

1. Technical Notes

1-1. Characteristics of Caisson-type Gravity Quay wall

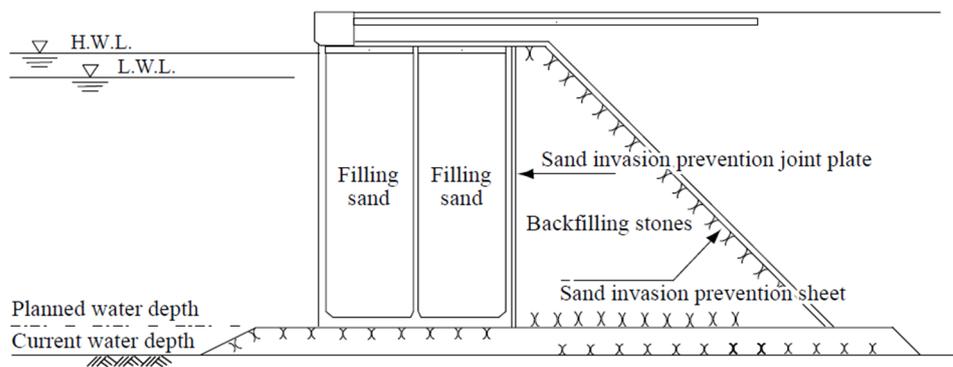
Gravity quay walls resist horizontal actions such as earth and water pressure by the weight of wall bodies. The structural characteristics of gravity quay walls include:

- ✓ Wall bodies are relatively firm and durable because they are constructed from concrete or similar materials.
- ✓ Utilizing precast concrete members in construction facilitates the process and helps to prevent reworking and accidents.
- ✓ Horizontal forces like earth and water pressure on wall bodies increase with deeper water depths in mooring facility installations, necessitating significantly heavier wall bodies. Consequently, ground improvements may be required for constructing gravity quay walls on soft ground with limited bearing capacity.
- ✓ To prevent wash-out of backfill materials between the caisson joints, prevention plates are installed between caissons and backfill stones, or sand invasion prevention sheets are laid on the backfill stones. It is crucial to consider potential damage to these prevention measures as they can lead to sagging of aprons and reclaimed land behind quay walls.
- ✓ The impact of earthquake ground motions on wall bodies is proportional to their weight. Designing wall bodies to resist strong earthquakes by increasing quay wall widths results in increased forces acting on them, making it challenging to prevent deformation during strong earthquakes. However, gravity quay walls, especially caisson-type, typically retain their stability even under deformation, offering advantages in serviceability post-earthquake.
- ✓ Gravity quay walls necessitate extensive onshore facilities, such as caisson and block fabrication yards, and specialized workboats like crane barges and tugboats. Consequently, constructing small-scale gravity quay walls is not economical without these facilities, particularly if the construction does not demand long periods.
- ✓ Gravity quay walls on soft cohesive ground should be designed with care due to potential consolidation of cohesive layers causing gradual settlement over time.

1-2. Basic Policy for Performance Verification

The members shall remain safe, and wall body shall maintain the stability under permanent states and earthquake ground motion. The stability verification for permanent states and earthquake motion is carried out using the partial factor method.

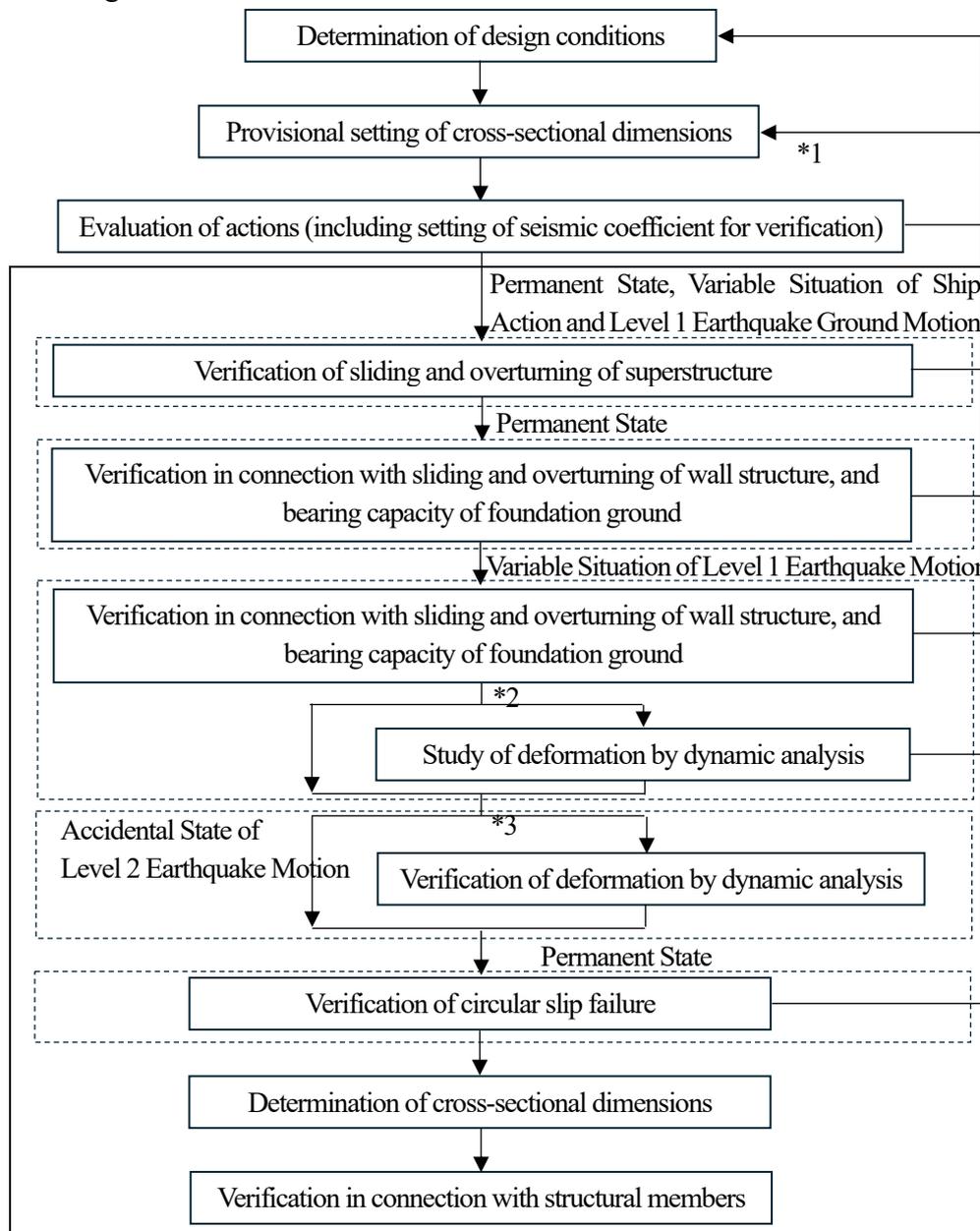
The seismic coefficient method based on the static equation of equilibrium can be used to examine gravity quay walls under a variable situation with respect to earthquake ground motions. By contrast, for gravity quay walls classified as high earthquake-resistance facilities, the deliberation of deformation amounts is preferably performed by nonlinear earthquake response analysis or other methods by taking into consideration the dynamic interaction between ground and structures.



Source: OCDI 2020

Figure 1.1- Example of Cross Section of Caisson-type Gravity Quay wall

The general performance verification procedure for caisson-type gravity quay wall is shown in Figure 1.2.



*1 As the effects of liquefaction, subsidence etc. are not included in this procedure, separate consideration is necessary.

*2 When necessary, study of deformation by dynamic analysis for Level 1 earthquake ground motion is possible. In high-earthquake-resistance facilities, study of deformation by dynamic analysis is preferable.

*3 For high-earthquake-resistance facilities, verification for Level 2 earthquake ground motion is performed.

Source: TCVN 11820-5-2021

Figure 1.2- Performance Verification Procedure for Gravity Quay walls

1-3. Design Conditions

(1) Setting of Design Conditions

Design conditions necessary for performance verification of the facility are set according to the nature of the facility and the situation in which it is installed. The main design conditions are as follows:

- ✓ Natural Conditions: Design tidal level, residual water level, seismic coefficient for verification, soil properties, wind speed, tidal currents, etc.
- ✓ Especially, the properties of the foundation ground are crucial conditions that affect the overall stability of the facility, so they should be determined based on thorough on-site investigation results.
- ✓ Operational Conditions: Planned length, planned water depth, planned top elevation, specifications of target vessels, berthing speed, live loads (cargo handled), loading equipment loads, mooring force, and the structure's service life.
- ✓ Construction Conditions: Caisson manufacturing location, manufacturing method, launching method, capabilities of work vessels, and supply capacity of materials.

(2) Approach to Structural Specifications

Structural specifications are determined either based on stability requirements or construction conditions. Below is the approach to determining these specifications:

1) Crown Height of the Quay wall and Caisson

- ✓ The crown height of a mooring facility shall be set at an appropriate height that is suitable for the main dimensions of the target vessel and the natural conditions of the surrounding area. The tidal level that is used as the datum in the determination of the crown height shall be the mean monthly-highest water level. The crown of berth (*CDMB*) is defined by both the following regulations in TCVN:

$$\begin{aligned}
 CDMB &= H_{1\%} + 1.5m \\
 CDMB &= H_{50\%} + 2.0m
 \end{aligned}
 \tag{1.1}$$

Where:

- $H_{1\%}$: water level of observation with the yield guarantee of 1% and
- and 50% according to the hourly level, which is determined in
- $H_{50\%}$ accordance with 5.3.2 of TCVN 11820-2:2017

- ✓ Table 1.1 is often used as a reference to determine the crown height in OCDI 2020. However, the reference tidal level should be H.W.L. (High Water Level).

Table 1.1- Standard Crown Heights of Quay wall

	Tide range of 3.0m or more	Tide range less than 3.0m
For Large Vessels (Water depth 4.5m or more)	+0.5 to 1.5m	+1.0 to 2.0m
For Small Vessels (Water depth less than 4.5m)	+0.3 to 1.0m	+0.5 to 1.5m

Source: OCDI 2020

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Table 2.1.2

- ✓ The top elevation of the caisson is determined considering the construction conditions, such as the caisson installation work, backfill sand placement, top cover concrete, and upper concrete pouring, as well as economic factors.

2) Design Water Depth and Construction Limits

- ✓ When footings are used, the design water depth is often set by adding the height of the footing to the planned water depth.
- ✓ In the absence of footings, 10 cm is typically added to the planned water depth to account for leveling tolerance of foundation mound.

3) Foundation Mound

The thickness of the foundation mound can be determined by examining the failures due to the insufficient bearing capacity of the foundation ground, the flatness of mound surfaces on which wall bodies are installed, the alleviation of local stress concentration in the ground. The minimum thickness values commonly used are shown in Table 1.2.

Table 1.2- Minimum Thickness of Foundation Mound

Minimum Thickness of Foundation Mound	
For Large Vessels (Water depth 4.5m or more)	At least 1.0m, but at least 3 times the average diameter of the rubble stone
For Small Vessels (Water depth less than 4.5m)	At least 0.5m, but at least 3 times the average diameter of the rubble stone

Source: OCDI 2020

The shoulder width of the foundation mound should ensure the necessary stability against bearing capacity failure from eccentric loads, prevent circular sliding of the foundation ground, prevent scouring, reduce stress concentrations, and account for wall body sliding due to earthquakes.

The slope of the rubble mound is often set to 1:2 on the seaward side and 1:1.5 on the harbor side, considering scouring prevention.

4) Width of the Caisson

The width of the caisson is determined by considering its stability against sliding, overturning, and reaction force against bottom slab. It is also necessary to ensure stability during floating conditions.

Using a footing reduces the design water depth of the caisson but allowing for a narrower caisson width. This adjustment not only improves stability against overturning but also decreases the reaction force on the bottom slab. The length of footings is typically set between 1.0 and 1.5 meters for reinforced concrete (RC) caissons, considering safety against bending moments and shear forces.

5) Length of the Caisson

The length of the caisson is determined by factors such as the construction facility's capabilities, jack-up or floating requirements, strength against bending moments and twisting caused by uneven settlement, and the total length of the wharf.

In general, increasing the length of the caisson reduces the number of installations required, making it more economical. However, excessively long caissons may have disadvantages regarding resistance to twisting and the structural integrity of the caisson as a beam. The slenderness ratio is usually kept below 1:2.

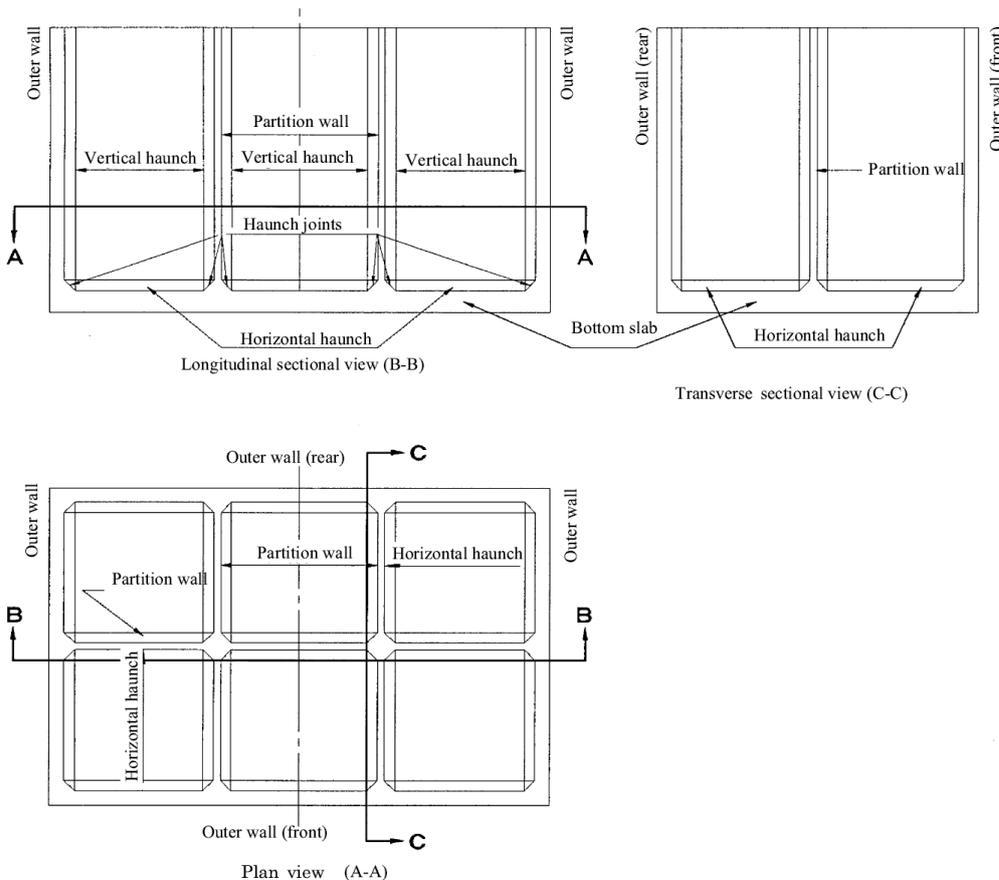
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6) Shape of the Caisson

The dimensions and shape of each caisson component are determined to ensure safety against external forces, bending and twisting caused by jack-up operations and uneven settlement, water pressure during launching, and localized loads during lifting.

When placing rail-mounted cargo handling machines like container cranes on top of the caisson, it is preferable to place them on the bulkheads to distribute the wheel loads evenly. Figure 1.3 indicates the names of various parts of the caisson. The thickness of the outer walls is generally 0.3 to 0.6 meters (for partition wall spacing within 5 meters). The bottom slab is typically 0.4 to 0.8 meters thick. The partition walls are usually 0.2 to 0.3 meters thick, and the haunches are often 0.2 to 0.3 meters thick.

If the toe pressure on the bottom slab becomes too high, it may not be feasible to use standard reinforcement. Therefore, it is crucial to pay attention to the toe pressure and slab thickness from the initial design stage.



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Source: OCDI 2002

Figure 1.3- Names of Various Parts of the Caisson

7) Superstructure

The shape of the superstructure shall be designed to ensure safety against the mooring force of vessels. In particular, the area where mooring bollards are installed is often partially widened and reinforced to secure the anchors.

1-4. Consideration of Applied Forces

(1) Seismic Forces

For structures such as gravity quay walls that are relatively stiff and whose amplitude of vibration is small compared to the ground motion during an earthquake, the seismic resistance must be considered using the seismic coefficient method.

The seismic coefficient is determined depending on geological conditions and ground

acceleration according to the earthquake classification criteria of TCVN 9386: 2012, and TCVN 11820 Part 2: 2025.

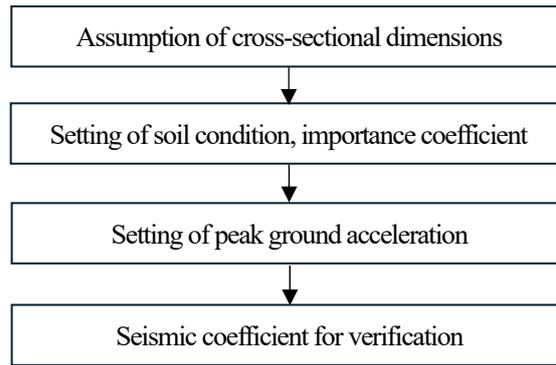


Figure 1.4- Example Flow for Setting Seismic Coefficient

The design earthquake coefficient is calculated according to the acceleration peak, so the reference a_gR on type A foundation (base rock) is taken from the acceleration zoning map of the territory of Vietnam or taken from the zoning map found in some regions approved by competent authorities.

Note: According to the design ground acceleration value $a_g = \gamma_i \times a_gR$, divided into three earthquake cases:

- ✓ Strong earthquake $a_g \geq 0.08g$, seismic resistance must be calculated and constructed
- ✓ Weak earthquake $0.04g \leq a_g < 0.08g$, only need to apply mitigation anti-seismic solutions
- ✓ Very weak earthquake $a_g < 0.04g$, no seismic design required

The earthquake coefficient is the horizontal earthquake coefficient determined by the equation below. The value of the coefficient must be expressed as a two-digit number by rounding if the third number is five or more or omitting if it is less than five.

Seismic coefficient (k_h) = regional seismic coefficient (k_{h1}) \times soil condition coefficient (γ_s) \times importance coefficient (γ_i)

The regional seismic coefficient shall be the values listed in Figure 1.5. The soil condition coefficient shall be the value listed in Table 1.3 corresponding to the type of subsoil given in Table 1.4, and the importance coefficient shall be the value listed in Table 1.5 corresponding to the characteristics of structures.

1) Regional seismic coefficient

The regional seismic coefficient is calculated according to the Equation (1.2):

$$k_{h1} = a_gR/g \tag{1.2}$$

Where:

- a_gR : reference peak ground acceleration (Figure 1.5)
- g : gravitational acceleration

The reference peak ground acceleration on type A ground, a_gR , for use in Vietnam given in TCVN 9386-1: 2012, Annex G, Part 1 or may be derived from zonation maps found in some regions approved by the relevant authorities.

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However, Figure 1.5 indicates acceleration for a return period of 500 years, and for a Level 1 performance verification with a return period of 75 years, the acceleration for a may be reduced. Since there are no acceleration maps with a return period of 75 years in Vietnam, Consultants are expected to set the design background acceleration after careful consideration and/or consultation with seismic experts.

Reference:

The PIANC WG34 report “Seismic Design Guidelines for Port Structures” recommends the application of seismic loading at two levels:

✓ Level 1:

The structures should not have significant residual deflections after the earthquake and should be immediately operational. The return period for the Level 1 event is 75 years, corresponding to a 50% probability of exceedance over the design life of 50 years.

✓ Level 2:

The structures should be operational following repairs. The structures should not buckle, collapse or overturn. The return period for the Level 2 event is 475 years, corresponding to a 10% probability of exceedance over the design life of 50 years.

2) Soil condition coefficient

Table 1.3- Soil Condition Coefficient γ_s

Type of subsoil	Type A	Type B	Type C
Soil condition coefficient	0.8	1.0	1.2

Source: TCVN 11820-2-2025

Table 1.4- Classification by Type of Subsoil

Type of subsoil	Gravel stratum	Ordinary sandy and cohesive soil	Weak soil
Thickness of quaternary strata			
5 meters or less	Type A	Type A	Type B
More than 5 meters and less than 25 meters	Type A	Type B	Type C
25 meters or greater	Type B	Type C	Type C

Source: TCVN 11820-2-2025

Note: In the above tables, "weak soil" is sandy soil with N value of SPT test less than 4 or cohesive soil with free lateral compressive strength less than 20 kN/m². When the ground consists of two or more layers, the soil type must be determined according to which layer has the greatest thickness. If the foundation soil consists of two or more layers of almost equal thickness, the soil type must be determined according to which soil layer has the largest soil condition coefficient value in those layers.

3) Importance coefficient

Reliability differentiation is implemented by classifying structures into different importance classes. An importance coefficient γ_I is assigned to each importance class. Wherever feasible this coefficient should be derived so as to correspond to a higher or lower value of the return period of the seismic event (with regard to the reference return period) as appropriate for the design of the specific category of structures. Terms to coefficients and importance coefficients are given in TCVN 11820 Part 2: 2025 and TCVN 9386-1:2012 Annex E, Part 1.

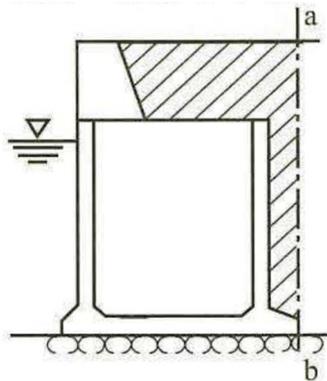
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(2) Surcharge Loads

The surcharge loads are applied by considering the utilization pattern of the quay. Both live and dead loads are considered. For general cargo piers, a load of about 10 to 20 kN/m² is commonly used. Regarding the variable conditions for earthquake motions, the loads are often considered as half of the permanent load conditions.

(3) Weight of the Wall Structure

The weight of the wall structure is taken as the portion of the wall in front of the vertical plane passing through the rear of the footing (shown in Figure 1.6). This weight includes the entire structural components in front of this plane.



Source: TCVN 11820-5-2021

Figure 1.6- The Weight of the Caisson Wall

(4) Residual Water Level

The residual water level is typically considered as one-third of the tidal range above the low water level (L.W.L) based on the average of the spring and neap tides. In general, gravity-type quay wall incorporate backfill materials such as rubble stones to reduce water levels and alleviate earth pressures over time. Therefore, one-third of the tidal range is used as the standard assumption for long-term permeability. However, in cases where poor permeability is expected initially, or a decline in permeability is anticipated over time, it is necessary to account for a residual water level that matches the actual conditions.

NOTE: In some cases, such as Singapore, where sand is used for backfill materials, half the tidal range is often applied.

(5) Dynamic Water Pressure

For structures submerged in water, or structures with internal spaces partially or entirely filled with water, dynamic water pressure is considered during an earthquake. The dynamic water pressure acting on the wall is calculated using the following formula:

$$p_{dwk} = \pm \frac{7}{8} c k_{hk} \gamma_w \sqrt{Hy} \quad (1.3)$$

Where:

- p_{dwk} : dynamic water pressure (kN/m²)
- k_{hk} : design seismic coefficient
- γ_w : unit weight of water (kN/m³)
- y : depth from the still water level to the point where dynamic water pressure is calculated (m)
- H : water depth (m), which can be considered as the installation depth of the caisson

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- c : correction coefficient (when $L/H \leq 1.5$, $c = L/1.5H$; when $L/H > 1.5$, $c = 1.0$)
- L : length of the space occupied by water in the direction of vibration (m)

For the quay wall, $c = 1.0$, and the resultant of the dynamic water pressure and the point of action can be calculated using Equation (1.4).

$$P_{dw} = \pm \frac{7}{12} k_{hk} \gamma_w h^2, \quad h_{dw} = \frac{3}{5} h \quad (1.4)$$

Where:

- p_{dw} : resultant force of dynamic water pressure (kN/m)
- h_{dw} : depth of the acting point of the dynamic water pressure resultant force from the still water level (m)

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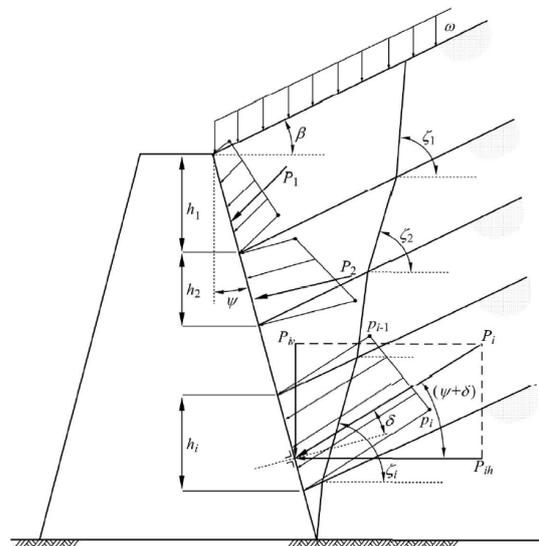
Dynamic water pressure resulting from pore water in the backfill materials, such as the caisson infill, rubble stones, and backfill soil, is generally not considered.

(6) Earth Pressure

The active earth pressure acting on the quay wall is calculated based on both permanent and variable situations associated with earthquake motions.

The active earth pressure for sandy soils under permanent states is calculated using Coulomb's earth pressure theory. During earthquake motions, the earth pressure is calculated using the formula proposed by Mononobe and Okabe.

The earth pressure below the water surface is evaluated using the apparent seismic coefficient according to Equation (1.6).



Source: TCVN 11820-4-1-2020

Figure 1.7- Earth Pressure

$$p_{ai} = K_{ai} \left[\sum_{j=1}^i \gamma_j h_j + \frac{\omega \cos \psi}{\cos(\psi - \beta)} \right] \cos \psi \quad (1.5)$$

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$$K_{ai} = \frac{\cos^2(\varphi_i - \psi - \theta)}{\cos\theta \cos^2\psi \cos(\delta + \psi + \theta) \left[1 + \frac{\sin(\varphi_i + \delta) \sin(\varphi_i - \beta - \theta)}{\cos(\delta + \psi + \theta) \cos(\psi - \beta)} \right]^2}$$

Where:

- p_a : active earth pressure, respectively, acting on the retaining wall in the i -th soil layer (kN/m²)
- φ : angle of shear resistance of the i -th soil layer (°)
- γ_i : unit weight of soil in each layer (kN/m³)
- h_i : thickness of each layer (m)
- K_a : coefficient of active earth pressure, respectively, in the i -th soil layer
- β : angle of ground surface to the horizontal plane (°)
- δ : angle of wall friction (°)
- ω : surcharge per unit area of the ground surface (kN/m²)
- θ : composite seismic angle (°)
 $\theta = \tan^{-1}k$ or $\theta = \tan^{-1}k'$
(In the permanent state, $\theta = 0$)
- k : seismic coefficient
- k' : apparent seismic coefficient

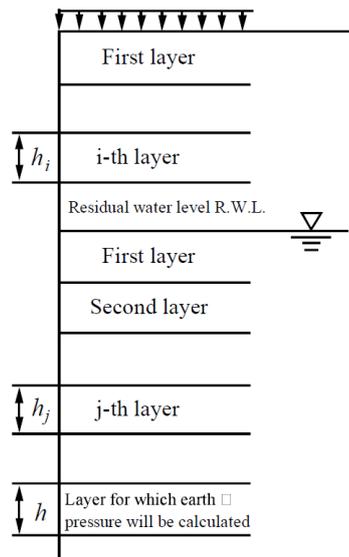
$$k' = \frac{2 \left(\sum \gamma_{ti} h_i + \sum \gamma_{satj} h_j + \omega \right) + \gamma_{sat} h}{2 \left\{ \sum \gamma_{ti} h_i + \sum (\gamma_{satj} - 10) h_j + \omega \right\} + (\gamma_{sat} - 10) h} k \quad (1.6)$$

Where:

- h_i : thickness of i -th soil layer above residual water level (m)
- h_j : thickness of j -th soil layer above the layer for which the earth pressure is being calculated below the residual water level (m)
- h : thickness of soil layer for which the earth pressure is being calculated below the residual water level (m)
- γ_{ti} : unit weight of soil in the i -th layer above the residual water level (kN/m³)
- γ_{satj} : saturated unit weight of soil in the j -th layer above the layer for which the earth pressure is being calculated below the residual water level (kN/m³)
- ω : surcharge per unit area of the ground surface (kN/m²)
- k : seismic coefficient
- k' : apparent seismic coefficient

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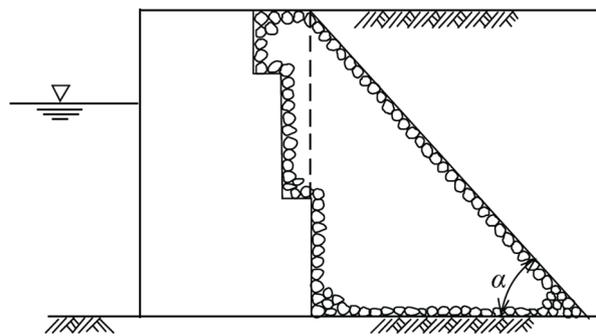
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Source: TCVN 11820-4-1-2020

Figure 1.8- Symbols for Apparent Seismic Coefficient

The wall friction angle used in calculating soil pressure strength is often set to 15° or less for both permanent state and earthquake motions.



