.0096	23.59	0.0116	28.50
	1	-0.0513	-126.04	-0.0086	-21.13
II	5	-0.0324	-79.60	-0.0054	-13.27
	4	0.0059	14.50	0.0059	14.50
	3	0.0116	28.50	0.0096	23.59
	2	0.0059	14.50	0.0059	14.50
	1	-0.0324	-79.60	-0.0054	-13.27
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0054	-13.27	-0.0324	-79.60
	3	-0.0086	-21.13	-0.0513	-126.04
	2	-0.0054	-13.27	-0.0324	-79.60
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$P = -62.16$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = -62.16 \times 4.200^2 \times X = -1{,}096.50 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = -62.16 \times 4.200^2 \times Y = -1{,}096.50 \times Y$ (kN·m)

Table 2.20- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0334	36.62	-0.0056	6.14
	4	0.0080	-8.77	0.0069	-7.57
	3	0.0103	-11.29	0.0103	-11.29
	2	0.0015	-1.64	0.0047	-5.15
	1	-0.0179	19.63	-0.0030	3.29
II	5	-0.0223	24.45	-0.0037	4.06

	4	0.0052	-5.70	0.0040	-4.39
	3	0.0058	-6.36	0.0048	-5.26
	2	0.0006	-0.66	0.0018	-1.97
	1	-0.0101	11.07	-0.0017	1.86
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0036	3.95	-0.0208	22.81
	3	-0.0043	4.71	-0.0257	28.18
	2	-0.0019	2.08	-0.0116	12.72
	1	0.0000	0.00	0.0000	0.00

- Combined uniformly distributed load and triangularly distributed load

Table 2.21- Summary of Sectional Force (Room A, Variable situation (upward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	-89.42	-14.99
	4	14.82	20.93
	3	39.32	39.32
	2	21.95	23.35
	1	-106.41	-17.84
II	5	-55.15	-9.21
	4	8.80	10.11
	3	22.14	18.33
	2	13.84	12.53
	1	-68.53	-11.41
III	5	0.00	0.00
	4	-9.32	-56.79
	3	-16.42	-97.86
	2	-11.19	-66.88
	1	0.00	0.00

ii) Room A: Variable situation, H.W.L, Wave crest (downward load)

P_1 = -122.55 (kN/m^2)

P_2 = -117.69 (kN/m^2)

L_X = 4.400 (m)

L_Y = 4.200 (m)

λ = 4.400/4.200 = 1.05, The coefficient table for λ =1.00 is applied.

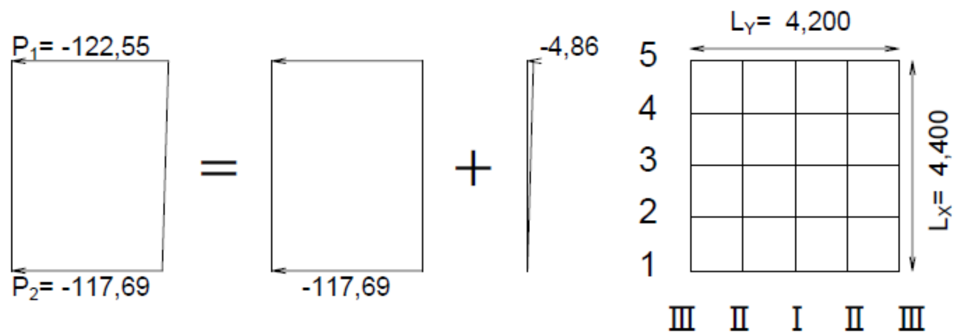


Figure 2.31- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

P = -117.69 (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X$ = -117.69 × 4.200^2 × X = -2,076.05 × X (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y$ = -117.69 × 4.200^2 × Y = -2,076.05 × Y (kN·m)

Table 2.22- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0513	106.50	-0.0086	17.85
	4	0.0096	-19.93	0.0116	-24.08
	3	0.0206	-42.77	0.0206	-42.77
	2	0.0096	-19.93	0.0116	-24.08
	1	-0.0513	106.50	-0.0086	17.85
II	5	-0.0324	67.26	-0.0054	11.21
	4	0.0059	-12.25	0.0059	-12.25
	3	0.0116	-24.08	0.0096	-19.93
	2	0.0059	-12.25	0.0059	-12.25
	1	-0.0324	67.26	-0.0054	11.21
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0054	11.21	-0.0324	67.26
	3	-0.0086	17.85	-0.0513	106.50
	2	-0.0054	11.21	-0.0324	67.26
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

P = -4.86 (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X$ = -4.86 × 4.200^2 × X = -85.73 × X (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y$ = -4.86 × 4.200^2 × Y = -85.73 × Y (kN·m)

Table 2.23- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0334	2.86	-0.0056	0.48
	4	0.0080	-0.69	0.0069	-0.59
	3	0.0103	-0.88	0.0103	-0.88
	2	0.0015	-0.13	0.0047	-0.40
	1	-0.0179	1.53	-0.0030	0.26
II	5	-0.0223	1.91	-0.0037	0.32
	4	0.0052	-0.45	0.0040	-0.34
	3	0.0058	-0.50	0.0048	-0.41
	2	0.0006	-0.05	0.0018	-0.15
	1	-0.0101	0.87	-0.0017	0.15
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0036	0.31	-0.0208	1.78
	3	-0.0043	0.37	-0.0257	2.20
	2	-0.0019	0.16	-0.0116	0.99
	1	0.0000	0.00	0.0000	0.00

- Combined uniformly distributed load and triangularly distributed load

Table 2.24- Summary of Sectional Force (Room A, Variable situation (downward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	109.36	18.33
	4	-20.62	-24.67
	3	-43.65	-43.65
	2	-20.06	-24.48
	1	108.03	18.11
II	5	69.17	11.53
	4	-12.70	-12.59
	3	-24.58	-20.34
	2	-12.30	-12.40
	1	68.13	11.36
III	5	0.00	0.00
	4	11.52	69.04
	3	18.22	108.70
	2	11.37	68.25
	1	0.00	0.00

iii) Room B: Permanent state, H.W.L (upward load)

$P_1 = 84.59$ (kN/m^2)

$P_2 = 94.42$ (kN/m^2)

$L_X = 4.300$ (m)

$L_Y = 4.200$ (m)

$\lambda = 4.300/4.200 = 1.02$, The coefficient table for $\lambda = 1.00$ is applied.

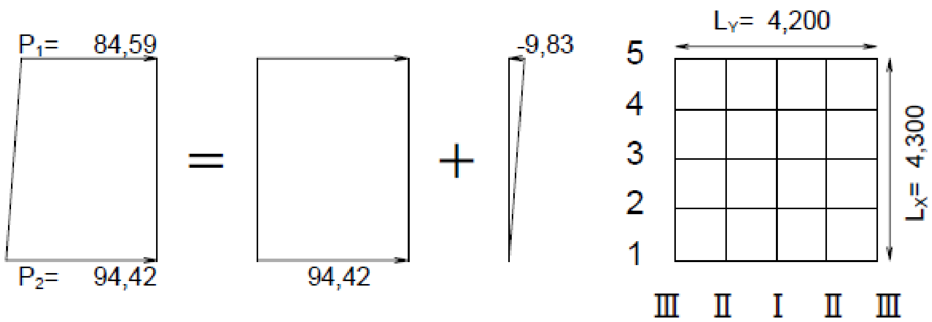


Figure 2.32- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$P = 94.42$ (kN/m^2)

$M_X = P \cdot L_Y^2 \cdot X = 94.42 \times 4.200^2 \times X = 1{,}665.57 \times X$ (kN·m)

$M_Y = P \cdot L_Y^2 \cdot Y = 94.42 \times 4.200^2 \times Y = 1{,}665.57 \times Y$ (kN·m)

Table 2.25- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0513	-85.44	-0.0086	-14.32
	4	0.0096	15.99	0.0116	19.32
	3	0.0206	34.31	0.0206	34.31

	2	0.0096	15.99	0.0116	19.32
	1	-0.0513	-85.44	-0.0086	-14.32
II	5	-0.0324	-53.96	-0.0054	-8.99
	4	0.0059	9.83	0.0059	9.83
	3	0.0116	19.32	0.0096	15.99
	2	0.0059	9.83	0.0059	9.83
	1	-0.0324	-53.96	-0.0054	-8.99
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0054	-8.99	-0.0324	-53.96
	3	-0.0086	-14.32	-0.0513	-85.44
	2	-0.0054	-8.99	-0.0324	-53.96
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$P = -9.83$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = -9.83 \times 4.200^2 \times X = -173.40 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = -9.83 \times 4.200^2 \times Y = -173.40 \times Y$ (kN·m)

Table 2.26- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0334	5.79	-0.0056	0.97
	4	0.0080	-1.39	0.0069	-1.20
	3	0.0103	-1.79	0.0103	-1.79
	2	0.0015	-0.26	0.0047	-0.81
	1	-0.0179	3.10	-0.0030	0.52
II	5	-0.0223	3.87	-0.0037	0.64
	4	0.0052	-0.90	0.0040	-0.69
	3	0.0058	-1.01	0.0048	-0.83
	2	0.0006	-0.10	0.0018	-0.31
	1	-0.0101	1.75	-0.0017	0.29
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0036	0.62	-0.0208	3.61
	3	-0.0043	0.75	-0.0257	4.46
	2	-0.0019	0.33	-0.0116	2.01
	1	0.0000	0.00	0.0000	0.00

- Combined uniformly distributed load and triangularly distributed load

Table 2.27- Summary of Sectional Force (Room B, Permanent state (upward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	-79.65	-13.35
	4	14.60	18.12
	3	32.52	32.52
	2	15.73	18.51
	1	-82.34	-13.80
II	5	-50.09	-8.35
	4	8.93	9.14
	3	18.31	15.16

	2	9.73	9.52
	1	-52.21	-8.70
III	5	0.00	0.00
	4	-8.37	-50.35
	3	-13.57	-80.98
	2	-8.66	-51.95
	1	0.00	0.00

iv) Room B: Variable situation, H.W.L, Wave crest (downward load)

$P_1 = 0.00$ (kN/m^2)

$P_2 = -79.86$ (kN/m^2)

$L_X = 4.300$ (m)

$L_Y = 4.200$ (m)

$\lambda = 4.300/4.200 = 1.02$, The coefficient table for $\lambda = 1.00$ is applied.

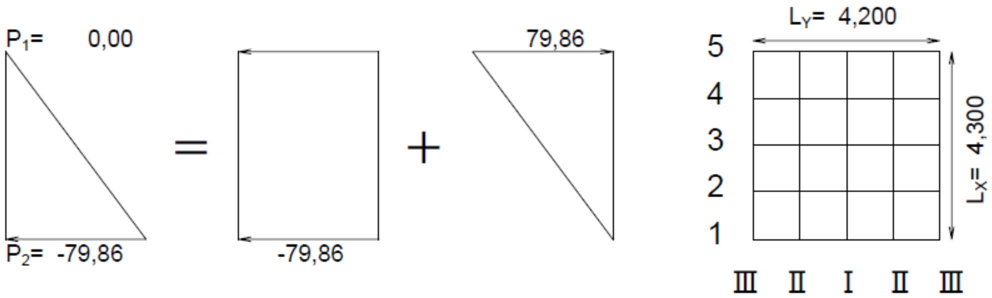


Figure 2.33- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$P = -79.86$ (kN/m^2)

$M_X = P \cdot L_Y^2 \cdot X = -79.86 \times 4.200^2 \times X = -1{,}408.73 \times X$ (kN·m)

$M_Y = P \cdot L_Y^2 \cdot Y = -79.86 \times 4.200^2 \times Y = -1{,}408.73 \times Y$ (kN·m)

Table 2.28- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0513	72.27	-0.0086	12.12
	4	0.0096	-13.52	0.0116	-16.34
	3	0.0206	-29.02	0.0206	-29.02
	2	0.0096	-13.52	0.0116	-16.34
	1	-0.0513	72.27	-0.0086	12.12
II	5	-0.0324	45.64	-0.0054	7.61
	4	0.0059	-8.31	0.0059	-8.31
	3	0.0116	-16.34	0.0096	-13.52
	2	0.0059	-8.31	0.0059	-8.31
	1	-0.0324	45.64	-0.0054	7.61
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0054	7.61	-0.0324	45.64
	3	-0.0086	12.12	-0.0513	72.27
	2	-0.0054	7.61	-0.0324	45.64
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$P = 79.86$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = 79.86 \times 4.200^2 \times X = 1{,}408.73 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = 79.86 \times 4.200^2 \times Y = 1{,}408.73 \times Y$ (kN·m)

Table 2.29- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0334	-47.05	-0.0056	-7.89
	4	0.0080	11.27	0.0069	9.72
	3	0.0103	14.51	0.0103	14.51
	2	0.0015	2.11	0.0047	6.62
	1	-0.0179	-25.22	-0.0030	-4.23
II	5	-0.0223	-31.41	-0.0037	-5.21
	4	0.0052	7.33	0.0040	5.63
	3	0.0058	8.17	0.0048	6.76
	2	0.0006	0.85	0.0018	2.54
	1	-0.0101	-14.23	-0.0017	-2.39
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0036	-5.07	-0.0208	-29.30
	3	-0.0043	-6.06	-0.0257	-36.20
	2	-0.0019	-2.68	-0.0116	-16.34
	1	0.0000	0.00	0.0000	0.00

- Combined uniformly distributed load and triangularly distributed load

Table 2.30- Summary of Sectional Force (Room B Variable situation (downward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	25.22	4.23
	4	-2.25	-6.62
	3	-14.51	-14.51
	2	-11.41	-9.72
	1	47.05	7.89
II	5	14.23	2.40
	4	-0.98	-2.68
	3	-8.17	-6.76
	2	-7.46	-5.77
	1	31.41	5.22
III	5	0.00	0.00
	4	2.54	16.34
	3	6.06	36.07
	2	4.93	29.30
	1	0.00	0.00

v) Room C: Variable situation, H.W.L, Wave crest (upward load)
$P_1 = 280.31$ (kN/m^2)
$P_2 = 108.37$ (kN/m^2)
$L_X = 4.400$ (m)

$L_Y = 4.200$ (m)
$\lambda = 4.400/4.200 = 1.05$, The coefficient table for $\lambda = 1.00$ is applied.

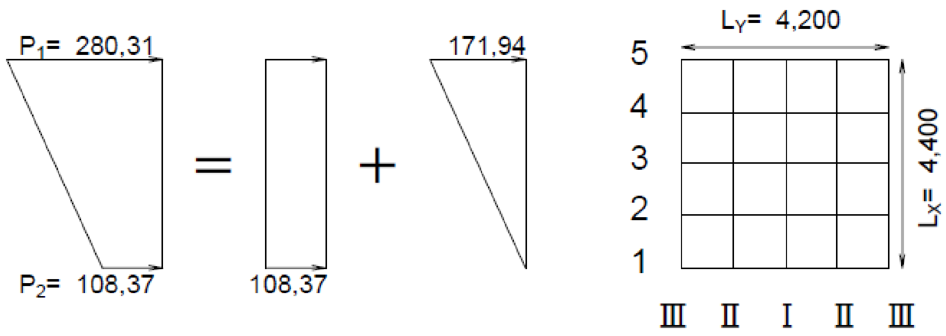


Figure 2.34- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$P = 108.37$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = 108.37 \times 4.200^2 \times X = 1{,}911.65 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = 108.37 \times 4.200^2 \times Y = 1{,}911.65 \times Y$ (kN·m)

Table 2.31- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0513	-98.07	-0.0086	-16.44
	4	0.0096	18.35	0.0116	22.18
	3	0.0206	39.38	0.0206	39.38
	2	0.0096	18.35	0.0116	22.18
	1	-0.0513	-98.07	-0.0086	-16.44
II	5	-0.0324	-61.94	-0.0054	-10.32
	4	0.0059	11.28	0.0059	11.28
	3	0.0116	22.18	0.0096	18.35
	2	0.0059	11.28	0.0059	11.28
	1	-0.0324	-61.94	-0.0054	-10.32
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0054	-10.32	-0.0324	-61.94
	3	-0.0086	-16.44	-0.0513	-98.07
	2	-0.0054	-10.32	-0.0324	-61.94
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$P = 171.94$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = 171.94 \times 4.200^2 \times X = 3{,}033.02 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = 171.94 \times 4.200^2 \times Y = 3{,}033.02 \times Y$ (kN·m)

Table 2.32- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0334	-101.30	-0.0056	-16.98
	4	0.0080	24.26	0.0069	20.93
	3	0.0103	31.24	0.0103	31.24
	2	0.0015	4.55	0.0047	14.26

	1	-0.0179	-54.29	-0.0030	-9.10
II	5	-0.0223	-67.64	-0.0037	-11.22
	4	0.0052	15.77	0.0040	12.13
	3	0.0058	17.59	0.0048	14.56
	2	0.0006	1.82	0.0018	5.46
	1	-0.0101	-30.63	-0.0017	-5.16
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0036	-10.92	-0.0208	-63.09
	3	-0.0043	-13.04	-0.0257	-77.95
	2	-0.0019	-5.76	-0.0116	-35.18
	1	0.0000	0.00	0.0000	0.00

- Combined uniformly distributed load and triangularly distributed load

Table 2.33- Summary of Sectional Force (Room C Variable situation (upward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	-199.37	-33.42
	4	42.61	43.11
	3	70.62	70.62
	2	22.90	36.44
	1	-152.36	-25.54
II	5	-129.58	-21.54
	4	27.05	23.41
	3	39.77	32.91
	2	13.10	16.74
	1	-92.57	-15.48
III	5	0.00	0.00
	4	-21.24	-125.03
	3	-29.48	-176.02
	2	-16.08	-97.12
	1	0.00	0.00

vi) Room C: Variable situation, H.W.L, Wave trough (downward load)

P_1 = -50.44 (kN/m^2)
P_2 = 0.00 (kN/m^2)
L_X = 4.400 (m)
L_Y = 4.200 (m)
λ = 4.400/4.200 = 1.05, The coefficient table for λ =1.00 is applied.

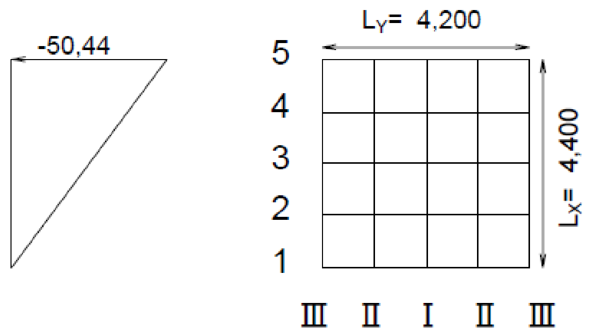


Figure 2.35- Design Model for Sectional Force Estimation

- Sectional forces induced by triangularly distributed load

P = -50.44 (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X$ = -50.44 × 4.200^2 × X = -889.76 × X (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y$ = -50.44 × 4.200^2 × Y = -889.76 × Y (kN·m)

Table 2.34- Summary of Sectional Force (Room C Variable situation (downward load))

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0334	29.72	-0.0056	4.98
	4	0.0080	-7.12	0.0069	-6.14
	3	0.0103	-9.16	0.0103	-9.16
	2	0.0015	-1.33	0.0047	-4.18
	1	-0.0179	15.93	-0.0030	2.67
II	5	-0.0223	19.84	-0.0037	3.29
	4	0.0052	-4.63	0.0040	-3.56
	3	0.0058	-5.16	0.0048	-4.27
	2	0.0006	-0.53	0.0018	-1.60
	1	-0.0101	8.99	-0.0017	1.51
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0036	3.20	-0.0208	18.51
	3	-0.0043	3.83	-0.0257	22.87
	2	-0.0019	1.69	-0.0116	10.32
	1	0.0000	0.00	0.0000	0.00

3) Sectional Force of Outer Wall (Front wall)

The sectional force of outer wall is estimated by the model of slabs supported on three sides or four sides according to constraint conditions and making calculations based on numerical tables. A positive bending moment is defined as causing tension on the inner side, while a negative bending moment causes tension on the outer side.

i) Floating condition

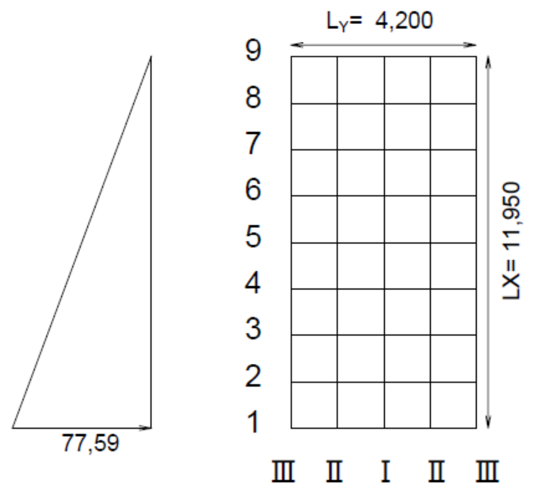


Figure 2.36- Design Model for Sectional Force Estimation

P_1 = 0.00 (kN/m^2)
P_2 = 77.59 (kN/m^2)

L_X = 11.950 (m)
L_Y = 4.200 (m)
λ = 11.950/4.200 = 2.85, The coefficient table for λ =2.75 is applied.