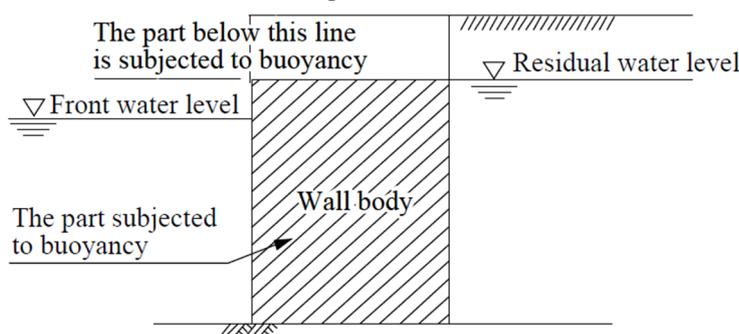
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Figure 1.9- Shape of Backfill

In cases where the shape of the backfill, as shown in Figure 1.9, is made with a slope angle that is gentler than the angle of repose α of the backfill material, drawn from the intersection of the vertical line passing through the base of the wall and the ground surface (assumed boundary plane), it can be considered as fully backfilled with backfill material.

(7) Buoyancy

Buoyancy is calculated as shown in Figure 1.10, assuming that the portion of the wall below the residual water level is submerged in water.



Source: TCVN 11820-5-2021

Figure 1.10- Assumption for Calculating Buoyancy

(8) Synthesis of Forces

The total vertical force is calculated as the wall weight excluding the surcharge loads and buoyancy. Additionally, the vertical component is included.

The total horizontal force includes the horizontal component of the earth pressure acting on the assumed boundary plane and the residual water pressure.

The performance verification in seismic motions, in addition to the above, the seismic force acting on the wall weight (without buoyancy) is added. The earth pressure is considered as the horizontal component of the earth pressure during earthquake motions.

Furthermore, if there are equipment loads, it is necessary to consider the horizontal forces of the crane legs during operation (permanent states), earthquake motions, and storm conditions, respectively.

1-5. Performance Verification Method

The performance requirement, performance criteria and performance verification items for Gravity-type Quaywalls are introduced in “Part 1 Performance-Based Design Approach” 8-5 (3) and (4).

(1) Stability Verification

For each design condition, performance verification of overall stability related to the gravity quay wall structure is conducted based on static equilibrium equations. The items for performance verification are listed in Table 1.6, which provides a guide for the overall stability verification (excluding accidental conditions).

Table 1.6- Performance Verification Items Related to Various Design Conditions

Design State/Situation	Circular Slip Failure	Sliding of Body	Overturning of Body	Bearing Capacity of Foundation
Permanent state related to self-weight	○			
Permanent state related to earth pressure		○	○	○
Variable situation related to Level 1 earthquake ground motion		○	○	○

1) Performance Verification for Overall Stability in Permanent State

In the permanent state, where the primary force is the self-weight of the quay wall structure and backfilling, performance verification for overall stability typically involves checking for circular slip failure. The verification of circular slip can be carried out using Equation (1.7).

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 1.7. For parts where a "-" is indicated in Table 1.7, values in parentheses can be used for convenience when performing the verification.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \tag{1.7}$$

$$S_k = \Sigma \{ (w'_k + q_k + q_{RWLk}) \sin \theta \}$$

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$$R_k = \Sigma [\{ c'_k s + (w'_k + q_k) \cos^2 \theta \cdot \tan \phi'_k \} \sec \theta]$$

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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'_k : characteristic value of effective weight of a segment (kN/m) (atmospheric weight when above the water surface or underwater weight when below the water surface)
- q_k : characteristic value of surcharge acting on a segment (kN/m)
- q_{RWLk} : characteristic value of weight of water, i.e., $\rho_w g \cdot (RWL - LWL) s$, in a segment corresponding to the difference in water levels between the residual water level (RWL) at the back of a facility and a tidal level (LWL) in front of a facility in a case where RWL is higher than LWL (kN/m)
- θ : angle between the bottom face of a segment and a horizontal plane ($^\circ$)
- c'_k : characteristic value of 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)
- ϕ'_k : characteristic value of apparent shear resistance angle based on effective stress ($^\circ$)

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

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

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Source: TCVN 11820-6-2023

2) Performance Verification for Sliding of Quay wall

The stability against sliding can be assessed using the Equation (1.8). 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 1.8.

In Table 1.8, values marked with a "-" indicate that the values in parentheses can be used for verification purposes for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \quad (1.8)$$

$$R_k = f_k (W_k + P_{Vk} - P_{Bk})$$

$$S_k = P_{Hk} + P_{Wk} + P_{dWk} + P_{Fk}$$

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Where:

- m : adjustment factor
- S_d : value to be used for design of the load term (kN/m)
- R_d : value to be used for design of the resistance term (kN/m)
- S_k : characteristic value of the load term (kN/m)
- R_k : characteristic value of the resistance term (kN/m)
- γ_S : partial factor multiplied by load term
- γ_R : partial factor multiplied by resistance term
- f_k : characteristic value of friction coefficient between the bottom face of a wall body and a foundation
- W_k : characteristic value of weight of the materials constituting a wall body (kN/m)
- P_{Vk} : characteristic value of resultant vertical earth pressure acting on a wall body (kN/m)
- P_{Bk} : characteristic value of buoyancy acting on wall body (kN/m)
- P_{Hk} : characteristic value of resultant horizontal earth pressure acting on a wall body (kN/m)
- P_{Wk} : characteristic value of resultant residual water pressure acting on a wall body (kN/m)
- P_{dWk} : characteristic value of resultant dynamic water pressure acting on a wall body (kN/m) (only during earthquake)
- P_{Fk} : characteristic value of inertia force acting on a wall body (kN/m) (only during earthquake)

Table 1.8- Partial Factors for Performance Verification for Sliding of Wall Body

Verification Conditions	Partial factor γ_R	Partial factor γ_S	Adjustment Factor m
Wall Sliding (Permanent State)	0.87	1.06	- (1.00)
Wall Sliding (Variable Situation: Level 1 Earthquake Ground Motion)	- (1.00)	- (1.00)	1.00

Source: TCVN 11820-5-2021

TCVN
11820
Part 5:
2021,
Bang 2

Additionally, if the caisson has rectangular cross-section footings on both the seaward and landward sides, the buoyancy characteristic value P_{Bk} can be calculated using the following equation:

$$P_{Bk} = \rho_w \cdot g \cdot \{(wl_k + h) \cdot B_c + 2h_f \cdot B_f\} \quad (1.9)$$

Where:

- ρ_w : unit weight of sea water (kN/m³)
- wl_k : characteristic value of tidal level (m)

TCVN
11820
Part 6:
2023,
Equation
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- h : depth of the wall installation (m)
- B_c : width of the wall body (m)
- h_f : height of footing (m)
- B_f : width of footing (m)

3) Performance Verification for Overturning of Breakwater Body

The stability of the overturning of quay wall can be evaluated using the Equation (1.10). 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 1.9. For values marked with "-", it indicates that values in parentheses can be conveniently used for verification.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \quad (1.10)$$

$$R_k = (aW_k + cP_{Vk} - bP_{Bk})$$

$$S_k = dP_{Hk} + eP_{wk} + hP_{dwk} + iP_{Fk}$$

TCVN
11820
Part 5:
2021,
Equation
(18)

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_k : characteristic value of weight of the materials constituting a wall body (kN/m)
- P_{Vk} : characteristic value of resultant vertical earth pressure acting on a wall body (kN/m)
- P_{Bk} : characteristic value of buoyancy acting on wall body (kN/m)
- P_{Hk} : characteristic value of resultant horizontal earth pressure acting on a wall body (kN/m)
- P_{wk} : characteristic value of resultant residual water pressure acting on a wall body (kN/m)
- P_{dwk} : characteristic value of resultant dynamic water pressure acting on a wall body (kN/m) (only during earthquake)
- P_{Fk} : characteristic value of inertia force acting on a wall body (kN/m) (only during earthquake)
- a : distance from the action line of resultant weight of wall to the front toe of a wall body (m)
- b : distance from the action line of buoyancy to the front toe of a wall body (m)
- c : distance from the action line of resultant vertical earth pressure to the front toe of a wall body (m)
- d : distance from the action line of resultant horizontal earth pressure to the bottom of a wall body (m)
- e : distance from the action line of resultant residual water pressure to the bottom of a wall body (m)
- h : distance from the action line of resultant dynamic water pressure to the bottom of a wall body (m), (only during

- earthquakes)
i : distance from the action line of inertial force to the bottom of a wall body (m) (only during earthquakes)

Table 1.9- Partial Factors for Performance Verification for Overturning of Wall Body

Verification Conditions	Partial factor γ_R	Partial factor γ_S	Adjustment Factor m
Wall Overturning (Permanent State)	0.99	1.23	- (1.00)
Wall Overturning (Variable Situation: Level 1 Earthquake Ground Motion)	- (1.00)	- (1.00)	1.10

Source: TCVN 11820-5-2021

TCVN
11820
Part 5:
2021,
Bang 3

4) Performance Verification for Bearing Capacity of the Foundation Ground

For evaluating the stability of the foundation ground can be examined using the following Equation (1.11), which is obtained through the simplified Bishop method. The partial factors in Equation (1.11) can be referenced from Table 1.10. For the parts in Table 1.10 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 (1.11), 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.

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

$$F_f = \frac{R_k(F_f)}{S_k} \quad (1.11)$$

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

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

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_{Hk} : characteristic value of horizontal action on a soil mass inside a slip failure circle (kN/m)
d : distance from the center of a slip failure circle to the action position of the forces *P_H* (m)

- c'_k : characteristic value of undrained shear strength for cohesive soil ground or apparent adhesion drained condition for sandy soil ground (kN/m²)
- s : width of a segment (m)
- w'_k : characteristic value of effective weight of a segment (kN/m) (atmospheric weight when above water surface or underwater weight when below water surface)
- q_k : characteristic value of surcharge acting on a segment (kN/m)
- ϕ'_k : characteristic value of apparent shear resistance angle based on effective stress (°)
- θ : angle between the bottom face of a segment and the horizontal plane (°)
- F_f : auxiliary parameter representing a ratio of the resistance term to a load term
- r : radius of a slip failure circle (m)

Table 1.10- Partial Factors for Performance Verification for Bearing Capacity Failure of the Foundation Ground

Verification Conditions	Partial factor γ_R	Partial factor γ_S	Adjustment Factor m
Foundation Bearing Capacity (Permanent State)	- (1.00)	- (1.00)	1.20
Foundation Bearing Capacity (Variable Situation: Level 1 Earthquake Ground Motion)	- (1.00)	- (1.00)	1.00

Source: Modified from TCVN 11820-6-2023

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TCVN
11820
Part 6:
2023
Bang 9

5) Performance Verification for Stability of Floating Condition

During construction, for the caisson to float and self-stabilize, it must satisfy Equation (1.12).

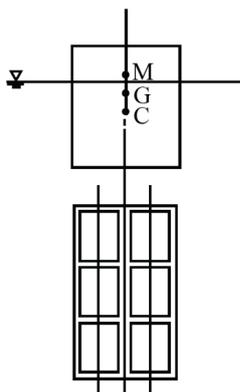
$$\frac{I}{V} - \overline{CG} = \overline{GM} \geq 0 \quad (1.12)$$

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)

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

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



Source: TCVN 11820-6-2023

Figure 1.11- Stability of Caisson

Equation (1.12) 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:

$$\frac{I}{V'} \left(I' - \sum i \right) - \overline{C'G'} > 0 \quad (1.13)$$

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

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

$$\frac{I'}{V'} - \overline{C'G'} > 0 \quad (1.14)$$

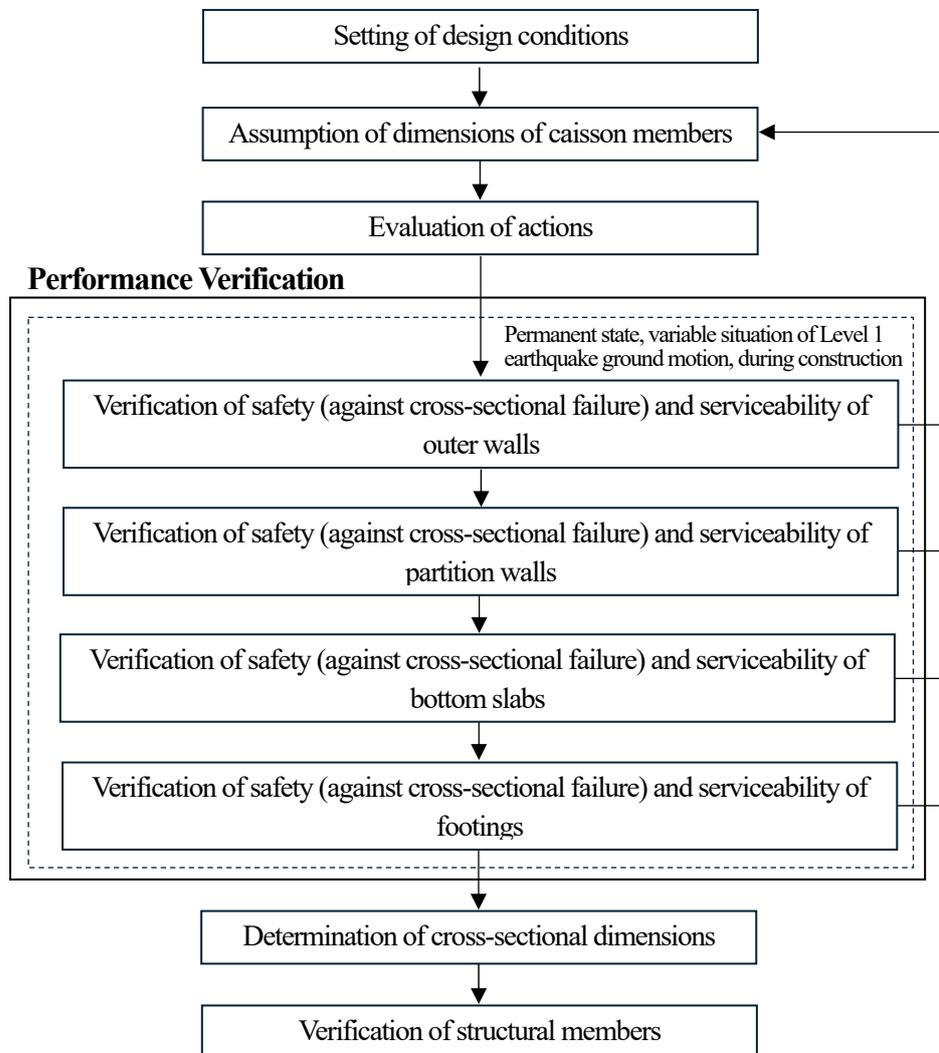
Where:

- V' : displacement volume for caisson with counter ballast (m³)
- I' : geometrical moment of inertia with respect to long axis at water level for caisson with counter ballast (m⁴)
- 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 (m⁴)

(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.

An example of the performance verification sequence for caissons is shown in Figure 1.12.



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TCVN
11820
Part 6:
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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.

Source: Modified from TCVN 11820-6-2023

Figure 1.12- Example of the Performance Verification Procedure for Caissons

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:

$$\gamma_i \cdot \frac{S_d}{R_d} \leq 1.0 \quad (1.15)$$

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 .

$$S_d = \sum \gamma_a \cdot S(F_d) \quad (1.16)$$

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 .

$$R_d = R(f_d) / \gamma_b \quad (1.17)$$

The design cross-section capacity M_{ud} for the bending moment of reinforced concrete can be calculated using Equation (1.18).

$$M_{ud} = A_s f_{yd} \cdot d \left(1 - \frac{\rho_w f_{yd}}{1.7 f'_{cd}} \right) / \gamma_b \quad (1.18)$$

Where:

- A_s : area of tension reinforcement (mm²)
- ρ_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 (1.19).

$$\frac{\gamma_i \cdot M_d}{M_{ud}} \leq 1.0 \quad (1.19)$$

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:

$$\frac{\gamma_i \cdot \sigma_{rd}}{f_{rd} / \gamma_b} \leq 1.0 \quad (1.20)$$

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

TCVN
11820
Part 11:
2025,
Equation
(12)

TCVN
11820
Part 11:
2025,
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TCVN
11820
Part 11:
2025
Equation
(40)

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:

$$\frac{\gamma_i \cdot S_{rd}}{R_{rd}} \leq 1.0 \quad (1.21)$$

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 .

In the typical caisson-type gravity quay wall, there is no repetitive load. Therefore, unless repeated waves are applied during construction, the safety verification of fatigue failure may be omitted.

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 (1.22).

$$\sigma'_c \leq 0.4f'_{ck} \quad (1.22)$$

TCVN
11820
Part 11:
2025,
Equation
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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.

$$\gamma_i w_d / w_a \leq 1.0 \quad (1.23)$$

TCVN
11820
Part 11:
2025,
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The design response value of crack width can be calculated using Equation (1.24).

$$w = 1.1 \times k_1 \times k_2 \times k_3 \times \{4 \times c + 0.7 \times (c_s - \phi)\} \times (\sigma_{se} / E_s + \varepsilon'_{csd}) \quad (1.24)$$

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 σ_{se} can be obtained using Equation (1.25) assuming the cross section is in the elastic range.

$$\sigma_{se} = M_d / (A_s j \cdot d) \quad (1.25)$$

Where:

- M_d : design value of the bending moment (N·mm)
- A_s : cross-sectional area of reinforcing bars (mm²)
- 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 (mm²)

The limit value of crack width w_a is generally set to the values shown in Table 1.11; 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 1.12.

Table 1.11- Limit Values of Crack Width w_a

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

Source: TCVN 11820-11-2025

TCVN
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Part 11:
2025,
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TCVN
11820
Part 11:
2025,
Table 3

Table 1.12- Standard Values for Cover

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 (1.26).

$$V_{dd} = \beta_d \cdot \beta_p \cdot \beta_a \cdot f_{dd} \cdot b_w \cdot d / \gamma_b \quad (1.26)$$

Where:

 V_{dd} : design shear compressive failure capacity (N)

$$f_{dd} : 0.19 \sqrt{f'_{cd}} \text{ (N/mm}^2\text{)}$$

$$\beta_d : \sqrt[4]{1000/d}, \text{ set to } 1.5 \text{ when } \beta_d > 1.5$$

$$\beta_p : (1 + \sqrt{100p_v})/2, \text{ set to } 1.5 \text{ when } \beta_p > 1.5$$

$$\beta_a : 5 / (1 + (a/d)^2)$$

 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) Partial Factors**

The partial factors 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. Regarding the floating condition, the duration of the load action is short compared to the other states, and its occurrence is only during construction. Therefore, the partial factors for serviceability states can be set at 0.5.

TCVN
11820
Part 11:
2025,
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Part 11:
2025,
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Table 1.13- 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_t		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 capacity carried 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	1.2	1.2	1.0
	Beams	1.1	1.1	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, 11820-11-2025

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 1.14. 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 1.14 cannot be used.

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Part 6:
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Part 11:
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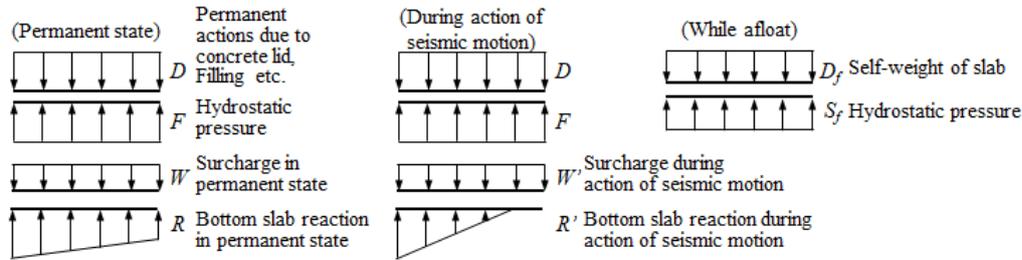
Table 1.14- Combinations of Actions and Load Factors

Situation	Design state/situation	Self-weight	Hydrostatic pressure	Internal water pressure	Internal earth pressure	Bottom slab reaction	Surcharge	Dynamic water pressure	Bottom slab reaction during action of seismic motion	Loads during construction		Remarks	
										Installation	Still water		
In service	Permanent state associated with self-weight	0.9 (1.0)	1.1 (1.0)			1.1 (1.0)	0.8 (0.5)					Bottom slab (Surcharge is equivalent to bottom slab reaction component)	
	Permanent state associated with internal earth pressure			1.1 (1.0)	1.1 (1.0)							Outer wall	
	Variable situation associated with Level 1 earthquake ground motion	1.0 (-)	1.0 (-)				1.0 (-)		1.0 (-)				Bottom slab (Surcharge is that during action of seismic 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 while afloat	
											1.1 (0.5)	Outer wall while afloat	
	Variable situation associated with water pressure during installation									1.1 (0.5)		Partition wall during installation	

Source: TCVN 11820-11-2025

ii) Load factors and combinations of actions of bottom slab

The design load for the bottom slab of the caisson for the quay wall is as shown in Figure 1.13. The load factor is provided in Table 1.15, and these factors can be multiplied to determine the load.



Permanent state: Situation in which surcharge is imposed

Source: TCVN 11820-11-2025

Figure 1.13- Actions on the Bottom Slab

Table 1.15- Combinations of Actions on the Bottom Slab and Load Factors

Design State/Situation	Safety (against cross-sectional failure)	Serviceability
Permanent state	$0.9D+1.1D_o+1.1F+0.8W$	$1.0D+1.0D_o+1.0F+0.5W$
Variable situation associated with Level 1 earthquake ground motion	$1.0D+1.0F+1.0R+1.0W'$	Not examined
Variable situation associated with water pressure while afloat during construction	$0.9D_f+1.1S_f$	$0.5D_f+0.5S_f$

Source: TCVN 11820-11-2025

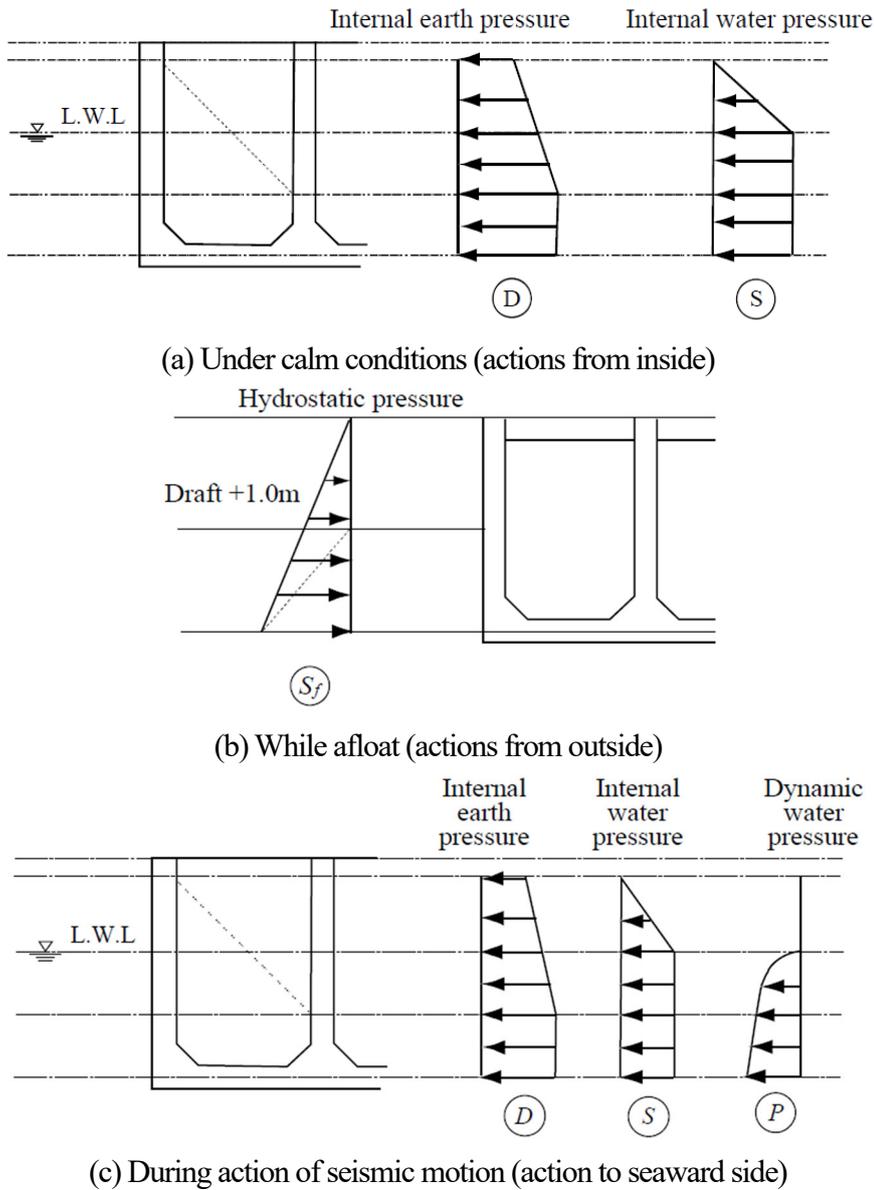
TCVN 11820 Part 11: 2025, Table 6

TCVN 11820 Part 11: 2025, Fig. 14

TCVN 11820 Part 11: 2025 Table 8

iii) Load factors and combinations of actions of outer wall

The design load for the outer wall of the quay wall caisson is shown in Figure 1.14, with load coefficients listed in Table 1.16.



Source: TCVN 11820-11-2025

Figure 1.14- Actions on the Outer Wall

Table 1.16- Combinations of Actions on the Outer Wall and Load Factors

Design State/Situation	Direction of action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with water pressure while afloat during construction	From outside of caisson	$1.1 S_f$	$0.5 S_f$
Permanent state associated with internal earth pressure	From inside of caisson	$1.1D+1.1S$	$1.0D+1.0S$
Variable situation associated with Level 1 earthquake ground motion		$1.0D+1.0S+1.0P$	Not examined

Source: TCVN 11820-11-2025

TCVN
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Part 11:
2025,
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TCVN
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Part 11:
2025,
Bang 7

iv) External forces during construction

During on-land construction in a yard, when lifting with jacks on a slipway or caisson foundation blocks, or when placing the caisson on a launch trolley, the caisson is considered as a beam under concentrated loads. This assumption is used for evaluating the external forces during construction.

v) 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 1.17.

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

Design State/Situation	Direction of Action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with water pressure during installation during construction	Direction of action due to hydrostatic head difference between chambers	1.1S _f	0.5S _f
Permanent state associated with internal earth pressure	Direction of extrusion of outer wall from partition wall	Maximum outward design load that acts on outer wall	Not examined
Permanent state associated with self-weight Variable situation associated with waves Variable situation associated with Level 1 earthquake ground motion	Direction of extrusion of bottom slab from partition wall	Maximum downward design load that acts on bottom slab	Not examined

TCVN
11820
Part 11:
2025
Bang 9

Source: TCVN 11820-11-2025

5) 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

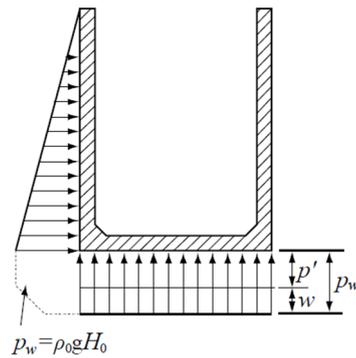
For forces acting on the bottom slab, the value used is the static water pressure at the bottom slab minus the weight of the bottom slab itself (see Figure 1.15).

$$p' = p_w - w = \rho_0 g \cdot H_0 - w \quad (1.27)$$

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)

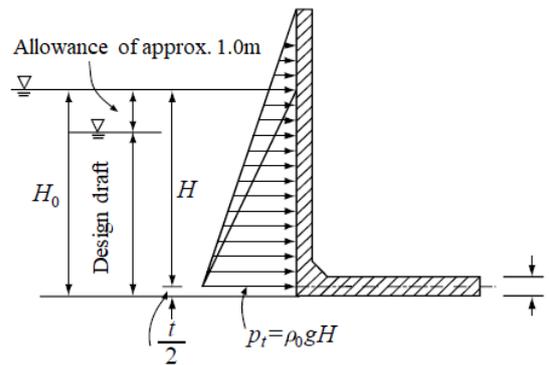


Source: TCVN 11820-6-2023

Figure 1.15- 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 1.16. 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 1.16- Water Pressure Acting on Outer Walls

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

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Part 6:
2023,
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Part III
Chapter 2
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to evaluate the loads based on the specific conditions.

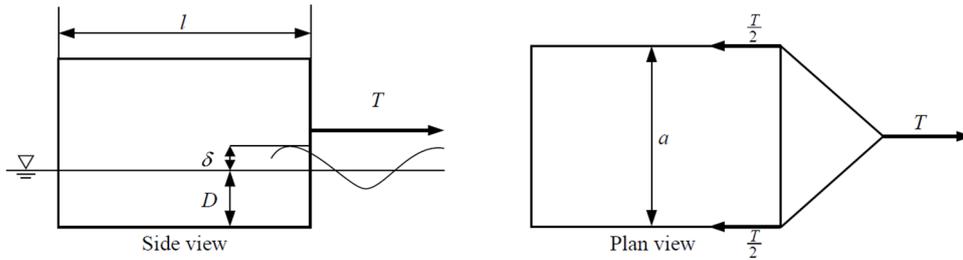
6) External Forces during Towing

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 (1.28). The schematic towing model is shown in Figure 1.17.