- Sectional forces induced by triangularly distributed load

P = 77.59 (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X$ = 77.59 × 4.200^2 × X = 1,368.69 × X (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y$ = 77.59 × 4.200^2 × Y = 1,368.69 × Y (kN·m)

Table 2.35- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0034	4.65
	8	0.0002	0.27	0.0060	8.21
	7	0.0015	2.05	0.0105	14.37
	6	0.0026	3.56	0.0157	21.49
	5	0.0037	5.06	0.0209	28.61
	4	0.0057	7.80	0.0257	35.18
	3	0.0101	13.82	0.0272	37.23
	2	0.0108	14.78	0.0173	23.68
	1	-0.0486	-66.52	-0.0081	-11.09
II	9	0.0000	0.00	0.0003	0.41
	8	-0.0002	-0.27	0.0013	1.78
	7	0.0003	0.41	0.0026	3.56
	6	0.0006	0.82	0.0039	5.34
	5	0.0009	1.23	0.0053	7.25
	4	0.0019	2.60	0.0068	9.31
	3	0.0045	6.16	0.0081	11.09
	2	0.0060	8.21	0.0068	9.31
	1	-0.0299	-40.92	-0.0050	-6.84
III	9	0.0000	0.00	-0.0021	-2.87
	8	-0.0019	-2.60	-0.0117	-16.01
	7	-0.0037	-5.06	-0.0220	-30.11
	6	-0.0054	-7.39	-0.0322	-44.07
	5	-0.0071	-9.72	-0.0427	-58.44
	4	-0.0086	-11.77	-0.0515	-70.49
	3	-0.0093	-12.73	-0.0555	-75.96
	2	-0.0068	-9.31	-0.0409	-55.98
	1	0.0000	0.00	0.0000	0.00

ii) Permanent state
P_1 = -43.17 (kN/m^2)
P_2 = -56.74 (kN/m^2)
L_X = 11.950 (m)
L_Y = 4.200 (m)
λ = 11.950/4.200 = 2.85, The coefficient table for λ =2.75 is applied.

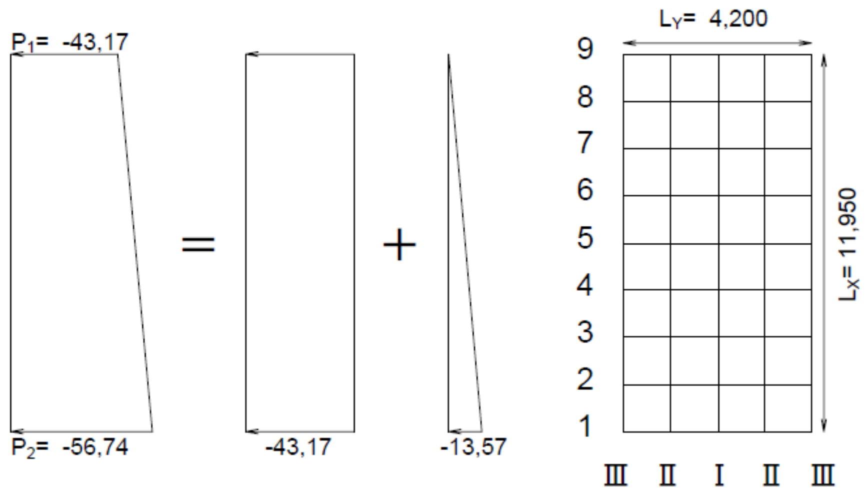


Figure 2.37- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$P = -43.17$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = -43.17 \times 4.200^2 \times X = -761.52 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = -43.17 \times 4.200^2 \times Y = -761.52 \times Y$ (kN·m)

Table 2.36- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0432	-32.90
	8	0.0061	-4.65	0.0414	-31.53
	7	0.0069	-5.25	0.0415	-31.60
	6	0.0070	-5.33	0.0417	-31.76
	5	0.0072	-5.48	0.0418	-31.83
	4	0.0084	-6.40	0.0413	-31.45
	3	0.0121	-9.21	0.0373	-28.40
	2	0.0110	-8.38	0.0213	-16.22
	1	-0.0566	43.10	-0.0094	7.16
II	9	0.0000	0.00	0.0105	-8.00
	8	0.0012	-0.91	0.0103	-7.84
	7	0.0018	-1.37	0.0104	-7.92
	6	0.0018	-1.37	0.0104	-7.92
	5	0.0018	-1.37	0.0105	-8.00
	4	0.0025	-1.90	0.0107	-8.15
	3	0.0050	-3.81	0.0107	-8.15
	2	0.0059	-4.49	0.0077	-5.86
	1	-0.0341	25.97	-0.0057	4.34
III	9	0.0000	0.00	-0.0863	65.72
	8	-0.0139	10.59	-0.0836	63.66
	7	-0.0137	10.43	-0.0820	62.44
	6	-0.0138	10.51	-0.0827	62.98
	5	-0.0141	10.74	-0.0845	64.35
	4	-0.0140	10.66	-0.0839	63.89
	3	-0.0127	9.67	-0.0763	58.10
	2	-0.0080	6.09	-0.0480	36.55
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

P = -13.57 (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X$ = -13.57 × 4.200^2 × X = -239.37 × X (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y$ = -13.57 × 4.200^2 × Y = -239.37 × Y (kN·m)

Table 2.37- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0034	-0.81
	8	0.0002	-0.05	0.0060	-1.44
	7	0.0015	-0.36	0.0105	-2.51
	6	0.0026	-0.62	0.0157	-3.76
	5	0.0037	-0.89	0.0209	-5.00
	4	0.0057	-1.36	0.0257	-6.15
	3	0.0101	-2.42	0.0272	-6.51
	2	0.0108	-2.59	0.0173	-4.14
	1	-0.0486	11.63	-0.0081	1.94
II	9	0.0000	0.00	0.0003	-0.07
	8	-0.0002	0.05	0.0013	-0.31
	7	0.0003	-0.07	0.0026	-0.62
	6	0.0006	-0.14	0.0039	-0.93
	5	0.0009	-0.22	0.0053	-1.27
	4	0.0019	-0.45	0.0068	-1.63
	3	0.0045	-1.08	0.0081	-1.94
	2	0.0060	-1.44	0.0068	-1.63
	1	-0.0299	7.16	-0.0050	1.20
III	9	0.0000	0.00	-0.0021	0.50
	8	-0.0019	0.45	-0.0117	2.80
	7	-0.0037	0.89	-0.0220	5.27
	6	-0.0054	1.29	-0.0322	7.71
	5	-0.0071	1.70	-0.0427	10.22
	4	-0.0086	2.06	-0.0515	12.33
	3	-0.0093	2.23	-0.0555	13.29
	2	-0.0068	1.63	-0.0409	9.79
	1	0.0000	0.00	0.0000	0.00

Table 2.38- Summary of Sectional Force (Front wall, Permanent State)

		M_X (kN·m)	M_Y (kN·m)
I	9	0.00	-33.71
	8	-4.70	-32.97
	7	-5.61	-34.11
	6	-5.95	-35.52
	5	-6.37	-36.83
	4	-7.76	-37.60
	3	-11.63	-34.91
	2	-10.97	-20.36
	1	54.73	9.10
II	9	0.00	-8.07
	8	-0.86	-8.15

	7	-1.44	-8.54
	6	-1.51	-8.85
	5	-1.59	-9.27
	4	-2.35	-9.78
	3	-4.89	-10.09
	2	-5.93	-7.49
	1	33.13	5.54
III	9	0.00	66.22
	8	11.04	66.46
	7	11.32	67.71
	6	11.80	70.69
	5	12.44	74.57
	4	12.72	76.22
	3	11.90	71.39
	2	7.72	46.34
	1	0.00	0.00

iii) Variable situation (Wave crest, L.W.L)

Based on the calculation results for both the slabs supported on three-sides and four-sides, the larger bending moment is adopted at each grid point. The coefficient for the four-sides fixed plate is calculated by dividing the coefficient for the 1/4 plate by 1/8.

$P_1 = 54.13$ (kN/m^2)
$P_2 = 33.19$ (kN/m^2)
$L_X = 11.950$ (m)
$L_Y = 4.200$ (m)
$\lambda = 11.950/4.200 = 2.85$, The coefficient table for $\lambda = 2.75$ is applied.

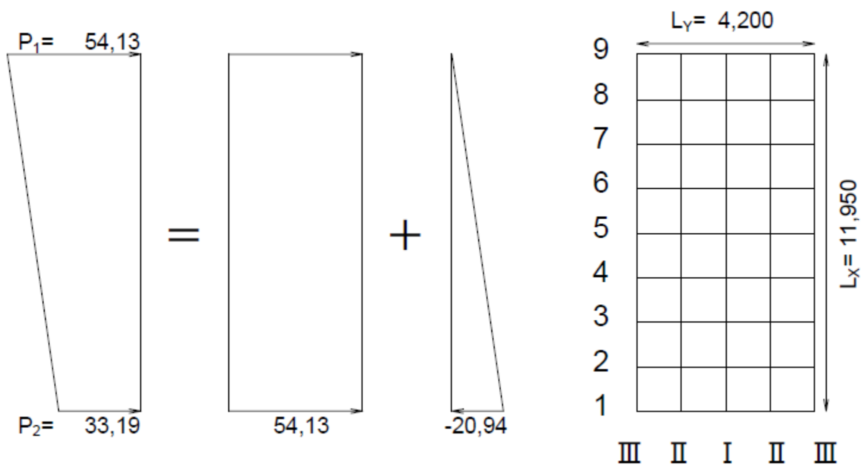


Figure 2.38- Design Model for Sectional Force Estimation (Supported on Three-sides)

- Sectional forces induced by uniformly distributed load

$P = 54.13$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = 54.13 \times 4.200^2 \times X = 954.85 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = 54.13 \times 4.200^2 \times Y = 954.85 \times Y$ (kN·m)

Table 2.39- Coefficient and Sectional Force (Supported on Three-sides)

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0432	41.25
	8	0.0061	5.82	0.0414	39.53
	7	0.0069	6.59	0.0415	39.63
	6	0.0070	6.68	0.0417	39.82
	5	0.0072	6.87	0.0418	39.91
	4	0.0084	8.02	0.0413	39.44
	3	0.0121	11.55	0.0373	35.62
	2	0.0110	10.50	0.0213	20.34
	1	-0.0566	-54.04	-0.0094	-8.98
II	9	0.0000	0.00	0.0105	10.03
	8	0.0012	1.15	0.0103	9.83
	7	0.0018	1.72	0.0104	9.93
	6	0.0018	1.72	0.0104	9.93
	5	0.0018	1.72	0.0105	10.03
	4	0.0025	2.39	0.0107	10.22
	3	0.0050	4.77	0.0107	10.22
	2	0.0059	5.63	0.0077	7.35
	1	-0.0341	-32.56	-0.0057	-5.44
III	9	0.0000	0.00	-0.0863	-82.40
	8	-0.0139	-13.27	-0.0836	-79.83
	7	-0.0137	-13.08	-0.0820	-78.30
	6	-0.0138	-13.18	-0.0827	-78.97
	5	-0.0141	-13.46	-0.0845	-80.68
	4	-0.0140	-13.37	-0.0839	-80.11
	3	-0.0127	-12.13	-0.0763	-72.66
	2	-0.0080	-7.64	-0.0480	-45.83
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$P = -20.94$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = -20.94 \times 4.200^2 \times X = -369.38 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = -20.94 \times 4.200^2 \times Y = -369.38 \times Y$ (kN·m)

Table 2.40- Coefficient and Sectional Force (Supported on Three-sides)

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0034	-1.26
	8	0.0002	-0.07	0.0060	-2.22
	7	0.0015	-0.55	0.0105	-3.88
	6	0.0026	-0.96	0.0157	-5.80
	5	0.0037	-1.37	0.0209	-7.72
	4	0.0057	-2.11	0.0257	-9.49
	3	0.0101	-3.73	0.0272	-10.05
	2	0.0108	-3.99	0.0173	-6.39
	1	-0.0486	17.95	-0.0081	2.99
II	9	0.0000	0.00	0.0003	-0.11
	8	-0.0002	0.07	0.0013	-0.48

	7	0.0003	-0.11	0.0026	-0.96
	6	0.0006	-0.22	0.0039	-1.44
	5	0.0009	-0.33	0.0053	-1.96
	4	0.0019	-0.70	0.0068	-2.51
	3	0.0045	-1.66	0.0081	-2.99
	2	0.0060	-2.22	0.0068	-2.51
	1	-0.0299	11.04	-0.0050	1.85
III	9	0.0000	0.00	-0.0021	0.78
	8	-0.0019	0.70	-0.0117	4.32
	7	-0.0037	1.37	-0.0220	8.13
	6	-0.0054	1.99	-0.0322	11.89
	5	-0.0071	2.62	-0.0427	15.77
	4	-0.0086	3.18	-0.0515	19.02
	3	-0.0093	3.44	-0.0555	20.50
	2	-0.0068	2.51	-0.0409	15.11
	1	0.0000	0.00	0.0000	0.00

Table 2.41 Summary of Sectional Force (Front wall, Wave Crest, L.W.L, Supported on Three-sides)

		M_X (kN·m)	M_Y (kN·m)
I	9	0.00	39.99
	8	5.75	37.31
	7	6.04	35.75
	6	5.72	34.02
	5	5.50	32.19
	4	5.91	29.95
	3	7.82	25.57
	2	6.51	13.95
	1	-36.09	-5.99
II	9	0.00	9.92
	8	1.22	9.35
	7	1.61	8.97
	6	1.50	8.49
	5	1.39	8.07
	4	1.69	7.71
	3	3.11	7.23
	2	3.41	4.84
	1	-21.52	-3.59
III	9	0.00	-81.62
	8	-12.57	-75.51
	7	-11.71	-70.17
	6	-11.19	-67.08
	5	-10.89	-64.91
	4	-10.19	-61.09
	3	-8.69	-52.36
	2	-5.13	-30.72
	1	0.00	0.00

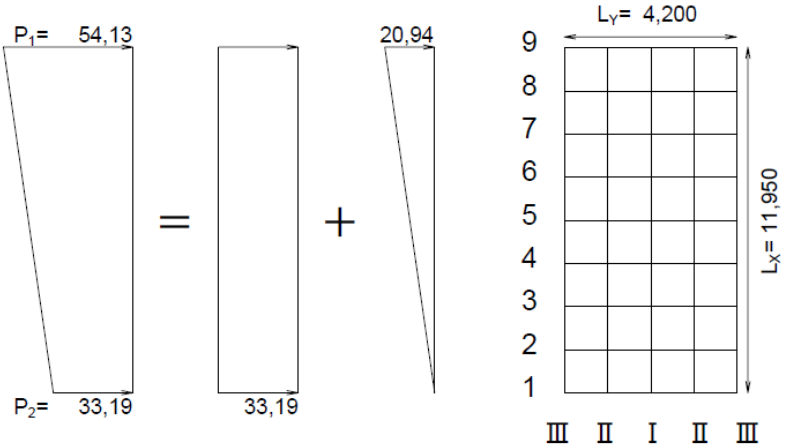


Figure 2.39- Design Model for Sectional Force Estimation (Supported on Four-sides)

- Sectional forces induced by uniformly distributed load

$P = 33.19$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = 33.19 \times 4.200^2 \times X = 585.47 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = 33.19 \times 4.200^2 \times Y = 585.47 \times Y$ (kN·m)

Table 2.42- Coefficient and Sectional Force (Supported on Four-sides)

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	-0.0568	-33.25	-0.0095	-5.56
	8	-0.0223	-13.06	0.0139	8.14
	7	0.0122	7.14	0.0373	21.84
	6	0.0098	5.74	0.0396	23.18
	5	0.0074	4.33	0.0419	24.53
	4	0.0098	5.74	0.0396	23.18
	3	0.0122	7.14	0.0373	21.84
	2	-0.0223	-13.06	0.0139	8.14
	1	-0.0568	-33.25	-0.0095	-5.56
II	9	-0.0344	-20.14	-0.0058	-3.40
	8	-0.0147	-8.61	0.0025	1.46
	7	0.0051	2.99	0.0108	6.32
	6	0.0036	2.11	0.0107	6.26
	5	0.0020	1.17	0.0106	6.21
	4	0.0036	2.11	0.0107	6.26
	3	0.0051	2.99	0.0108	6.32
	2	-0.0147	-8.61	0.0025	1.46
	1	-0.0344	-20.14	-0.0058	-3.40
III	9	0.0000	0.00	0.0000	0.00
	8	-0.0065	-3.81	-0.0388	-22.72
	7	-0.0130	-7.61	-0.0775	-45.37
	6	-0.0135	-7.90	-0.0807	-47.25
	5	-0.0140	-8.20	-0.0839	-49.12
	4	-0.0135	-7.90	-0.0807	-47.25
	3	-0.0130	-7.61	-0.0775	-45.37
	2	-0.0065	-3.81	-0.0388	-22.72
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$P = 20.94$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = 20.94 \times 4.200^2 \times X = 369.38 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = 20.94 \times 4.200^2 \times Y = 369.38 \times Y$ (kN·m)

Table 2.43- Coefficient and Sectional Force (Supported on Four-sides)

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	-0.0488	-18.03	-0.0081	-2.99
	8	-0.0193	-7.13	0.0096	3.55
	7	0.0102	3.77	0.0272	10.05
	6	0.0070	2.59	0.0241	8.90
	5	0.0037	1.37	0.0210	7.76
	4	0.0028	1.03	0.0155	5.73
	3	0.0019	0.70	0.0100	3.69
	2	-0.0031	-1.15	0.0044	1.63
	1	-0.0080	-2.96	-0.0013	-0.48
II	9	-0.0302	-11.16	-0.0050	-1.85
	8	-0.0129	-4.77	0.0016	0.59
	7	0.0045	1.66	0.0082	3.03
	6	0.0028	1.03	0.0068	2.51
	5	0.0010	0.37	0.0053	1.96
	4	0.0008	0.30	0.0040	1.48
	3	0.0006	0.22	0.0026	0.96
	2	-0.0018	-0.66	0.0010	0.37
	1	-0.0042	-1.55	-0.0007	-0.26
III	9	0.0000	0.00	0.0000	0.00
	8	-0.0048	-1.77	-0.0287	-10.60
	7	-0.0096	-3.55	-0.0573	-21.17
	6	-0.0083	-3.07	-0.0496	-18.32
	5	-0.0070	-2.59	-0.0419	-15.48
	4	-0.0052	-1.92	-0.0311	-11.49
	3	-0.0034	-1.26	-0.0202	-7.46
	2	-0.0017	-0.63	-0.0101	-3.73
	1	0.0000	0.00	0.0000	0.00

Table 2.44 Summary of Sectional Force (Front wall, Wave Crest, L.W.L, Supported on Four-sides)

		M_X (kN·m)	M_Y (kN·m)
I	9	-51.28	-8.55
	8	-20.19	11.69
	7	10.91	31.89
	6	8.33	32.08
	5	5.70	32.29
	4	6.77	28.91
	3	7.84	25.53
	2	-14.21	9.77
	1	-36.21	-6.04
II	9	-31.30	-5.25

	8	-13.38	2.05
	7	4.65	9.35
	6	3.14	8.77
	5	1.54	8.17
	4	2.41	7.74
	3	3.21	7.28
	2	-9.27	1.83
	1	-21.69	-3.66
III	9	0.00	0.00
	8	-5.58	-33.32
	7	-11.16	-66.54
	6	-10.97	-65.57
	5	-10.79	-64.60
	4	-9.82	-58.74
	3	-8.87	-52.83
	2	-4.44	-26.45
	1	0.00	0.00

Table 2.45 Summary of Design Sectional Force (Front wall, Wave Crest, L.W.L, Supported on Three-sides and/or Four sides)

		M_X (kN·m)	M_Y (kN·m)
I	9	-51.28	39.99, -8.55
	8	5.75, -20.19	37.31
	7	10.91	35.75
	6	8.33	34.02
	5	5.70	32.29
	4	6.77	29.95
	3	7.84	25.57
	2	6.51, -14.21	13.95
	1	-36.21	-6.04
II	9	-31.30	9.92, -5.25
	8	1.22, -13.38	9.35
	7	4.65	9.35
	6	3.14	8.77
	5	1.54	8.17
	4	2.41	7.74
	3	3.21	7.28
	2	3.41, -9.27	4.84
	1	-21.69	-3.66
III	9	0.00	-81.62
	8	-12.57	-75.51
	7	-11.71	-70.17
	6	-11.19	-67.08
	5	-10.84	-64.91
	4	-10.19	-61.09
	3	-8.87	-52.83
	2	-5.13	-30.72
	1	0.00	0.00

iv)Variable situation (Wave trough, L.W.L)

P_1 = -55.65 (kN/m^2)
P_2 = -84.37 (kN/m^2)
L_X = 11.950 (m)
L_Y = 4.200 (m)
λ = 11.950/4.200 = 2.85, The coefficient table for λ =2.75 is applied.

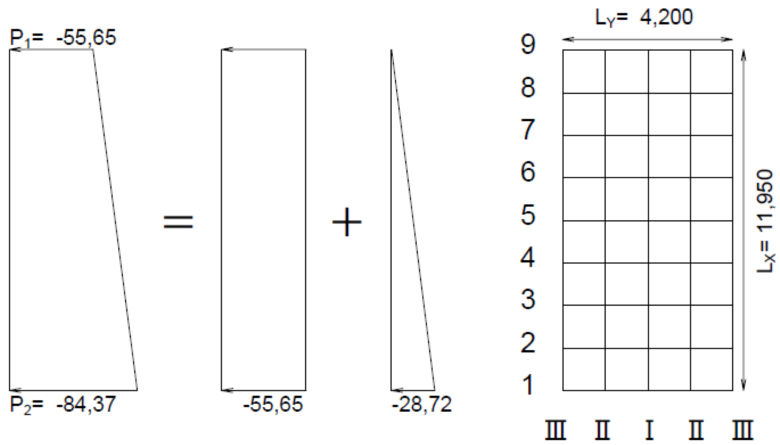


Figure 2.40- Design Model for Sectional Force Estimation (Supported on Three-sides)

- Sectional forces induced by uniformly distributed load

P = -55.65 (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X$ = -55.65 × 4.200^2 × X = -981.67 × X (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y$ = -55.65 × 4.200^2 × Y = -981.67 × Y (kN·m)

Table 2.46- Coefficient and Sectional Force (Supported on Three-sides)

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0432	-42.41
	8	0.0061	-5.99	0.0414	-40.64
	7	0.0069	-6.77	0.0415	-40.74
	6	0.0070	-6.87	0.0417	-40.94
	5	0.0072	-7.07	0.0418	-41.03
	4	0.0084	-8.25	0.0413	-40.54
	3	0.0121	-11.88	0.0373	-36.62
	2	0.0110	-10.80	0.0213	-20.91
	1	-0.0566	55.56	-0.0094	9.23
II	9	0.0000	0.00	0.0105	-10.31
	8	0.0012	-1.18	0.0103	-10.11
	7	0.0018	-1.77	0.0104	-10.21
	6	0.0018	-1.77	0.0104	-10.21
	5	0.0018	-1.77	0.0105	-10.31
	4	0.0025	-2.45	0.0107	-10.50
	3	0.0050	-4.91	0.0107	-10.50
	2	0.0059	-5.79	0.0077	-7.56
	1	-0.0341	33.47	-0.0057	5.60
III	9	0.0000	0.00	-0.0863	84.72

	8	-0.0139	13.65	-0.0836	82.07
	7	-0.0137	13.45	-0.0820	80.50
	6	-0.0138	13.55	-0.0827	81.18
	5	-0.0141	13.84	-0.0845	82.95
	4	-0.0140	13.74	-0.0839	82.36
	3	-0.0127	12.47	-0.0763	74.90
	2	-0.0080	7.85	-0.0480	47.12
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

P = -28.72 (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X$ = -28.72 × 4.200^2 × X = -506.62 × X (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y$ = -28.72 × 4.200^2 × Y = -506.62 × Y (kN·m)

Table 2.47- Coefficient and Sectional Force (Supported on Three-sides)

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0034	-1.72
	8	0.0002	-0.10	0.0060	-3.04
	7	0.0015	-0.76	0.0105	-5.32
	6	0.0026	-1.32	0.0157	-7.95
	5	0.0037	-1.87	0.0209	-10.59
	4	0.0057	-2.89	0.0257	-13.02
	3	0.0101	-5.12	0.0272	-13.78
	2	0.0108	-5.47	0.0173	-8.76
	1	-0.0486	24.62	-0.0081	4.10
II	9	0.0000	0.00	0.0003	-0.15
	8	-0.0002	0.10	0.0013	-0.66
	7	0.0003	-0.15	0.0026	-1.32
	6	0.0006	-0.30	0.0039	-1.98
	5	0.0009	-0.46	0.0053	-2.69
	4	0.0019	-0.96	0.0068	-3.45
	3	0.0045	-2.28	0.0081	-4.10
	2	0.0060	-3.04	0.0068	-3.45
	1	-0.0299	15.15	-0.0050	2.53
III	9	0.0000	0.00	-0.0021	1.06
	8	-0.0019	0.96	-0.0117	5.93
	7	-0.0037	1.87	-0.0220	11.15
	6	-0.0054	2.74	-0.0322	16.31
	5	-0.0071	3.60	-0.0427	21.63
	4	-0.0086	4.36	-0.0515	26.09
	3	-0.0093	4.71	-0.0555	28.12
	2	-0.0068	3.45	-0.0409	20.72
	1	0.0000	0.00	0.0000	0.00

Table 2.48 Summary of Sectional Force (Front wall, Wave Trough, L.W.L, Supported on Three-sides)

		M_X (kN·m)	M_Y (kN·m)
I	9	0.00	-44.13
	8	-6.09	-43.68
	7	-7.53	-46.06
	6	-8.19	-48.89
	5	-8.94	-51.62
	4	-11.14	-53.56
	3	-17.00	-50.40
	2	-16.27	-29.67
	1	80.18	13.33
II	9	0.00	-10.46
	8	-1.08	-10.77
	7	-1.92	-11.53
	6	-2.07	-12.19
	5	-2.23	-13.00
	4	-3.41	-13.95
	3	-7.19	-14.60
	2	-8.83	-11.01
	1	48.62	8.13
III	9	0.00	85.78
	8	14.61	88.00
	7	15.32	91.65
	6	16.29	97.49
	5	17.44	104.58
	4	18.10	108.45
	3	17.18	103.02
	2	11.30	67.84
	1	0.00	0.00

4) Sectional Force of Outer Wall (Side wall)

The sectional force of outer wall is estimated by the model of slabs supported on three sides or four sides according to constraint conditions and making calculations based on numerical tables. A positive bending moment is defined as causing tension on the inner side, while a negative bending moment causes tension on the outer side.

i) Floating condition

$P_1 = 0.00$ (kN/m^2)
$P_2 = 77.59$ (kN/m^2)
$L_X = 11.950$ (m)
$L_Y = 4.400$ (m)
$\lambda = 11.950/4.400 = 2.72$, The coefficient table for $\lambda = 2.75$ is applied.

- Sectional forces induced by triangularly distributed load

$P = 77.59$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = 77.59 \times 4.400^2 \times X = 1{,}502.14 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = 77.59 \times 4.400^2 \times Y = 1{,}502.14 \times Y$ (kN·m)

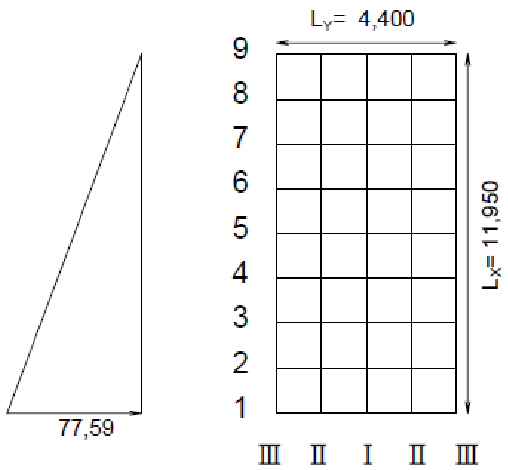


Figure 2.41- Design Model for Sectional Force Estimation

Table 2.49- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0034	5.11
	8	0.0002	0.30	0.0060	9.01
	7	0.0015	2.25	0.0105	15.77
	6	0.0026	3.91	0.0157	23.58
	5	0.0037	5.56	0.0209	31.39
	4	0.0057	8.56	0.0257	38.60
	3	0.0101	15.17	0.0272	40.86
	2	0.0108	16.22	0.0173	25.99
	1	-0.0486	-73.00	-0.0081	-12.17
II	9	0.0000	0.00	0.0003	0.45
	8	-0.0002	-0.30	0.0013	1.95
	7	0.0003	0.45	0.0026	3.91
	6	0.0006	0.90	0.0039	5.86
	5	0.0009	1.35	0.0053	7.96
	4	0.0019	2.85	0.0068	10.21
	3	0.0045	6.76	0.0081	12.17
	2	0.0060	9.01	0.0068	10.21
	1	-0.0299	-44.91	-0.0050	-7.51
III	9	0.0000	0.00	-0.0021	-3.15
	8	-0.0019	-2.85	-0.0117	-17.58
	7	-0.0037	-5.56	-0.0220	-33.05
	6	-0.0054	-8.11	-0.0322	-48.37
	5	-0.0071	-10.67	-0.0427	-64.14
	4	-0.0086	-12.92	-0.0515	-77.36
	3	-0.0093	-13.97	-0.0555	-83.37
	2	-0.0068	-10.21	-0.0409	-61.44
	1	0.0000	0.00	0.0000	0.00

ii) Permanent state

P_1 = -42.77 (kN/m^2)

P_2 = -55.42 (kN/m^2)

L_X = 11.950 (m)

$L_Y = 4.400$ (m)
$\lambda = 11.950/4.400 = 2.72$, The coefficient table for $\lambda = 2.75$ is applied.

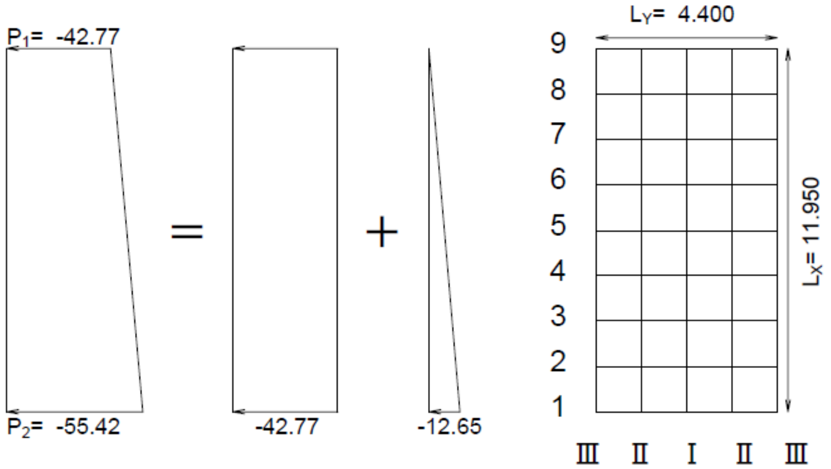


Figure 2.42- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$P = -42.77$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = -42.77 \times 4.400^2 \times X = -828.03 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = -42.77 \times 4.400^2 \times Y = -828.03 \times Y$ (kN·m)

Table 2.50- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0432	-35.77
	8	0.0061	-5.05	0.0414	-34.28
	7	0.0069	-5.71	0.0415	-34.36
	6	0.0070	-5.80	0.0417	-34.53
	5	0.0072	-5.96	0.0418	-34.61
	4	0.0084	-6.96	0.0413	-34.20
	3	0.0121	-10.02	0.0373	-30.89
	2	0.0110	-9.11	0.0213	-17.64
	1	-0.0566	46.87	-0.0094	7.78
II	9	0.0000	0.00	0.0105	-8.69
	8	0.0012	-0.99	0.0103	-8.53
	7	0.0018	-1.49	0.0104	-8.61
	6	0.0018	-1.49	0.0104	-8.61
	5	0.0018	-1.49	0.0105	-8.69
	4	0.0025	-2.07	0.0107	-8.86
	3	0.0050	-4.14	0.0107	-8.86
	2	0.0059	-4.89	0.0077	-6.38
	1	-0.0341	28.24	-0.0057	4.72
III	9	0.0000	0.00	-0.0863	71.46
	8	-0.0139	11.51	-0.0836	69.22
	7	-0.0137	11.34	-0.0820	67.90
	6	-0.0138	11.43	-0.0827	68.48
	5	-0.0141	11.68	-0.0845	69.97
	4	-0.0140	11.59	-0.0839	69.47
	3	-0.0127	10.52	-0.0763	63.18

	2	-0.0080	6.62	-0.0480	39.75
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

P = -12.65 (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X$ = -12.65 × 4.400^2 × X = -244.90 × X (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y$ = -12.65 × 4.400^2 × Y = -244.90 × Y (kN·m)

Table 2.51- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0034	-0.83
	8	0.0002	-0.05	0.0060	-1.47
	7	0.0015	-0.37	0.0105	-2.57
	6	0.0026	-0.64	0.0157	-3.84
	5	0.0037	-0.91	0.0209	-5.12
	4	0.0057	-1.40	0.0257	-6.29
	3	0.0101	-2.47	0.0272	-6.66
	2	0.0108	-2.64	0.0173	-4.24
	1	-0.0486	11.90	-0.0081	1.98
II	9	0.0000	0.00	0.0003	-0.07
	8	-0.0002	0.05	0.0013	-0.32
	7	0.0003	-0.07	0.0026	-0.64
	6	0.0006	-0.15	0.0039	-0.96
	5	0.0009	-0.22	0.0053	-1.30
	4	0.0019	-0.47	0.0068	-1.67
	3	0.0045	-1.10	0.0081	-1.98
	2	0.0060	-1.47	0.0068	-1.67
	1	-0.0299	7.32	-0.0050	1.22
III	9	0.0000	0.00	-0.0021	0.51
	8	-0.0019	0.47	-0.0117	2.87
	7	-0.0037	0.91	-0.0220	5.39
	6	-0.0054	1.32	-0.0322	7.89
	5	-0.0071	1.74	-0.0427	10.46
	4	-0.0086	2.11	-0.0515	12.61
	3	-0.0093	2.28	-0.0555	13.59
	2	-0.0068	1.67	-0.0409	10.02
	1	0.0000	0.00	0.0000	0.00

Table 2.52 Summary of Sectional Force (Side wall, Permanent State)

		M_X (kN·m)	M_Y (kN·m)
I	9	0.00	-36.60
	8	-5.10	-35.75
	7	-6.08	-36.93
	6	-6.44	-38.37
	5	-6.87	-39.73
	4	-8.36	-40.49
	3	-12.49	-37.55
	2	-11.75	-21.88
	1	58.77	9.76

II	9	0.00	-8.76
	8	-0.94	-8.85
	7	-1.56	-9.25
	6	-1.64	-9.57
	5	-1.71	-9.99
	4	-2.54	-10.53
	3	-5.24	-10.84
	2	-6.36	-8.05
	1	35.56	5.94
III	9	0.00	71.97
	8	11.98	72.09
	7	12.25	73.29
	6	12.75	76.37
	5	13.42	80.43
	4	13.70	82.08
	3	12.80	76.77
	2	8.29	49.77
	1	0.00	0.00

5) Correction Method of Unbalanced Bending Moments between Outer walls

At the intersection between the side wall and the front/rear walls (i.e., at the outer wall corner), lateral bending moments are unbalanced. This imbalance occurs because the slab is modeled as a plate with three fixed sides and one free side.

Therefore, the bending moment at the corners shall be redistributed and corrected based on the rigidity ratio between the adjacent slabs. Specifically, the following corrections are made at the corners (III-axis) and the center of the span (I-axis):

- Stiffness ratio

Since the EI of the side wall, front wall, and rear wall is the same, the stiffness ratio K_1 and K_2 are:

$K_1 = 1 / L_{y1} = 1 / 4.400 = 0.227$

$K_2 = 1 / L_{y2} = 1 / 4.200 = 0.238$

- Redistributed ratio

Side wall direction: $\alpha = K_1 / (K_1 + K_2) = 0.227 / (0.227 + 0.238) = 0.488$

Front and Rear wall direction: $\beta = K_2 / (K_1 + K_2) = 0.238 / (0.227 + 0.238) = 0.512$

- Corrected moment at corners (III-axis)

$\Delta M = M_1 - M_2$

When $M_1 > M_2$

$M_1' = M_1 - \alpha\Delta M = M_1 - 0.488 \times \Delta M$

$M_2' = M_2 + \beta\Delta M = M_2 + 0.512 \times \Delta M$

However, for the outer reinforcement, no correction is applied, and the larger value is used.

For the inner reinforcement, if the corrected moment is smaller than the original moment, the original (uncorrected) moment is used to ensure safety.

- Moment correction at mid-span (I-axis)

The correction amount at mid-span is applied as 50% of the correction made at III-axis. However, if the corrected moment is smaller than the original moment, the original (uncorrected) value is used to ensure safety.

Corrected moment

$M_1' = M_1 - 1/2 \cdot \Delta M \cdot e_1 = M_1 - 0.244 \times \Delta M$

$M_2' = M_2 + 1/2 \cdot \Delta M \cdot e_2 = M_2 + 0.256 \times \Delta M$

i) Corrected moment at floating condition

- Moment correction at corners (III-axis)

No.	Side wall			Unbalanced moment	Front wall		
	Corrected moment	$\Delta M \cdot \alpha$	Original moment		Original moment	$\Delta M \cdot \beta$	Corrected moment
9	-1.37	0.06	-1.43	0.12	-1.31	-0.06	-1.37
8	-7.64	0.35	-7.99	0.71	-7.28	-0.36	-7.64
7	-14.37	0.65	-15.02	1.33	-13.69	-0.68	-14.37
6	-21.03	0.96	-21.99	1.96	-20.03	-1.00	-21.03
5	-27.90	1.26	-29.16	2.59	-26.57	-1.33	-27.90
4	-33.64	1.53	-35.17	3.13	-32.04	-1.60	-33.64
3	-36.26	1.64	-37.90	3.37	-34.53	-1.73	-36.26
2	-26.72	1.21	-27.93	2.48	-25.45	-1.27	-26.72
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00

- Moment correction at mid-span (I-axis)

No.	Side wall			Unbalanced moment	Front wall		
	Corrected moment	$1/2 \cdot \Delta M \cdot \alpha$	Original moment		Original moment	$1/2 \cdot \Delta M \cdot \beta$	Corrected moment
9	2.35	0.03	2.32	0.06	2.12	-0.03	2.09
8	4.27	0.17	4.10	0.35	3.73	-0.18	3.55
7	7.49	0.32	7.17	0.66	6.53	-0.34	6.19
6	11.20	0.48	10.72	0.98	9.77	-0.50	9.27
5	14.90	0.63	14.27	1.29	13.00	-0.66	12.34
4	18.31	0.76	17.55	1.56	15.99	-0.80	15.19
3	19.39	0.82	18.57	1.68	16.92	-0.86	16.06
2	12.42	0.61	11.81	1.24	10.76	-0.63	10.13
1	-5.53	0.00	-5.53	0.00	-5.04	0.00	-5.04

ii) Corrected moment at permanent state

- Moment correction at corners (III-axis)

No.	Side wall			Unbalanced moment	Front wall		
	Corrected moment	$\Delta M \cdot \alpha$	Original moment		Original moment	$\Delta M \cdot \beta$	Corrected moment
9	62.88	-2.55	65.43	5.23	60.20	2.68	62.88
8	63.04	-2.49	65.53	5.11	60.42	2.62	63.04
7	64.15	-2.47	66.62	5.07	61.55	2.60	64.15
6	66.90	-2.52	69.42	5.17	64.25	2.65	66.90
5	70.51	-2.60	73.11	5.33	67.78	2.73	70.51

4	72.01	-2.61	74.62	5.34	69.28	2.73	72.01
3	67.40	-2.39	69.79	4.90	64.89	2.51	67.40
2	43.72	-1.52	45.24	3.11	42.13	1.59	43.72
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00

- Moment correction at mid-span (I-axis)

No.	Side wall			Unbalanced moment	Front wall		
	Corrected moment	$1/2{\cdot}\Delta M{\cdot}\alpha$	Original moment		Original moment	$1/2{\cdot}\Delta M{\cdot}\beta$	Corrected moment
9	-34.56	-1.28	-33.28	2.62	-30.64	1.34	-29.30
8	-33.75	-1.25	-32.50	2.56	-29.97	1.31	-28.66
7	-34.82	-1.24	-33.58	2.54	-31.02	1.30	-29.72
6	-36.15	-1.26	-34.89	2.58	-32.28	1.32	-30.96
5	-37.41	-1.30	-36.11	2.66	-33.48	1.36	-32.12
4	-38.11	-1.30	-36.81	2.67	-34.18	1.37	-32.81
3	-35.34	-1.20	-34.14	2.45	-31.74	1.25	-30.49
2	-20.64	-0.76	-19.88	1.56	-18.51	0.80	-17.71
1	8.88	0.00	8.88	0.00	8.27	0.00	8.27

Other sectional forces and moments, such as serviceability and compressive stress, are calculated in the same method, but are omitted here.