Source: TCVN 11820-6-2023

Figure 1.17- Tension Force during Towing

$$T = \frac{\rho_0 C_D V^2 A}{2} \quad (1.28)$$

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 : wetted surface area on caisson front side (m²), $A = a \cdot (D + \delta)$
- a : width of caisson (m)
- D : draft (m)
- δ : water level on front side (m)
- ρ_0 : density of sea water (t/m³)
- 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.

TCVN
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Part 6:
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7) External Forces during Installation

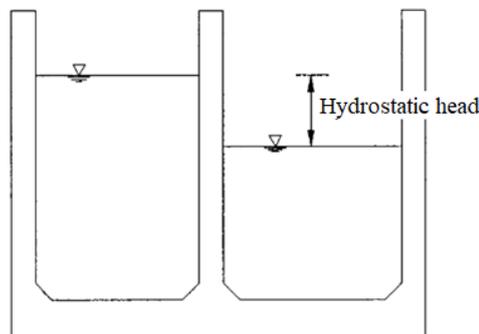
i) Bottom Slab and Outer Wall

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 1.18.

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.



Source: OCDI 2002

Figure 1.18- Head Difference Between Chambers

8) 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/m³.
- ✓ 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/m³ for plain concrete and 24.0kN/m³ 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

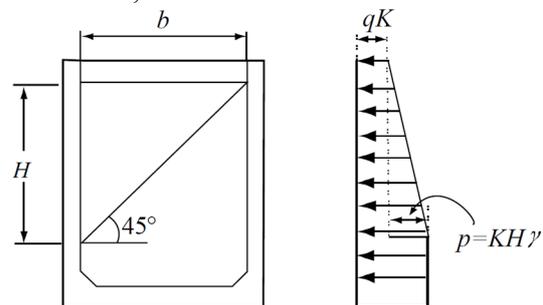
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Part IV
Chapter 1
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calculation is 24.0kN/m^3 .

- ✓ 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 1.19. 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

Figure 1.19- Internal Earth Pressure Distribution

Where:

- q : load transmitted from the top to the backfill (kN/m^2)
- γ' : submerged unit weight of the backfill material (in kN/m^3). 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.

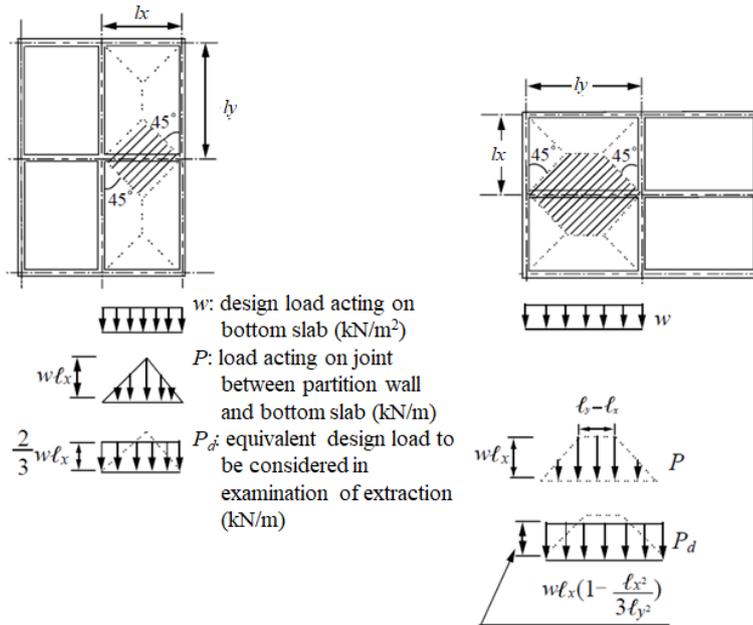
- ✓ Internal Water Pressure: The internal water pressure is determined by the head difference between the water level inside the caisson and the Low Water Level (L.W.L).

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 1.20.
- ✓ 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

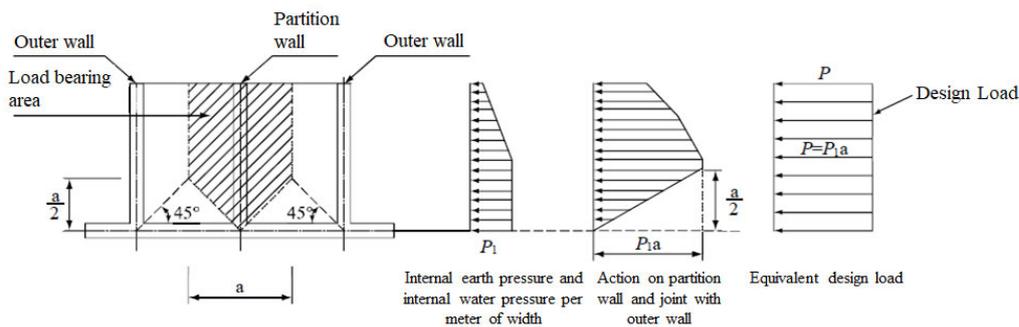
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11820
Part 6:
2023,
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the external walls and the partition walls as Figure 1.21.



Source: OCDI 2020

Figure 1.20- Actions used in Examination of Extrusion Base Slab from Partition Wall

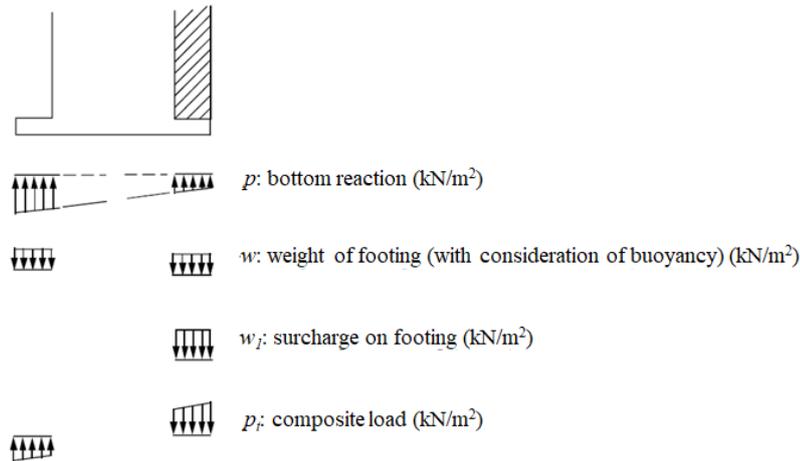


Source: OCDI 2020

Figure 1.21- Actions Used in Examination of Extrusion of Outer Wall from Partition Wall

iv) Footing

- ✓ The design loads acting on the footing are based on the load distribution shown in Figure 1.22.
- ✓ 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/m^3 .
- ✓ 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.



Source: OCDI 2020

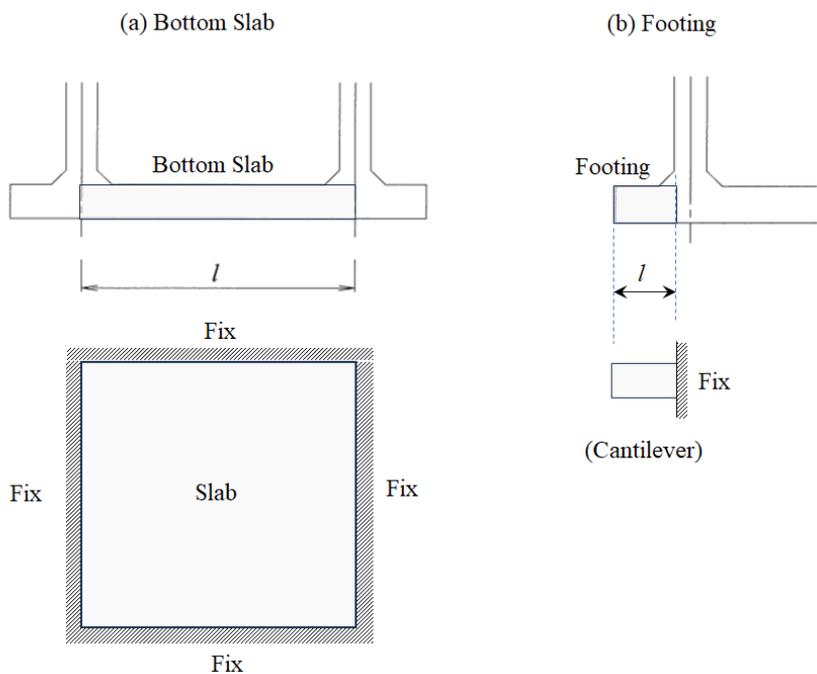
Figure 1.22- Actions on Footing

9) 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 1.23.



Source: TCVN 11820-11-2025

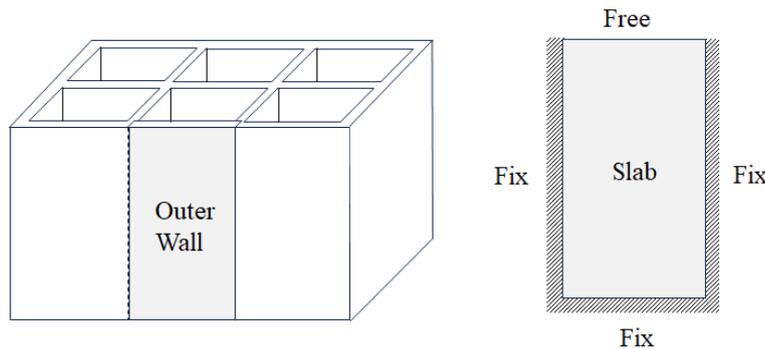
Figure 1.23- Span Used for Bottom Slab and Footing Design

ii) Outer Walls

- ✓ The design of the outer walls is calculated as a three-side fixed, one-side free plate

as Figure 1.24. 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 1.25.
- ✓ The distributed moment for the bending moment of the outer wall, shown in Figure 1.25, becomes as described in Equation (1.29) after distribution.



Source: TCVN 11820-11-2025

Figure 1.24- Modeling of Outer Wall

$$\begin{aligned}
 M'_{BA} &= M_{BA} - (M_{BA} - M_{BC}) \frac{K_a}{K_a + K_b} \\
 M'_{BC} &= M_{BC} + (M_{BA} - M_{BC}) \frac{K_b}{K_a + K_b} \\
 M'_a &= M_a - \frac{1}{2} (M_{BA} - M_{BC}) \frac{K_a}{K_a + K_b} \\
 M'_b &= M_b + \frac{1}{2} (M_{BA} - M_{BC}) \frac{K_b}{K_a + K_b} \\
 M'_{AB} &= M_{AB} \\
 M'_{CB} &= M_{CB}
 \end{aligned} \tag{1.29}$$

Where:

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

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

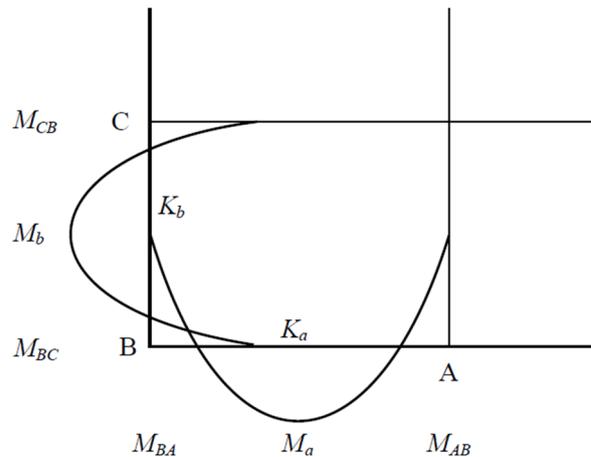
$$K_a = EI_a/l_a; K_b = EI_b/l_b$$

TCVN
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Part 11:
2025,
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Part 11:
2025,
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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:
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Fig. 17

Source: TCVN 11820-11-2025

Figure 1.25- 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

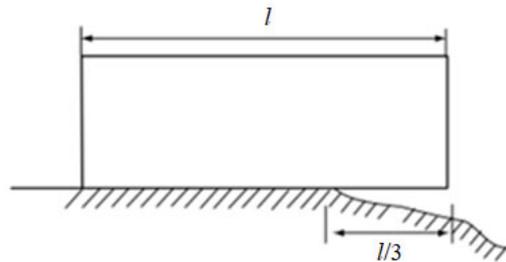
- ✓ The footing section should be calculated as a cantilever plate.
- ✓ The section used for calculations related to bending and shear in the footing will be at the front face of the wall as the Figure 1.23 A-A section. However, for examining shear failure due to diagonal tension, the section can be 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 1.23 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.

10) 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 1.26.



Source: TCVN 11820-6-2023

Figure 1.26- Examination of Actions due to Uneven Ground Support

For the safety verification of members against differential settlement, Equation (1.30) 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:

$$\frac{\gamma_i \cdot M_d}{M_{ud}} \leq 1.0 \quad (1.30)$$

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 (1.31):

$$M_{ud} = f_{bk} Z / \gamma_b \quad (1.31)$$

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.

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Part 6:
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2. Design Example

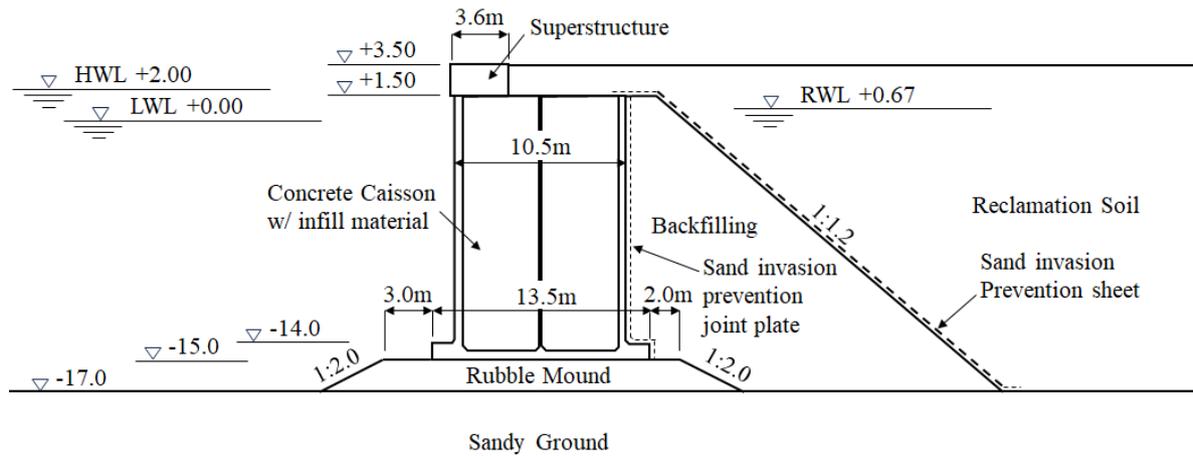


Figure 2.1- Typical Cross Section of Caisson Quay wall

2-1. Design Conditions

In the design standards, 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) Planned Conditions

1) Design Vessels

Container ship, heavy cargo carrier (multi-purpose berth) 50,000 DWT
Berthing velocity $V = 0.10$ m/s

2) Type of Quay wall

Container handling quay wall as international port

3) Water Depth and Crown Height

Planned water depth: -14.00 m
Design water depth: -15.00 m (considering footing thickness of 1.0 m)
Crown height: +3.50 m (1.5 m above H.W.L.)

4) Surcharge load

As surcharges, the following values are used for permanent states and earthquake motion (variable situations of earthquake ground motion).

Permanent state	Variable situation Earthquake motion
30 kN/m ²	15 kN/m ²

(2) Natural Conditions

1) Tide Levels

H.W.L. + 2.00 m
L.W.L. ±0.00 m
R.W.L. +0.67 m (1/3 of tide level difference)

2) Ground Conditions

Ground surface	+3.50
Reclamation soil	$\varphi = 30^\circ$ $\gamma_t = 18 \text{ kN/m}^3, \gamma_{sat} = 20 \text{ kN/m}^3$ $\gamma' = 10 \text{ kN/m}^3$ $\bar{N} = 9$
Original ground	-17.00
Sandy soil	$\varphi = 35^\circ$ $\gamma_t = 18 \text{ kN/m}^3, \gamma_{sat} = 20 \text{ kN/m}^3$ $\gamma' = 10 \text{ kN/m}^3$ $\bar{N} = 25$
	-30.00

- Rubble mound
 $\varphi = 40^\circ$
 $\gamma_{sat} = 20 \text{ kN/m}^3, \gamma' = 10 \text{ kN/m}^3$
- Backfilling stones
 $\varphi = 40^\circ$
 $\gamma_t = 18 \text{ kN/m}^3, \gamma_{sat} = 20 \text{ kN/m}^3$
 $\gamma' = 10 \text{ kN/m}^3$

(3) Friction Coefficient between Caisson and Rubble Mound

$$f = 0.6$$

(4) 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_{hl}) \times soil condition coefficient (γ_s) \times importance coefficient (γ_i) = $0.08 \times 1.2 \times 1.0 = 0.096$

0.10 of the seismic coefficients is assumed in this design example.

(5) 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.

(6) Partial Factor

1) Permanent State

- i) Sliding of caisson body
 - $\gamma_R = 0.87$ (resistance term)
 - $\gamma_S = 1.06$ (load term)
 - $m = 1.00$ (adjustment factor)
- ii) Overturning of caisson body
 - $\gamma_R = 0.99$ (resistance term)
 - $\gamma_S = 1.23$ (load term)
 - $m = 1.00$ (adjustment factor)
- iii) Bearing capacity against eccentric inclined loads
 - $\gamma_R = 1.00$ (resistance term)
 - $\gamma_S = 1.00$ (load term)
 - $m = 1.20$ (adjustment factor)
- iv) Circular slip failure (for sandy soil)
 - $\gamma_R = 0.83$ (resistance term)
 - $\gamma_S = 1.01$ (load term)
 - $m = 1.00$ (adjustment factor)

2) Variable Situation: Level 1 Earthquake Ground Motion

- i) Sliding of caisson body
 - $\gamma_R = 1.00$ (resistance term)
 - $\gamma_S = 1.00$ (load term)
 - $m = 1.00$ (adjustment factor)
- ii) Overturning of caisson body
 - $\gamma_R = 1.00$ (resistance term)
 - $\gamma_S = 1.00$ (load term)
 - $m = 1.10$ (adjustment factor)
- iii) Bearing capacity against eccentric inclined loads
 - $\gamma_R = 1.00$ (resistance term)
 - $\gamma_S = 1.00$ (load term)
 - $m = 1.00$ (adjustment factor)

2-2. Design Specifications

(1) Determination of the Top Level of Caisson

The top level of a caisson in the nearshore area will be D.L.+ 1.50 m, as the calmness inside harbour is maintained during construction period.

(2) Thickness of Cover Concrete

Since calm will be maintained within the harbor during the construction period, the thickness of the cover concrete will be 0.3m.

2-3. Assumptions of Design Cross-sections

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

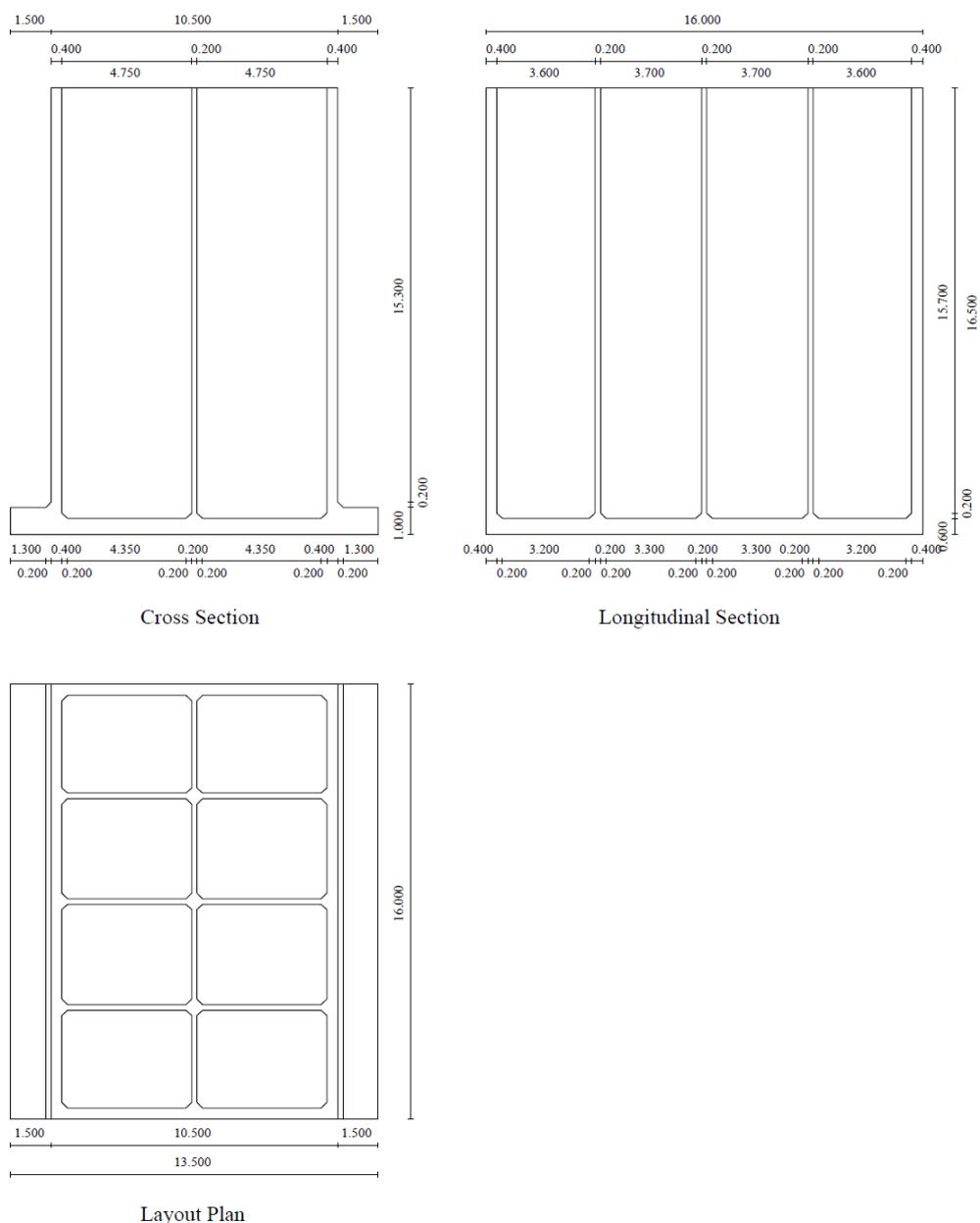


Figure 2.2- Caisson Dimension (Unit: m)

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)	x (m)	$W \cdot x$ (kN·m)
			W (kN)	y (m)	$W \cdot y$ (kN·m)
Superstructure 1	0.100 × 1.900 × 16.000	1	3.040	1.450	99.621
			68.704	17.550	1,205.755
Superstructure 2	0.100 × 0.100 × 16.000 ×1/2	1	0.080	1.467	2.652
			1.808	16.567	29.953

Superstructure 3	3.500 × 2.000 × 16.000	1	112.000 2,531.200	3.250 17.500	8,226.400 44,296.000
Total			115.120 2,601.712	3.201 17.501	8,328.673 45,531.708
Weight per meter W (kN/m) ($L=16.000$ m)			162.607		

Note: $\gamma=22.6$ (kN/m³)

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	10.500 × 16.000 × 0.600	1	100.800 2,419.200	6.750 0.300	16,329.600 725.760
Side Wall Longitudinal	0.400 × 16.000 × 15.900	2	203.520 4,884.480	6.750 8.550	32,970.240 41,762.304
Side Wall Transverse	9.700 × 0.400 × 15.900	2	123.384 2,961.216	6.750 8.550	19,988.208 25,318.397
Partition Longitudinal	0.200 × 15.200 × 15.900	1	48.336 1,160.064	6.750 8.550	7,830.432 9,918.547
Partition Transverse	9.500 × 0.200 × 15.900	3	90.630 2,175.120	6.750 8.550	14,682.060 18,597.276
Vertical haunch	0.200 × 0.200 × 15.900 ×1/2	32	10.176 244.224	6.750 8.550	1,648.512 2,088.115
Horizontal haunch Longitudinal	0.200 × 13.000 × 0.200 ×1/2	4	1.040 24.960	6.750 0.667	168.480 16.648
Horizontal haunch Transverse	8.700 × 0.200 × 0.200 ×1/2	8	1.392 33.408	6.750 0.667	225.504 22.283
Corner haunch	0.200 × 0.200 × 0.200 ×1/3	32	0.085 2.040	6.750 0.675	13.770 1.377
Footing	1.500 × 16.000 × 1.000	2	48.000 1,152.000	6.750 0.500	7,776.000 576.000
Footing haunch	0.200 × 16.000 × 0.200 ×1/2	2	0.640 15.360	6.750 1.067	103.680 16.389
Total			628.003 15,072.072	6.750 6.571	101,736.486 99,043.096
Weight per meter W (kN/m) ($L=16.000$ m)			942.005		

Note: $\gamma=24.0$ (kN/m³)

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	4.750 × 3.600 × 0.300	4	20.520 463.752	6.750 16.350	3,130.326 7,582.345
Cover	4.750 × 3.700 × 0.300	4	21.090 476.634	6.750 16.350	3,217.280 7,792.966
Vertical haunch	0.200 × 0.200 × 0.300 ×1/2	32	-0.192 -4.339	6.750 16.350	-29.288 -70.943
Total			41.418 936.047	6.750 16.350	6,318.318 15,304.368
Weight per meter W (kN/m) ($L=16.000$ m)			58.503		

Note: $\gamma=22.6$ (kN/m³)

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	4.750 × 3.600 × 15.600	4	1,067.040 21,340.800	6.750 8.400	144,050.400 179,262.720
Infill material	4.750 × 3.700 × 15.600	4	1,096.680 21,933.600	6.750 8.400	148,051.800 184,242.240
Vertical haunch	0.200 × 0.200 × 15.600 ×1/2	32	-9.984 -199.680	6.750 8.400	-1,347.840 -1,677.312
Horizontal haunch Transverse	4.350 × 0.200 × 0.200 ×1/2	16	-1.392 -27.840	6.750 0.667	-187.920 -18.569
Horizontal haunch Longitudinal	0.200 × 3.200 × 0.200 ×1/2	8	-0.512 -10.240	6.750 0.667	-69.120 -6.830
Horizontal haunch Longitudinal	0.200 × 3.300 × 0.200 ×1/2	8	-0.528 -10.560	6.750 0.667	-71.280 -7.044
Corner haunch	0.200 × 0.200 × 0.200 ×1/3	32	-0.085 -1.700	6.750 0.667	-11.475 -1.134
Total			2,151.219 43,024.380	6.750 8.409	290,414.565 361,794.071
Weight per meter W (kN/m) ($L=16.000$ m)			2,689.024		

Note: $\gamma_{sat}=20.0$ (kN/m³)

5) Overburden Soil

The weights and moments of the overburden soil are presented in Table 2.6.

Table 2.6- Characteristic Values of Overburden Soil 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)
Overburden soil	8.500 × 2.000 × 16.000	1	272.000 4,896.000	9.250 17.500	45,288.000 85,680.000
Total			272.000 4,896.000	9.250 17.500	45,288.000 85,680.000
Weight per meter W (kN/m) ($L=16.000$ m)			306.000		

Note: $\gamma_t=18.0$ (kN/m³)

6) Backfill Material

The weights and moments of the backfill material are presented in Table 2.7.

Table 2.7- Characteristic Values of Backfill 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)
Backfill 1	1.500 × 0.830 × 16.000	1	19.920 358.560	12.750 16.085	4,571.640 5,767.438
Backfill 2	0.200 × 14.470 × 16.000	1	46.304 926.080	12.100 8.435	11,205.568 7,811.485
Backfill 3	0.200 × 0.200 × 16.000 ×1/2	1	0.320 6.400	12.133 1.133	77.651 7.251
Backfill 4	1.300 × 14.670 × 16.000 ×1/2	1	152.568 6,102.720	12.850 8.335	78,419.952 50,866.171
Total			219.112 7,393.760	12.751 8.717	94,274.811 64,452.345
Weight per meter W (kN/m) ($L=16.000$ m)			462.110		

Note: $\gamma_t=18.0$ (kN/m³), $\gamma_{sat}=20.0$ (kN/m³)

(3) Characteristic Value of Design Load

1) Quay wall Body Weight and Moment

The characteristic values of the quay wall body weight and moment per 1.0 meter are shown in Table 2.8 and Figure 2.3.

Table 2.8- Characteristic Values of Quay wall Weight and Moment

No	Name	W (kN/m)	x (m)	$W \cdot x$ (kN·m/m)
A	Superstructure	162.607	3.201	520.505
B	Caisson	942.005	6.750	6,358.534
B	Cover concrete	58.503	6.750	394.895
B	Infill material	2,689.024	6.750	18,150.912
C	Overburden soil	306.000	9.250	2,830.500
D	Backfill material	462.110	12.751	5,892.365
Point a (-15.000m)		4,620.249		34,147.711

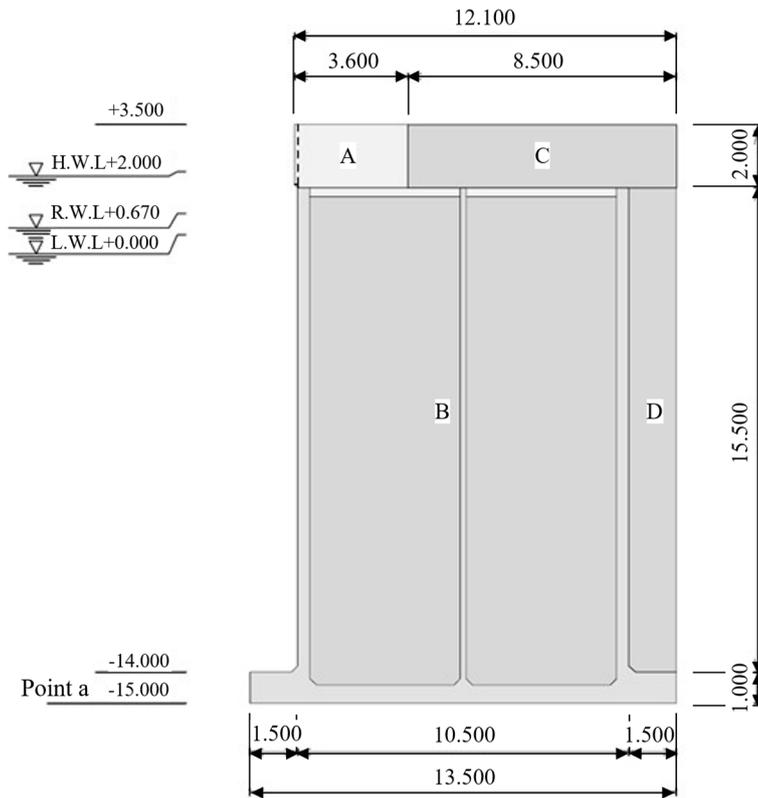


Figure 2.3- Assumed Quay wall Dimension

2) Inertial Force and Moment at Earthquake

The characteristic values of the inertial force and moment are shown in Table 2.9.

Table 2.9- Characteristic Values of Inertial Force and Moment at Earthquake

No	Name	W (kN/m)	k_h	P_E (kN/m)	y (m)	$P_E y$ (kN·m/m)
A	Superstructure	162.607	0.100	16.261	17.501	284.584
B	Caisson	942.005	0.100	94.201	6.571	618.995
B	Cover concrete	58.503	0.100	5.850	16.350	95.648
B	Infill material	2,689.024	0.100	268.902	8.409	2,261.197
C	Overburden soil	306.000	0.100	30.600	17.500	535.500
D	Backfilling material	462.110	0.100	46.211	8.717	402.821
Point a (-15.000m)				462.025		4,198.745

3) Buoyancy and Moment

The characteristic values of buoyancy and moment below the residual water level (RWL) are shown in Table 2.10.

Table 2.10- Characteristic Values of Buoyancy and Moment

B: Caisson

Name	Dimensions (m)	Nos	P_B (kN/m)	x (m)	$P_B \cdot x$ (kN·m/m)
Caisson	10.500 × 15.670 × 10.100	1	1,661.804	5.250	8,724.471
Footing	1.500 × 1.000 × 10.100	2	30.300	5.250	159.075
Footing haunch	0.200 × 0.200 × 10.100 × 1/2	2	0.404	5.250	2.121
Total			1,692.508	5.250	8,885.667

D: Backfilling

No	Base × Height × Unit weight	P_B (kN/m)	x (m)	$P_B \cdot x$ (kN·m/m)
1	0.200 × 14.470 × 16.000 × 10.100	467.670	10.600	4,957.302
2	1/2 × 0.200 × 0.200 × 16.000 × 10.100	3.232	10.633	34.366
3	1.300 × 14.670 × 16.000 × 10.100	3,081.874	11.350	34,979.270
Total		3,552.776	11.251	39,970.938
Weight per meter W(kN/m) (L=16.000m)		222.049		

Total buoyancy taking into account footing width

No	Name	P_B (kN/m)	x (m)	$P_B \cdot x$ (kN·m/m)
	B Caisson	1,692.508	6.750	11,424.429
	D Backfilling material	222.049	12.751	2,831.347
	Point a (-15.000m)	1,914.557		14,255.776

4) Surcharge Load and Moment

The characteristic values of surcharge load and moment are shown in Table 2.11.

Table 2.11- Characteristic Values of Surcharge Load and Moment

Permanent state

	No	Load × Acting width w (kN/m ²) × b (m)	P_{VS} (kN/m)	x (m)	$P_{VS} \cdot x$ (kN·m/m)
Point a (-15.000m)	1	30.000 × 12.100	363.000	7.450	2,704.350

Variable situation: Level 1 earthquake ground motion - Vertical force

	No	Load × Acting width w (kN/m ²) × b (m)	P_{VS} (kN/m)	x (m)	$P_{VS} \cdot x$ (kN·m/m)
Point a (-15.000m)	1	15.000 × 12.100	181.500	7.450	1,352.175

Variable situation: Level 1 earthquake ground motion - Inertial force

	No	V_j (kN/m) k_h	P_{HS} (kN/m)	y (m)	$P_{HS} \cdot y$ (kN·m/m)
Point a (-15.000m)	1	181.500 × 0.100	18.150	18.500	335.775

(4) Earth Pressure and Moment

1) Equation of Earth Pressure

Earth pressure is calculated by the following equation.

$$P_a = K_a \left[\sum \gamma_i h_i + \frac{w}{\cos \beta} \right]$$

$$K_a = \frac{\cos^2(\varphi - \theta)}{\cos \theta \cos(\delta + \theta) \left[1 + \sqrt{\frac{\sin(\varphi + \theta) \sin(\varphi - \beta - \theta)}{\cos(\delta + \theta) \cos \beta}} \right]^2}$$

Where:

- P_a : intensity of active earth pressure (kN/m²)
- ϕ : shear resistance angle of soil (°)
- γ_i : unit weight of soil (kN/m³)
- h_i : thickness of a layer (m)
- K_a : coefficient of active earth pressure
- β : angle of ground surface to the horizontal (°)
- δ : angle of wall friction (°)
- ω : load per unit area of ground surface (kN/m²)
- θ : composite seismic angle (°)
 $\theta = \tan^{-1}k$ or $\theta = \tan^{-1}k'$ (In the permanent state, $\theta = 0$)
- k : seismic coefficient
- k' : apparent seismic coefficient

The apparent seismic coefficient below water level is given by the following equation.

$$k' = \frac{2 \left(\sum \gamma_{ti} h_i + \sum \gamma_{satj} h_j + \omega \right) + \gamma_{sat} h}{2 \left\{ \sum \gamma_{ti} h_i + \sum (\gamma_{satj} - 10) h_j + \omega \right\} + (\gamma_{sat} - 10) h} k$$

Where:

- h_i : thickness of i -th soil layer above residual water level (m)
- h_j : thickness of j -th soil layer above the layer where a soil layer below the residual water level is to be obtained (m)
- h : thickness of a soil layer below the residual water level, where the soil layer is to be obtained (m)
- γ_{ti} : unit weight of soil above the residual water level (kN/m³)
- γ_{satj} : unit weight in air of soil saturated with water (kN/m³)
- ω : load per unit area of ground surface (kN/m²)
- k : seismic coefficient
- k' : apparent seismic coefficient

2) Characteristic Values of Earth Pressure and Moment (Permanent state)

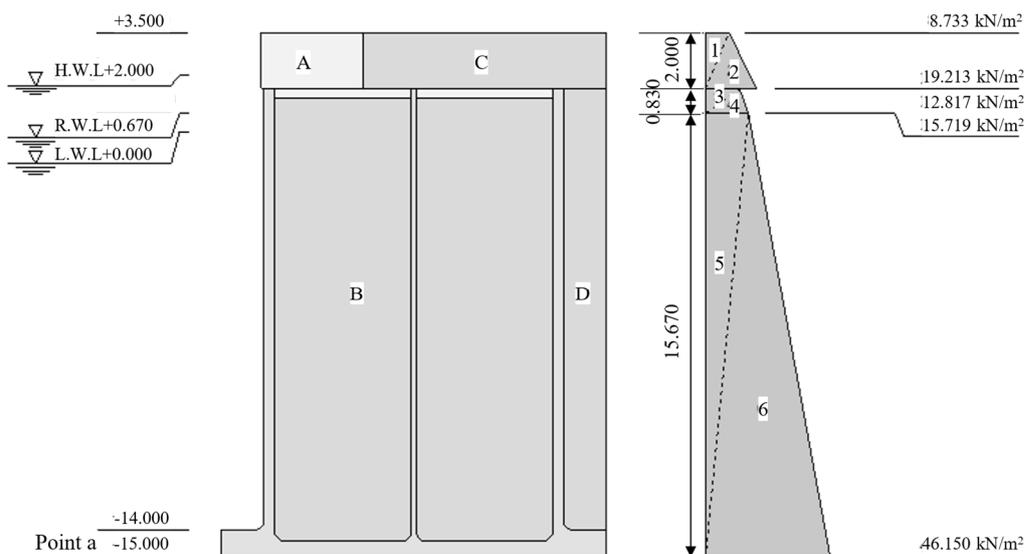


Figure 2.4- Active Earth Pressure (Permanent state)

Table 2.12- Characteristic Values of Active Earth Pressure (Permanent state)

Level (m)	h (m)	γ (kN/m ³)	γh (kN/m ²)	$\Sigma\gamma h$ (kN/m ²)	ψ (deg)	$\omega\cos\psi/\cos(\psi-\beta)$	$K_a\cos(\delta+\psi)$	P_a (kN/m ²)
3.500	0.000	18.0	0.000	0.000	0.0	30.000	0.2911	8.733
1.500	2.000	18.0	36.000	36.000	0.0	30.000	0.2911	19.213
1.500	2.000	18.0	36.000	36.000	0.0	30.000	0.1942	12.817
0.670	0.830	18.0	14.940	50.940	0.0	30.000	0.1942	15.719
0.670	0.830	10.0	14.940	50.940	0.0	30.000	0.1942	15.719
-15.000	15.670	10.0	156.700	207.640	0.0	30.000	0.1942	46.150

Table 2.13- Characteristic Values of Total Earth Pressure and Moment (Permanent state)

Horizontal forces and moments

No.	Calculation formula P_a (kN/m ²) h (m)	P_H (kN/m)	Arm length y (m)	Moment M_{PH} (kN·m/m)
1	$1/2 \times 8.733 \times 2.000$	8.733	17.833	155.736
2	$1/2 \times 19.213 \times 2.000$	19.213	17.167	329.830
3	$1/2 \times 12.817 \times 0.830$	5.319	16.223	86.290
4	$1/2 \times 15.719 \times 0.830$	6.523	15.947	104.022
5	$1/2 \times 15.719 \times 15.670$	123.158	10.447	1,286.632
6	$1/2 \times 46.150 \times 15.670$	361.585	5.223	1,888.558
Total		524.531		3,851.068

Vertical forces and moments

No.	Horizontal force P_H (kN/m)	$\tan(\psi+\delta)$	Vertical force P_V (kN/m)	Arm length x (m)	Moment M_{PV} (kN·m/m)
1	8.733	0.268	2.340	13.500	31.590
2	19.213	0.268	5.149	13.500	69.512
3	5.319	0.268	1.425	13.500	19.238
4	6.523	0.268	1.748	13.500	23.598
5	123.158	0.268	33.006	13.500	445.581
6	361.585	0.268	96.905	13.500	1,308.218
Total			140.573		1,897.737

3) Characteristic Values of Earth Pressure and Moment (Variable situation: Level 1 Earthquake Ground Motion)

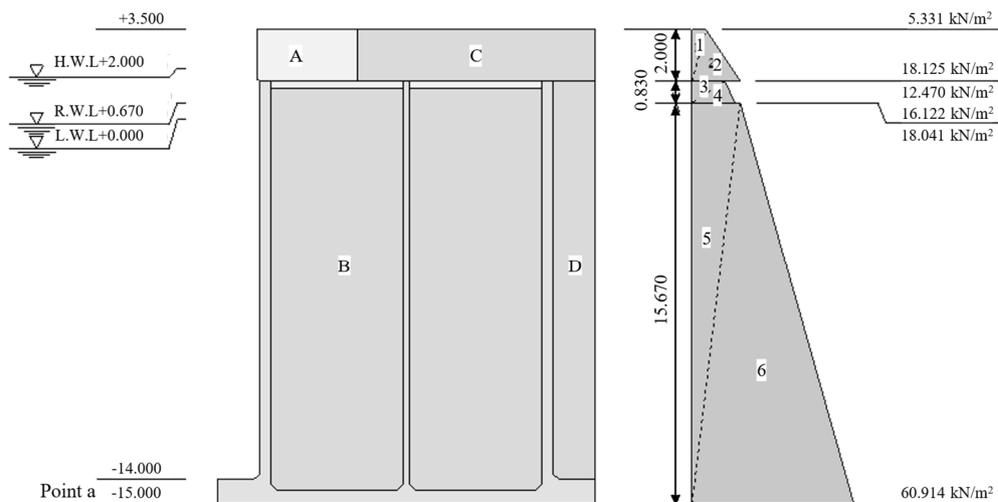


Figure 2.5- Active Earth Pressure (Variable situation: Level 1 Earthquake Ground Motion)

**Table 2.14- Characteristic Values of Active Earth Pressure
(Variable situation: Level 1 Earthquake Ground Motion)**

Earth pressure coefficient

Earth Pressure Level (m)	β (degrees)	ϕ (degrees)	δ (degrees)	ψ (degrees)	k or k'	θ (degrees)	K_a
3.500 to 1.500	0.0	30.0	15.0	0.0	0.10	5.71	0.3679
1.500 to 0.670	0.0	40.0	15.0	0.0	0.10	5.71	0.2531
0.670 to -15.000	0.0	40.0	15.0	0.0	0.15	8.53	0.2833

Earth pressure

Level (m)	h (m)	γ (kN/m ³)	γh (kN/m ²)	$\Sigma \gamma h$ (kN/m ²)	ψ (deg)	$\omega \cos \psi / \cos(\psi - \beta)$	$K_a \cos(\delta + \psi)$	P_a (kN/m ²)
3.500	0.000	18.0	0.000	0.000	0.0	15.000	0.3554	5.331
1.500	2.000	18.0	36.000	36.000	0.0	15.000	0.3554	18.125
1.500	2.000	18.0	36.000	36.000	0.0	15.000	0.2445	12.470
0.670	0.830	18.0	14.940	50.940	0.0	15.000	0.2445	16.122
0.670	0.830	10.0	14.940	50.940	0.0	15.000	0.2736	18.041
-15.000	15.670	10.0	156.700	207.640	0.0	15.000	0.2736	60.914

**Table 2.15- Characteristic Values of Total Earth Pressure and Moment
(Variable situation: Level 1 Earthquake Ground Motion)**

Horizontal forces and moments

No.	Calculation formula P_a (kN/m ²) h (m)	P_H (kN/m)	Arm length y (m)	Moment M_{PH} (kN·m/m)
1	$1/2 \times 5.331 \times 2.000$	5.331	17.833	95.068
2	$1/2 \times 18.125 \times 2.000$	18.125	17.167	311.152
3	$1/2 \times 12.470 \times 0.830$	5.175	16.223	83.954
4	$1/2 \times 16.122 \times 0.830$	6.691	15.947	106.701
5	$1/2 \times 18.041 \times 15.670$	141.351	10.447	1,476.694
6	$1/2 \times 60.914 \times 15.670$	477.261	5.223	2,492.734
Total		653.934		4,566.303

Vertical forces and moments

No.	Horizontal force P_H (kN/m)	$\tan(\psi + \delta)$	Vertical force P_V (kN/m)	Arm length x (m)	Moment M_{PV} (kN·m/m)
1	5.331	0.268	1.429	13.500	19.292
2	18.125	0.268	4.858	13.500	65.583
3	5.175	0.268	1.387	13.500	18.725
4	6.691	0.268	1.793	13.500	24.206
5	141.351	0.268	37.882	13.500	511.407
6	477.261	0.268	127.906	13.500	1,726.731
Total			175.255		2,365.944

4) Residual Water Pressure and Moment

Residual water pressure considers the water pressure due to the difference of the water levels between the mean monthly lowest water level (L.W.L.) and the residual water level (R.W.L.).

$$p_w = \gamma_w h_w$$

Where:

p_w : residual water pressure (kN/m²)

h_w : residual water level; in case the water level in the backfilling material or the backfilling soil is higher than the water level on the front side of the structure, the maximum water level difference at that time is used (m)

γ_w : unit weight of water (kN/m³)

$$\gamma_w = \rho_0 g = 10.1 \text{ (kN/m}^3\text{)}$$

Water level difference = 0.670 - 0.000 = 0.670 (m)
 Residual water pressure $p_w = 10.100 \times 0.670 = 6.767$ (kN/m²)

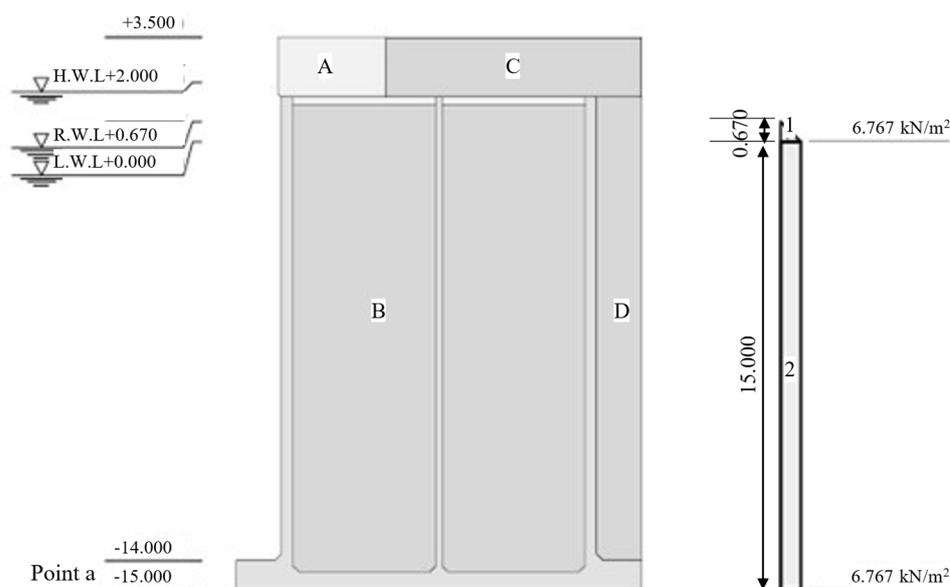


Figure 2.6- Residual Water Pressure

Table 2.16- Characteristic Value of Residual Water Pressure and Moment

No	Calculation formula	P_w (kN/m)	y (m)	Moment M_{WH} (kN·m/m)
1	$1/2 \times 6.767 \times 0.670$	2.267	15.223	34.511
2	6.767×15.000	101.505	7.500	761.288
Point a (-15.000m)		103.772		795.799

5) Dynamic Water Pressure and Moment

Under variable situations associated with earthquake motion, the dynamic water pressure at the front side of the quaywall is directed toward the sea. Dynamic water pressure is obtained by the following equation (Westergaard's approximate formula).

$$p_{dw} = \frac{7}{8} k \gamma_w \sqrt{Hy}$$

$$P_{dw} = \pm \frac{7}{12} k \gamma_w H^2, \quad h_{dw} = \frac{3}{5} H$$

Where:

- p_{dw} : dynamic water pressure (kN/m²)
- k : horizontal seismic coefficient = 0.10
- γ_w : unit weight of water (seawater) (kN/m³)
- H : height of structure below the still water level (m) (caisson installation water depth at L.W.L.) = 15.0 (m)
- y : depth of the dynamic water pressure calculation level from the still water level (m)
- P_{dwk} : resultant force of dynamic water pressure (kN/m)
- h_{dw} : depth of the acting point of the dynamic water pressure resultant force from the still water level (m)

$$P_{dwk} = 7/12 \times 0.10 \times 10.1 \times 15.00^2 = 132.563 \text{ (kN/m)}$$

$$h_{dw} = 3/5 \times 15.00 = 9.00 \text{ (m) (elevation -9.00 m)}$$

$$P_{dwk} \cdot y = 132.563 \times (15.00 - 9.00) = 795.378 \text{ (kN}\cdot\text{m/m)}$$

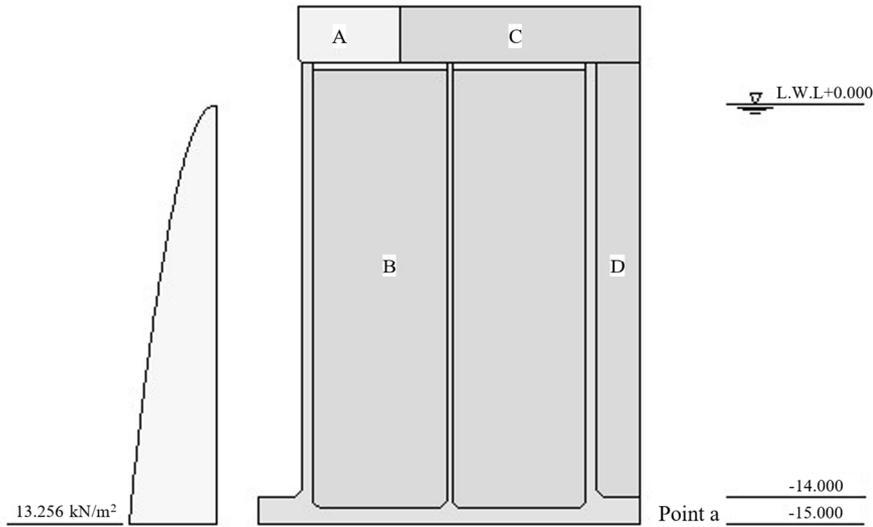


Figure 2.7- Dynamic Water Pressure

2-5. Verification of Stability

(1) Verification of Sliding/Overturning of Wall Body

1) Permanent state (without surcharge)

Point a (-15.000m)

	V (kN/m)	H (kN/m)	M_V (kN·m/m)	M_H (kN·m/m)
Earth pressure	140.573	524.531	1,897.737	3,851.068
Residual water pressure		103.772		795.799
Wall weight	4,620.249		34,147.711	
Buoyancy	-1,914.557		-14,255.776	
Total	2,846.265	628.303	21,789.672	4,646.867

i) Verification of Sliding Stability

$$\text{Resistance term } R_d = \gamma_R \cdot f \cdot V = 0.87 \times 0.60 \times 2,846.265 = 1,485.750 \text{ (kN/m)}$$

$$\text{Load term } S_d = \gamma_S \cdot H = 1.06 \times 628.303 = 666.001 \text{ (kN/m)}$$

$$m \cdot S_d / R_d = 1.00 \times 666.001 / 1,485.750 = 0.449 \leq 1.00 \text{ O.K.}$$

ii) Verification of Overturning Stability

$$\text{Resistance term } R_d = \gamma_R \cdot M_V = 0.99 \times 21,789.672 = 21,571.775 \text{ (kN}\cdot\text{m/m)}$$

$$\text{Load term } S_d = \gamma_S \cdot M_H = 1.23 \times 4,646.867 = 5,715.646 \text{ (kN}\cdot\text{m/m)}$$

$$m \cdot S_d / R_d = 1.00 \times 5,715.646 / 21,571.775 = 0.265 \leq 1.00 \text{ O.K.}$$

2) Permanent state (with surcharge)

Point a (-15.000m)

	V (kN/m)	H (kN/m)	M_V (kN·m/m)	M_H (kN·m/m)
Earth pressure	140.573	524.531	1897.737	3,851.068
Residual water pressure		103.772		795.799
Wall weight	4,620.249		34,147.711	
Buoyancy	-1,914.557		-14,255.776	
Surcharge load	363.000		2,704.350	
Total	3,209.265	628.303	24,494.022	4,646.867

i) Verification of Sliding Stability

$$\text{Resistance term } R_d = \gamma_R \cdot f \cdot V = 0.87 \times 0.60 \times 3,209.265 = 1,675.236 \text{ (kN/m)}$$

$$\text{Load term } S_d = \gamma_S \cdot H = 1.06 \times 628.303 = 666.001 \text{ (kN/m)}$$

$$m \cdot S_d / R_d = 1.00 \times 666.001 / 1,675.236 = 0.398 \leq 1.00 \quad \text{O.K.}$$

ii) Verification of Overturning Stability

$$\text{Resistance term } R_d = \gamma_R \cdot M_V = 0.99 \times 24,494.022 = 24,249.082 \text{ (kN}\cdot\text{m/m)}$$

$$\text{Load term } S_d = \gamma_S \cdot M_H = 1.23 \times 4,646.867 = 5,715.646 \text{ (kN}\cdot\text{m/m)}$$

$$m \cdot S_d / R_d = 1.00 \times 5,715.646 / 24,249.082 = 0.236 \leq 1.00 \quad \text{O.K.}$$

3) Variable situation: Level 1 Earthquake Ground Motion (without surcharge)

Point a (-15.000m)

	V (kN/m)	H (kN/m)	M_V (kN·m/m)	M_H (kN·m/m)
Earth pressure	175.255	653.934	2,365.944	4,566.303
Residual water pressure	—	103.772	—	795.799
Wall weight	4,620.249	—	34,147.711	—
Buoyancy	-1,914.557	—	-14,255.776	—
Inertia force	—	462.025	—	4,198.745
Dynamic Water Pressure	—	132.563	—	795.378
Total	2,880.947	1,352.294	22,257.879	10,356.225

i) Verification of Sliding Stability

$$\text{Resistance term } R_d = \gamma_R \cdot f \cdot V = 1.00 \times 0.60 \times 2,880.947 = 1,728.568 \text{ (kN/m)}$$

$$\text{Load term } S_d = \gamma_S \cdot H = 1.00 \times 1,352.294 = 1,352.294 \text{ (kN/m)}$$

$$m \cdot S_d / R_d = 1.00 \times 1,352.294 / 1,728.568 = 0.783 \leq 1.00 \quad \text{O.K.}$$

ii) Verification of Overturning Stability

$$\text{Resistance term } R_d = \gamma_R \cdot M_V = 1.00 \times 22,257.879 = 22,257.879 \text{ (kN}\cdot\text{m/m)}$$

$$\text{Load term } S_d = \gamma_S \cdot M_H = 1.00 \times 10,356.225 = 10,356.225 \text{ (kN}\cdot\text{m/m)}$$

$$m \cdot S_d / R_d = 1.00 \times 10,356.225 / 22,257.879 = 0.512 \leq 1.00 \quad \text{O.K.}$$

4) Variable situation: Level 1 Earthquake Ground Motion (with surcharge)

Point a (-15.000m)

	V (kN/m)	H (kN/m)	M_V (kN·m/m)	M_H (kN·m/m)
Earth pressure	175.255	653.934	2,365.944	4,566.303
Residual water pressure	—	103.772	—	795.799
Wall weight	4,620.249	—	34,147.711	—
Buoyancy	-1,914.557	—	-14,255.776	—
Inertia force	—	462.025	—	4,198.745
Dynamic Water Pressure	—	132.563	—	795.378
Surcharge load	181.500	18.150	1,352.175	335.775
Total	3,062.447	1,370.444	23,610.054	10,692.000

i) Verification of Sliding Stability

$$\text{Resistance term } R_d = \gamma_R \cdot f \cdot V = 1.00 \times 0.60 \times 3,062.447 = 1,837.468 \text{ (kN/m)}$$

$$\text{Load term } S_d = \gamma_S \cdot H = 1.00 \times 1,370.444 = 1,370.444 \text{ (kN/m)}$$

$$m \cdot S_d / R_d = 1.00 \times 1,370.444 / 1,837.468 = 0.746 \leq 1.00 \quad \text{O.K.}$$

ii) Verification of Overturning Stability

Resistance term $R_d = \gamma_R \cdot M_V = 1.00 \times 23,610.054 = 23,610.054 \text{ (kN}\cdot\text{m/m)}$

Load term $S_d = \gamma_S \cdot M_H = 1.00 \times 10,692.000 = 10,692.000 \text{ (kN}\cdot\text{m/m)}$

$m \cdot S_d / R_d = 1.00 \times 10,692.000 / 23,610.054 = 0.499 \leq 1.00 \text{ O.K.}$

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

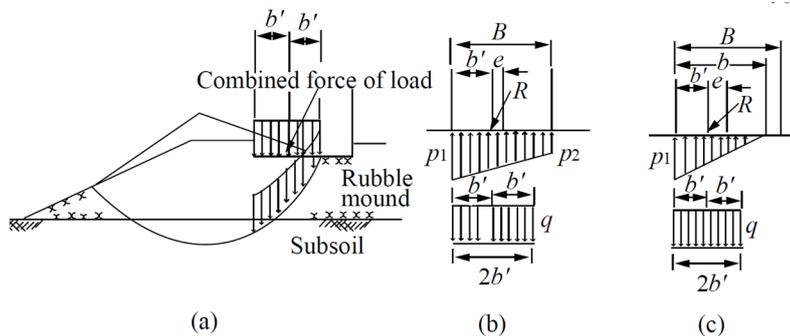


Figure 2.8- Analysis of Bearing Capacity for Eccentric and Inclined Actions

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 $C=20.0 \text{ kN/m}^2$ and internal friction angle $\phi=35^\circ$. For foundation soils, a standard value of $\phi=40^\circ$ is used for sandy soils with an N-value less than 10, and $\phi=45^\circ$ 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 2.17- Characteristic Values of Ground Conditions for Verification of Bearing Capacity (for Bishop's method)