(4) Reinforcement Concrete Design Verification

1) Verification of Safety of Structural Members

i) Bending moment

It shall be verified that the design cross-section capacity (M_{ud}) is greater than or equal to the design value of the bending moment (M_d) according to Equation (1.37).

$$\frac{\gamma_i \cdot M_d}{M_{ud}} \leq 1.0$$

The design cross-section capacity M_{ud} for the bending moment of reinforced concrete can be calculated.

$$M_{ud} = A_s f_{yd} \cdot d \left(1 - \frac{p_w \cdot f_{yd}}{1.7 f'_{cd}}\right) / \gamma_b$$

Where:

A_s : area of tension reinforcement (mm²)
p_w : reinforcing bar ratio ($=A_s/(b_w \cdot d)$)
f'_{cd} : design compressive strength of concrete
$f'_{cd} = f'_{ck}/\gamma_c = 30.0 / 1.30 = 23.1$ (N/mm²)
f'_{yd} : design yield strength of tensile reinforcement
$f_{yd} = f_{yk}/\gamma_s = 345.0 / 1.00 = 345$ (N/mm²)
d : effective height (mm)
γ_i : structure factor (1.0)
γ_b : member factor (1.1)
γ_c : material factor for steel (1.3)
γ_s : material factor for concrete (1.0)

2) Verification of Serviceability of Structural Members

i) Crack width

It shall be verified that the design response value of crack width (w) is less than or equal to the design limit value of the crack width (w_a).

$$\gamma_i w_d / w_a \leq 1.0$$

The design response value of crack width can be calculated.

$$w = 1.1 \times k_1 \times k_2 \times k_3 \times \{4 \times c + 0.7 \times (c_s - \varphi)\} \times (\sigma_{se} / E_s + \varepsilon'_{csd})$$

Where:

w	:	design response value of crack width (mm)
k_1	:	coefficient expressing the influence of the surface profile of reinforcing bars on crack width (when deformed bars = 1.0)
k_2	:	coefficient expressing the influence of concrete quality on crack width, $k_2 = 15/(f_c' + 20) + 0.7$
f_c'	:	compressive strength of concrete (N/mm²). It can normally be the design value of the compressive strength f'_{cd}
k_3	:	coefficient expressing the influence of the number of layers on the tensile bars, $k_3 = 5(n+2)/(7n+8)$
n	:	number of layers of tension bars
c	:	concrete cover (mm)
c_s	:	distance between the centers of reinforcing bars (mm)
φ	:	diameter of the tension reinforcing bar, nominal diameter of the smallest reinforcing bar (mm)
E_s	:	Young's modulus of reinforcing bars (200 kN/mm²)
ε'_{csd}	:	value considering the increase in crack width due to concrete shrinkage and creep , on the order of 0.00010
σ_{se}	:	stress increment of the reinforcing bars near the surface (N/mm²)

Table 2.53- Limit Values of Crack Width w_a

Reinforcement	Environmental classification	Crack width limit value (mm)
Outer Lower (Outer) reinforcement	Severe corrosion environment	0.0035c
Upper (Inner) reinforcement	Other corrosion environment	0.0040c

ii) Concrete compressive stress in permanent state

It shall be verified by the following equation.

$$\sigma'_c \leq 0.4 f'_{ck}$$

Where:

σ'_c	:	compressive stress generated in concrete by a permanent action (N/mm²)
f'_{ck}	:	characteristic value of compressive strength of concrete (N/mm²)

iii) Verification of shear stress

Design shear compressive failure capacity can be calculated using the following equation.

$$V_{dd}=\beta_d \cdot \beta_p \cdot \beta_a \cdot f_{dd} \cdot b_w \cdot d / \gamma_b$$

Where:

V_{dd}	:	design shear compressive failure capacity (N)
f_{dd}	:	$0.19\sqrt{f'_{cd}}$ (N/mm²)
β_d	:	$\sqrt[4]{1000/d}$, set to 1.5 when $\beta_d > 1.5$
β_p	:	$(1+\sqrt{100p_v})/2$, set to 1.5 when $\beta_p > 1.5$
β_a	:	$5/(1+(a/d)^2)$
b_w	:	width of web (mm)
d	:	loading point in the case of simple beams; effective depth (mm) at the support of cantilever beams
a	:	distance from the support frontal surface to the loading point (mm)
p_v	:	$A_s/(b_w \cdot d)$
A_s	:	cross-sectional area of reinforcing bars at tension side (mm²)
f'_{cd}	:	design compression strength of concrete (N/mm²)
γ_b	:	may generally be set to 1.3

3) Bottom Slab (Room A)

i) Safety verification (Room A / Transverse direction / Upper reinforcement)

B=100cm

NO.		M_d (kN·m)	d (cm)	A_{sn} (cm²)	Dia. (mm)	Pitch (cm)	A_s (cm²)	M_{ud} (kN·m)	γ_i	$\gamma_i \cdot M_d / M_{ud}$
I	5	109.36	64.0	6.04	D13	20.0	6.34	126.154	1.10	0.95
	4	14.82	64.0	0.81	D13	20.0	6.34	126.154	1.10	0.13
	3	39.32	64.0	2.16	D13	20.0	6.34	126.154	1.10	0.34
	2	21.95	64.0	1.20	D13	20.0	6.34	126.154	1.10	0.19
	1	108.03	64.0	5.97	D13	20.0	6.34	126.154	1.10	0.94
II	5	69.17	64.0	3.81	D13	20.0	6.34	126.154	1.10	0.60
	4	8.80	64.0	0.48	D13	20.0	6.34	126.154	1.10	0.08
	3	22.14	64.0	1.22	D13	20.0	6.34	126.154	1.10	0.19
	2	13.84	64.0	0.76	D13	20.0	6.34	126.154	1.10	0.12
	1	68.13	64.0	3.75	D13	20.0	6.34	126.154	1.10	0.59
III	5	0.00	64.0	0.00	D13	20.0	6.34	126.154	—	0.00
	4	11.52	64.0	0.63	D13	20.0	6.34	126.154	1.10	0.10
	3	18.22	64.0	1.00	D13	20.0	6.34	126.154	1.10	0.16
	2	11.37	64.0	0.62	D13	20.0	6.34	126.154	1.10	0.10
	1	0.00	64.0	0.00	D13	20.0	6.34	126.154	—	0.00

ii) Crack width of serviceability verification (Room A / Transverse direction / Upper reinforcement)

Young's Modulus Ratio N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	σ_{se} (N/mm^2)	Crack width w (cm)	Limit crack width w_a	w / w_a
I	5	7.55	64.0	D13	20.0	6.34	19.327	0.0072	0.0040 × 5.0 = 0.0200	0.36
	4	9.79	64.0	D13	20.0	6.34	25.061	0.0082		0.41
	3	25.06	64.0	D13	20.0	6.34	64.150	0.0153		0.77
	2	13.62	64.0	D13	20.0	6.34	34.865	0.0100		0.50
	1	14.09	64.0	D13	20.0	6.34	36.069	0.0102		0.51
II	5	4.26	64.0	D13	20.0	6.34	10.905	0.0056	0.0040 × 5.0 = 0.0200	0.28
	4	5.85	64.0	D13	20.0	6.34	14.975	0.0064		0.32
	3	14.12	64.0	D13	20.0	6.34	36.145	0.0102		0.51
	2	8.57	64.0	D13	20.0	6.34	21.938	0.0076		0.38
	1	9.41	64.0	D13	20.0	6.34	24.088	0.0080		0.40
III	5	0.00	64.0	D13	20.0	6.34	0.000	0.0036	0.0040 × 5.0 = 0.0200	0.18
	4	0.76	64.0	D13	20.0	6.34	1.945	0.0040		0.20
	3	1.82	64.0	D13	20.0	6.34	4.659	0.0045		0.23
	2	1.48	64.0	D13	20.0	6.34	3.789	0.0043		0.22
	1	0.00	64.0	D13	20.0	6.34	0.000	0.0036		0.18

iii) Compression stress of serviceability verification (Room A / Transverse direction / Upper reinforcement)

γ_i=1.00, N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	J	$\gamma_i \cdot \sigma_c'$ (N/mm^2)		$0.4 \cdot f'_{ck}$ (N/mm^2)
I	5	0.00	64.0	D13	20.0	6.34	0.963	0.00	≤	12.00
	4	7.10	64.0	D13	20.0	6.34	0.963	0.32	≤	12.00
	3	16.33	64.0	D13	20.0	6.34	0.963	0.74	≤	12.00
	2	8.15	64.0	D13	20.0	6.34	0.963	0.37	≤	12.00
	1	0.00	64.0	D13	20.0	6.34	0.963	0.00	≤	12.00
II	5	0.00	64.0	D13	20.0	6.34	0.963	0.00	≤	12.00
	4	4.31	64.0	D13	20.0	6.34	0.963	0.20	≤	12.00
	3	9.19	64.0	D13	20.0	6.34	0.963	0.42	≤	12.00
	2	5.05	64.0	D13	20.0	6.34	0.963	0.23	≤	12.00
	1	0.00	64.0	D13	20.0	6.34	0.963	0.00	≤	12.00
III	5	0.00	64.0	D13	20.0	6.34	0.963	0.00	≤	12.00
	4	0.00	64.0	D13	20.0	6.34	0.963	0.00	≤	12.00
	3	0.00	64.0	D13	20.0	6.34	0.963	0.00	≤	12.00
	2	0.00	64.0	D13	20.0	6.34	0.963	0.00	≤	12.00
	1	0.00	64.0	D13	20.0	6.34	0.963	0.00	≤	12.00

iv) Safety verification (Room A / Transverse direction / Lower reinforcement)

B=100cm

NO.		M_d (kN·m)	d (cm)	A_{sn} (cm^2)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	M_{ud} (kN·m)	γ_i	$\gamma_i \cdot M_d / M_{ud}$
I	5	89.42	68.6	4.60	D16	20.0	9.93	210.927	1.10	0.47*
	4	20.62	62.0	1.17	D13	20.0	6.34	122.179	1.10	0.19
	3	43.65	62.0	2.48	D13	20.0	6.34	122.179	1.10	0.39
	2	20.06	62.0	1.14	D13	20.0	6.34	122.179	1.10	0.18
	1	106.41	68.6	5.48	D13, D13	10.0	12.67	268.170	1.10	0.44*
II	5	55.15	68.6	2.83	D13	20.0	6.34	135.305	1.10	0.45
	4	12.70	62.0	0.72	D13	20.0	6.34	122.179	1.10	0.11
	3	24.58	62.0	1.39	D13	20.0	6.34	122.179	1.10	0.22
	2	12.30	62.0	0.70	D13	20.0	6.34	122.179	1.10	0.11
	1	68.53	68.6	3.52	D16	20.0	9.93	210.927	1.10	0.36*
III	5	0.00	68.6	0.00	D13	20.0	6.34	135.305	—	0.00
	4	9.32	68.6	0.48	D13	20.0	6.34	135.305	1.10	0.08
	3	16.42	68.6	0.84	D13	20.0	6.34	135.305	1.10	0.13
	2	11.19	68.6	0.57	D13	20.0	6.34	135.305	1.10	0.09
	1	0.00	68.6	0.00	D13	20.0	6.34	135.305	—	0.00

*: reinforcement is determined by serviceability verification

v) Crack width of serviceability verification (Room A / Transverse direction / Lower reinforcement)

Young's Modulus Ratio *N*=7.1, *B*=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	σ_{se} (N/mm^2)	Crack width w (cm)	Limit crack width w_a	w / w_a
I	5	57.84	68.6	D16	20.0	9.93	88.866	0.0245	0.0035 × 7.0 = 0.0245	1.00
	4	0.68	62.0	D13	20.0	6.34	1.798	0.0049		0.20
	3	4.35	62.0	D13	20.0	6.34	11.501	0.0071		0.29
	2	3.42	62.0	D13	20.0	6.34	9.042	0.0066		0.27
	1	66.98	68.6	D13, D13	10.0	12.67	81.104	0.0190		0.78
II	5	35.82	68.6	D13	20.0	6.34	85.433	0.0238	0.0035 × 7.0 = 0.0245	0.97
	4	0.30	62.0	D13	20.0	6.34	0.793	0.0047		0.19
	3	2.44	62.0	D13	20.0	6.34	6.451	0.0060		0.24
	2	2.24	62.0	D13	20.0	6.34	5.922	0.0059		0.24
	1	43.02	68.6	D16	20.0	9.93	66.096	0.0194		0.79
III	5	0.00	68.6	D13	20.0	6.34	0.000	0.0045	0.0035 × 7.0 = 0.0245	0.18
	4	6.04	68.6	D13	20.0	6.34	14.406	0.0078		0.32
	3	10.46	68.6	D13	20.0	6.34	24.948	0.0102		0.42
	2	7.04	68.6	D13	20.0	6.34	16.791	0.0083		0.34
	1	0.00	68.6	D13	20.0	6.34	0.000	0.0045		0.18

vi) Compression stress of serviceability verification (Room A / Transverse direction / Lower reinforcement)

γ_i=1.00, N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	J	$\gamma_i \cdot \sigma_c'$ (N/mm^2)		$0.4 \cdot f'_{ck}$ (N/mm^2)
I	5	39.42	68.6	D16	20.0	9.93	0.955	1.31	≤	12.00
	4	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	3	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	2	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	1	41.92	68.6	D13, D13	10.0	12.67	0.950	1.25	≤	12.00
II	5	24.70	68.6	D13	20.0	6.34	0.964	1.01	≤	12.00
	4	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	3	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	2	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	1	26.67	68.6	D16	20.0	9.93	0.955	0.89	≤	12.00
III	5	0.00	68.6	D13	20.0	6.34	0.964	0.00	≤	12.00
	4	4.14	68.6	D13	20.0	6.34	0.964	0.17	≤	12.00
	3	6.82	68.6	D13	20.0	6.34	0.964	0.28	≤	12.00
	2	4.41	68.6	D13	20.0	6.34	0.964	0.18	≤	12.00
	1	0.00	68.6	D13	20.0	6.34	0.964	0.00	≤	12.00

vii) Safety verification (Room A / Longitudinal direction / Upper reinforcement)

B=100cm

NO.		M_d (kN·m)	d (cm)	A_{sn} (cm^2)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	M_{ud} (kN·m)	γ_i	$\gamma_i \cdot M_d / M_{ud}$
I	5	18.33	62.0	1.04	D13	20.0	6.34	122.179	1.10	0.17
	4	20.93	62.0	1.19	D13	20.0	6.34	122.179	1.10	0.19
	3	39.32	62.0	2.23	D13	20.0	6.34	122.179	1.10	0.35
	2	23.35	62.0	1.32	D13	20.0	6.34	122.179	1.10	0.21
	1	18.11	62.0	1.03	D13	20.0	6.34	122.179	1.10	0.16
II	5	11.53	62.0	0.65	D13	20.0	6.34	122.179	1.10	0.10
	4	10.11	62.0	0.57	D13	20.0	6.34	122.179	1.10	0.09
	3	18.33	62.0	1.04	D13	20.0	6.34	122.179	1.10	0.17
	2	12.53	62.0	0.71	D13	20.0	6.34	122.179	1.10	0.11
	1	11.36	62.0	0.64	D13	20.0	6.34	122.179	1.10	0.10
III	5	0.00	62.0	0.00	D13	20.0	6.34	122.179	—	0.00
	4	69.04	62.0	3.93	D13	20.0	6.34	122.179	1.10	0.62
	3	108.70	62.0	6.20	D13	20.0	6.34	122.179	1.10	0.98
	2	68.25	62.0	3.88	D13	20.0	6.34	122.179	1.10	0.61
	1	0.00	62.0	0.00	D13	20.0	6.34	122.179	—	0.00

viii) Crack width of serviceability verification (Room A / Longitudinal direction / Upper reinforcement)

Young's Modulus Ratio N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	σ_{se} (N/mm^2)	Crack width w (cm)	Limit crack width w_a	w / w_a
I	5	1.27	62.0	D13	20.0	6.34	3.358	0.0053	0.0040 × 70 = 0.028	0.19
	4	13.47	62.0	D13	20.0	6.34	35.613	0.0126		0.45
	3	25.06	62.0	D13	20.0	6.34	66.256	0.0195		0.70
	2	14.77	62.0	D13	20.0	6.34	39.050	0.0134		0.48
	1	2.36	62.0	D13	20.0	6.34	6.240	0.0059		0.21
II	5	0.72	62.0	D13	20.0	6.34	1.904	0.0050	0.0040 × 70 = 0.028	0.18
	4	6.56	62.0	D13	20.0	6.34	17.344	0.0084		0.30
	3	11.68	62.0	D13	20.0	6.34	30.881	0.0115		0.41
	2	7.86	62.0	D13	20.0	6.34	20.781	0.0092		0.33
	1	1.56	62.0	D13	20.0	6.34	4.124	0.0055		0.20
III	5	0.00	62.0	D13	20.0	6.34	0.000	0.0045	0.0040 × 70 = 0.0280	0.16
	4	4.90	62.0	D13	20.0	6.34	12.955	0.0075		0.27
	3	10.80	62.0	D13	20.0	6.34	28.554	0.0110		0.39
	2	8.78	62.0	D13	20.0	6.34	23.213	0.0098		0.35
	1	0.00	62.0	D13	20.0	6.34	0.000	0.0045		0.16

ix) Compression stress of serviceability verification (Room A / Longitudinal direction / Upper reinforcement)

γ_i=1.00, N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	J	$\gamma_i \cdot \sigma_c'$ (N/mm^2)		$0.4 \cdot f'_{ck}$ (N/mm^2)
I	5	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	4	9.02	62.0	D13	20.0	6.34	0.962	0.43	≤	12.00
	3	16.33	62.0	D13	20.0	6.34	0.962	0.78	≤	12.00
	2	9.37	62.0	D13	20.0	6.34	0.962	0.45	≤	12.00
	1	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
II	5	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	4	4.50	62.0	D13	20.0	6.34	0.962	0.21	≤	12.00
	3	7.62	62.0	D13	20.0	6.34	0.962	0.36	≤	12.00
	2	4.86	62.0	D13	20.0	6.34	0.962	0.23	≤	12.00
	1	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
III	5	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	4	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	3	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	2	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	1	0.00	62.0	D13	20.0	6.34	0.962	0.00	≤	12.00

x) Safety verification (Room A / Longitudinal direction / Lower reinforcement)