	Saturated weight $\gamma_{sat} \text{ (kN/m}^3\text{)}$	Weight in water $\gamma' \text{ (kN/m}^3\text{)}$	Shear resistance angle $\phi'_k \text{ (}^\circ\text{)}$	Cohesion	
				$c'_k \text{ (kN/m}^2\text{)}$	Primary coefficient of cohesion
Foundation Rubble mound	20.00	10.00	35.00 ($\tan \phi'_k = 0.700$)	20.00	0.00
Sandy soil	20.00	10.00	40.00 ($\tan \phi'_k = 0.839$)	0.00	0.00

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2) Load Condition

The critical load condition at variable situation without surcharge load is indicated as follows:

Variable situation: Level 1 Earthquake Ground Motion (without surcharge)

	V (kN/m)	H (kN/m)	M_V (kN·m/m)	M_H (kN·m/m)
Earth pressure	175.255	653.934	2,365.944	4,566.303
Residual water pressure	—	103.772	—	795.799
Wall weight	4,620.249	—	34,147.711	—
Buoyancy	-1,914.557	—	-14,255.776	—
Inertia force	—	462.025	—	4,198.745
Dynamic Water Pressure	—	132.563	—	795.378
Total	2,880.947	1,352.294	22,257.879	10,356.225

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

$$x = \frac{\Sigma M}{\Sigma V} = \frac{\Sigma M_V - \Sigma M_H}{\Sigma V} = \frac{22,257.879 - 10,356.225}{2,880.947} = 4.131 \text{ (m)}$$

$$e = B/2 - x = 13.500/2 - 4.131 = 2.619 \text{ (m)}$$

$$e > B/6 = 13.500 / 6 = 2.250 \text{ (m)}$$

Therefore, the distribution is triangular, and

$$p_1 = \frac{2\Sigma V}{3(B/2 - e)} = \frac{2 \times 2,880.947}{3 \times (13.500/2 - 2.619)} = 464.932 \text{ (kN/m}^2\text{)}$$

$$\text{Width of distribution } b' = 3(B/2 - e) = 3 \times (13.500/2 - 2.619) = 12.393 \text{ (m)}$$

Calculation of width of load

$$2b' = \frac{2\Sigma M}{\Sigma V} = \frac{2(\Sigma V \cdot x - \Sigma H \cdot y)}{\Sigma V} = \frac{2 \times (22,257.879 - 10,356.225)}{2,880.947} = 8.262 \text{ (m)}$$

Calculation of average value of uniformly-distributed load

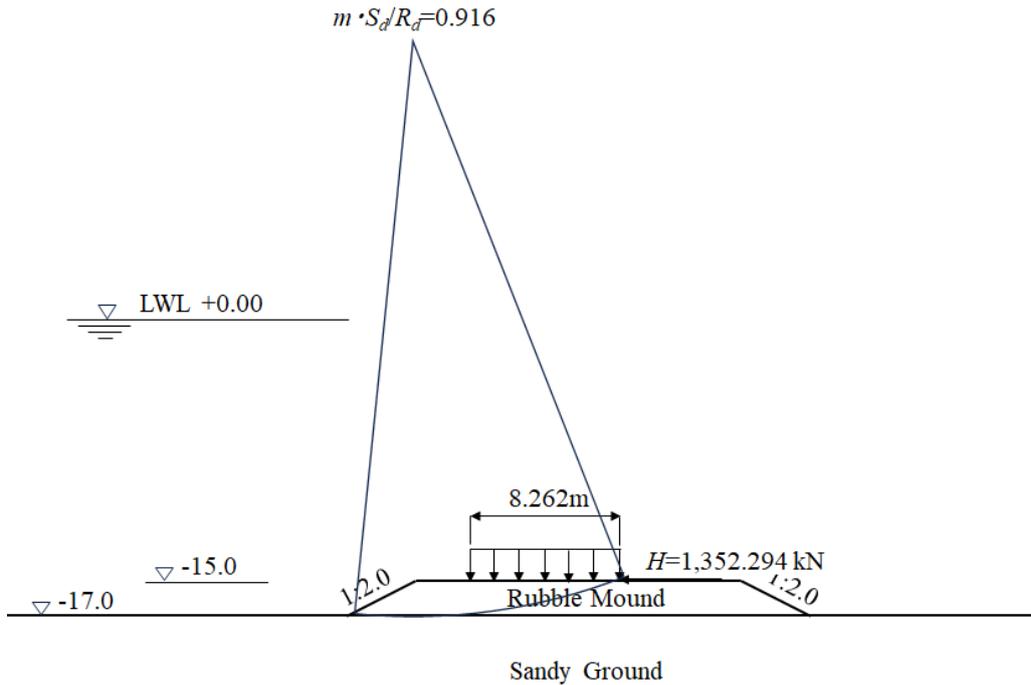
$$q = \frac{\Sigma V}{2b'} = \frac{2,880.947}{8.262} = 348.698 \text{ (kN/m}^2\text{)}$$

Horizontal force

$$H = 1,352.294 \text{ (kN/m)}$$

3) Verification Results

In verification of bearing capacity of the foundation ground, the adjustment factor m is considered. Table 2.18 shows the load conditions and verification results. Figure 2.9 shows the verification results for the case of variable situation associated with Level 1 earthquake ground motion not considering surcharge, which is the most dangerous condition.



**Figure 2.9- Verification of Bearing Capacity by Bishop's Method
(Variable Situation: Level 1 Earthquake Ground Motion without surcharge)**

**Table 2.18- Load Conditions and Verification Results for Verification
of Bearing Capacity of Foundation Ground**

State/Situation		Converted uniformly- distributed load q (kN/m ²)	Load width $2b'$ (m)	Horizontal load H (kN)	Resistance term R_d (kN·m)	Load term S_d (kN·m)	Adjustme nt factor m	Load- resistance ratio $m \cdot S_d / R_d$
Permanent state	Without surcharge	236.284	12.046	628.303	95,926.0	55,995.0	1.20	0.584
	With surcharge	259.482	12.368	628.303	104,949.2	59,859.1	1.20	0.570
Variable situation: Level 1 earthquake ground motion	Without surcharge	348.698	8.262	1,352.294	70,550.5	64,599.5	1.00	0.916
	With surcharge	363.022	8.436	1,370.444	85,132.6	75,610.6	1.00	0.888

(3) Verification of Circular Slip Failure of Foundation Ground

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

Table 2.19- Characteristic Values of Ground Conditions for Verification of Circular Slip Failure of Foundation Ground

	Saturated weight γ (kN/m ³)	Wet weight γ_t (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	20.00	18.00	10.00	40.00	0.00	0.00
Sandy soil	20.00	18.00	10.00	35.00	0.00	0.00
Wall body	20.93	20.93	10.93	40.00	0.00	0.00
Superstructure	22.60	22.60	-	40.00	0.00	0.00
Backfilling stones	20.00	18.00	10.00	40.00	0.00	0.00
Backfilling soil	20.00	18.00	10.00	40.00	0.00	0.00

Table 2.20- Verification of Circular Slip Failure of Foundation Ground

State	Resistance term	Load term	Adjustment factor	Load-resistance ratio	
	R_d (kN.m)	S_d (kN.m)	m	$m \cdot S_d / R_d$	
Permanent state	Without surcharge	79,494.6	72,417.7	1.0	0.911
	With surcharge	84,886.6	80,552.5	1.0	0.949

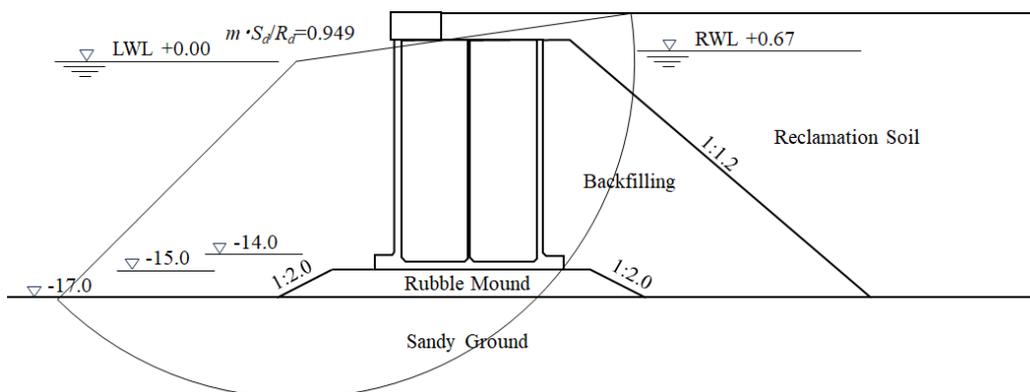


Figure 2.10- Verification of Circular Slip Failure Stability of Foundation Ground (Permanent State, with Surcharge)

(4) Verification of Stability under Floating Condition

Since the stability against buoyancy is not secured with the shape of this caisson, sand ballast is applied to the verification.

1) Ballast Weight and Moment

Ballast thickness $t = 1.51$ (m)

Unit weight of infill sand during floating condition $\gamma_t = 18.0$ kN/m³

Table 2.21- Characteristic Values of Ballast Weight and Moment

Name	Dimensions (m)	Nos.	V (m ³)	x (m)	$W \cdot x$ (kN·m)
			W (kN)	y (m)	$W \cdot y$ (kN·m)
Infill sand	4.750 × 3.600 × 1.510	4	103.284	6.750	12,549.006
			1,859.112	1.355	2,519.097
	4.750 × 3.700 × 1.510	4	106.153	6.750	12,897.590

			1,910.754	1.355	2,589.072
Vertical haunch	0.200 × 0.200 × 1.510 ×1/2	32	-0.966 -17.388	6.750 1.355	-117.369 -23.561
Horizontal haunch	4.350 × 0.200 × 0.200 ×1/2	16	-1.392 -25.056	6.750 0.667	-169.128 -16.712
Horizontal and vertical haunch	0.200 × 3.200 × 0.200 ×1/2	8	-0.512 -9.216	6.750 0.667	-62.208 -6.147
	0.200 × 3.300 × 0.200 ×1/2	8	-0.528 -9.504	6.750 0.667	-64.152 -6.339
Corner haunch	0.200 × 0.200 × 0.200 ×1/3	32	-0.085 -1.530	6.750 0.675	-10.328 -1.033
Total			205.954	6.750	25,023.411
			3,707.172	1.363	5,054.377

2) Total Weight, Center of Gravity and Moment

Table 2.22- Characteristic Values of Total Weight and Moment

	W (kN)	x (m)	y (m)	$W \cdot x$ (kN·m)	$W \cdot y$ (kN·m)
Caisson	15,072.072	6.750	6.571	101,736.486	99,043.096
Ballast	3,707.172	6.750	1.363	25,023.411	5,054.377
Total	18,779.244	6.750	5.543	126,759.897	104,097.473

3) Draft Calculation

$$d = \frac{W - V_f \times \gamma_w}{B \times L \times \gamma_w} = \frac{18,779.244 - 48.640 \times 10.1}{10.5 \times 16 \times 10.1} = 10.778 \text{ (m)}$$

Where:

- d : draft depth (m)
- W : total weight of caisson and ballast (kN)
- V_f : displacement volume of footing (m^3)
- γ_w : unit weight of seawater (kN/m^3)
- B : width of caisson body (m)
- L : length of caisson (m)

The freeboard height (f) above water level:

$$f = H - d = 16.500 - 10.778 = 5.722 \geq 1.00 \text{ (m)} \quad \text{O.K}$$

4) Stability Verification under Floating Condition

Table 2.23- Characteristic Values of Buoyancy and Moment

Name	Dimensions (m)	Nos.	V (m^3)	x (m)	$W \cdot x$ (kN·m)
			W (kN)	y (m)	$W \cdot y$ (kN·m)
Caisson	10.500 × 16.000 × 10.778	1	1,810.704	6.750	123,444.743
			18,288.110	5.389	98,554.625
Footing	1.500 × 16.000 × 1.000	2	48.000	6.750	3,272.400
			484.800	0.500	242.400
Footing haunch	0.200 × 16.000 × 0.200 ×1/2	2	0.640	6.750	43.632
			6.464	1.067	6.897
Total			1,859.344	6.750	126,760.775
			18,779.374	5.261	98,803.922

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

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
 V', I', C', G' : corresponding values or positions during the application of the counter ballast

$$I = L \times B^3 / 12 = 16.000 \times 10.500^3 / 12 = 1,543.500 \text{ (m}^4\text{)}$$

$$\overline{GM} = I/V - \overline{CG} = 1,543.500 / 1,859.344 - (5.543 - 5.261) = 0.548 \text{ (m)} \geq 0$$

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

$$\overline{GM} = 0.548 \text{ (m)} \geq 0.05d = 0.539 \text{ (m)} \quad \text{O.K.}$$

2-6. Summary of Load-Resistance Ratio

Table 2.24 shows the load-resistance ratios obtained in a stability verification considering partial factors based on the study presented above.

Table 2.24- Summary of Load-Resistance Ratio

*Shaded cells show the critical load-resistance ratio of each verification.

State/Situation	Unit	Resistance term	Load term	Adjustment factor	Load-resistance ratio	
Permanent state (without surcharge)	Sliding of wall body	kN/m	1,485.750	666.001	1.0	0.45 ≤ 1.0
	Overturning of wall body	kN·m/m	21,571.775	5,715.646	1.0	0.27 ≤ 1.0
	Bearing capacity failure of foundation ground	kN·m	95,926.000	55,995.000	1.2	0.58 ≤ 1.0
	Circular slip failure of foundation ground	kN·m	74,494.600	72,417.700	1.0	0.91 ≤ 1.0
Permanent state (with surcharge)	Sliding of wall body	kN/m	1,675.236	666.001	1.0	0.40 ≤ 1.0
	Overturning of wall body	kN·m/m	24,249.082	5,715.646	1.0	0.24 ≤ 1.0
	Bearing capacity failure of foundation ground	kN·m	104,949.200	59,859.100	1.2	0.39 ≤ 1.0
	Circular slip failure of foundation ground	kN·m	84,886.600	80,552.500	1.0	0.95 ≤ 1.0
Variable situation: Level 1 earthquake ground motion (without surcharge)	Sliding of wall body	kN/m	1,728.568	1,352.294	1.0	0.78 ≤ 1.0
	Overturning of wall body	kN·m/m	22,257.879	10,356.225	1.0	0.51 ≤ 1.0
	Bearing capacity failure of foundation ground	kN·m	70,550.500	64,599.500	1.0	0.92 ≤ 1.0
Various situation: Level 1 earthquake ground motion (with surcharge)	Sliding of wall body	kN/m	1,837.468	1,370.444	1.0	0.75 ≤ 1.0
	Overturning of wall body	kN·m/m	23,610.054	10,692.000	1.0	0.50 ≤ 1.0
	Bearing capacity failure of foundation ground	kN·m	85,132.600	75,610.600	1.0	0.89 ≤ 1.0

2-7. Safety Verification of Uneven Ground Support

(1) Bending Moment Calculation

- Cantilever length: $l/3 = 16.0/3 = 5.33$ (m)
- Caisson weight: 3,209.27 (kN/m)
- Bending moment: $1/2 \times 3,209.27 \times 5.33^2 = 45,585.92$ (kN·m)

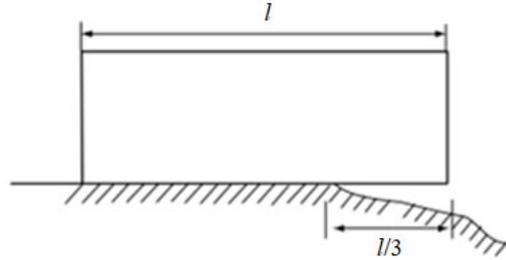


Figure 2.11- Examination of Actions due to Uneven Ground Support

(2) Section Modulus

1) Neutral axis of cross section

The calculated distance y from the bottom of the caisson to the center of gravity is presented in Table 2.25.

Table 2.25- Neutral Axis of Cross Section

Name	Area A (m ²)	Distance y (m)	Ay (m ³)
Bottom slab	$10.50 \times 0.60 = 6.30$	$1/2 \times 0.60 = 0.30$	1.89
Footing	$1.50 \times 1.00 \times 2 = 3.00$	$1/2 \times 1.00 = 0.50$	1.50
Footing haunch	$1/2 \times 0.20 \times 0.20 \times 2 = 0.04$	$1.00 + 1/3 \times 0.20 = 1.07$	0.04
Haunch	$1/2 \times 0.20^2 \times 4 = 0.08$	$0.60 + 1/3 \times 0.20 = 0.67$	0.05
Side wall	$0.40 \times 15.90 \times 2 = 12.72$	$0.60 + 1/2 \times 15.90 = 8.55$	108.76
Partition wall	$0.20 \times 15.90 \times 1 = 3.18$	$0.60 + 1/2 \times 15.90 = 8.55$	27.19
Total	25.32		139.43

$$y_c = Ay/A = 139.43/25.32 = 5.51 \text{ (m)}, y_l = 16.50 - 5.51 = 10.99 \text{ (m)}$$

2) Section Modulus around Neutral Axis

The section modulus around neutral axis is presented in Table 2.26.

Table 2.26- Section Modulus around Neutral Axis

Name	I_o (m ⁴)	A (m ²)	$yo = y_c - y$ (m)	Ayo^2 (m ⁴)	$I = I_o + Ayo^2$ (m ⁴)
Bottom slab	$1/12 \times 10.50 \times 0.60^3 = 0.19$	6.30	$5.51 - 0.30 = 5.21$	171.01	171.20
Footing	$1/12 \times 1.50 \times 1.00^3 \times 2 = 0.25$	3.00	$5.51 - 0.50 = 5.01$	75.30	75.55
Footing haunch	$1/36 \times 0.20 \times 0.20^3 \times 2 = 0.00$	0.04	$5.51 - 1.07 = 4.44$	0.79	0.79
Haunch	$1/36 \times 0.20^4 \times 4 = 0.00$	0.08	$5.51 - 0.67 = 4.84$	1.87	1.87
Side wall	$1/12 \times 0.40 \times 15.90^3 \times 2 = 267.98$	12.72	$5.51 - 8.55 = -3.04$	117.55	385.53
Partition wall	$1/12 \times 0.20 \times 15.90^3 \times 1 = 66.99$	3.18	$5.51 - 8.55 = -3.04$	29.39	96.38
Total		25.32			731.32

i) Compression

$$y = y_c = 5.51 \text{ (m)}$$

$$Z_c = 731.32 / 5.51 = 132.73 \text{ (m}^3\text{)}$$

ii) Tension

$$y = y_t = 10.99 \text{ (m)}$$

$$Z_t = 731.32 / 10.99 = 66.54 \text{ (m}^3\text{)}$$

3) Safety Verification

The section modulus around neutral axis is presented in Table 2.26.

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

Where:

- γ_i : structural coefficient (1.0)
- M_d : design value of bending moment (kN·m)
- M_{ud} : design moment capacity (kN·m)
 - M_{udc} : design bending moment capacity = $f_{bk} \cdot Z / \gamma_b$
 - M_{udt} : design tensile moment capacity = $f_{tk} \cdot Z / \gamma_b$
- Z : section modulus of caisson (m³)
- γ_b : member coefficient (1.1)
- f_{bk} : design bending strength of the concrete (N/mm²), calculated as $f_{bk} = 0.42 f'_{ck}{}^{2/3} / \gamma_c$
- f'_{ck} : characteristic value of compressive strength of concrete (N/mm²)
- γ_c : material coefficient (1.3)
- f_{tk} : design tensile strength of the concrete (N/mm²), calculated as $f_{tk} = 0.23 f'_{ck}{}^{2/3} / \gamma_c$

i) Compression

$$f_{bk} = 0.42 \times f'_{ck}{}^{2/3} / \gamma_c$$

$$= 0.42 \times 30.00^{2/3} / 1.30 = 3.12$$

$$M_{udc} = f_{bk} \times Z_c / \gamma_c$$

$$= 3.12 \times 132.73 / 1.10 = 376.38 \times 10^3 \text{ (kN·m)}$$

$$\gamma_i \cdot M_d / M_{udc} = 1.00 \times 455,585.92 / (376.38 \times 10^3) = 0.12 \leq 1.0 \quad \text{O.K}$$

ii) Tension

$$f_{tk} = 0.23 \times f'_{ck}{}^{2/3} / \gamma_c$$

$$= 0.23 \times 30.00^{2/3} / 1.30 = 1.71$$

$$M_{udt} = f_{tk} \times Z_t / \gamma_c$$

$$= 1.71 \times 66.54 / 1.10 = 103.33 \times 10^3 \text{ (kN·m)}$$

$$\gamma_i \cdot M_d / M_{udt} = 1.00 \times 455,585.92 / (103.33 \times 10^3) = 0.44 \leq 1.0 \quad \text{O.K}$$

2-8. Verification of Structural Members

(1) Design Conditions

1) Material Conditions

Since the materials that can be procured vary depending on the construction conditions, this casebook applies to the materials specified in OCDI 2020. Therefore, consultants shall verify the applicable materials.

i) Concrete

- Design characteristic strength $f_{ck} = 30 \text{ N/mm}^2$
- Design compressive strength $f_{cd} = f_{ck}/\gamma_c = 30/1.3 = 23.1 \text{ N/mm}^2$
- Young's modulus $E_c = 28 \text{ kN/mm}^2$

ii) Reinforcement (SD345)

- Design tensile yield strength $f_{yd} = 345 \text{ N/mm}^2$
- Young's modulus $E_s = 200 \text{ kN/mm}^2$

2) Load Factor and Combination

i) Bottom slab

Table 2.27- Combinations of Actions on the Bottom Slab and Load Factors

Design State/Situation	Safety (against cross-sectional failure)	Serviceability
Permanent state	$0.9D+1.1D_o+1.1F+0.8W$	$1.0D+1.0D_o+1.0F+0.5W$
Variable situation associated with Level 1 earthquake ground motion	$1.0D+1.0F+1.0R'+1.0W'$	Not examined
Variable situation associated with water pressure while afloat during construction	$0.9D_f+1.1S_f$	$0.5D_f+0.5S_f$

Where:

- D : self-weight (kN/m²)
- D_o : bottom reaction force in permanent state (kN/m²)
- F : water pressure (kN/m²)
- W : surcharge load in permanent state (kN/m²)
- R' : bottom reaction force in variable situation (kN/m²)
- W' : surcharge load in variable situation (kN/m²)
- D_f : self-weight of slab (kN/m²)
- S_f : hydrostatic pressure while afloat (kN/m²)

ii) Outer wall

Table 2.28- Combinations of Actions on the Outer Wall and Load Factors

Design State/Situation	Direction of action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with water pressure while afloat during construction	From outside of caisson	$1.1 S_f$	$0.5S_f$
Permanent state associated with internal earth pressure	From inside of caisson	$1.1D+1.1S$	$1.0D+1.0S$
Variable situation associated with Level 1 earthquake ground motion		$1.0D+1.0S+1.0P_{dw}$	Not examined

Where:

- D : internal earth pressure (kN/m²)
- S : internal water pressure (kN/m²)
- S_f : hydrostatic pressure while afloat (kN/m²)
- P_{dw} : Dynamic water pressure (kN/m²)

iii) Partition wall

Table 2.29- Combinations of Actions and Load Factors of the Partition Wall

Design State/Situation	Direction of Action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with water pressure during installation during construction	Direction of action due to hydrostatic head difference between chambers	$1.1S_f$	$0.5S_f$
Permanent state associated with internal earth pressure	Direction of extrusion of outer wall from partition wall	Maximum outward design load that acts on outer wall	Not examined
Permanent state associated with self-weight Variable situation associated with waves Variable situation associated with Level 1 earthquake ground motion	Direction of extrusion of bottom slab from partition wall	Maximum downward design load that acts on bottom slab	Not examined

Where:

S_f : hydrostatic head difference between chambers (kN/m²)

3) Limit Values of Crack Width

Table 2.30- Limit Values of Crack Width

Location of reinforcement bar	Crack width limit value (mm)	Minimum cover (mm)
Lower part of bottom slab, Outer face of outer wall, Footing	$0.0035c$	70
Upper part of bottom slab, Inner face of outer wall, Partition wall	$0.0040c$	50

Where:

c : concrete cover (mm)

4) Target Members for Structural Verification

In this casebook, the verification of structural members will be conducted on the critical members among the bottom slab, outer wall, partition wall, and footing. The target members for structural verification are presented in Figure 2.11. In the structural verification, the reinforcement for the load acting from the inside of the outer wall is determined based on serviceability, and therefore the safety (cross section failure) verification is omitted.

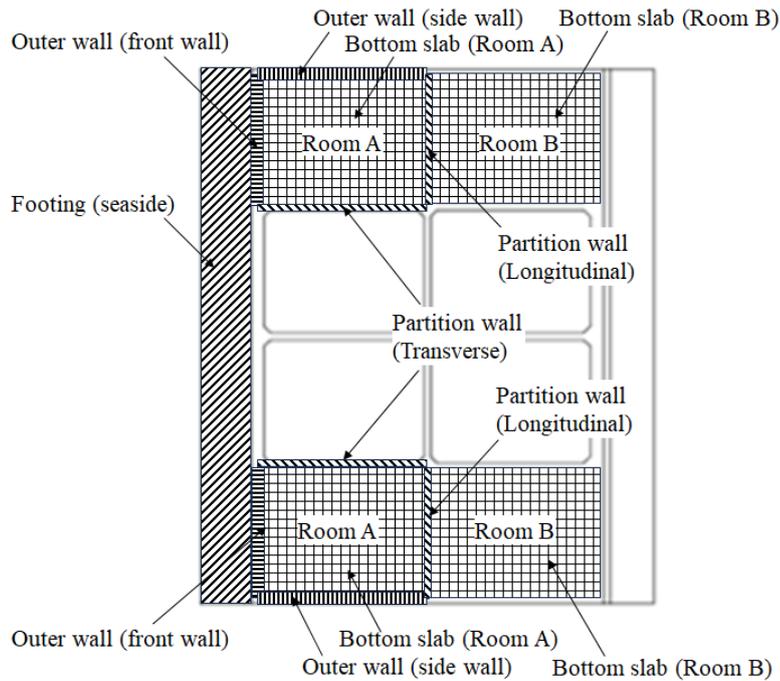


Figure 2.12- Target Members for Structural Verification

(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 and ballast.

$$S_f = (10.778 + 1.000 - 0.000) \times 10.10 = 118.96 \text{ (kN/m}^2\text{)}$$

$$D_f = (-0.600 \times 24.00) + (-1.510 \times 18.00) = -41.58 \text{ (kN/m}^2\text{)}$$

- Safety (against cross-sectional failure)

$$P = 1.1 \times S_f + 0.9 \times D_f = 93.43 \text{ (kN/m}^2\text{)}$$

- Serviceability

$$P = 0.5 \times S_f + 0.5 \times D_f = 38.69 \text{ (kN/m}^2\text{)}$$

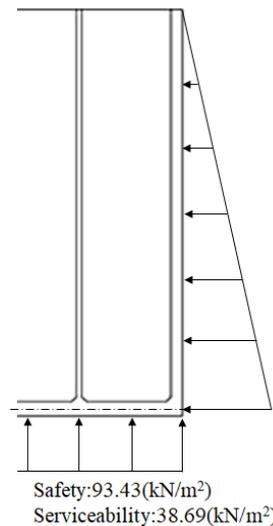


Figure 2.13- Design Load at Floating Condition

ii) After completion

- ✓ Load calculation under permanent state
- Self-weight of each chamber: D

$$D = \text{Cover concrete weight} + \text{Self-weight of Infill sand} + \text{Self-weight of Bottom slab}$$

$$= 0.300 \times 22.60 + 15.600 \times 20.00 + 0.600 \times 24.00 = 333.18 \text{ (kN/m}^2\text{)}$$

- Self-weight of footing: D

$$D = 1.000 \times (22.60 - 10.10) = 13.90 \text{ (kN/m}^2\text{)}$$

- Bottom reaction force: Do

$$\text{Seaside: } Do = 278.96 \text{ (kN/m}^2\text{)}, \text{ Landside: } Do = 142.71 \text{ (kN/m}^2\text{)}, \text{ Width: } B = 13.5 \text{ (m)}$$

- Water pressure: F

$$F = (\text{R.W.L} - \text{Installed depth}) \times \gamma_w$$

$$= (0.670 - (-15.000)) \times 10.10 = 158.27 \text{ (kN/m}^2\text{)}$$

- Surcharge load: W

$$W = 30.00 \text{ (kN/m}^2\text{)}$$

- ✓ Design load under permanent state

The combined load in the permanent state based on Table 2.25 is distributed as shown in Figure 2.13. The design load in the permanent state taking into account the load conditions during floating is presented in Figure 2.14. The unit of length is (m) and unit of load is (kN/m²) in Figure 2.13 and 2.14.

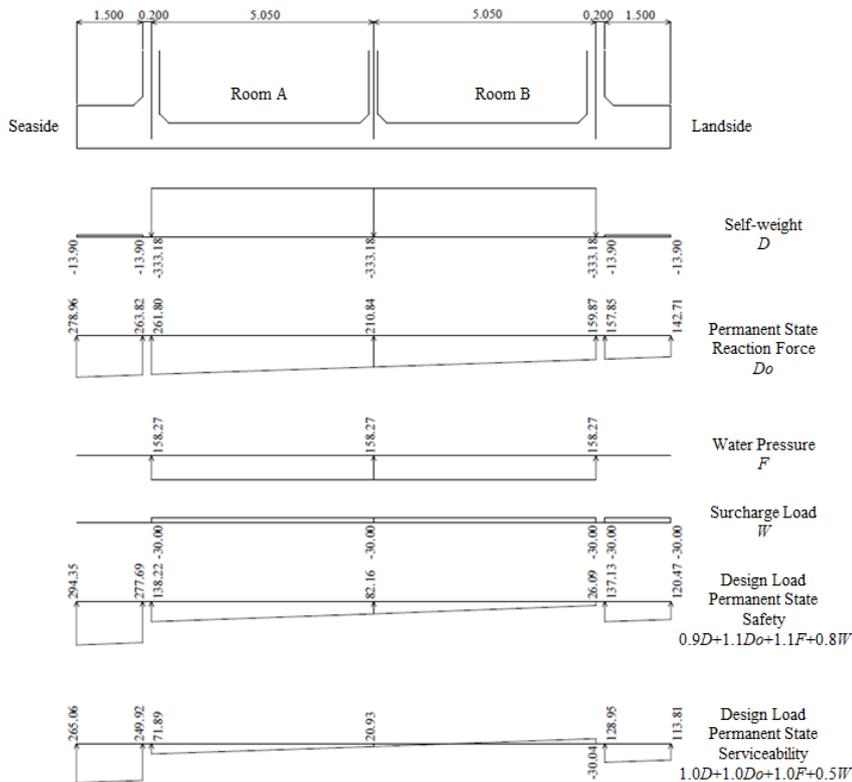


Figure 2.14- Combined Load in Permanent States

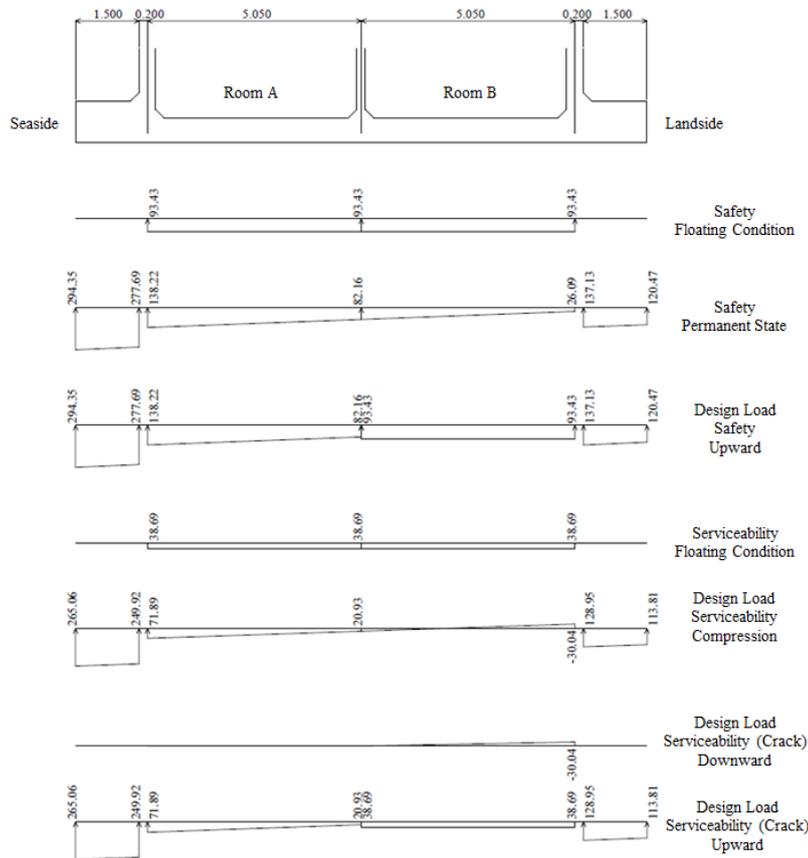


Figure 2.15- Design Load in Permanent States and Floating Condition

✓ Load calculation under Level 1 seismic ground motion

- Self-weight of each chamber: D

$$D = \text{Cover concrete weight} + \text{Self-weight of Infill sand} + \text{Self-weight of Bottom slab} \\ = 0.300 \times 22.60 + 15.600 \times 20.00 + 0.600 \times 24.00 = 333.18 \text{ (kN/m}^2\text{)}$$

- Self-weight of footing: D

$$D = 1.000 \times (22.60 - 10.10) = 13.90 \text{ (kN/m}^2\text{)}$$

- Bottom reaction force: D_o

$$\text{Seaside: } R' = 464.93 \text{ (kN/m}^2\text{)}, \text{ Landside: } R' = 0.00 \text{ (kN/m}^2\text{)}, \text{ Width: } B = 12.393 \text{ (m)}$$

- Water pressure: F

$$F = (\text{R.W.L} - \text{Installed depth}) \times \gamma_w \\ = (0.670 - (-15.000)) \times 10.10 = 158.27 \text{ (kN/m}^2\text{)}$$

- Surcharge load: W

$$W = 15.00 \text{ (kN/m}^2\text{)}$$

The following presents the conversion of irregular load distributions under Level 1 seismic ground motion into equivalent uniform and triangular load patterns.

- Calculation of $\sum A$

$$1/2 \times (-14.160 - 28.900) \times 0.393 = -8.46 \text{ (kN/ m}^2\text{)}$$

$$1/2 \times (-28.900 - 28.900) \times 1.017 = -31.99 \text{ (kN/ m}^2\text{)}$$

$$\sum A = -40.45 \text{ (kN/m}^2\text{)}$$

- Equivalent load

$$P_1 = -28.90 \text{ (kN/m}^2\text{)}$$

$$P_2 = (2 \times \sum A / L) - P_1$$

$$= 2 \times (-40.45) / 1.500 - (-28.90) = -25.03 \text{ (kN/m}^2\text{)}$$

- ✓ Design load under Level 1 earthquake ground motion

The combined load in the Level 1 earthquake ground motion based on Table 2.25 is distributed as shown in Figure 2.15.

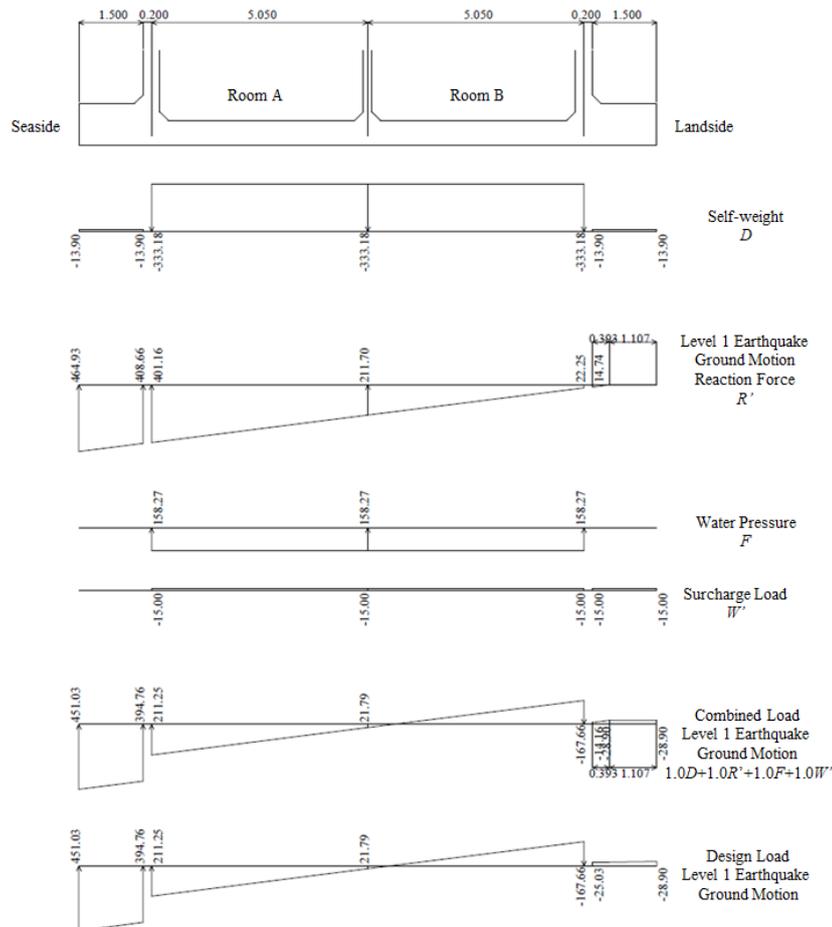


Figure 2.16- Design Load in Level 1 Earthquake Ground Motion

2) Outer Wall

- i) Floating condition (front wall, rear wall and side walls)

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

$$S_f = (10.778 + 1.000 - 0.600 / 2) \times 10.10 = 115.93 \text{ (kN/m}^2\text{)}$$

- Safety (against cross-sectional failure)

$$P = 1.1 \times S_f = 127.52 \text{ (kN/m}^2\text{)}$$

- Serviceability

$$P = 0.5 \times S_f = 57.97 \text{ (kN/m}^2\text{)}$$

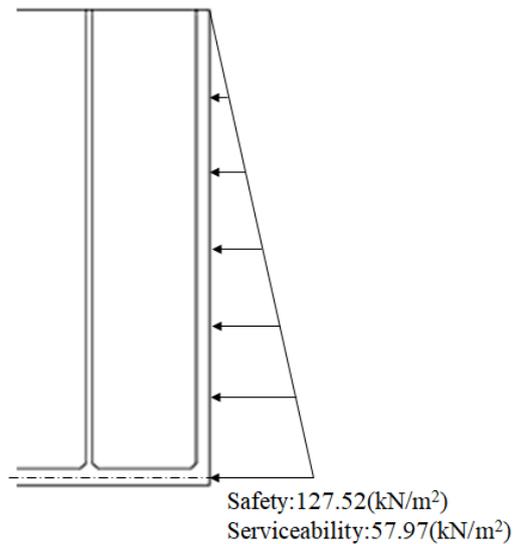


Figure 2.17- Design Load at Floating Condition

ii) After completion (Side wall)

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/m³). The internal water pressure will take into account the water level difference from the bottom of the cover concrete to the LWL.

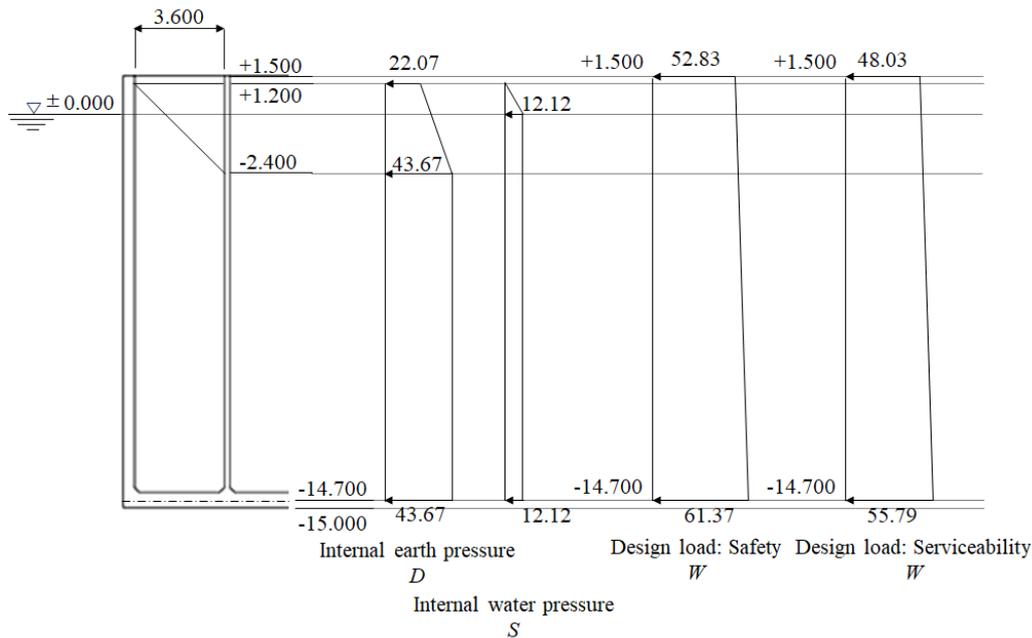


Figure 2.18- Design Load after Completion (Side wall)

- ✓ Load calculation under permanent state
- Internal earth pressure: *D*

$$P_1 = (\text{Surcharge Load} + \text{Equipment Load} + \text{Cover concrete weight}) \times K$$

$$P_2 = P_1 + \text{Inside dimension} \times \text{Unit weight of infill sand} \times K$$

K : earth pressure coefficient at rest of the filling sand

$$P_1 = (30.00 + 0.00 + 0.300 \times 22.60) \times 0.60 = 22.07 \text{ (kN/m}^2\text{)}$$

$$P_2 = 22.07 + (3.600 \times 10.00) \times 0.60 = 43.67 \text{ (kN/m}^2\text{)}$$

$$D = 1/2 \times (22.07 + 43.67) \times 3.600 + 43.67 \times 12.300 = 655.47 \text{ (kN/m)}$$

- Internal water pressure: S

$$P = 1.200 \times 10.10 = 12.12 \text{ (kN/m}^2\text{)}$$

$$S = 1/2 \times 12.12 \times 1.200 + 12.12 \times 14.700 = 185.44 \text{ (kN/m)}$$

✓ 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)

$$\sum P = 1.1 \times D + 1.1 \times S = 925.00 \text{ (kN/m)}$$

Trapezoidal load (Bottom)

$$P_1 = 43.67 \times 1.1 + 12.12 \times 1.1 = 61.37 \text{ (kN/m}^2\text{)}$$

Trapezoidal load (Top)

$$P_2 = (2 \times \sum P / L) - P_1$$

$$= 2 \times 925.00 / 16.200 - 61.37 = 52.83 \text{ (kN/m}^2\text{)}$$

- Serviceability

$$\sum P = 1.0 \times D + 1.0 \times S = 840.91 \text{ (kN/m)}$$

Trapezoidal load (Bottom)

$$P_1 = 43.67 \times 1.0 + 12.12 \times 1.0 = 55.79 \text{ (kN/m}^2\text{)}$$

Trapezoidal load (Top)

$$P_2 = (2 \times \sum P / L) - P_1$$

$$= 2 \times 840.91 / 16.200 - 55.79 = 48.03 \text{ (kN/m}^2\text{)}$$

iii) Outer wall (Front wall and Rear wall)

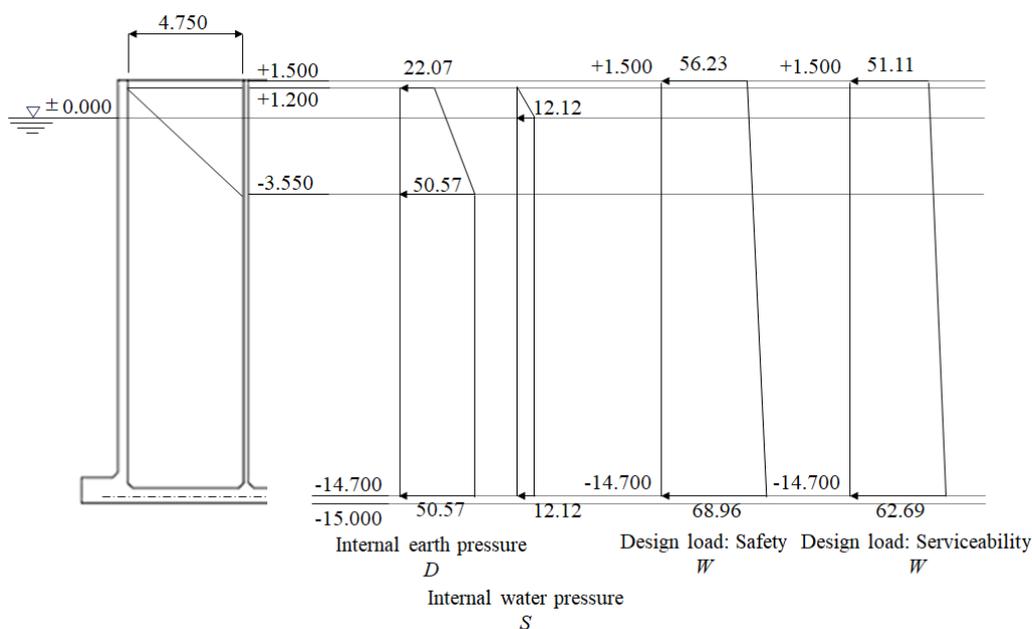


Figure 2.19- Design Load after Completion (Front and Rear wall)

✓ Load calculation under permanent state

- Internal earth pressure: D

$$P_1 = (\text{Surcharge Load} + \text{Equipment Load} + \text{Cover concrete weight}) \times K$$

$$P_2 = P_1 + \text{Inside dimension} \times \text{Unit weight of infill sand} \times K$$

K : earth pressure coefficient at rest of the filling sand

$$P_1 = (30.00 + 0.00 + 0.300 \times 22.60) \times 0.60 = 22.07 \text{ (kN/m}^2\text{)}$$

$$P_2 = 22.07 + (4.750 \times 10.00) \times 0.60 = 50.57 \text{ (kN/m}^2\text{)}$$

$$D = 1/2 \times (22.07 + 50.57) \times 4.750 + 50.57 \times 11.150 = 736.38 \text{ (kN/m)}$$

- Internal water pressure: S

$$P = 1.200 \times 10.10 = 12.12 \text{ (kN/m}^2\text{)}$$

$$S = 1/2 \times 12.12 \times 1.200 + 12.12 \times 14.700 = 185.44 \text{ (kN/m)}$$

✓ 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)

$$\Sigma P = 1.1 \times D + 1.1 \times S = 1,014.00 \text{ (kN/m)}$$

Trapezoidal load (Bottom)

$$P_1 = 50.57 \times 1.1 + 12.12 \times 1.1 = 68.96 \text{ (kN/m}^2\text{)}$$

Trapezoidal load (Top)

$$P_2 = (2 \times \Sigma P / L) - P_1 \\ = 2 \times 1014.00 / 16.200 - 68.96 = 56.23 \text{ (kN/m}^2\text{)}$$

- Serviceability

$$\Sigma P = 1.0 \times D + 1.0 \times S = 921.82 \text{ (kN/m)}$$

Trapezoidal load (Bottom)

$$P_1 = 50.57 \times 1.0 + 12.12 \times 1.0 = 62.69 \text{ (kN/m}^2\text{)}$$

Trapezoidal load (Top)

$$P_2 = (2 \times \Sigma P / L) - P_1 \\ = 2 \times 921.82 / 16.200 - 62.69 = 51.11 \text{ (kN/m}^2\text{)}$$

3) Partition Wall

When pouring water into the chamber, maximum water level difference of 1.0 m is required between the chambers. Therefore, the design load of the partition wall includes a water level difference of 1.0 m.

$$S_f = 1.000 \times 10.10 = 10.10 \text{ (kN/m}^2\text{)}$$

- Safety (against cross-sectional failure)

$$Q = 1.1 \times S_f = 11.11 \text{ (kN/m}^2\text{)}$$

- Serviceability

$$Q = 0.5 \times S_f = 5.05 \text{ (kN/m}^2\text{)}$$

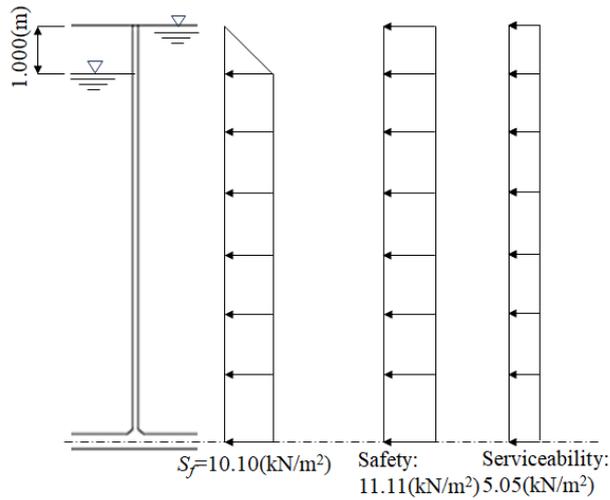


Figure 2.20- Design Load at Installation Condition

- ✓ Pull-out force of partition wall and side wall

For the pull-out of side walls, the maximum intensity of the combined load per unit length, as determined in the side wall design, shall be used. The analysis is conducted for side walls in the longitudinal direction, while the calculation of side wall in the perpendicular direction is omitted here.

$$L = (5.050 + 5.050) / 2 = 5.050 \text{ (m)}$$

$$T_d = 61.37 \times 5.050 = 309.92 \text{ (kN/m)}$$

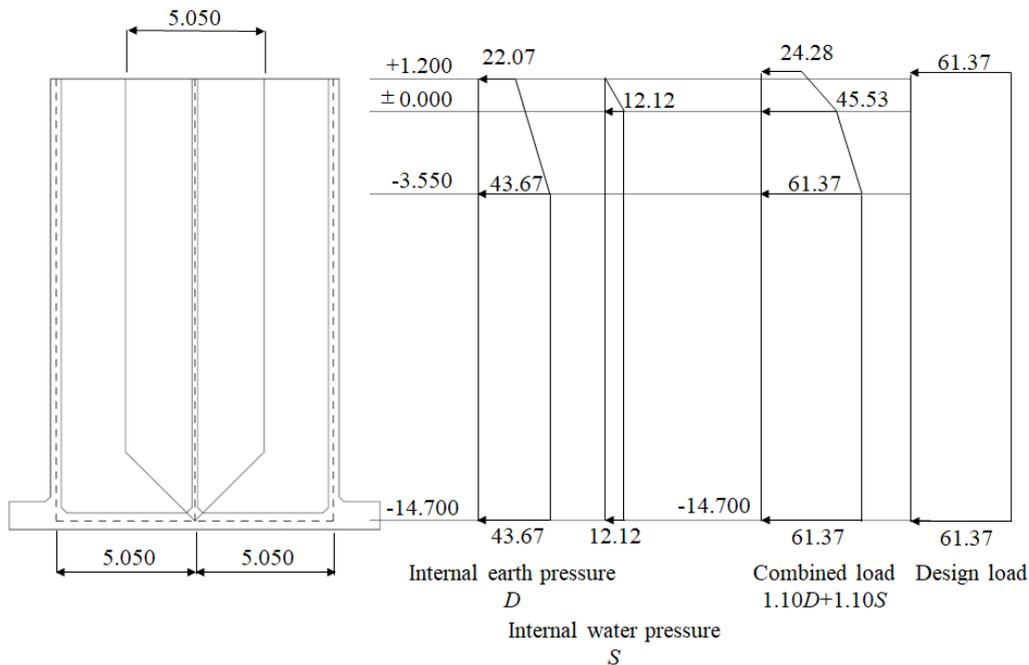


Figure 2.21- Design Load in Permanent State (Partition Wall and Side Wall)

Pull-out force of partition wall and side wall in the longitudinal direction

$$T_d = 268.94 \text{ (kN/m)}$$

- ✓ Pull-out force of partition wall and bottom slab

The maximum downward load acting on the bottom slab in Figure 2.18 is considered as the pull-out force.

$$W = (167.66 + 0.000) / 2 = 83.830 \text{ (kN/ m}^2\text{)}$$

$$L_X = 3.900 \text{ (m)}$$

$$L_Y = 5.050 \text{ (m)}$$

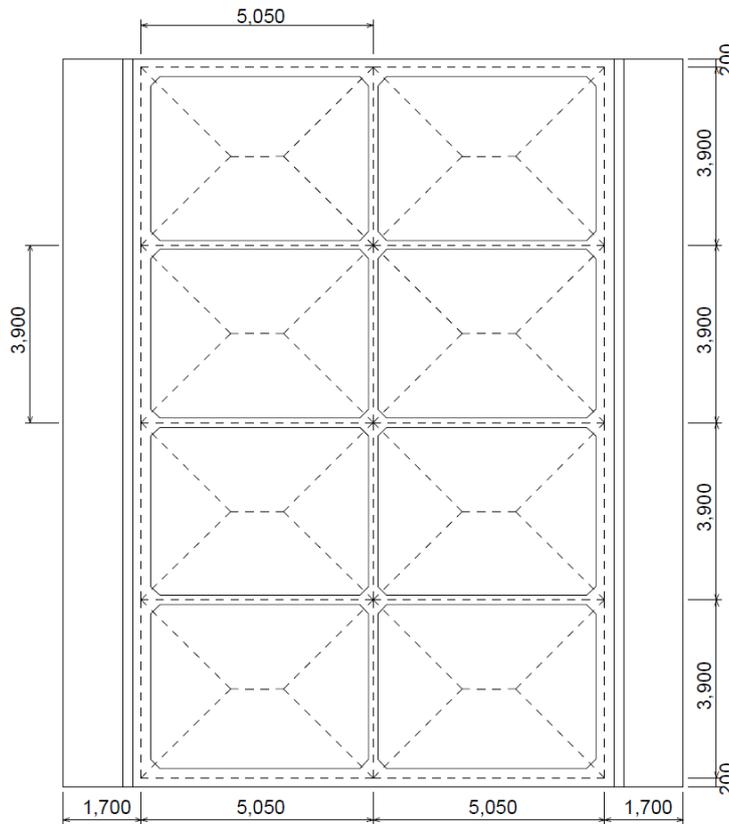


Figure 2.22- Design Load in Level 1 Earthquake Ground Motion (Partition Wall and Bottom Slab)

- Design load between partition wall in the perpendicular direction and bottom slab

$$P_1 = W \cdot L_X \cdot (1 - L_X^2 / (3 \times L_Y^2))$$

$$= 83.830 \times 3.900 \times (1 - 3.900^2 / (3 \times 5.050^2)) = 261.940 \text{ (kN/m)}$$

- Design load between partition wall in the longitudinal direction and bottom slab

$$P_2 = 2 / 3 \cdot W \cdot L_X$$

$$= 2 / 3 \times 83.830 \times 3.900 = 217.960 \text{ (kN/m)}$$

(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 2.22 and 2.23.

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 2.24.

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

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Part VI
Chapter 4
Table 2.1.1
Table 2.1.2
Table 2.1.3

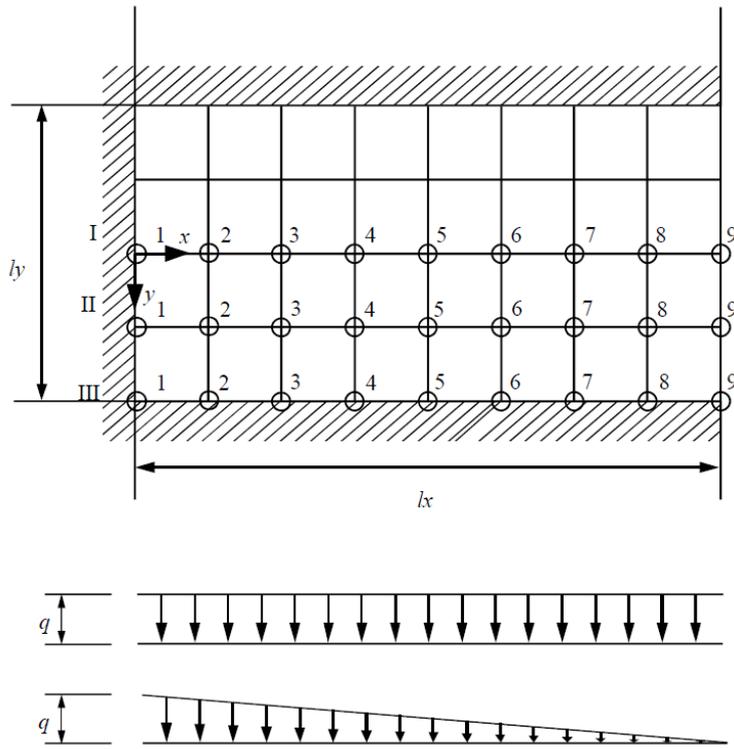


Figure 2.24- Slab Supported on Three Sides and Free on One Side (8 Equal Parts)

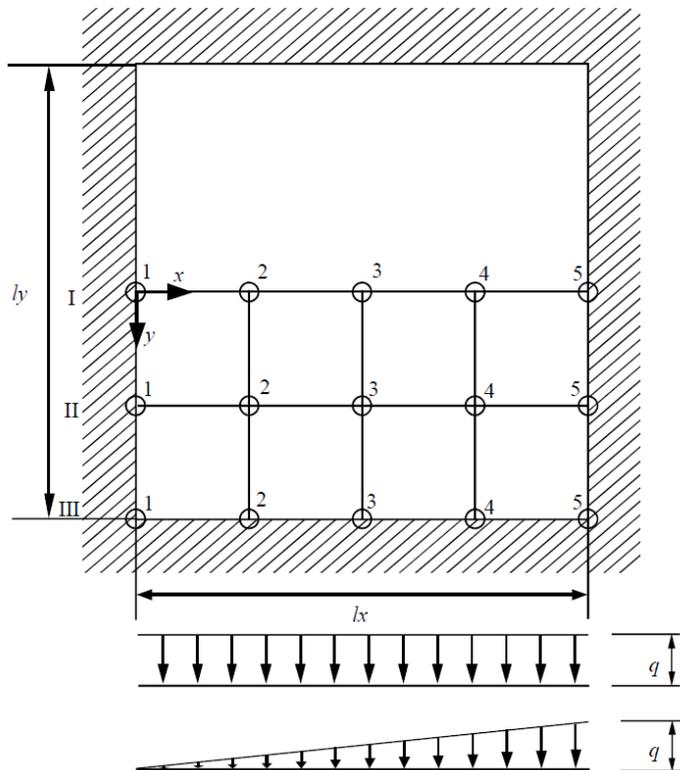


Figure 2.25- Slab Supported on Four Sides (4 Equal Parts)

2) Safety Verification of Bottom Slab

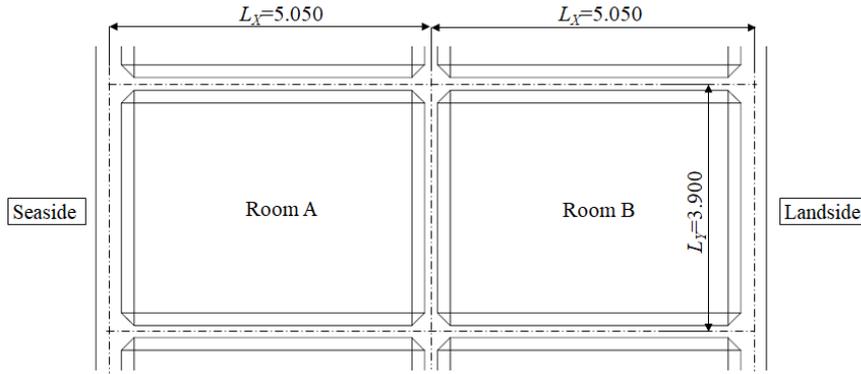


Figure 2.26- Design Span of Bottom Slab

The sectional force of bottom slab is estimated by the model of slabs fixed on four sides according to constraint conditions and making calculations based on numerical tables.

i) Room A: Permanent state (upward load)

$$P_1 = 82.16 \text{ (kN/m}^2\text{)}$$

$$P_2 = 138.22 \text{ (kN/m}^2\text{)}$$

$$L_X = 5.050 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 5.050/3.900 = 1.29$, The coefficient table for $\lambda = 1.25$ is applied.