B=100cm

NO.		M_d (kN·m)	d (cm)	A_{sn} (cm²)	Dia. (mm)	Pitch (cm)	A_s (cm²)	M_{ud} (kN·m)	γ_i	$\gamma_i \cdot M_d / M_{ud}$
I	5	14.99	66.6	0.79	D13	20.0	6.34	131.326	1.10	0.13
	4	24.67	60.0	1.45	D13	20.0	6.34	118.196	1.10	0.23
	3	43.65	60.0	2.56	D13	20.0	6.34	118.196	1.10	0.41
	2	24.48	60.0	1.43	D13	20.0	6.34	118.196	1.10	0.23
	1	17.84	66.6	0.94	D13	20.0	6.34	131.326	1.10	0.15
II	5	9.21	66.6	0.49	D13	20.0	6.34	131.326	1.10	0.08
	4	12.59	60.0	0.74	D13	20.0	6.34	118.196	1.10	0.12
	3	20.34	60.0	1.19	D13	20.0	6.34	118.196	1.10	0.19
	2	12.40	60.0	0.73	D13	20.0	6.34	118.196	1.10	0.12
	1	11.41	66.6	0.60	D13	20.0	6.34	131.326	1.10	0.10
III	5	0.00	66.6	0.00	D13	20.0	6.34	131.326	—	0.00
	4	56.79	66.6	3.00	D13	20.0	6.34	131.326	1.10	0.48
	3	97.86	66.6	5.19	D13, D13	10.0	12.67	260.236	1.10	0.41
	2	66.88	66.6	3.54	D16	20.0	9.93	204.704	1.10	0.36
	1	0.00	66.6	0.00	D13	20.0	6.34	131.326	—	0.00

xi) Crack width of serviceability verification (Room A / Longitudinal direction / Lower reinforcement)

Young's Modulus Ratio N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm²)	σ_{se} (N/mm²)	Crack width w (cm)	Limit crack width w_a	w / w_a
I	5	9.69	66.6	D13	20.0	6.34	23.819	0.0118	0.0035 × 9.0 = 0.0315	0.37
	4	1.98	60.0	D13	20.0	6.34	5.413	0.0069		0.22
	3	4.35	60.0	D13	20.0	6.34	11.893	0.0086		0.27
	2	2.91	60.0	D13	20.0	6.34	7.956	0.0076		0.24
	1	11.23	66.6	D13	20.0	6.34	27.604	0.0129		0.41
II	5	5.98	66.6	D13	20.0	6.34	14.699	0.0094	0.0035 × 9.0 = 0.0315	0.30
	4	0.80	60.0	D13	20.0	6.34	2.187	0.0060		0.19
	3	2.03	60.0	D13	20.0	6.34	5.550	0.0069		0.22
	2	1.73	60.0	D13	20.0	6.34	4.730	0.0067		0.21
	1	7.16	66.6	D13	20.0	6.34	17.600	0.0102		0.32
III	5	0.00	66.6	D13	20.0	6.34	0.000	0.0054	0.0035 × 9.0 = 0.0315	0.17
	4	36.70	66.6	D13	20.0	6.34	90.212	0.0298		0.95
	3	62.38	66.6	D13, D13	10.0	12.67	77.852	0.0227		0.72
	2	42.13	66.6	D16	20.0	9.93	66.712	0.0233		0.74
	1	0.00	66.6	D13	20.0	6.34	0.000	0.0054		0.17

xii) Compression stress of serviceability verification (Room A / Longitudinal direction / Lower reinforcement)

γ_i=1.00, N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	J	$\gamma_i \cdot \sigma_c'$ (N/mm^2)		$0.4 \cdot f'_{ck}$ (N/mm^2)
I	5	6.61	66.6	D13	20.0	6.34	0.963	0.28	≤	12.00
	4	0.00	60.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	3	0.00	60.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	2	0.00	60.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	1	7.03	66.6	D13	20.0	6.34	0.963	0.30	≤	12.00
II	5	4.12	66.6	D13	20.0	6.34	0.963	0.18	≤	12.00
	4	0.00	60.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	3	0.00	60.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	2	0.00	60.0	D13	20.0	6.34	0.962	0.00	≤	12.00
	1	4.45	66.6	D13	20.0	6.34	0.963	0.19	≤	12.00
III	5	0.00	66.6	D13	20.0	6.34	0.963	0.00	≤	12.00
	4	24.94	66.6	D13	20.0	6.34	0.963	1.07	≤	12.00
	3	40.66	66.6	D13, D13	10.0	12.67	0.950	1.28	≤	12.00
	2	26.43	66.6	D16	20.0	9.93	0.955	0.92	≤	12.00
	1	0.00	66.6	D13	20.0	6.34	0.963	0.00	≤	12.00

The verification of Room B and Room C can be calculated in the same method, but are omitted here.

4) Front Wall

i) Safety verification (Lateral reinforcement / Inner reinforcement)

B=100cm

NO.		M_d (kN·m)	d (cm)	A_{sn} (cm^2)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	M_{ud} (kN·m)	γ_i	$\gamma_i \cdot M_d$ $/M_{ud}$
I	9	39.99	34.0	4.17	D13	20.0	6.34	66.503	1.10	0.66
	8	37.31	34.0	3.89	D13	20.0	6.34	66.503	1.10	0.62
	7	35.75	34.0	3.72	D13	20.0	6.34	66.503	1.10	0.59
	6	34.02	34.0	3.54	D13	20.0	6.34	66.503	1.10	0.56
	5	32.29	34.0	3.36	D13	20.0	6.34	66.503	1.10	0.53
	4	35.18	34.0	3.66	D13	20.0	6.34	66.503	1.10	0.58
	3	37.23	34.0	3.88	D13	20.0	6.34	66.503	1.10	0.62
	2	23.68	34.0	2.46	D13	20.0	6.34	66.503	1.10	0.39
	1	13.33	34.0	1.38	D13	20.0	6.34	66.503	1.10	0.22
II	9	9.92	34.0	1.03	D13	20.0	6.34	66.503	1.10	0.16
	8	9.35	34.0	0.97	D13	20.0	6.34	66.503	1.10	0.15
	7	9.35	34.0	0.97	D13	20.0	6.34	66.503	1.10	0.15
	6	8.77	34.0	0.91	D13	20.0	6.34	66.503	1.10	0.15
	5	8.17	34.0	0.84	D13	20.0	6.34	66.503	1.10	0.14
	4	9.31	34.0	0.96	D13	20.0	6.34	66.503	1.10	0.15
	3	11.09	34.0	1.15	D13	20.0	6.34	66.503	1.10	0.18
	2	9.31	34.0	0.96	D13	20.0	6.34	66.503	1.10	0.15
	1	8.13	34.0	0.84	D13	20.0	6.34	66.503	1.10	0.13
III	9	85.78	34.0	9.06	D16, D16	10.0	19.86	200.914	1.10	0.47*
	8	88.00	34.0	9.30	D16, D16	10.0	19.86	200.914	1.10	0.48*
	7	91.65	34.0	9.70	D16, D16	10.0	19.86	200.914	1.10	0.50*
	6	97.49	34.0	10.33	D16, D19	10.0	24.26	242.472	1.10	0.44*
	5	104.58	34.0	11.11	D16, D19	10.0	24.26	242.472	1.10	0.47*
	4	108.45	34.0	11.53	D16, D19	10.0	24.26	242.472	1.10	0.49*
	3	103.02	34.0	10.94	D16, D19	10.0	24.26	242.472	1.10	0.47*
	2	67.84	34.0	7.13	D13, D16	10.0	16.27	166.196	1.10	0.45*
	1	0.00	34.0	0.00	D13	20.0	6.34	66.503	—	0.00
I'	9	39.99	34.0	4.17	D13	20.0	6.34	66.503	1.10	0.66
	8	37.31	34.0	3.89	D13	20.0	6.34	66.503	1.10	0.62
	7	35.75	34.0	3.72	D13	20.0	6.34	66.503	1.10	0.59
	6	34.02	34.0	3.54	D13	20.0	6.34	66.503	1.10	0.56
	5	32.29	34.0	3.36	D13	20.0	6.34	66.503	1.10	0.53
	4	35.18	34.0	3.66	D13	20.0	6.34	66.503	1.10	0.58
	3	37.23	34.0	3.88	D13	20.0	6.34	66.503	1.10	0.62
	2	23.68	34.0	2.46	D13	20.0	6.34	66.503	1.10	0.39
	1	13.33	34.0	1.38	D13	20.0	6.34	66.503	1.10	0.22
III'	9	85.78	34.0	9.06	D16, D16	10.0	19.86	200.914	1.10	0.47*
	8	88.00	34.0	9.30	D16, D16	10.0	19.86	200.914	1.10	0.48*
	7	91.65	34.0	9.70	D16, D16	10.0	19.86	200.914	1.10	0.50*
	6	97.49	34.0	10.33	D16, D19	10.0	24.26	242.472	1.10	0.44*
	5	104.58	34.0	11.11	D16, D19	10.0	24.26	242.472	1.10	0.47*
	4	108.45	34.0	11.53	D16, D19	10.0	24.26	242.472	1.10	0.49*
	3	103.02	34.0	10.94	D16, D19	10.0	24.26	242.472	1.10	0.47*
	2	67.84	34.0	7.13	D13, D16	10.0	16.27	166.196	1.10	0.45*
	1	0.00	34.0	0.00	D13	20.0	6.34	66.503	—	0.00

*: reinforcement is determined by serviceability verification

I', III': corrected moment at corners

ii) Crack width of serviceability verification (Lateral reinforcement / Inner reinforcement)

Young's Modulus Ratio N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	σ_{se} (N/mm^2)	Crack width w (cm)	Limit crack width w_a	w / w_a
I	9	2.12	34.0	D13	20.0	6.34	10.352	0.0055	0.0040 × 5.0 = 0.0200	0.28
	8	3.73	34.0	D13	20.0	6.34	18.214	0.0070		0.35
	7	6.53	34.0	D13	20.0	6.34	31.886	0.0094		0.47
	6	9.77	34.0	D13	20.0	6.34	47.707	0.0123		0.62
	5	13.00	34.0	D13	20.0	6.34	63.479	0.0152		0.76
	4	15.99	34.0	D13	20.0	6.34	78.079	0.0179		0.90
	3	16.92	34.0	D13	20.0	6.34	82.620	0.0187		0.94
	2	10.76	34.0	D13	20.0	6.34	52.541	0.0132		0.66
	1	10.25	34.0	D13	20.0	6.34	50.051	0.0128		0.64
II	9	0.19	34.0	D13	20.0	6.34	0.928	0.0038	0.0040 × 5.0 = 0.0200	0.19
	8	0.81	34.0	D13	20.0	6.34	3.955	0.0044		0.22
	7	1.62	34.0	D13	20.0	6.34	7.910	0.0051		0.26
	6	2.43	34.0	D13	20.0	6.34	11.866	0.0058		0.29
	5	3.30	34.0	D13	20.0	6.34	16.114	0.0066		0.33
	4	4.23	34.0	D13	20.0	6.34	20.655	0.0074		0.37
	3	5.04	34.0	D13	20.0	6.34	24.610	0.0081		0.41
	2	4.23	34.0	D13	20.0	6.34	20.655	0.0074		0.37
	1	6.25	34.0	D13	20.0	6.34	30.519	0.0092		0.46
III	9	70.94	34.0	D16, D16	10.0	19.86	114.588	0.0192	0.0040 × 5.0 = 0.0200	0.96
	8	71.82	34.0	D16, D16	10.0	19.86	116.010	0.0194		0.97
	7	73.83	34.0	D16, D16	10.0	19.86	119.256	0.0198		0.99
	6	77.68	34.0	D16, D19	10.0	24.26	103.556	0.0176		0.88
	5	82.52	34.0	D16, D19	10.0	24.26	110.008	0.0185		0.93
	4	84.86	34.0	D16, D19	10.0	24.26	113.128	0.0190		0.95
	3	79.97	34.0	D16, D19	10.0	24.26	106.609	0.0180		0.90
	2	52.23	34.0	D13, D16	10.0	16.27	102.220	0.0176		0.88
	1	0.00	34.0	D13	20.0	6.34	0.000	0.0036		0.18
I'	9	2.12	34.0	D13	20.0	6.34	10.352	0.0055	0.0040 × 5.0 = 0.0200	0.28
	8	3.73	34.0	D13	20.0	6.34	18.214	0.0070		0.35
	7	6.53	34.0	D13	20.0	6.34	31.886	0.0094		0.47
	6	9.77	34.0	D13	20.0	6.34	47.707	0.0123		0.62
	5	13.00	34.0	D13	20.0	6.34	63.479	0.0152		0.76
	4	15.99	34.0	D13	20.0	6.34	78.079	0.0179		0.90
	3	16.92	34.0	D13	20.0	6.34	82.620	0.0187		0.94
	2	10.76	34.0	D13	20.0	6.34	52.541	0.0132		0.66
	1	10.25	34.0	D13	20.0	6.34	50.051	0.0128		0.64
III'	9	70.94	34.0	D16, D16	10.0	19.86	114.588	0.0192	0.0040 × 5.0 = 0.0200	0.96
	8	71.82	34.0	D16, D16	10.0	19.86	116.010	0.0194		0.97
	7	73.83	34.0	D16, D16	10.0	19.86	119.256	0.0198		0.99
	6	77.68	34.0	D16, D19	10.0	24.26	103.556	0.0176		0.88
	5	82.52	34.0	D16, D19	10.0	24.26	110.008	0.0185		0.93
	4	84.86	34.0	D16, D19	10.0	24.26	113.128	0.0190		0.95
	3	79.97	34.0	D16, D19	10.0	24.26	106.609	0.0180		0.90
	2	52.23	34.0	D13, D16	10.0	16.27	102.220	0.0176		0.88
	1	0.00	34.0	D13	20.0	6.34	0.000	0.0036		0.18

I', III': corrected moment at corners

iii) Compression stress of serviceability verification (Lateral reinforcement / Inner reinforcement)

γ_i=1.00, N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	J	$\gamma_i \cdot \sigma_c'$ (N/mm^2)		$0.4 \cdot f'_{ck}$ (N/mm^2)
I	9	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	8	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	7	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	6	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	5	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	4	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	3	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	2	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	1	8.27	34.0	D13	20.0	6.34	0.950	1.01	≤	12.00
II	9	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	8	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	7	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	6	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	5	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	4	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	3	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	2	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	1	5.04	34.0	D13	20.0	6.34	0.950	0.61	≤	12.00
III	9	60.20	34.0	D16, D16	10.0	19.86	0.917	4.55	≤	12.00
	8	60.42	34.0	D16, D16	10.0	19.86	0.917	4.57	≤	12.00
	7	61.55	34.0	D16, D16	10.0	19.86	0.917	4.65	≤	12.00
	6	64.25	34.0	D16, D19	10.0	24.26	0.909	4.50	≤	12.00
	5	67.78	34.0	D16, D19	10.0	24.26	0.909	4.75	≤	12.00
	4	69.28	34.0	D16, D19	10.0	24.26	0.909	4.85	≤	12.00
	3	64.89	34.0	D16, D19	10.0	24.26	0.909	4.55	≤	12.00
	2	42.13	34.0	D13, D16	10.0	16.27	0.924	3.44	≤	12.00
	1	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
I’	9	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	8	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	7	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	6	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	5	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	4	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	3	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	2	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00
	1	8.27	34.0	D13	20.0	6.34	0.950	1.01	≤	12.00
III’	9	62.88	34.0	D16, D16	10.0	19.86	0.917	4.76	≤	12.00
	8	63.04	34.0	D16, D16	10.0	19.86	0.917	4.77	≤	12.00
	7	64.15	34.0	D16, D16	10.0	19.86	0.917	4.85	≤	12.00
	6	66.90	34.0	D16, D19	10.0	24.26	0.909	4.69	≤	12.00
	5	70.51	34.0	D16, D19	10.0	24.26	0.909	4.94	≤	12.00
	4	72.01	34.0	D16, D19	10.0	24.26	0.909	5.04	≤	12.00
	3	67.40	34.0	D16, D19	10.0	24.26	0.909	4.72	≤	12.00
	2	43.72	34.0	D13, D16	10.0	16.27	0.924	3.57	≤	12.00
	1	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤	12.00

I’, III’: corrected moment at corners

iv) Safety verification (Lateral reinforcement / Outer reinforcement)

B=100cm

NO.		M_d (kN·m)	d (cm)	A_{sn} (cm^2)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	M_{ud} (kN·m)	γ_i	$\gamma_i \cdot M_d / M_{ud}$
I	9	44.13	32.0	4.90	D13, D13	10.0	12.67	122.737	1.10	0.40*
	8	43.68	32.0	4.85	D13, D13	10.0	12.67	122.737	1.10	0.39*
	7	46.06	32.0	5.12	D13, D13	10.0	12.67	122.737	1.10	0.41*
	6	48.89	32.0	5.44	D13, D13	10.0	12.67	122.737	1.10	0.44*
	5	51.62	32.0	5.75	D13, D13	10.0	12.67	122.737	1.10	0.46*
	4	53.56	32.0	5.97	D13, D16	10.0	16.27	156.004	1.10	0.38*
	3	50.40	32.0	5.61	D13, D13	10.0	12.67	122.737	1.10	0.45*
	2	29.67	32.0	3.28	D16	20.0	9.93	96.947	1.10	0.34*
	1	11.09	38.6	1.01	D13	20.0	6.34	75.648	1.10	0.16
II	9	10.46	32.0	1.15	D13	20.0	6.34	62.524	1.10	0.18
	8	10.77	32.0	1.18	D13	20.0	6.34	62.524	1.10	0.19
	7	11.53	32.0	1.27	D13	20.0	6.34	62.524	1.10	0.20
	6	12.19	32.0	1.34	D13	20.0	6.34	62.524	1.10	0.21
	5	13.00	32.0	1.43	D13	20.0	6.34	62.524	1.10	0.23
	4	13.95	32.0	1.54	D13	20.0	6.34	62.524	1.10	0.25
	3	14.60	32.0	1.61	D13	20.0	6.34	62.524	1.10	0.26
	2	11.01	32.0	1.21	D13	20.0	6.34	62.524	1.10	0.19
	1	6.84	38.6	0.62	D13	20.0	6.34	62.524	1.10	0.10
III	9	81.62	38.6	7.55	D16	20.0	9.93	117.502	1.10	0.76
	8	75.51	38.6	6.97	D16	20.0	9.93	117.502	1.10	0.71
	7	70.17	38.6	6.47	D16	20.0	9.93	117.502	1.10	0.66
	6	67.08	38.6	6.18	D13	20.0	6.34	75.648	1.10	0.98
	5	64.91	38.6	5.98	D16	20.0	9.93	117.502	1.10	0.61*
	4	70.49	38.6	6.50	D16	20.0	9.93	117.502	1.10	0.66
	3	75.96	38.6	7.01	D13, D13	10.0	12.67	148.968	1.10	0.56*
	2	55.98	38.6	5.15	D16	20.0	9.93	117.502	1.10	0.52*
	1	0.00	38.6	0.00	D13	20.0	6.34	75.648	—	0.00
I’	9	44.13	32.0	4.90	D13, D13	10.0	12.67	122.737	1.10	0.40*
	8	43.68	32.0	4.85	D13, D13	10.0	12.67	122.737	1.10	0.39*
	7	46.06	32.0	5.12	D13, D13	10.0	12.67	122.737	1.10	0.41*
	6	48.89	32.0	5.44	D13, D13	10.0	12.67	122.737	1.10	0.44*
	5	51.62	32.0	5.75	D13, D13	10.0	12.67	122.737	1.10	0.46*
	4	53.56	32.0	5.97	D13, D16	10.0	16.27	156.004	1.10	0.38*
	3	50.40	32.0	5.61	D13, D13	10.0	12.67	122.737	1.10	0.45*
	2	29.67	32.0	3.28	D16	20.0	9.93	96.947	1.10	0.34*
	1	11.09	38.6	1.01	D13	20.0	6.34	75.648	1.10	0.16
III’	9	81.62	38.6	7.55	D16	20.0	9.93	117.502	1.10	0.76
	8	75.51	38.6	6.97	D16	20.0	9.93	117.502	1.10	0.71
	7	70.17	38.6	6.47	D16	20.0	9.93	117.502	1.10	0.66
	6	67.08	38.6	6.18	D16	20.0	9.93	117.502	1.10	0.63*
	5	64.91	38.6	5.98	D16	20.0	9.93	117.502	1.10	0.61*
	4	74.01	38.6	6.83	D13, D13	10.0	12.67	148.968	1.10	0.55*
	3	79.75	38.6	7.37	D13, D13	10.0	12.67	148.968	1.10	0.59*
	2	58.78	38.6	5.41	D16	20.0	9.93	117.502	1.10	0.55*
	1	0.00	38.6	0.00	D13	20.0	6.34	75.648	—	0.00