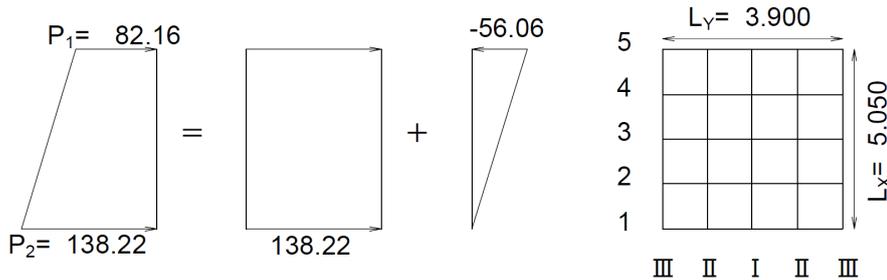


Figure 2.27- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = 138.22 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 138.22 \times 3.900^2 \times X = 2,102.33 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_X^2 \cdot Y = 138.22 \times 5.050^2 \times Y = 3,500.00 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.31- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0559	-117.52	-0.0093	-19.55
	4	0.0119	25.02	0.0181	38.05
	3	0.0189	39.73	0.0285	62.02
	2	0.0119	25.02	0.0181	38.05
	1	-0.0559	-117.52	-0.0093	-19.55
II	5	-0.0343	-72.11	-0.0057	-11.98
	4	0.0067	14.09	0.0074	15.56
	3	0.0097	20.39	0.0108	22.71
	2	0.0067	14.09	0.0074	15.56
	1	-0.0343	-72.11	-0.0057	-11.98

III	5	0.0000	0.00	0.0000	0.00
	4	-0.0074	-15.56	-0.0442	-92.92
	3	-0.0111	-23.34	-0.0664	-139.59
	2	-0.0074	-15.56	-0.0442	-92.92
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$$P = -56.06 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L \cdot Y^2 \cdot X = -56.06 \times 3.900^2 \times X = -852.67 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L^2 \cdot Y = -56.06 \times 3.900^2 \times Y = -852.67 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.32- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0389	33.17	-0.0065	5.54
	4	0.0102	-8.70	0.0111	-9.46
	3	0.0095	-8.10	0.0147	-12.53
	2	0.0017	-1.45	0.0069	-5.88
	1	-0.0171	14.58	-0.0029	2.47
II	5	-0.0251	21.40	-0.0042	3.58
	4	0.0061	-5.20	0.0053	-4.52
	3	0.0048	-4.09	0.0054	-4.60
	2	0.0006	-0.51	0.0021	-1.79
	1	-0.0092	7.84	-0.0015	1.28
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0049	4.18	-0.0291	24.81
	3	-0.0055	4.69	-0.0332	28.31
	2	-0.0025	2.13	-0.0151	12.88
	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 A, Permanent state (upward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	-84.35	-14.01
	4	16.32	28.59
	3	31.63	49.49
	2	23.57	32.17
	1	-102.94	-17.08
II	5	-50.71	-8.40
	4	8.89	11.04
	3	16.30	18.11
	2	13.58	13.77
	1	-64.27	-10.70
III	5	0.00	0.00
	4	-11.38	-68.11
	3	-18.65	-111.28
	2	-13.43	-80.04
	1	0.00	0.00

ii) Room B: Floating condition (upward load)

$$P_1 = 93.43 \text{ (kN/m}^2\text{)}$$

$$P_2 = 93.43 \text{ (kN/m}^2\text{)}$$

$$L_X = 5.050 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 5.050/3.900 = 1.29$, The coefficient table for $\lambda = 1.25$ is applied.

- Sectional forces induced by uniformly distributed load

$$P = 93.43 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 93.43 \times 3.900^2 \times X = 1,421.07 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_X^2 \cdot Y = 93.43 \times 5.050^2 \times Y = 1,421.07 \times Y \text{ (kN}\cdot\text{m)}$$

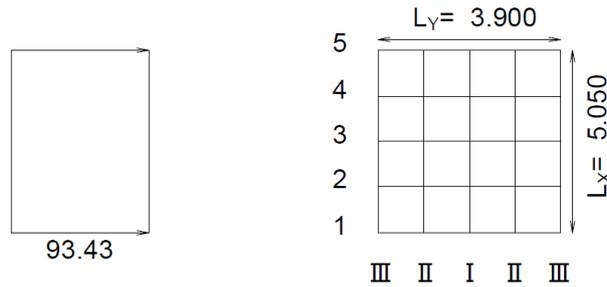


Figure 2.28- Design Model for Sectional Force Estimation

Table 2.34- Coefficient and Sectional Force (Room B, Floating condition (upward load))

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0559	-79.44	-0.0093	-13.22
	4	0.0119	16.91	0.0181	25.72
	3	0.0189	26.86	0.0285	41.92
	2	0.0119	16.91	0.0181	25.72
	1	-0.0559	-79.44	-0.0093	-13.22
II	5	-0.0343	-48.74	-0.0057	-8.10
	4	0.0067	9.52	0.0074	10.52
	3	0.0097	13.78	0.0108	15.35
	2	0.0067	9.52	0.0074	10.52
	1	-0.0343	-48.74	-0.0057	-8.10
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0074	-10.52	-0.0442	-62.81
	3	-0.0111	-15.77	-0.0664	-94.36
	2	-0.0074	-10.52	-0.0442	-62.81
	1	0.0000	0.00	0.0000	0.00

iii) Room A: Level 1 earthquake ground motion (upward load)

$$P_1 = 21.79 \text{ (kN/m}^2\text{)}$$

$$P_2 = 211.25 \text{ (kN/m}^2\text{)}$$

$$L_X = 5.050 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 5.050/3.900 = 1.29$, The coefficient table for $\lambda = 1.25$ is applied.

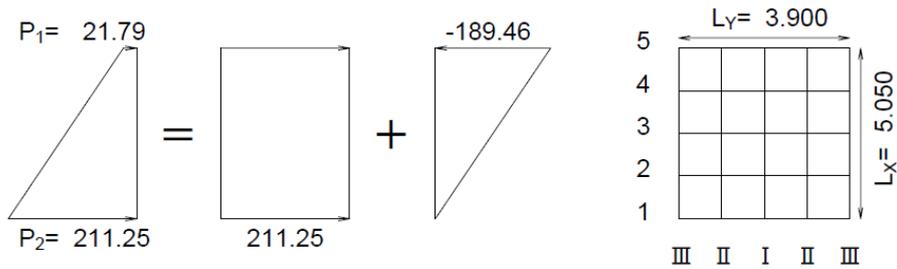


Figure 2.29- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = 211.25 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 211.25 \times 3.900^2 \times X = 3,213.11 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = 211.25 \times 3.900^2 \times Y = 3,213.11 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.35- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0559	-179.61	-0.0093	-29.88
	4	0.0119	38.24	0.0181	58.16
	3	0.0189	60.73	0.0285	94.79
	2	0.0119	38.24	0.0181	58.16
	1	-0.0559	-179.61	-0.0093	-29.88
II	5	-0.0343	-110.21	-0.0057	-18.31
	4	0.0067	21.53	0.0074	23.78
	3	0.0097	31.17	0.0108	34.70
	2	0.0067	21.53	0.0074	23.78
	1	-0.0343	-110.21	-0.0057	-18.31
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0074	-23.78	-0.0442	-142.02
	3	-0.0111	-35.67	-0.0664	-213.35
	2	-0.0074	-23.78	-0.0442	-142.02
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$$P = -189.46 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = -189.46 \times 3.900^2 \times X = -2,881.69 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = -189.46 \times 3.900^2 \times Y = -2,881.69 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.36- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0389	112.10	-0.0065	18.73
	4	0.0102	-29.39	0.0111	-31.99
	3	0.0095	-27.38	0.0147	-42.36
	2	0.0017	-4.90	0.0069	-19.88
	1	-0.0171	49.28	-0.0029	8.36
II	5	-0.0251	72.33	-0.0042	12.10
	4	0.0061	-17.58	0.0053	-15.27
	3	0.0048	-13.83	0.0054	-15.56

	2	0.0006	-1.73	0.0021	-6.05
	1	-0.0092	26.51	-0.0015	4.32
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0049	14.12	-0.0291	83.86
	3	-0.0055	15.85	-0.0332	95.67
	2	-0.0025	7.20	-0.0151	43.51
	1	0.0000	0.00	0.0000	0.00

- Combined uniformly distributed load and triangularly distributed load

Table 2.37- Summary of Sectional Force (Room A, Level 1 Earthquake Ground Motion (upward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	-67.51	-11.15
	4	8.85	26.17
	3	33.35	52.43
	2	-33.34	38.28
	1	-130.33	-21.52
II	5	-37.88	-6.21
	4	3.95	8.51
	3	17.34	19.14
	2	19.80	17.73
	1	-83.70	-13.99
III	5	0.00	0.00
	4	-9.66	-58.16
	3	-19.82	-117.68
	2	-16.58	-98.51
	1	0.00	0.00

iv) Room B: Level 1 earthquake ground motion (upward load)

$$P_1 = 0.00 \text{ (kN/m}^2\text{)}$$

$$P_2 = 21.79 \text{ (kN/m}^2\text{)}$$

$$L_X = 5.050 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 5.050/3.900 = 1.29$, The coefficient table for $\lambda = 1.25$ is applied.

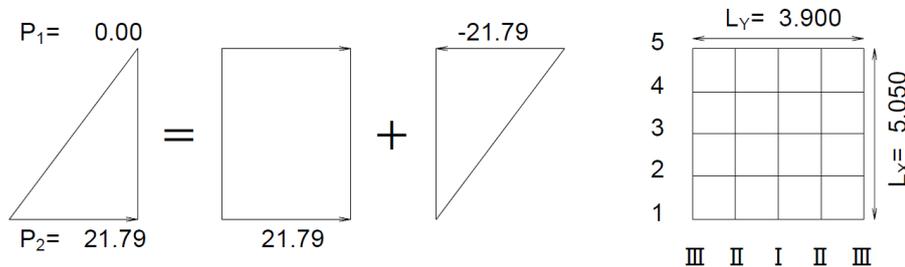


Figure 2.30- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = 21.79 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 21.79 \times 3.900^2 \times X = 331.43 \times X \text{ (kN·m)}$$

$$M_Y = P \cdot L_X^2 \cdot Y = 21.79 \times 5.050^2 \times Y = 551.43 \times Y \text{ (kN·m)}$$

Table 2.38- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0559	-18.53	-0.0093	-3.08
	4	0.0119	3.94	0.0181	6.00
	3	0.0189	6.26	0.0285	9.78
	2	0.0119	3.94	0.0181	6.00
	1	-0.0559	-18.53	-0.0093	-3.08
II	5	-0.0343	-11.37	-0.0057	-1.89
	4	0.0067	2.22	0.0074	2.45
	3	0.0097	3.21	0.0108	3.58
	2	0.0067	2.22	0.0074	2.45
	1	-0.0343	-11.37	-0.0057	-1.89
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0074	-2.45	-0.0442	-14.65
	3	-0.0111	-3.68	-0.0664	-22.01
	2	-0.0074	-2.45	-0.0442	-14.65
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$$P = -21.79 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L^2 \cdot X = -21.79 \times 3.900^2 \times X = -331.43 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L^2 \cdot Y = -21.79 \times 3.900^2 \times Y = -331.43 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.39- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0389	12.89	-0.0065	2.15
	4	0.0102	-3.38	0.0111	-3.68
	3	0.0095	-3.15	0.0147	-4.87
	2	0.0017	-0.56	0.0069	-2.29
	1	-0.0171	5.67	-0.0029	0.96
II	5	-0.0251	8.32	-0.0042	1.39
	4	0.0061	-2.02	0.0053	-1.76
	3	0.0048	-1.59	0.0054	-1.79
	2	0.0006	-0.20	0.0021	-0.70
	1	-0.0092	3.05	-0.0015	0.50
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0049	1.62	-0.0291	9.64
	3	-0.0055	1.82	-0.0332	11.00
	2	-0.0025	0.83	-0.0151	5.00
	1	0.0000	0.00	0.0000	0.00

- Combined uniformly distributed load and triangularly distributed load

Table 2.40- Summary of Sectional Force (Room B, Level 1 Earthquake Ground Motion (upward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	-5.64	-0.93
	4	0.56	2.32

	3	3.11	4.91
	2	3.38	3.71
	1	-12.86	-2.12
II	5	-3.05	-0.50
	4	0.20	0.69
	3	1.62	1.79
	2	2.02	1.75
	1	-8.32	-1.39
III	5	0.00	0.00
	4	-0.83	-5.01
	3	-1.86	-11.01
	2	-1.62	-9.65
	1	0.00	0.00

v) Room B: Level 1 earthquake ground motion (downward load)

$$P_1 = -167.66 \text{ (kN/m}^2\text{)}$$

$$P_2 = 0.00 \text{ (kN/m}^2\text{)}$$

$$L_X = 5.050 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 5.050/3.900 = 1.29$, The coefficient table for $\lambda = 1.25$ is applied.

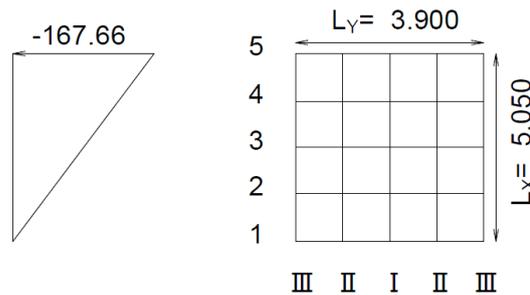


Figure 2.31- Design Model for Sectional Force Estimation

- Sectional forces induced by triangularly distributed load

$$P = -167.66 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = -167.66 \times 3.900^2 \times X = -2,550.11 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = -167.66 \times 3.900^2 \times Y = -2,550.11 \times Y \text{ (kN}\cdot\text{m)}$$

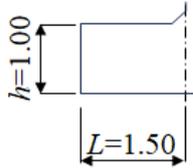
Table 2.41- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0389	99.20	-0.0065	16.58
	4	0.0102	-26.01	0.0111	-28.31
	3	0.0095	-24.23	0.0147	-37.49
	2	0.0017	-4.34	0.0069	-17.60
	1	-0.0171	43.61	-0.0029	7.40
II	5	-0.0251	64.01	-0.0042	10.71
	4	0.0061	-15.56	0.0053	-13.52
	3	0.0048	-12.24	0.0054	-13.77
	2	0.0006	-1.53	0.0021	-5.36
	1	-0.0092	23.46	-0.0015	3.83
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0049	12.50	-0.0291	74.21

	3	-0.0055	14.03	-0.0332	84.66
	2	-0.0025	6.38	-0.0151	38.51
	1	0.0000	0.00	0.0000	0.00

3) Safety Verification of Footing

The sectional forces acting on the footing are calculated by modeling it as a cantilever beam fixed at the front face of the front wall.



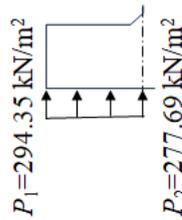
- Bending moment

$$M_d = 1/6 \cdot L^2 \cdot (2 \cdot P_1 + P_2)$$

- Shear force

$$V_d = 1/2 \cdot L \cdot (P_1 + P_2)$$

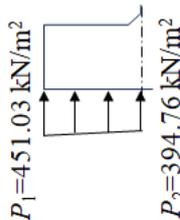
- i) Permanent state



$$M_d = 1/6 \times 1.50^2 \times (2 \times 294.35 + 277.69) = 324.90 \text{ (kN}\cdot\text{m/m)}$$

$$V_d = 1/2 \times 1.50 \times (294.35 + 277.69) = 429.03 \text{ (kN/m)}$$

- ii) Level 1 earthquake ground motion



$$M_d = 1/6 \times 1.50^2 \times (2 \times 451.03 + 394.76) = 486.31 \text{ (kN}\cdot\text{m/m)}$$

$$V_d = 1/2 \times 1.50 \times (451.03 + 394.76) = 634.34 \text{ (kN/m)}$$

4) Safety Verification of Outer Wall

The sectional force of outer wall is estimated by the model of slabs fixed on three sides and free on one side according to constraint conditions and making calculations based on numerical tables.

- i) Floating condition (Side wall)

$$P_1 = 0.00 \text{ (kN/m}^2\text{)}$$

$$P_2 = 127.52 \text{ (kN/m}^2\text{)}$$

$$L_X = 16.200 \text{ (m)}$$

$$L_Y = 5.050 \text{ (m)}$$

$\lambda = 16.200/5.050 = 3.21$, The coefficient table for $\lambda = 3.25$ is applied.

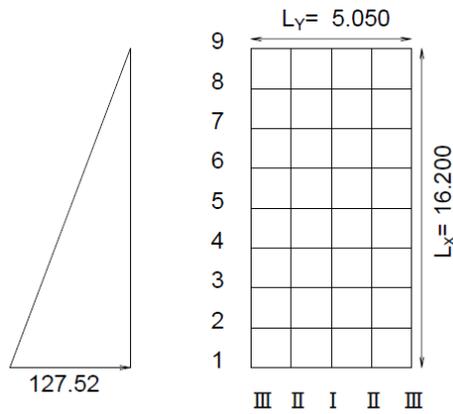


Figure 2.32- Design Model for Sectional Force Estimation

- Sectional forces induced by triangularly distributed load

$$P = 127.52 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 127.52 \times 5.050^2 \times X = 3,252.08 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = 127.52 \times 5.050^2 \times Y = 3,252.08 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.42- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0029	9.43
	8	0.0004	1.30	0.0057	18.54
	7	0.0017	5.53	0.0105	34.15
	6	0.0026	8.46	0.0156	50.73
	5	0.0035	11.38	0.0209	67.97
	4	0.0048	15.61	0.0261	84.88
	3	0.0085	27.64	0.0293	95.29
	2	0.0122	39.68	0.0212	68.94
	1	-0.0496	-161.30	-0.0083	-26.99
II	9	0.0000	0.00	0.0003	0.98
	8	-0.0001	-0.33	0.0013	4.23
	7	0.0004	1.30	0.0026	8.46
	6	0.0006	1.95	0.0039	12.68
	5	0.0008	2.60	0.0052	16.91
	4	0.0014	4.55	0.0066	21.46
	3	0.0034	11.06	0.0082	26.67
	2	0.0064	20.81	0.0077	25.04
	1	-0.0302	-98.21	-0.0050	-16.26
III	9	0.0000	0.00	-0.0008	-2.60
	8	-0.0019	-6.18	-0.0112	-36.42
	7	-0.0037	-12.03	-0.0224	-72.85
	6	-0.0054	-17.56	-0.0325	-105.69
	5	-0.0071	-23.09	-0.0428	-139.19
	4	-0.0086	-27.97	-0.0516	-167.81
	3	-0.0096	-31.22	-0.0578	-187.97
	2	-0.0080	-26.02	-0.0479	-155.77
	1	0.0000	0.00	0.0000	0.00

- ii) Floating condition (Front wall)

$$P_1 = 0.00 \text{ (kN/m}^2\text{)}$$

$$P_2 = 127.52 \text{ (kN/m}^2\text{)}$$

$$L_X = 16.200 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 16.200/3.900 = 4.15$, The coefficient table for $\lambda = 4.25$ is applied.

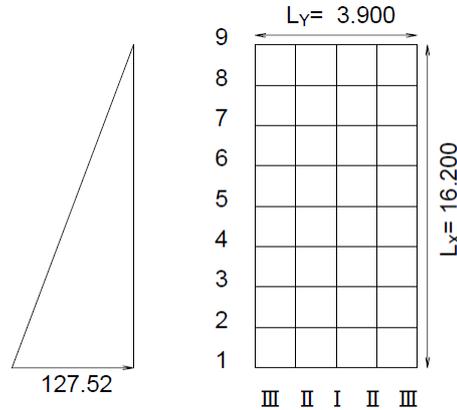


Figure 2.33- Design Model for Sectional Force Estimation

- Sectional forces induced by triangularly distributed load

$$P = 127.52 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 127.52 \times 3.900^2 \times X = 1,939.58 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = 127.52 \times 3.900^2 \times Y = 1,939.58 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.43- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0022	4.27
	8	0.0006	1.16	0.0054	10.47
	7	0.0018	3.49	0.0105	20.37
	6	0.0027	5.24	0.0157	30.45
	5	0.0034	6.59	0.0209	40.54
	4	0.0043	8.34	0.0261	50.62
	3	0.0064	12.41	0.0309	59.93
	2	0.0123	23.86	0.0274	53.14
	1	-0.0507	-98.34	-0.0085	-16.49
II	9	0.0000	0.00	0.0004	0.78
	8	0.0000	0.00	0.0012	2.33
	7	0.0003	0.58	0.0027	5.24
	6	0.0005	0.97	0.0039	7.56
	5	0.0007	1.36	0.0052	10.09
	4	0.0011	2.13	0.0065	12.61
	3	0.0023	4.46	0.0080	15.52
	2	0.0059	11.44	0.0088	17.07
	1	-0.0301	-58.38	-0.0050	-9.70
III	9	0.0000	0.00	0.0018	3.49
	8	-0.0018	-3.49	-0.0108	-20.95
	7	-0.0039	-7.56	-0.0234	-45.39
	6	-0.0056	-10.86	-0.0334	-64.78
	5	-0.0072	-13.96	-0.0434	-84.18
	4	-0.0085	-16.49	-0.0507	-98.34

	2	-0.0097	-18.81	-0.0580	-112.50
	1	0.0000	0.00	0.0000	0.00

iii) Permanent state (Side wall)

$$P_1 = -52.83 \text{ (kN/m}^2\text{)}$$

$$P_2 = -61.37 \text{ (kN/m}^2\text{)}$$

$$L_X = 16.200 \text{ (m)}$$

$$L_Y = 5.050 \text{ (m)}$$

$\lambda = 16.200/5.050 = 3.21$, The coefficient table for $\lambda = 3.25$ is applied.

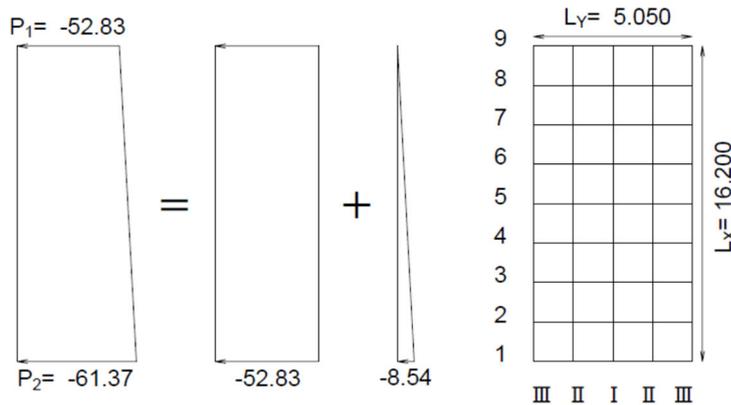


Figure 2.34- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = -52.83 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = -52.83 \times 5.050^2 \times X = -1,347.30 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_X^2 \cdot Y = -52.83 \times 16.200^2 \times Y = -138,432.00 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.44- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0432	-58.20
	8	0.0064	-8.62	0.0414	-55.78
	7	0.0071	-9.57	0.0416	-56.05
	6	0.0070	-9.43	0.0417	-56.18
	5	0.0070	-9.43	0.0418	-56.32
	4	0.0075	-10.10	0.0417	-56.18
	3	0.0104	-14.01	0.0395	-53.22
	2	0.0128	-17.25	0.0256	-34.49
	1	-0.0565	76.12	-0.0094	12.66
II	9	0.0000	0.00	0.0105	-14.15
	8	0.0014	-1.89	0.0103	-13.88
	7	0.0019	-2.56	0.0104	-14.01
	6	0.0019	-2.56	0.0104	-14.01
	5	0.0017	-2.29	0.0105	-14.15
	4	0.0019	-2.56	0.0106	-14.28
	3	0.0039	-5.25	0.0108	-14.55
	2	0.0067	-9.03	0.0088	-11.86
	1	-0.0338	45.54	-0.0056	7.54
III	9	0.0000	0.00	-0.0872	117.48

	8	-0.0140	18.86	-0.0837	112.77
	7	-0.0136	18.32	-0.0816	109.94
	6	-0.0137	18.46	-0.0823	110.88
	5	-0.0141	19.00	-0.0846	113.98
	4	-0.0141	19.00	-0.0845	113.85
	3	-0.0132	17.78	-0.0791	106.57
	2	-0.0092	12.40	-0.0553	74.51
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$$P = -8.54 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L^2 \cdot X = -8.54 \times 5.050^2 \times X = -217.79 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L^2 \cdot Y = -8.54 \times 5.050^2 \times Y = -217.79 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.45- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0029	-0.63
	8	0.0004	-0.09	0.0057	-1.24
	7	0.0017	-0.37	0.0105	-2.29
	6	0.0026	-0.57	0.0156	-3.40
	5	0.0035	-0.76	0.0209	-4.55
	4	0.0048	-1.05	0.0261	-5.68
	3	0.0085	-1.85	0.0293	-6.38
	2	0.0122	-2.66	0.0212	-4.62
	1	-0.0496	10.80	-0.0083	1.81
II	9	0.0000	0.00	0.0003	-0.07
	8	-0.0001	0.02	0.0013	-0.28
	7	0.0004	-0.09	0.0026	-0.57
	6	0.0006	-0.13	0.0039	-0.85
	5	0.0008	-0.17	0.0052	-1.13
	4	0.0014	-0.30	0.0066	-1.44
	3	0.0034	-0.74	0.0082	-1.79
	2	0.0064	-1.39	0.0077	-1.68
	1	-0.0302	6.58	-0.0050	1.09
III	9	0.0000	0.00	-0.0008	0.17
	8	-0.0019	0.41	-0.0112	2.44
	7	-0.0037	0.81	-0.0224	4.88
	6	-0.0054	1.18	-0.0325	7.08
	5	-0.0071	1.55	-0.0428	9.32
	4	-0.0086	1.87	-0.0516	11.24
	3	-0.0096	2.09	-0.0578	12.59
	2	-0.0080	1.74	-0.0479	10.43
	1	0.0000	0.00	0.0000	0.00

Table 2.46- Summary of Sectional Force (Side wall, Permanent State)

		M_X (kN·m)	M_Y (kN·m)
I	9	0.00	-58.83
	8	-8.71	-57.02

	7	-9.94	-58.34
	6	-10.00	-59.58
	5	-10.19	-60.87
	4	-11.15	-61.86
	3	-15.86	-59.60
	2	-19.91	-39.11
	1	86.92	14.47
II	9	0.00	-14.22
	8	-1.87	-14.16
	7	-2.65	-14.58
	6	-2.69	-14.86
	5	-2.46	-15.28
	4	-2.86	-15.72
	3	-5.99	-16.34
	2	-10.42	-13.54
	1	52.12	8.63
III	9	0.00	117.65
	8	19.27	115.21
	7	19.13	114.82
	6	19.64	117.96
	5	20.55	123.30
	4	20.87	125.09
	3	19.87	119.16
	2	14.14	84.94
	1	0.00	0.00

iv) Permanent state (Front and Rear wall)

$$P_1 = -56.23 \text{ (kN/m}^2\text{)}$$

$$P_2 = -68.96 \text{ (kN/m}^2\text{)}$$

$$L_X = 16.200 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 16.200/3.900 = 4.15$, The coefficient table for $\lambda = 4.25$ is applied.