*: reinforcement is determined by serviceability verification

I’, III’: corrected moment at corners

v) Crack width of serviceability verification (Lateral reinforcement / Outer reinforcement)

Young's Modulus Ratio *N*=7.1, *B*=100cm

NO.		M_S (kN·m)	*d* (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	σ_{se} (N/mm^2)	Crack width *w* (cm)	Limit crack width w_a	w / w_a
I	9	36.26	32.0	D13, D13	10.0	12.67	96.189	0.0218	0.0035 × 7.0 = 0.0245	0.89
	8	35.63	32.0	D13, D13	10.0	12.67	94.518	0.0215		0.88
	7	37.16	32.0	D13, D13	10.0	12.67	98.577	0.0222		0.91
	6	39.00	32.0	D13, D13	10.0	12.67	103.458	0.0232		0.95
	5	40.75	32.0	D13, D13	10.0	12.67	108.100	0.0240		0.98
	4	41.88	32.0	D13, D16	10.0	16.27	87.274	0.0201		0.82
	3	39.12	32.0	D13, D13	10.0	12.67	103.776	0.0232		0.95
	2	22.90	32.0	D16	20.0	9.93	76.911	0.0218		0.89
	1	5.04	38.6	D13	20.0	6.34	21.613	0.0094		0.38
II	9	8.65	32.0	D13	20.0	6.34	44.946	0.0147	0.0035 × 7.0 = 0.0245	0.60
	8	8.81	32.0	D13	20.0	6.34	45.778	0.0149		0.61
	7	9.31	32.0	D13	20.0	6.34	48.376	0.0155		0.63
	6	9.72	32.0	D13	20.0	6.34	50.506	0.0159		0.65
	5	10.25	32.0	D13	20.0	6.34	53.260	0.0166		0.68
	4	10.90	32.0	D13	20.0	6.34	56.638	0.0173		0.71
	3	11.31	32.0	D13	20.0	6.34	58.768	0.0178		0.73
	2	8.46	32.0	D13	20.0	6.34	43.959	0.0145		0.59
	1	3.11	38.6	D13	20.0	6.34	13.337	0.0075		0.31
III	9	1.31	38.6	D16	20.0	9.93	3.628	0.0053	0.0035 × 7.0 = 0.0245	0.22
	8	7.28	38.6	D16	20.0	9.93	20.160	0.0090		0.37
	7	13.69	38.6	D16	20.0	9.93	37.911	0.0130		0.53
	6	20.03	38.6	D13	20.0	6.34	85.896	0.0239		0.98
	5	26.57	38.6	D16	20.0	9.93	73.579	0.0210		0.86
	4	32.04	38.6	D16	20.0	9.93	88.727	0.0245		1.00
	3	34.53	38.6	D13, D13	10.0	12.67	75.480	0.0179		0.73
	2	25.45	38.6	D16	20.0	9.93	70.477	0.0203		0.83
	1	0.00	38.6	D13	20.0	6.34	0.000	0.0045		0.18
I'	9	36.26	32.0	D13, D13	10.0	12.67	96.189	0.0218	0.0035 × 7.0 = 0.0245	0.89
	8	35.63	32.0	D13, D13	10.0	12.67	94.518	0.0215		0.88
	7	37.16	32.0	D13, D13	10.0	12.67	98.577	0.0222		0.91
	6	39.00	32.0	D13, D13	10.0	12.67	103.458	0.0232		0.95
	5	40.75	32.0	D13, D13	10.0	12.67	108.100	0.0240		0.98
	4	41.88	32.0	D13, D16	10.0	16.27	87.274	0.0201		0.82
	3	39.12	32.0	D13, D13	10.0	12.67	103.776	0.0232		0.95
	2	22.90	32.0	D16	20.0	9.93	76.911	0.0218		0.89
	1	5.04	38.6	D13	20.0	6.34	21.613	0.0094		0.38
III'	9	1.37	38.6	D16	20.0	9.93	3.794	0.0054	0.0035 × 7.0 = 0.0245	0.22
	8	7.64	38.6	D16	20.0	9.93	21.157	0.0093		0.38
	7	14.37	38.6	D16	20.0	9.93	39.794	0.0134		0.55
	6	21.03	38.6	D16	20.0	9.93	58.237	0.0176		0.72
	5	27.90	38.6	D16	20.0	9.93	77.262	0.0219		0.89
	4	33.64	38.6	D13, D13	10.0	12.67	73.534	0.0175		0.71
	3	36.26	38.6	D13, D13	10.0	12.67	79.261	0.0186		0.76
	2	26.72	38.6	D16	20.0	9.93	73.994	0.0211		0.86
	1	0.00	38.6	D13	20.0	6.34	0.000	0.0045		0.18

I', III': corrected moment at corners

vi) Compression stress of serviceability verification (Lateral reinforcement / Outer reinforcement)

γ_i=1.00, N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	J	$\gamma_i \cdot \sigma_c'$ (N/mm^2)		$0.4 \cdot f'_{ck}$ (N/mm^2)
I	9	30.64	32.0	D13, D13	10.0	12.67	0.930	3.05	≤	12.00
	8	29.97	32.0	D13, D13	10.0	12.67	0.930	2.99	≤	12.00
	7	31.02	32.0	D13, D13	10.0	12.67	0.930	3.09	≤	12.00
	6	32.28	32.0	D13, D13	10.0	12.67	0.930	3.22	≤	12.00
	5	33.48	32.0	D13, D13	10.0	12.67	0.930	3.34	≤	12.00
	4	34.18	32.0	D13, D16	10.0	16.27	0.922	3.08	≤	12.00
	3	31.74	32.0	D13, D13	10.0	12.67	0.930	3.16	≤	12.00
	2	18.51	32.0	D16	20.0	9.93	0.937	2.04	≤	12.00
	1	0.00	38.6	D13	20.0	6.34	0.953	0.00	≤	12.00
II	9	7.34	32.0	D13	20.0	6.34	0.949	0.98	≤	12.00
	8	7.41	32.0	D13	20.0	6.34	0.949	0.99	≤	12.00
	7	7.77	32.0	D13	20.0	6.34	0.949	1.04	≤	12.00
	6	8.05	32.0	D13	20.0	6.34	0.949	1.07	≤	12.00
	5	8.42	32.0	D13	20.0	6.34	0.949	1.12	≤	12.00
	4	8.89	32.0	D13	20.0	6.34	0.949	1.19	≤	12.00
	3	9.17	32.0	D13	20.0	6.34	0.949	1.22	≤	12.00
	2	6.81	32.0	D13	20.0	6.34	0.949	0.91	≤	12.00
	1	0.00	38.6	D13	20.0	6.34	0.953	0.00	≤	12.00
III	9	0.00	38.6	D16	20.0	9.93	0.942	0.00	≤	12.00
	8	0.00	38.6	D16	20.0	9.93	0.942	0.00	≤	12.00
	7	0.00	38.6	D16	20.0	9.93	0.942	0.00	≤	12.00
	6	0.00	38.6	D13	20.0	6.34	0.953	0.00	≤	12.00
	5	0.00	38.6	D16	20.0	9.93	0.942	0.00	≤	12.00
	4	0.00	38.6	D16	20.0	9.93	0.942	0.00	≤	12.00
	3	0.00	38.6	D13, D13	10.0	12.67	0.935	0.00	≤	12.00
	2	0.00	38.6	D16	20.0	9.93	0.942	0.00	≤	12.00
	1	0.00	38.6	D13	20.0	6.34	0.953	0.00	≤	12.00
I’	9	30.64	32.0	D13, D13	10.0	12.67	0.930	3.05	≤	12.00
	8	29.97	32.0	D13, D13	10.0	12.67	0.930	2.99	≤	12.00
	7	31.02	32.0	D13, D13	10.0	12.67	0.930	3.09	≤	12.00
	6	32.28	32.0	D13, D13	10.0	12.67	0.930	3.22	≤	12.00
	5	33.48	32.0	D13, D13	10.0	12.67	0.930	3.34	≤	12.00
	4	34.18	32.0	D13, D16	10.0	16.27	0.922	3.08	≤	12.00
	3	31.74	32.0	D13, D13	10.0	12.67	0.930	3.16	≤	12.00
	2	18.51	32.0	D16	20.0	9.93	0.937	2.04	≤	12.00
	1	0.00	38.6	D13	20.0	6.34	0.953	0.00	≤	12.00
III’	9	0.00	38.6	D16	20.0	9.93	0.942	0.00	≤	12.00
	8	0.00	38.6	D16	20.0	9.93	0.942	0.00	≤	12.00
	7	0.00	38.6	D16	20.0	9.93	0.942	0.00	≤	12.00
	6	0.00	38.6	D16	20.0	9.93	0.942	0.00	≤	12.00
	5	0.00	38.6	D16	20.0	9.93	0.942	0.00	≤	12.00
	4	0.00	38.6	D13, D13	10.0	12.67	0.935	0.00	≤	12.00
	3	0.00	38.6	D13, D13	10.0	12.67	0.935	0.00	≤	12.00
	2	0.00	38.6	D16	20.0	9.93	0.942	0.00	≤	12.00
	1	0.00	38.6	D13	20.0	6.34	0.953	0.00	≤	12.00

I’, III’: corrected moment at corners

vii) Safety verification (Vertical reinforcement / Inner reinforcement)

B=100cm

NO.		M_d (kN·m)	d (cm)	A_{sn} (cm^2)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	M_{ud} (kN·m)	γ_i	$\gamma_i \cdot M_d$ /M_{ud}
I	9	0.00	32.0	0.00	D13	20.0	6.34	62.524	—	0.00
	8	5.75	32.0	0.63	D13	20.0	6.34	62.524	1.10	0.10
	7	10.91	32.0	1.20	D13	20.0	6.34	62.524	1.10	0.19
	6	8.33	32.0	0.92	D13	20.0	6.34	62.524	1.10	0.15
	5	5.70	32.0	0.63	D13	20.0	6.34	62.524	1.10	0.10
	4	7.80	32.0	0.86	D13	20.0	6.34	62.524	1.10	0.14
	3	13.82	32.0	1.52	D13	20.0	6.34	62.524	1.10	0.24
	2	14.78	32.0	1.63	D13	20.0	6.34	62.524	1.10	0.26
	1	80.18	32.0	9.01	D13, D16	10.0	16.27	156.004	1.10	0.57*
II	9	0.00	32.0	0.00	D13	20.0	6.34	62.524	—	0.00
	8	1.22	32.0	0.13	D13	20.0	6.34	62.524	1.10	0.02
	7	4.65	32.0	0.51	D13	20.0	6.34	62.524	1.10	0.08
	6	3.14	32.0	0.34	D13	20.0	6.34	62.524	1.10	0.06
	5	1.54	32.0	0.17	D13	20.0	6.34	62.524	1.10	0.03
	4	2.60	32.0	0.29	D13	20.0	6.34	62.524	1.10	0.05
	3	6.16	32.0	0.68	D13	20.0	6.34	62.524	1.10	0.11
	2	8.21	32.0	0.90	D13	20.0	6.34	62.524	1.10	0.14
	1	48.62	32.0	5.41	D13, D13	10.0	12.67	122.737	1.10	0.44*
III	9	0.00	32.0	0.00	D13	20.0	6.34	62.524	—	0.00
	8	14.61	32.0	1.61	D13	20.0	6.34	62.524	1.10	0.26
	7	15.32	32.0	1.69	D13	20.0	6.34	62.524	1.10	0.27
	6	16.29	32.0	1.79	D13	20.0	6.34	62.524	1.10	0.29
	5	17.44	32.0	1.92	D13	20.0	6.34	62.524	1.10	0.31
	4	18.10	32.0	1.99	D13	20.0	6.34	62.524	1.10	0.32
	3	17.18	32.0	1.89	D13	20.0	6.34	62.524	1.10	0.30
	2	11.30	32.0	1.24	D13	20.0	6.34	62.524	1.10	0.20
	1	0.00	32.0	0.00	D13	20.0	6.34	62.524	—	0.00

*: reinforcement is determined by serviceability verification

viii) Crack width of serviceability verification (Vertical reinforcement / Inner reinforcement)

Young's Modulus Ratio N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	σ_{se} (N/mm^2)	Crack width w (cm)	Limit crack width w_a	w / w_a
I	9	0.00	32.0	D13	20.0	6.34	0.000	0.0045	0.0040 × 7.0 = 0.0280	0.16
	8	0.12	32.0	D13	20.0	6.34	0.624	0.0047		0.17
	7	0.93	32.0	D13	20.0	6.34	4.832	0.0056		0.20
	6	1.62	32.0	D13	20.0	6.34	8.418	0.0064		0.23
	5	2.30	32.0	D13	20.0	6.34	11.951	0.0072		0.26
	4	3.55	32.0	D13	20.0	6.34	18.446	0.0087		0.31
	3	6.28	32.0	D13	20.0	6.34	32.632	0.0119		0.43
	2	6.72	32.0	D13	20.0	6.34	34.918	0.0124		0.44
	1	61.71	32.0	D13, D16	10.0	16.27	128.598	0.0279		1.00
II	9	0.00	32.0	D13	20.0	6.34	0.000	0.0045	0.0040 × 7.0 = 0.0280	0.16
	8	0.00	32.0	D13	20.0	6.34	0.000	0.0045		0.16
	7	0.19	32.0	D13	20.0	6.34	0.987	0.0047		0.17
	6	0.37	32.0	D13	20.0	6.34	1.923	0.0050		0.18
	5	0.56	32.0	D13	20.0	6.34	2.910	0.0052		0.19
	4	1.18	32.0	D13	20.0	6.34	6.131	0.0059		0.21
	3	2.80	32.0	D13	20.0	6.34	14.549	0.0078		0.28
	2	3.73	32.0	D13	20.0	6.34	19.381	0.0089		0.32
	1	37.38	32.0	D13, D13	10.0	12.67	99.160	0.0224		0.80
III	9	0.00	32.0	D13	20.0	6.34	0.000	0.0045	0.0040 × 7.0 = 0.0280	0.16
	8	11.93	32.0	D13	20.0	6.34	61.989	0.0185		0.66
	7	12.34	32.0	D13	20.0	6.34	64.120	0.0190		0.68
	6	12.97	32.0	D13	20.0	6.34	67.393	0.0198		0.71
	5	13.76	32.0	D13	20.0	6.34	71.498	0.0207		0.74
	4	14.17	32.0	D13	20.0	6.34	73.629	0.0212		0.76
	3	13.33	32.0	D13	20.0	6.34	69.264	0.0202		0.72
	2	8.70	32.0	D13	20.0	6.34	45.206	0.0147		0.53
	1	0.00	32.0	D13	20.0	6.34	0.000	0.0045		0.16

ix) Compression stress of serviceability verification (Vertical reinforcement / Inner reinforcement)

γ_i=1.00, N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	J	$\gamma_i \cdot \sigma_c'$ (N/mm^2)		$0.4 \cdot f'_{ck}$ (N/mm^2)
I	9	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	8	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	7	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	6	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	5	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	4	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	3	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	2	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	1	49.76	32.0	D13, D16	10.0	16.27	0.922	4.49	≤	12.00
II	9	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	8	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	7	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	6	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	5	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	4	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	3	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	2	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	1	30.11	32.0	D13, D13	10.0	12.67	0.930	3.00	≤	12.00
III	9	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00
	8	10.03	32.0	D13	20.0	6.34	0.949	1.34	≤	12.00
	7	10.29	32.0	D13	20.0	6.34	0.949	1.37	≤	12.00
	6	10.73	32.0	D13	20.0	6.34	0.949	1.43	≤	12.00
	5	11.31	32.0	D13	20.0	6.34	0.949	1.51	≤	12.00
	4	11.56	32.0	D13	20.0	6.34	0.949	1.54	≤	12.00
	3	10.81	32.0	D13	20.0	6.34	0.949	1.44	≤	12.00
	2	7.02	32.0	D13	20.0	6.34	0.949	0.94	≤	12.00
	1	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤	12.00

x) Safety verification (Vertical reinforcement / Outer reinforcement)

B=100cm

NO.		M_d (kN·m)	d (cm)	A_{sn} (cm^2)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	M_{ud} (kN·m)	γ_i	$\gamma_i \cdot M_d$ $/M_{ud}$
I	9	51.28	30.0	6.10	D13	20.0	6.34	58.548	1.10	0.96
	8	20.19	30.0	2.38	D13	20.0	6.34	58.548	1.10	0.38
	7	7.53	30.0	0.88	D13	20.0	6.34	58.548	1.10	0.14
	6	8.19	30.0	0.96	D13	20.0	6.34	58.548	1.10	0.15
	5	8.94	30.0	1.05	D13	20.0	6.34	58.548	1.10	0.17
	4	11.14	30.0	1.31	D13	20.0	6.34	58.548	1.10	0.21
	3	17.00	30.0	2.00	D13	20.0	6.34	58.548	1.10	0.32
	2	16.27	30.0	1.91	D13	20.0	6.34	58.548	1.10	0.31
	1	66.52	36.6	6.48	D16	20.0	9.93	111.274	1.10	0.66
II	9	31.30	30.0	3.70	D13	20.0	6.34	58.548	1.10	0.59
	8	13.38	30.0	1.57	D13	20.0	6.34	58.548	1.10	0.25
	7	1.92	30.0	0.22	D13	20.0	6.34	58.548	1.10	0.04
	6	2.07	30.0	0.24	D13	20.0	6.34	58.548	1.10	0.04
	5	2.23	30.0	0.26	D13	20.0	6.34	58.548	1.10	0.04
	4	3.41	30.0	0.40	D13	20.0	6.34	58.548	1.10	0.06
	3	7.19	30.0	0.84	D13	20.0	6.34	58.548	1.10	0.14
	2	9.27	30.0	1.09	D13	20.0	6.34	58.548	1.10	0.17
	1	40.92	36.6	3.96	D13	20.0	6.34	71.671	1.10	0.63
III	9	0.00	36.6	0.00	D13	20.0	6.34	71.671	—	0.00
	8	12.57	36.6	1.21	D13	20.0	6.34	71.671	1.10	0.19
	7	11.71	36.6	1.13	D13	20.0	6.34	71.671	1.10	0.18
	6	11.19	36.6	1.08	D13	20.0	6.34	71.671	1.10	0.17
	5	10.84	36.6	1.04	D13	20.0	6.34	71.671	1.10	0.17
	4	11.77	36.6	1.13	D13	20.0	6.34	71.671	1.10	0.18
	3	12.73	36.6	1.22	D13	20.0	6.34	71.671	1.10	0.20
	2	9.31	36.6	0.89	D13	20.0	6.34	71.671	1.10	0.14
	1	0.00	36.6	0.00	D13	20.0	6.34	71.671	—	0.00

xi) Crack width of serviceability verification (Vertical reinforcement / Outer reinforcement)

Young's Modulus Ratio N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	σ_{se} (N/mm^2)	Crack width w (cm)	Limit crack width w_a	w / w_a
I	9	0.00	30.0	D13	20.0	6.34	0.000	0.0054	0.0035 × 9.0 = 0.0315	0.17
	8	5.03	30.0	D13	20.0	6.34	27.924	0.0129		0.41
	7	6.10	30.0	D13	20.0	6.34	33.864	0.0145		0.46
	6	6.54	30.0	D13	20.0	6.34	36.306	0.0152		0.48
	5	7.05	30.0	D13	20.0	6.34	39.137	0.0160		0.51
	4	8.67	30.0	D13	20.0	6.34	48.131	0.0184		0.58
	3	13.10	30.0	D13	20.0	6.34	72.723	0.0250		0.79
	2	12.42	30.0	D13	20.0	6.34	68.948	0.0240		0.76
	1	30.24	36.6	D16	20.0	9.93	88.450	0.0292		0.93
II	9	0.00	30.0	D13	20.0	6.34	0.000	0.0054	0.0035 × 9.0 = 0.0315	0.17
	8	0.92	30.0	D13	20.0	6.34	5.107	0.0068		0.22
	7	1.57	30.0	D13	20.0	6.34	8.716	0.0078		0.25
	6	1.66	30.0	D13	20.0	6.34	9.215	0.0079		0.25
	5	1.76	30.0	D13	20.0	6.34	9.770	0.0080		0.25
	4	2.65	30.0	D13	20.0	6.34	14.711	0.0094		0.30
	3	5.52	30.0	D13	20.0	6.34	30.644	0.0137		0.43
	2	6.73	30.0	D13	20.0	6.34	37.361	0.0155		0.49
	1	18.60	36.6	D13	20.0	6.34	84.226	0.0282		0.90
III	9	0.00	36.6	D13	20.0	6.34	0.000	0.0054	0.0035 × 9.0 = 0.0315	0.17
	8	1.18	36.6	D13	20.0	6.34	5.343	0.0068		0.22
	7	2.30	36.6	D13	20.0	6.34	10.415	0.0082		0.26
	6	3.36	36.6	D13	20.0	6.34	15.215	0.0095		0.30
	5	4.42	36.6	D13	20.0	6.34	20.015	0.0108		0.34
	4	5.35	36.6	D13	20.0	6.34	24.226	0.0119		0.38
	3	5.79	36.6	D13	20.0	6.34	26.219	0.0125		0.40
	2	4.23	36.6	D13	20.0	6.34	19.155	0.0106		0.34
	1	0.00	36.6	D13	20.0	6.34	0.000	0.0054		0.17

xii) Compression stress of serviceability verification (Vertical reinforcement / Outer reinforcement)

γ_i=1.00, N=7.1, B=100cm

NO.		M_S (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	J	$\gamma_i \cdot \sigma_c'$ (N/mm^2)		$0.4 \cdot f'_{ck}$ (N/mm^2)
I	9	0.00	30.0	D13	20.0	6.34	0.947	0.00	≤	12.00
	8	4.26	30.0	D13	20.0	6.34	0.947	0.63	≤	12.00
	7	5.11	30.0	D13	20.0	6.34	0.947	0.76	≤	12.00
	6	5.42	30.0	D13	20.0	6.34	0.947	0.80	≤	12.00
	5	5.79	30.0	D13	20.0	6.34	0.947	0.86	≤	12.00
	4	7.05	30.0	D13	20.0	6.34	0.947	1.04	≤	12.00
	3	10.58	30.0	D13	20.0	6.34	0.947	1.56	≤	12.00
	2	9.96	30.0	D13	20.0	6.34	0.947	1.47	≤	12.00
	1	0.00	36.6	D16	20.0	9.93	0.941	0.00	≤	12.00
II	9	0.00	30.0	D13	20.0	6.34	0.947	0.00	≤	12.00
	8	0.79	30.0	D13	20.0	6.34	0.947	0.12	≤	12.00
	7	1.32	30.0	D13	20.0	6.34	0.947	0.20	≤	12.00
	6	1.38	30.0	D13	20.0	6.34	0.947	0.20	≤	12.00
	5	1.45	30.0	D13	20.0	6.34	0.947	0.21	≤	12.00
	4	2.14	30.0	D13	20.0	6.34	0.947	0.32	≤	12.00
	3	4.44	30.0	D13	20.0	6.34	0.947	0.66	≤	12.00
	2	5.39	30.0	D13	20.0	6.34	0.947	0.80	≤	12.00
	1	0.00	36.6	D13	20.0	6.34	0.952	0.00	≤	12.00
III	9	0.00	36.6	D13	20.0	6.34	0.952	0.00	≤	12.00
	8	0.00	36.6	D13	20.0	6.34	0.952	0.00	≤	12.00
	7	0.00	36.6	D13	20.0	6.34	0.952	0.00	≤	12.00
	6	0.00	36.6	D13	20.0	6.34	0.952	0.00	≤	12.00
	5	0.00	36.6	D13	20.0	6.34	0.952	0.00	≤	12.00
	4	0.00	36.6	D13	20.0	6.34	0.952	0.00	≤	12.00
	3	0.00	36.6	D13	20.0	6.34	0.952	0.00	≤	12.00
	2	0.00	36.6	D13	20.0	6.34	0.952	0.00	≤	12.00
	1	0.00	36.6	D13	20.0	6.34	0.952	0.00	≤	12.00

(5) Fatigue Failure

If the calculated fatigue life under the load corresponding to the maximum wave height exceeds 2×10^6 cycles, further safety verification regarding fatigue failure is not required. If the fatigue life is equal to or less than 2×10^6 cycles, fatigue life shall be calculated for each wave height class. For those classes with fatigue life not exceeding 2×10^6 cycles, a detailed fatigue strength verification shall be performed.

This calculation shall be conducted for structural members subject to wave action, such as the bottom slab and the face wall (longitudinal direction). The following provides an example of the verification method for the bottom slab.

Design tidal level: +1.00 (m)
Design wave height: $H_D = 6.50$ (m)

1) Design Load

i) Variable situation (Wave motion, Wave crest)

✓ Load calculation

- Self-weight of each chamber: D

D = Cover concrete weight + Self-weight of Infill sand + Self-weight of Bottom slab
= 0.500 × 22.60 + 11.100 × 20.00 + 0.700 × 24.00 = 250.10 (kN/m^2)

- Water pressure: F

F = (M.W.L - Installed depth) × γ_w
= (1.000 - (-9.800)) × 10.10 = 109.08 (kN/m^2)

- Bottom reaction force: R

Seaside: R =4.14 (kN/m^2), Landside: R = 316.81 (kN/m^2) , Width: B = 13.50 (m)

- Uplift force: U

U = 32.88 (kN/m^2)

✓ Design load

The combined load in the wave motion is distributed as shown in Figure 2.43.

Seaside: $W = 1.0D + 1.0F + 1.0R + 1.0U$
= - 1.0 × 250.10 + 1.0 × 109.08 +1.0 × 4.14 + 1.0× 32.88
= -104.00 (kN/m^2)

Landside: $W = 1.0D + 1.0F + 1.0R + 1.0U$
= - 1.0 × 250.10 + 1.0 × 109.08 +1.0 × 316.81 + 1.0× 0.00
= 175.79 (kN/m^2)

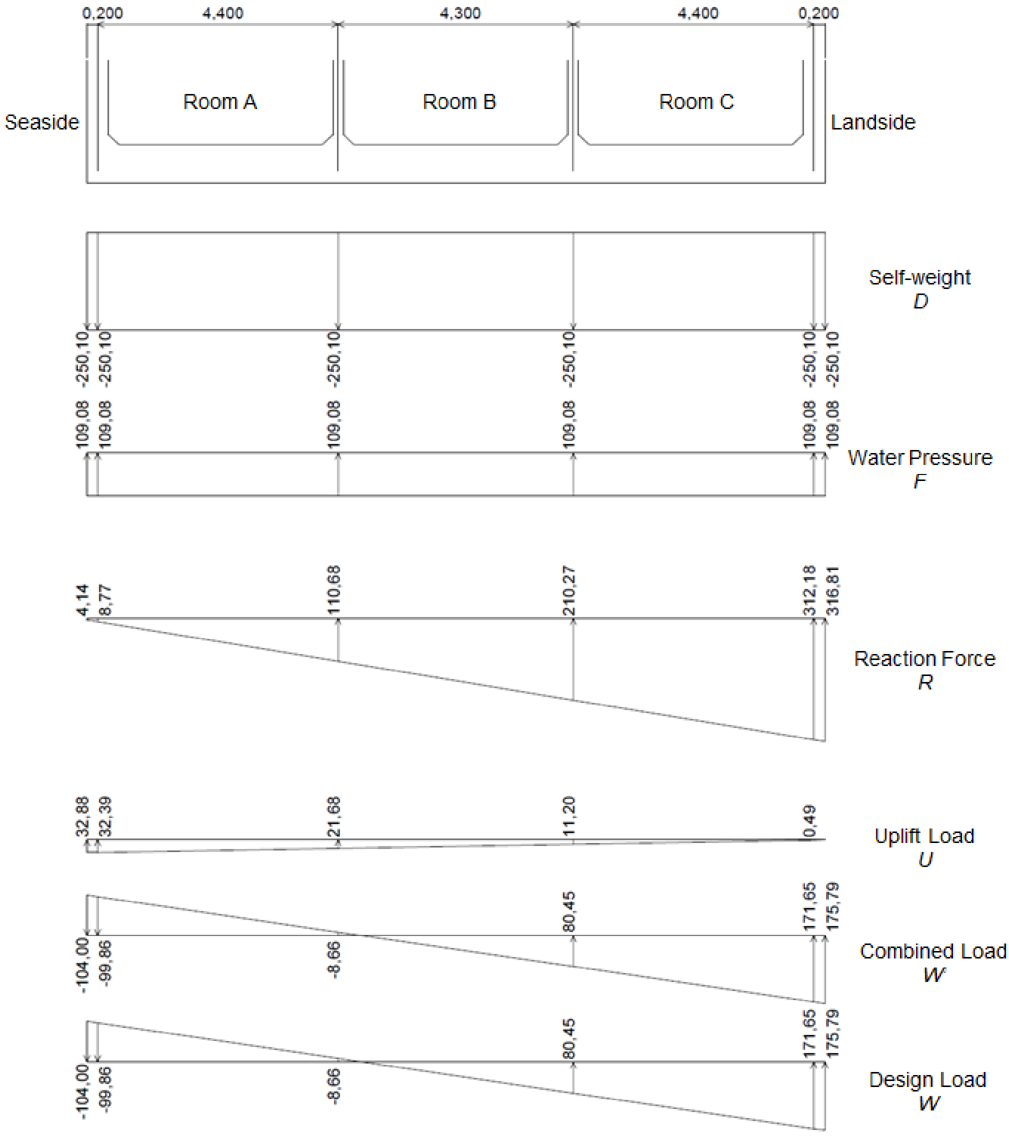


Figure 2.43- Design Load in Wave Motion (Wave Crest, Fatigue Failure)

ii) Variable situation (Wave motion, Wave trough)

✓ Load calculation

- Self-weight of each chamber: D

D = Cover concrete weight + Self-weight of Infill sand + Self-weight of Bottom slab
$= 0.500 \times 22.60 + 11.100 \times 20.00 + 0.700 \times 24.00 = 250.10$ (kN/m^2)

- Water pressure: F

F = (M.W.L - Installed depth) $\times \gamma_w$
$= (1.000 - (-9.800)) \times 10.10 = 109.08$ (kN/m^2)

- Bottom reaction force: R

Seaside: R =269.72 (kN/m^2), Landside: R = 116.95 (kN/m^2) , Width: B = 13.50 (m)

- Uplift force: U

U = -32.84 (kN/m^2)

✓ Design load

The combined load in the wave motion is distributed as shown in Figure 2.44.

Seaside: $W = 1.0D + 1.0F + 1.0R + 1.0U$

$= -1.0 \times 250.10 + 1.0 \times 109.08 + 1.0 \times 269.72 + 1.0 \times (-32.84)$

$= 95.86$ (kN/m^2)

Landside: $W = 1.0D + 1.0F + 1.0R + 1.0U$

$= -1.0 \times 250.10 + 1.0 \times 109.08 + 1.0 \times 116.95 + 1.0 \times 0.00$

$= -24.07$ (kN/m^2)

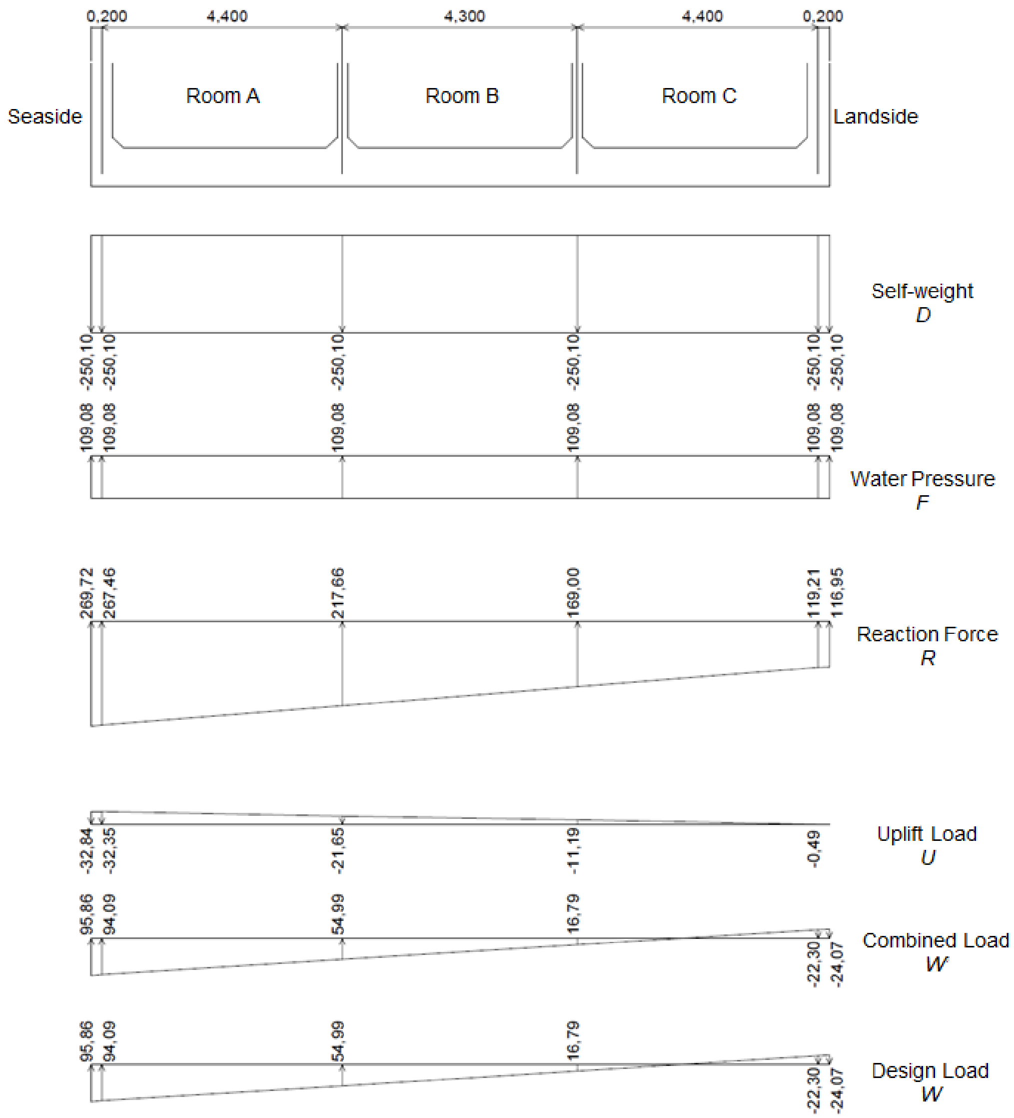


Figure 2.44- Design Load in Wave Motion (Wave Trough, Fatigue Failure)

iii) Permanent state

✓ Load calculation

- Self-weight of each chamber: D

D = Cover concrete weight + Self-weight of Infill sand + Self-weight of Bottom slab

$= 0.500 \times 22.60 + 11.100 \times 20.00 + 0.700 \times 24.00 = 250.10$ (kN/m^2)

- Water pressure: F

F = (M.W.L - Installed depth) × γ_w

= (1.000 - (-9.800)) × 10.10 = 109.08 (kN/m^2)

- Bottom reaction force: R

Seaside: R =190.91 (kN/m^2), Landside: R = 162.92 (kN/m^2) , Width: B = 13.50 (m)

✓ Design load

The combined load in the permanent state is distributed as shown in Figure 2.45.

Seaside: W = 1.0D + 1.0F +1.0R

= - 1.0 × 250.10 + 1.0 × 109.08 +1.0 × 190.91

= 49.89 (kN/m^2)

Landside: W = 1.0D + 1.0F +1.0R

= - 1.0 × 250.10 + 1.0 × 109.08 +1.0 × 162.92

= 21.90 (kN/m^2)

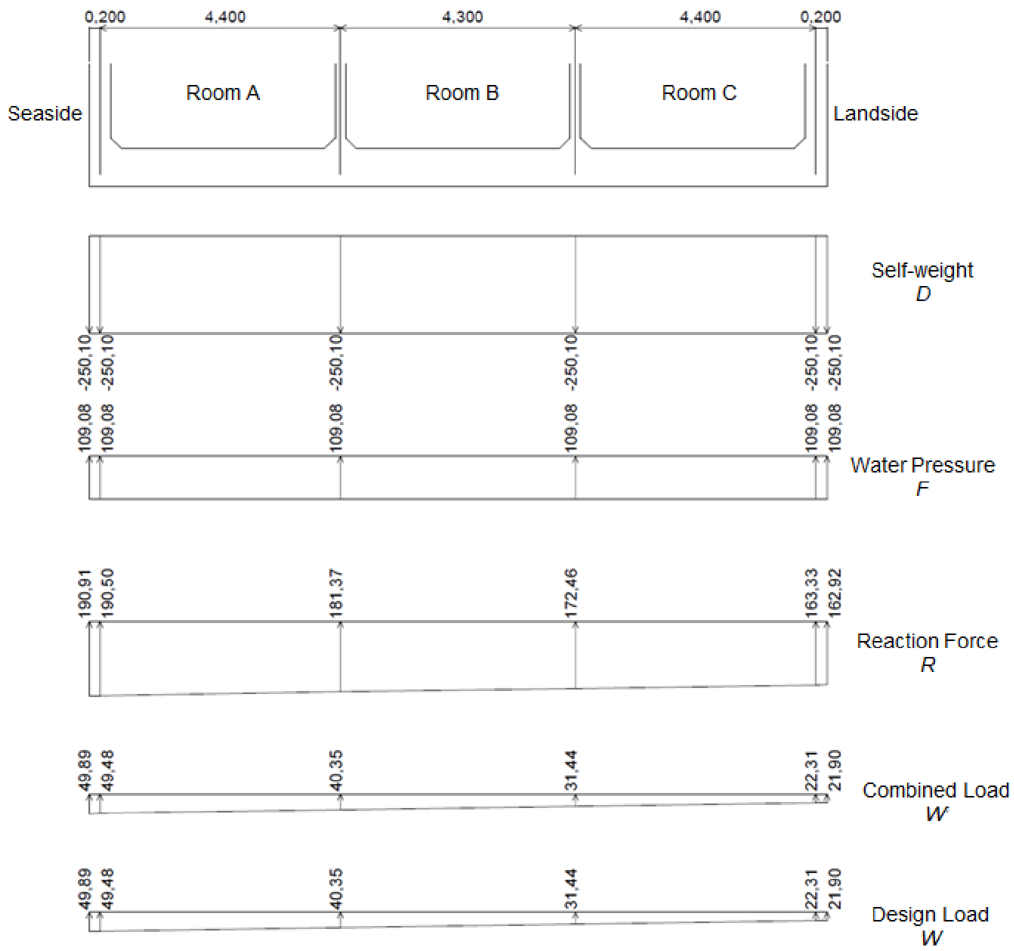


Figure 2.45- Design Load in Permanent State (Fatigue Failure)

2) Sectional Force

i) Room A: Variable situation, Wave crest (downward load)

P_1 = -8.66 (kN/m^2)

P_2 = -99.86 (kN/m^2)

L_X = 4.400 (m)

L_Y = 4.200 (m)

$\lambda = 4.400/4.200 = 1.05$, The coefficient table for $\lambda = 1.00$ is applied.

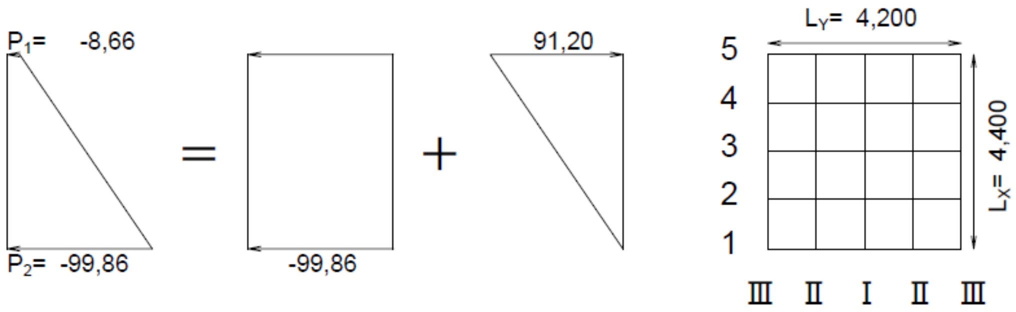


Figure 2.46- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$P = -99.86$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = -99.86 \times 4.200^2 \times X = -1{,}761.53 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = -99.86 \times 4.200^2 \times Y = -1{,}761.53 \times Y$ (kN·m)

Table 2.54- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0513	90.37	-0.0086	15.15
	4	0.0096	-16.91	0.0116	-20.43
	3	0.0206	-36.29	0.0206	-36.29
	2	0.0096	-16.91	0.0116	-20.43
	1	-0.0513	90.37	-0.0086	15.15
II	5	-0.0324	57.07	-0.0054	9.51
	4	0.0059	-10.39	0.0059	-10.39
	3	0.0116	-20.43	0.0096	-16.91
	2	0.0059	-10.39	0.0059	-10.39
	1	-0.0324	57.07	-0.0054	9.51
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0054	9.51	-0.0324	57.07
	3	-0.0086	15.15	-0.0513	90.37
	2	-0.0054	9.51	-0.0324	57.07
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$P = 91.20$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = 91.20 \times 4.200^2 \times X = 1{,}608.77 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = 91.20 \times 4.200^2 \times Y = 1{,}608.77 \times Y$ (kN·m)

Table 2.55- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0334	-53.73	-0.0056	-9.01
	4	0.0080	12.87	0.0069	11.10
	3	0.0103	16.57	0.0103	16.57
	2	0.0015	2.41	0.0047	7.56
	1	-0.0179	-28.80	-0.0030	-4.83

II	5	-0.0223	-35.88	-0.0037	-5.95
	4	0.0052	8.37	0.0040	6.44
	3	0.0058	9.33	0.0048	7.72
	2	0.0006	0.97	0.0018	2.90
	1	-0.0101	-16.25	-0.0017	-2.73
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0036	-5.79	-0.0208	-33.46
	3	-0.0043	-6.92	-0.0257	-41.35
	2	-0.0019	-3.06	-0.0116	-18.66
	1	0.0000	0.00	0.0000	0.00

- Combined uniformly distributed load and triangularly distributed load

Table 2.56- Summary of Sectional Force (Room A, Variable situation (downward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	36.64	6.14
	4	-4.04	-9.33
	3	-19.72	-19.72
	2	-14.50	-12.87
	1	61.57	10.32
II	5	21.19	3.56
	4	-2.02	-3.95
	3	-11.10	-9.19
	2	-9.42	-7.49
	1	40.82	6.78
III	5	0.00	0.00
	4	3.72	23.61
	3	8.23	49.02
	2	6.45	38.41
	1	0.00	0.00

ii) Room A: Variable situation, Wave trough (upward load)

P_1 = 54.99 (kN/m^2)

P_2 = 94.09 (kN/m^2)

L_X = 4.400 (m)

L_Y = 4.200 (m)

λ = 4.400/4.200 = 1.05, The coefficient table for λ =1.00 is applied.

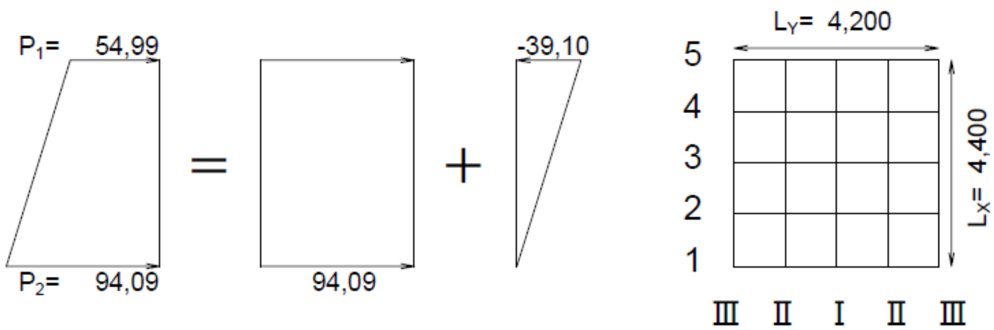


Figure 2.47- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$P = 94.09$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = 94.09 \times 4.200^2 \times X = 1{,}659.75 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = 94.09 \times 4.200^2 \times Y = 1{,}659.75 \times Y$ (kN·m)

Table 2.57- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0513	-85.15	-0.0086	-14.27
	4	0.0096	15.93	0.0116	19.25
	3	0.0206	34.19	0.0206	34.19
	2	0.0096	15.93	0.0116	19.25
	1	-0.0513	-85.15	-0.0086	-14.27
II	5	-0.0324	-53.78	-0.0054	-8.96
	4	0.0059	9.79	0.0059	9.79
	3	0.0116	19.25	0.0096	15.93
	2	0.0059	9.79	0.0059	9.79
	1	-0.0324	-53.78	-0.0054	-8.96
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0054	-8.96	-0.0324	-53.78
	3	-0.0086	-14.27	-0.0513	-85.15
	2	-0.0054	-8.96	-0.0324	-53.78
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$P = -39.10$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = -39.10 \times 4.200^2 \times X = -689.72 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = -39.10 \times 4.200^2 \times Y = -689.72 \times Y$ (kN·m)

Table 2.58- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0334	23.04	-0.0056	3.86
	4	0.0080	-5.52	0.0069	-4.76
	3	0.0103	-7.10	0.0103	-7.10
	2	0.0015	-1.03	0.0047	-3.24
	1	-0.0179	12.35	-0.0030	2.07
II	5	-0.0223	15.38	-0.0037	2.55
	4	0.0052	-3.59	0.0040	-2.76
	3	0.0058	-4.00	0.0048	-3.31
	2	0.0006	-0.41	0.0018	-1.24
	1	-0.0101	6.97	-0.0017	1.17
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0036	2.48	-0.0208	14.35
	3	-0.0043	2.97	-0.0257	17.73
	2	-0.0019	1.31	-0.0116	8.00
	1	0.0000	0.00	0.0000	0.00

- Combined uniformly distributed load and triangularly distributed load

Table 2.59- Summary of Sectional Force (Room A, Variable situation (upward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	-62.11	-10.41
	4	10.41	14.49
	3	27.09	27.09
	2	14.90	16.01
	1	-72.80	-12.20
II	5	-38.40	-6.41
	4	6.20	7.03
	3	15.25	12.62
	2	9.38	8.55
	1	-46.81	-7.79
III	5	0.00	0.00
	4	-6.48	-39.43
	3	-11.30	-67.42
	2	-7.65	-45.78
	1	0.00	0.00

iii) Room A: Permanent state (upward load)

$P_1 = 40.35$ (kN/m^2)
$P_2 = 49.48$ (kN/m^2)
$L_X = 4.400$ (m)
$L_Y = 4.200$ (m)
$\lambda = 4.400/4.200 = 1.05$, The coefficient table for $\lambda = 1.00$ is applied.

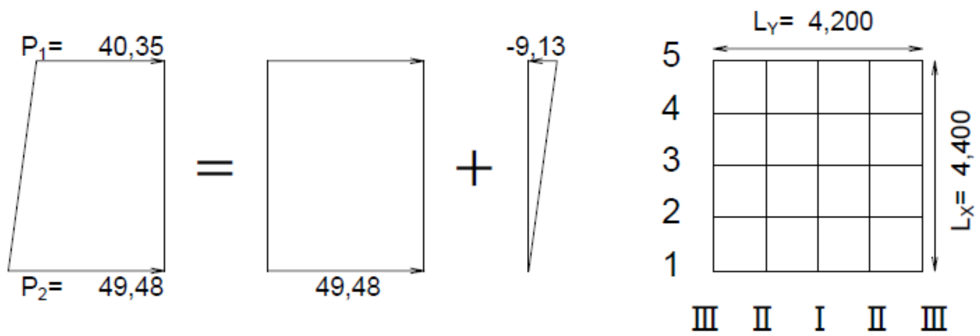


Figure 2.48- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$P = 49.48$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = 49.48 \times 4.200^2 \times X = 872.83 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = 49.48 \times 4.200^2 \times Y = 872.83 \times Y$ (kN·m)

Table 2.60- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0513	-44.78	-0.0086	-7.51
	4	0.0096	8.38	0.0116	10.12
	3	0.0206	17.98	0.0206	17.98

	2	0.0096	8.38	0.0116	10.12
	1	-0.0513	-44.78	-0.0086	-7.51
II	5	-0.0324	-28.28	-0.0054	-4.71
	4	0.0059	5.15	0.0059	5.15
	3	0.0116	10.12	0.0096	8.38
	2	0.0059	5.15	0.0059	5.15
	1	-0.0324	-28.28	-0.0054	-4.71
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0054	-4.71	-0.0324	-28.28
	3	-0.0086	-7.51	-0.0513	-44.78
	2	-0.0054	-4.71	-0.0324	-28.28
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$P = -9.13$ (kN/m^2)
$M_X = P \cdot L_Y^2 \cdot X = -9.13 \times 4.200^2 \times X = -161.05 \times X$ (kN·m)
$M_Y = P \cdot L_Y^2 \cdot Y = -9.13 \times 4.200^2 \times Y = -161.05 \times Y$ (kN·m)

Table 2.61- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0334	5.38	-0.0056	0.90
	4	0.0080	-1.29	0.0069	-1.11
	3	0.0103	-1.66	0.0103	-1.66
	2	0.0015	-0.24	0.0047	-0.76
	1	-0.0179	2.88	-0.0030	0.48
II	5	-0.0223	3.59	-0.0037	0.60
	4	0.0052	-0.84	0.0040	-0.64
	3	0.0058	-0.93	0.0048	-0.77
	2	0.0006	-0.10	0.0018	-0.29
	1	-0.0101	1.63	-0.0017	0.27
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0036	0.58	-0.0208	3.35
	3	-0.0043	0.69	-0.0257	4.14
	2	-0.0019	0.31	-0.0116	1.87
	1	0.0000	0.00	0.0000	0.00

- Combined uniformly distributed load and triangularly distributed load

Table 2.62- Summary of Sectional Force (Room A, Permanent state (upward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	-39.40	-6.61
	4	7.09	9.01
	3	16.32	16.32
	2	8.14	9.36
	1	-41.90	-7.03
II	5	-24.69	-4.11
	4	4.31	4.51
	3	9.19	7.61

	2	5.05	4.86
	1	-26.65	-4.44
III	5	0.00	0.00
	4	-4.13	-24.93
	3	-6.82	-40.64
	2	-4.40	-26.41
	1	0.00	0.00

3) Stress Calculation

i) Transverse direction (Upper part, Midspan)

	$N = 7.1, B = 100$ (cm), $As = 6.34$ (cm^2), $As' = 6.34$ (cm^2), $d = 64.0$ (cm), $d' = 8.0$ (cm)							
	M (kN·m)	P	P'	k	σ_C (N/mm^2)	σ_C' (N/mm^2)	σ_S (Inner) (N/mm^2)	σ_S (Outer) (N/mm^2)
Wave crest	0.00	0.000991	0.000991	0.113	0.000	0.000	0.000	0.000
Wave trough	27.09	0.000991	0.000991	0.113	1.231	0.923	68.606	-0.928
Permanent	16.32	0.000991	0.000991	0.113	0.742	0.557	41.353	-0.559

ii) Transverse direction (Lower part, Midspan)

	$N = 7.1, B = 100$ (cm), $As = 6.34$ (cm^2), $As' = 6.34$ (cm^2), $d = 62.0$ (cm), $d' = 8.0$ (cm)							
	M (kN·m)	P	P'	k	σ_C (N/mm^2)	σ_C' (N/mm^2)	σ_S (Inner) (N/mm^2)	σ_S (Outer) (N/mm^2)
Wave crest	-19.72	0.001023	0.001023	0.113	-0.927	-0.695	-0.945	-51.663
Wave trough	0.00	0.001023	0.001023	0.113	0.000	0.000	0.000	0.000
Permanent	0.00	0.001023	0.001023	0.113	0.000	0.000	0.000	0.000

iii) Transverse direction (Upper part, Support)

	$N = 7.1, B = 100$ (cm), $As = 6.34$ (cm^2), $As' = 9.93$ (cm^2), $d = 64.0$ (cm), $d' = 8.0$ (cm)							
	M (kN·m)	P	P'	k	σ_C (N/mm^2)	σ_C' (N/mm^2)	σ_S (Inner) (N/mm^2)	σ_S (Outer) (N/mm^2)
Wave crest	36.64	0.000991	0.001552	0.113	1.677	1.258	93.462	-1.264
Wave trough	0.00	0.000991	0.001552	0.113	0.000	0.000	0.000	0.000
Permanent	0.00	0.000991	0.001552	0.113	0.000	0.000	0.000	0.000

iv) Transverse direction (Lower part, Support)

	$N = 7.1, B = 100$ (cm), $As = 9.93$ (cm^2), $As' = 6.34$ (cm^2), $d = 68.6$ (cm), $d' = 12.6$ (cm)							
	M (kN·m)	P	P'	k	σ_C (N/mm^2)	σ_C' (N/mm^2)	σ_S (Inner) (N/mm^2)	σ_S (Outer) (N/mm^2)
Wave crest	0.00	0.001448	0.000924	0.136	0.000	0.000	0.000	0.000
Wave trough	-62.11	0.001448	0.000924	0.136	-2.094	-1.571	5.212	-94.452
Permanent	-39.40	0.001448	0.000924	0.136	-1.328	-0.996	3.305	-59.901

v) Longitudinal direction (Upper part, Midspan)

	$N = 7.1, B = 100$ (cm), $As = 6.34$ (cm^2), $As' = 6.34$ (cm^2), $d = 62.0$ (cm), $d' = 10.0$ (cm)							
	M (kN·m)	P	P'	k	σ_C (N/mm^2)	σ_C' (N/mm^2)	σ_S (Inner) (N/mm^2)	σ_S (Outer) (N/mm^2)
Wave crest	0.00	0.001023	0.001023	0.116	0.000	0.000	0.000	0.000
Wave trough	27.09	0.001023	0.001023	0.116	1.320	0.990	71.421	-3.659
Permanent	16.32	0.001023	0.001023	0.116	0.795	0.596	43.015	-2.204

vi) Longitudinal direction (Lower part, Midspan)

	$N = 7.1, B = 100$ (cm), $As = 6.34$ (cm^2), $As' = 6.34$ (cm^2), $d = 60.0$ (cm), $d' = 8.0$ (cm)							
	M (kN·m)	P	P'	k	σ_C (N/mm^2)	σ_C' (N/mm^2)	σ_S (Inner) (N/mm^2)	σ_S (Outer) (N/mm^2)
Wave crest	-19.72	0.001057	0.001057	0.116	-1.000	-0.750	1.061	-54.107
Wave trough	0.00	0.001057	0.001057	0.116	0.000	0.000	0.000	0.000
Permanent	0.00	0.001057	0.001057	0.116	0.000	0.000	0.000	0.000

vii) Longitudinal direction (Upper part, Support)

	$N = 7.1, B = 100$ (cm), $As = 6.34$ (cm^2), $As' = 12.67$ (cm^2), $d = 62.0$ (cm), $d' = 10.0$ (cm)							
	M (kN·m)	P	P'	k	σ_C (N/mm^2)	σ_C' (N/mm^2)	σ_S (Inner) (N/mm^2)	σ_S (Outer) (N/mm^2)
Wave crest	49.02	0.001023	0.002044	0.119	2.415	1.811	126.942	-6.094
Wave trough	0.00	0.001023	0.002044	0.119	0.000	0.000	0.000	0.000
Permanent	0.00	0.001023	0.002044	0.119	0.000	0.000	0.000	0.000

viii) Longitudinal direction (Lower part, Support)

	$N = 7.1, B = 100$ (cm), $As = 12.67$ (cm^2), $As' = 6.34$ (cm^2), $d = 66.6$ (cm), $d' = 14.6$ (cm)							
	M (kN·m)	P	P'	k	σ_C (N/mm^2)	σ_C' (N/mm^2)	σ_S (Inner) (N/mm^2)	σ_S (Outer) (N/mm^2)
Wave crest	0.00	0.001902	0.000952	0.154	0.000	0.000	0.000	0.000
Wave trough	-67.42	0.001902	0.000952	0.154	-2.147	-1.610	6.456	-83.741
Permanent	-40.64	0.001902	0.000952	0.154	-1.294	-0.971	3.891	-50.471

4) Verification of Fatigue Failure

i) Concrete (Upper part)

	$k_1 = 0.85$, $f_d = 23.1$ (N/mm^2), $\gamma_C = 1.3$, $K = 10$			
	Point	σ_{rd} (N/mm^2)	σ_p (N/mm^2)	Ni (cycle)
Transverse direction	Midspan	| -0.695 - 0.000 | = 0.695	0.000	$4.426 \times 10^9 > 2.0 \times 10^6$
	Support	| 0.000 - (-1.571) | = 1.571	0.000	$1.585 \times 10^9 > 2.0 \times 10^6$
Longitudinal direction	Midspan	| -0.750 - 0.000 | = 0.750	0.000	$4.150 \times 10^9 > 2.0 \times 10^6$
	Support	| 0.000 - (-1.610) | = 1.610	0.000	$1.514 \times 10^9 > 2.0 \times 10^6$

ii) Concrete (Lower part)

	$k_1 = 0.85$, $f_d = 23.1$ (N/mm^2), $\gamma_C = 1.3$, $K = 10$			
	Point	σ_{rd} (N/mm^2)	σ_p (N/mm^2)	Ni (cycle)
Transverse direction	Midspan	| 0.000 - 0.923 | = 0.923	0.000	$3.388 \times 10^9 > 2.0 \times 10^6$
	Support	| 1.258 - 0.000 | = 1.258	0.000	$2.287 \times 10^9 > 2.0 \times 10^6$
Longitudinal direction	Midspan	| 0.000 - 0.990 | = 0.990	0.000	$3.132 \times 10^9 > 2.0 \times 10^6$
	Support	| 1.811 - 0.000 | = 1.811	0.000	$1.196 \times 10^9 > 2.0 \times 10^6$

iii) Reinforcement bar (Upper part)

	$k_0 = 1.0$, $f_{ud} = 467$ (N/mm^2), $\gamma_S = 1.05$, $k = 0.12$					
	Point	σ_{rd} (N/mm^2)	φ (mm)	α	σ_{sp} (N/mm^2)	Ni (cycle)
Transverse direction	Midspan	$\lvert -0.945 - 68.606 \rvert = 69.551$	12.7	0.772	41.353	$3.616 \times 10^9 > 2.0 \times 10^6$
	Support	$\lvert 93.462 - 5.212 \rvert = 88.250$	12.7	0.772	3.305	$1.015 \times 10^9 > 2.0 \times 10^6$
Longitudinal direction	Midspan	$\lvert 1.061 - 71.421 \rvert = 70.360$	12.7	0.772	43.015	$3.179 \times 10^9 > 2.0 \times 10^6$
	Support	$\lvert 126.942 - 6.456 \rvert = 120.486$	12.7	0.772	3.891	$7.498 \times 10^9 > 2.0 \times 10^6$

iv) Reinforcement bar (Lower part)

	$k_0 = 1.0$, $f_{ud} = 467$ (N/mm^2), $\gamma_S = 1.05$, $k = 0.12$					
	Point	σ_{rd} (N/mm^2)	φ (mm)	α	σ_{sp} (N/mm^2)	Ni (cycle)
Transverse direction	Midspan	$\lvert -51.663 - (-0.928) \rvert = 50.735$	12.7	0.772	-0.559	$1.074 \times 10^9 > 2.0 \times 10^6$
	Support	$\lvert -1.264 - (-94.452) \rvert = 93.188$	15.9	0.762	-59.901	$1.798 \times 10^9 > 2.0 \times 10^6$
Longitudinal direction	Midspan	$\lvert -54.107 - (-3.659) \rvert = 50.448$	12.7	0.772	-2.204	$1.094 \times 10^9 > 2.0 \times 10^6$
	Support	$\lvert -6.094 - (-83.741) \rvert = 77.647$	12.7	0.772	-50.471	$1.206 \times 10^9 > 2.0 \times 10^6$

Since fatigue life is not reached even after 2.0×10^6 cycles, there is no need to consider safety verification of fatigue failure.

-End-