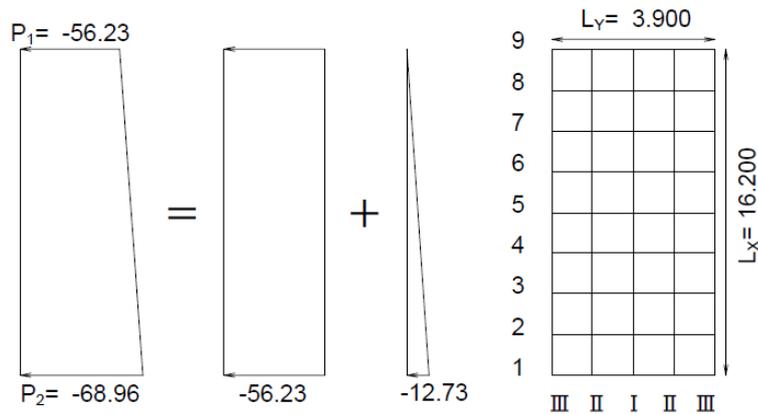


Figure 2.35- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = -56.23 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = -56.23 \times 3.900^2 \times X = -855.26 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = -56.23 \times 3.900^2 \times Y = -855.26 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.47- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0433	-37.03
	8	0.0068	-5.82	0.0415	-35.49
	7	0.0072	-6.16	0.0416	-35.58
	6	0.0071	-6.07	0.0417	-35.66
	5	0.0069	-5.90	0.0417	-35.66
	4	0.0068	-5.82	0.0418	-35.75
	3	0.0081	-6.93	0.0413	-35.32
	2	0.0134	-11.46	0.0321	-27.45
	1	-0.0560	47.89	-0.0093	7.95
II	9	0.0000	0.00	0.0105	-8.98
	8	0.0016	-1.37	0.0104	-8.89
	7	0.0022	-1.88	0.0105	-8.98
	6	0.0020	-1.71	0.0104	-8.89
	5	0.0015	-1.28	0.0104	-8.89
	4	0.0015	-1.28	0.0104	-8.89
	3	0.0026	-2.22	0.0106	-9.07
	2	0.0065	-5.56	0.0101	-8.64
	1	-0.0325	27.80	-0.0054	4.62
III	9	0.0000	0.00	-0.0891	76.20
	8	-0.0141	12.06	-0.0844	72.18
	7	-0.0135	11.55	-0.0807	69.02
	6	-0.0136	11.63	-0.0816	69.79
	5	-0.0142	12.14	-0.0852	72.87
	4	-0.0141	12.06	-0.0848	72.53
	3	-0.0133	11.37	-0.0801	68.51
	2	-0.0108	9.24	-0.0651	55.68
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$$P = -12.73 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L^2 \cdot X = -12.73 \times 3.900^2 \times X = -193.62 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L^2 \cdot Y = -12.73 \times 3.900^2 \times Y = -193.62 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.48- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0022	-0.43
	8	0.0006	-0.12	0.0054	-1.05
	7	0.0018	-0.35	0.0105	-2.03
	6	0.0027	-0.52	0.0157	-3.04
	5	0.0034	-0.66	0.0209	-4.05
	4	0.0043	-0.83	0.0261	-5.05
	3	0.0064	-1.24	0.0309	-5.98
	2	0.0123	-2.38	0.0274	-5.31
	1	-0.0507	9.82	-0.0085	1.65
II	9	0.0000	0.00	0.0004	-0.08

	8	0.0000	0.00	0.0012	-0.23
	7	0.0003	-0.06	0.0027	-0.52
	6	0.0005	-0.10	0.0039	-0.76
	5	0.0007	-0.14	0.0052	-1.01
	4	0.0011	-0.21	0.0065	-1.26
	3	0.0023	-0.45	0.0080	-1.55
	2	0.0059	-1.14	0.0088	-1.70
	1	-0.0301	5.83	-0.0050	0.97
III	9	0.0000	0.00	0.0018	-0.35
	8	-0.0018	0.35	-0.0108	2.09
	7	-0.0039	0.76	-0.0234	4.53
	6	-0.0056	1.08	-0.0334	6.47
	5	-0.0072	1.39	-0.0434	8.40
	4	-0.0085	1.65	-0.0507	9.82
	3	-0.0097	1.88	-0.0580	11.23
	2	-0.0097	1.88	-0.0580	11.23
	1	0.0000	0.00	0.0000	0.00

Table 2.49- Summary of Sectional Force (Side wall, Permanent State)

		M_x (kN·m)	M_y (kN·m)
I	9	0.00	-37.46
	8	-5.94	-36.54
	7	-6.51	-37.61
	6	-6.59	-38.70
	5	-6.56	-39.71
	4	-6.65	-40.80
	3	-8.17	-41.30
	2	-13.84	-32.76
	1	57.71	9.60
II	9	0.00	-9.06
	8	-1.37	-9.12
	7	-1.94	-9.50
	6	-1.81	-9.65
	5	-1.42	-9.90
	4	-1.49	-10.15
	3	-2.67	-10.62
	2	-6.70	-10.34
	1	33.63	5.59
III	9	0.00	75.85
	8	12.41	74.27
	7	12.31	73.55
	6	12.71	76.26
	5	13.53	81.27
	4	13.71	82.35
	3	13.25	79.74
	2	11.12	66.91
	1	0.00	0.00

v) 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:

$$K_1 = 1 / L_{y1} = 1 / 5.050 = 0.198$$

$$K_2 = 1 / L_{y2} = 1 / 3.900 = 0.256$$

- Redistributed ratio

$$\text{Side wall direction: } \alpha = K_1 / (K_1 + K_2) = 0.198 / (0.198 + 0.256) = 0.436$$

$$\text{Front and Rear wall direction: } \beta = K_2 / (K_1 + K_2) = 0.256 / (0.198 + 0.256) = 0.564$$

- Corrected moment at corners (III-axis)

$$\Delta M = M_1 - M_2$$

When $M_1 > M_2$

$$M_1' = M_1 - \alpha \Delta M = M_1 - 0.436 \times \Delta M$$

$$M_2' = M_2 + \beta \Delta M = M_2 + 0.564 \times \Delta M$$

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

$$M_1' = M_1 - 1/2 \cdot \Delta M \cdot e_1 = M_1 - 0.218 \times \Delta M$$

$$M_2' = M_2 + 1/2 \cdot \Delta M \cdot e_2 = M_2 + 0.282 \times \Delta M$$

- vi) 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	-2.60	2.66	-2.60	6.09	3.49	-3.43	3.49
8	-36.42	6.74	-36.42	15.47	-20.95	-8.73	-36.42
7	-72.85	11.97	-72.85	27.46	-45.39	-15.49	-72.85
6	-105.69	17.84	-105.69	40.91	-64.78	-23.07	-105.69
5	-139.19	23.98	-139.19	55.01	-84.18	-31.03	-139.19
4	-167.81	30.29	-167.81	69.47	-98.34	-39.18	-167.81
3	-187.97	32.90	-187.97	75.47	-112.50	-42.57	-187.97

2	-155.77	18.87	-155.77	43.27	-112.50	-24.40	-155.77
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	10.76	1.33	9.43	6.09	4.27	-1.72	4.27
8	21.91	3.37	18.54	15.47	10.47	-4.36	10.47
7	40.14	5.99	34.15	27.46	20.37	-7.74	20.37
6	59.65	8.92	50.73	40.91	30.45	-11.54	30.45
5	79.96	11.99	67.97	55.01	40.54	-15.51	40.54
4	100.02	15.14	84.88	69.47	50.62	-19.59	50.62
3	111.74	16.45	95.29	75.47	59.93	-21.28	59.93
2	78.37	9.43	68.94	43.27	53.14	-12.20	53.14
1	-26.99	0.00	-26.99	0.00	-16.49	0.00	-16.49

vii) 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	117.65	-18.22	117.65	41.80	75.85	23.58	99.43
8	115.21	-17.85	115.21	40.94	74.27	23.09	97.36
7	114.82	-17.99	114.82	41.27	73.55	23.28	96.83
6	117.96	-18.18	117.96	41.70	76.26	23.52	99.78
5	123.30	-18.33	123.30	42.03	81.27	23.70	104.97
4	125.09	-18.63	125.09	42.74	82.35	24.11	106.46
3	119.16	-17.19	119.16	39.42	79.74	22.23	101.97
2	84.94	-7.86	84.94	18.03	66.91	10.17	77.08
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	-67.94	-9.11	-58.83	41.80	-37.46	11.79	-37.46
8	-65.94	-8.92	-57.02	40.94	-36.54	11.55	-36.54
7	-67.34	-9.00	-58.34	41.27	-37.61	11.64	-37.61
6	-68.67	-9.09	-59.58	41.70	-38.70	11.76	-38.70
5	-70.03	-9.16	-60.87	42.03	-39.71	11.85	-39.71
4	-71.18	-9.32	-61.86	42.74	-40.80	12.05	-40.80
3	-68.19	-8.59	-59.60	39.42	-41.30	11.12	-41.30
2	-43.04	-3.93	-39.11	18.03	-32.76	5.08	-32.76
1	14.47	0.00	14.47	0.00	9.60	0.00	9.60

5) Safety Verification of Partition Wall

The sectional force of partition wall is estimated by the model of slabs fixed on three

sides and free on one side according to constraint conditions and making calculations based on numerical tables. The sign convention for bending moments is as follows:

- Positive (+) when tension occurs on the inner side,
- Negative (-) when tension occurs on the outer side.

i) Transverse direction

$$P_1 = 11.11 \text{ (kN/m}^2\text{)}$$

$$P_2 = 11.11 \text{ (kN/m}^2\text{)}$$

$$L_X = 16.200 \text{ (m)}$$

$$L_Y = 5.050 \text{ (m)}$$

$\lambda = 16.200/5.050 = 3.21$, The coefficient table for $\lambda = 3.25$ is applied.

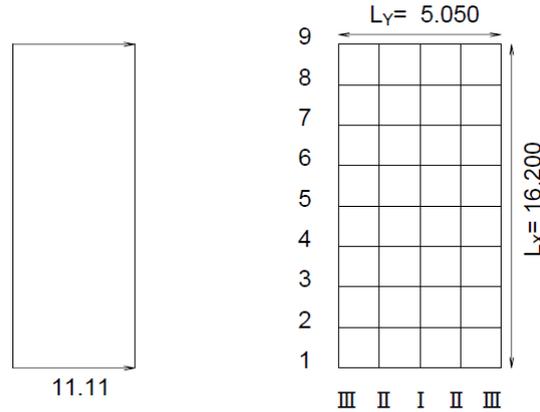


Figure 2.36- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = 11.11 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 11.11 \times 5.050^2 \times X = 283.33 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = 11.11 \times 5.050^2 \times Y = 283.33 \times Y \text{ (kN}\cdot\text{m)}$$

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	12.24
	8	0.0064	1.81	0.0414	11.73
	7	0.0071	2.01	0.0416	11.79
	6	0.0070	1.98	0.0417	11.81
	5	0.0070	1.98	0.0418	11.84
	4	0.0075	2.12	0.0417	11.81
	3	0.0104	2.95	0.0395	11.19
	2	0.0128	3.63	0.0256	7.25
	1	-0.0565	-16.01	-0.0094	-2.66
II	9	0.0000	0.00	0.0105	2.97
	8	0.0014	0.40	0.0103	2.92
	7	0.0019	0.54	0.0104	2.95
	6	0.0019	0.54	0.0104	2.95
	5	0.0017	0.48	0.0105	2.97
	4	0.0019	0.54	0.0106	3.00
	3	0.0039	1.10	0.0108	3.06
	2	0.0067	1.90	0.0088	2.49
	1	-0.0338	-9.58	-0.0056	-1.59

III	9	0.0000	0.00	-0.0872	-24.71
	8	-0.0140	-3.97	-0.0837	-23.71
	7	-0.0136	-3.85	-0.0816	-23.12
	6	-0.0137	-3.88	-0.0823	-23.32
	5	-0.0141	-3.99	-0.0846	-23.97
	4	-0.0141	-3.99	-0.0845	-23.94
	3	-0.0132	-3.74	-0.0791	-22.41
	2	-0.0092	-2.61	-0.0553	-15.67
	1	0.0000	0.00	0.0000	0.00

ii) Longitudinal direction

$$P_1 = 11.11 \text{ (kN/m}^2\text{)}$$

$$P_2 = 11.11 \text{ (kN/m}^2\text{)}$$

$$L_X = 16.200 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 16.200/3.900 = 4.15$, The coefficient table for $\lambda = 4.25$ is applied.

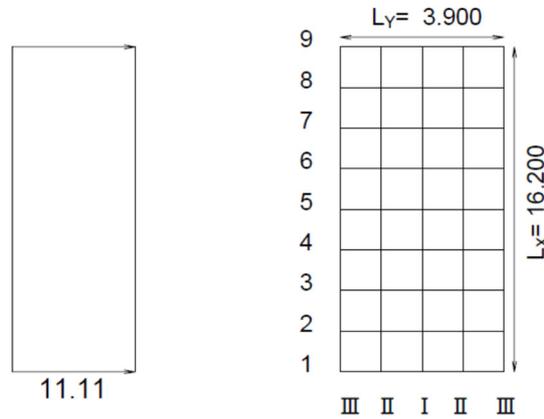


Figure 2.37- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = 11.11 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 11.11 \times 3.900^2 \times X = 168.98 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_X^2 \cdot Y = 11.11 \times 16.200^2 \times Y = 2918.8 \times Y \text{ (kN}\cdot\text{m)}$$

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	0.0433	7.32
	8	0.0068	1.15	0.0415	7.01
	7	0.0072	1.22	0.0416	7.03
	6	0.0071	1.20	0.0417	7.05
	5	0.0069	1.17	0.0417	7.05
	4	0.0068	1.15	0.0418	7.06
	3	0.0081	1.37	0.0413	6.98
	2	0.0134	2.26	0.0321	5.42
	1	-0.0560	-9.46	-0.0093	-1.57
II	9	0.0000	0.00	0.0105	1.77
	8	0.0016	0.27	0.0104	1.76
	7	0.0022	0.37	0.0105	1.77

	6	0.0020	0.34	0.0104	1.76
	5	0.0015	0.25	0.0104	1.76
	4	0.0015	0.25	0.0104	1.76
	3	0.0026	0.44	0.0106	1.79
	2	0.0065	1.10	0.0101	1.71
	1	-0.0325	-5.49	-0.0054	-0.91
III	9	0.0000	0.00	-0.0891	-15.06
	8	-0.0141	-2.38	-0.0844	-14.26
	7	-0.0135	-2.28	-0.0807	-13.46
	6	-0.0136	-2.30	-0.0816	-13.79
	5	-0.0142	-2.40	-0.0852	-14.40
	4	-0.0141	-2.38	-0.0848	-14.33
	3	-0.0133	-2.25	-0.0801	-13.54
	2	-0.0108	-1.82	-0.0651	-11.00
	1	0.0000	0.00	0.0000	0.00

(4) Sectional Force for Serviceability Verification
1) Serviceability Verification of Bottom Slab

The sectional force of bottom slab is estimated by the model of slabs fixed on four sides according to constraint conditions and making calculations based on numerical tables. The sign convention for bending moments is as follows:

- Positive (+) when tension occurs on the upper side,
- Negative (-) when tension occurs on the lower side.

i) Room A: Permanent state (upward load)

$$P_1 = 20.93 \text{ (kN/m}^2\text{)}$$

$$P_2 = 71.89 \text{ (kN/m}^2\text{)}$$

$$L_X = 5.050 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 5.050/3.900 = 1.29$, The coefficient table for $\lambda = 1.25$ is applied.

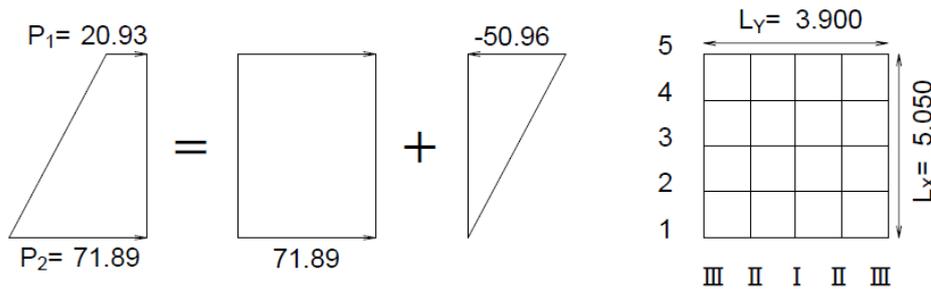


Figure 2.38- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = 71.89 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 71.89 \times 3.900^2 \times X = 1,093.45 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_X^2 \cdot Y = 71.89 \times 5.050^2 \times Y = 1,838.45 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.52- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0559	-61.12	-0.0093	-10.17
	4	0.0119	13.01	0.0181	19.79
	3	0.0189	20.67	0.0285	32.26
	2	0.0119	13.01	0.0181	19.79
	1	-0.0559	-61.12	-0.0093	-10.17
II	5	-0.0343	-37.51	-0.0057	-6.23
	4	0.0067	7.33	0.0074	8.09
	3	0.0097	10.61	0.0108	11.81
	2	0.0067	7.33	0.0074	8.09
	1	-0.0343	-37.51	-0.0057	-6.23
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0074	-8.09	-0.0442	-48.33
	3	-0.0111	-12.14	-0.0664	-72.61
	2	-0.0074	-8.09	-0.0442	-48.33
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$$P = -50.96 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = -50.96 \times 3.900^2 \times X = -775.10 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_X^2 \cdot Y = -50.96 \times 3.900^2 \times Y = -775.10 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.53- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0389	30.15	-0.0065	5.04
	4	0.0102	-7.91	0.0111	-8.60
	3	0.0095	-7.36	0.0147	-11.39
	2	0.0017	-1.32	0.0069	-5.35
	1	-0.0171	13.25	-0.0029	2.25
II	5	-0.0251	19.46	-0.0042	3.26
	4	0.0061	-4.73	0.0053	-4.11
	3	0.0048	-3.72	0.0054	-4.19
	2	0.0006	-0.47	0.0021	-1.63
	1	-0.0092	7.13	-0.0015	1.16
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0049	3.80	-0.0291	22.56
	3	-0.0055	4.26	-0.0332	25.73
	2	-0.0025	1.94	-0.0151	11.70
	1	0.0000	0.00	0.0000	0.00

- Combined uniformly distributed load and triangularly distributed load

Table 2.54- Summary of Sectional Force (Room A, Permanent state (upward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	-30.97	-5.13
	4	5.10	11.19
	3	13.31	20.87
	2	11.69	14.44
	1	-47.87	-7.92
II	5	-18.05	-2.97
	4	2.60	3.98
	3	6.89	7.62
	2	6.86	6.46
	1	-30.38	-5.07
III	5	0.00	0.00
	4	-4.29	-25.77
	3	-7.88	-46.88
	2	-6.15	-36.63
	1	0.00	0.00

ii) Room B: Floating condition (upward load)

$$P_1 = 38.69 \text{ (kN/m}^2\text{)}$$

$$P_2 = 38.69 \text{ (kN/m}^2\text{)}$$

$$L_X = 5.050 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 5.050/3.900 = 1.29$, The coefficient table for $\lambda = 1.25$ is applied.

- Sectional forces induced by uniformly distributed load

$$P = 38.69 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 38.69 \times 3.900^2 \times X = 588.47 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_X^2 \cdot Y = 38.69 \times 5.050^2 \times Y = 988.47 \times Y \text{ (kN}\cdot\text{m)}$$

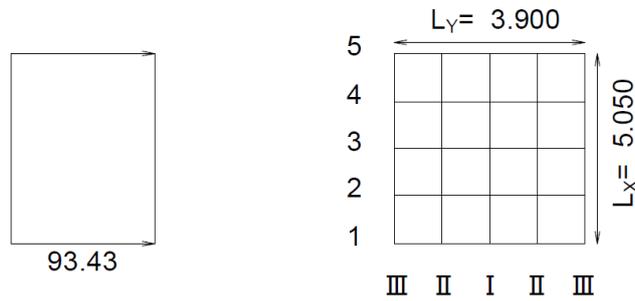


Figure 2.39- Design Model for Sectional Force Estimation

Table 2.55- Coefficient and Sectional Force (Room B, Floating condition (upward load))

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0559	-32.90	-0.0093	-5.47
	4	0.0119	7.00	0.0181	10.65
	3	0.0189	11.12	0.0285	17.36
	2	0.0119	7.00	0.0181	10.65
	1	-0.0559	-32.90	-0.0093	-5.47
II	5	-0.0343	-20.18	-0.0057	-3.35
	4	0.0067	3.94	0.0074	4.35
	3	0.0097	5.71	0.0108	6.36
	2	0.0067	3.94	0.0074	4.35
	1	-0.0343	-20.18	-0.0057	-3.35
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0074	-4.35	-0.0442	-26.01
	3	-0.0111	-6.53	-0.0664	-39.07
	2	-0.0074	-4.35	-0.0442	-26.01
	1	0.0000	0.00	0.0000	0.00

iii) Room B: Permanent state (downward load)

$$P_1 = -30.04 \text{ (kN/m}^2\text{)}$$

$$P_2 = 0.00 \text{ (kN/m}^2\text{)}$$

$$L_X = 5.050 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 5.050/3.900 = 1.29$, The coefficient table for $\lambda = 1.25$ is applied.

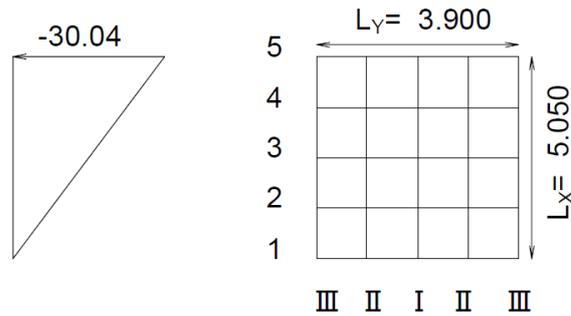


Figure 2.40- Design Model for Sectional Force Estimation

- Sectional forces induced by triangularly distributed load

$$P = -30.04 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = -30.04 \times 3.900^2 \times X = -456.91 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_X^2 \cdot Y = -30.04 \times 5.050^2 \times Y = -761.15 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.56- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0389	17.77	-0.0065	2.97
	4	0.0102	-4.66	0.0111	-5.07
	3	0.0095	-4.34	0.0147	-6.72
	2	0.0017	-0.78	0.0069	-3.15
	1	-0.0171	7.81	-0.0029	1.33
II	5	-0.0251	11.47	-0.0042	1.92
	4	0.0061	-2.79	0.0053	-2.42
	3	0.0048	-2.19	0.0054	-2.47
	2	0.0006	-0.27	0.0021	-0.96
	1	-0.0092	4.20	-0.0015	0.69
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0049	2.24	-0.0291	13.30
	3	-0.0055	2.51	-0.0332	15.17
	2	-0.0025	1.14	-0.0151	6.90
	1	0.0000	0.00	0.0000	0.00

2) Serviceability Verification of Bottom Slab (Compression Stress)

The sectional force of bottom slab is estimated by the model of slabs fixed on four sides according to constraint conditions and making calculations based on numerical tables. The sign convention for bending moments is as follows:

- Positive (+) when tension occurs on the upper side,
- Negative (-) when tension occurs on the lower side.

i) Room A: Permanent state (upward load)

$$P_1 = 20.93 \text{ (kN/m}^2\text{)}$$

$$P_2 = 71.89 \text{ (kN/m}^2\text{)}$$

$$L_X = 5.050 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 5.050/3.900 = 1.29$, The coefficient table for $\lambda = 1.25$ is applied.

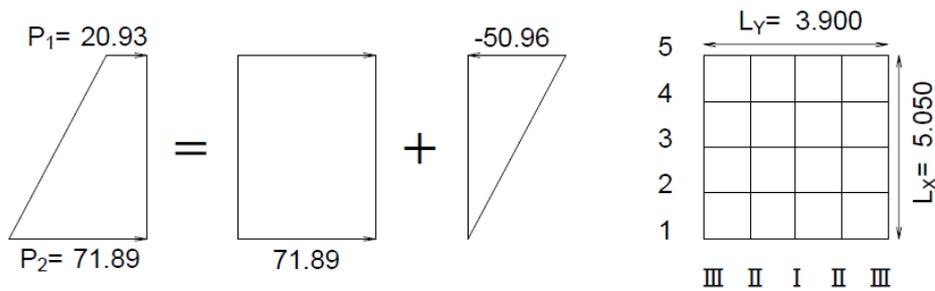


Figure 2.41- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = 71.89 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 71.89 \times 3.900^2 \times X = 1,093.45 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = 71.89 \times 3.900^2 \times Y = 1,093.45 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.57- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0559	-61.12	-0.0093	-10.17
	4	0.0119	13.01	0.0181	19.79
	3	0.0189	20.67	0.0285	32.26
	2	0.0119	13.01	0.0181	19.79
	1	-0.0559	-61.12	-0.0093	-10.17
II	5	-0.0343	-37.51	-0.0057	-6.23
	4	0.0067	7.33	0.0074	8.09
	3	0.0097	10.61	0.0108	11.81
	2	0.0067	7.33	0.0074	8.09
	1	-0.0343	-37.51	-0.0057	-6.23
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0074	-8.09	-0.0442	-48.33
	3	-0.0111	-12.14	-0.0664	-72.61
	2	-0.0074	-8.09	-0.0442	-48.33
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$$P = -50.96 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = -50.96 \times 3.900^2 \times X = -775.10 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = -50.96 \times 3.900^2 \times Y = -775.10 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.58- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0389	30.15	-0.0065	5.04
	4	0.0102	-7.91	0.0111	-8.60
	3	0.0095	-7.36	0.0147	-11.39
	2	0.0017	-1.32	0.0069	-5.35
	1	-0.0171	13.25	-0.0029	2.25
II	5	-0.0251	19.46	-0.0042	3.26
	4	0.0061	-4.73	0.0053	-4.11
	3	0.0048	-3.72	0.0054	-4.19

	2	0.0006	-0.47	0.0021	-1.63
	1	-0.0092	7.13	-0.0015	1.16
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0049	3.80	-0.0291	22.56
	3	-0.0055	4.26	-0.0332	25.73
	2	-0.0025	1.94	-0.0151	11.70
	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, Permanent State (upward load))

		M_x (kN·m)	M_y (kN·m)
I	5	-30.97	-5.13
	4	5.10	11.19
	3	13.31	20.87
	2	11.69	14.44
	1	-47.87	-7.92
II	5	-18.05	-2.97
	4	2.60	3.98
	3	6.89	7.62
	2	6.86	6.46
	1	-30.38	-5.07
III	5	0.00	0.00
	4	-4.29	-25.77
	3	-7.88	-46.88
	2	-6.15	-36.63
	1	0.00	0.00

ii) Room B: Permanent state (upward load)

$$P_1 = 0.00 \text{ (kN/m}^2\text{)}$$

$$P_2 = 20.93 \text{ (kN/m}^2\text{)}$$

$$L_x = 5.050 \text{ (m)}$$

$$L_y = 3.900 \text{ (m)}$$

$\lambda = 5.050/3.900 = 1.29$, The coefficient table for $\lambda = 1.25$ is applied.

- Sectional forces induced by uniformly distributed load

$$P = 20.93 \text{ (kN/m}^2\text{)}$$

$$M_x = P \cdot L_y^2 \cdot X = 20.93 \times 3.900^2 \times X = 318.35 \times X \text{ (kN}\cdot\text{m)}$$

$$M_y = P \cdot L_x^2 \cdot Y = 20.93 \times 5.050^2 \times Y = 534.35 \times Y \text{ (kN}\cdot\text{m)}$$

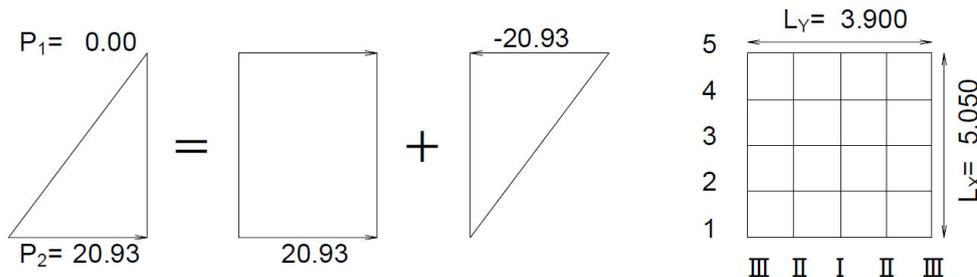


Figure 2.42- Design Model for Sectional Force Estimation

Table 2.60- Coefficient and Sectional Force (Room B, Permanent State (upward load))

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0559	-17.80	-0.0093	-2.96
	4	0.0119	3.79	0.0181	5.76
	3	0.0189	6.02	0.0285	9.39
	2	0.0119	3.79	0.0181	5.76
	1	-0.0559	-17.80	-0.0093	-2.96
II	5	-0.0343	-10.92	-0.0057	-1.81
	4	0.0067	2.13	0.0074	2.36
	3	0.0097	3.09	0.0108	3.44
	2	0.0067	2.13	0.0074	2.36
	1	-0.0343	-10.92	-0.0057	-1.81
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0074	-2.36	-0.0442	-14.07
	3	-0.0111	-3.53	-0.0664	-21.14
	2	-0.0074	-2.36	-0.0442	-14.07
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$$P = -20.93 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L \cdot Y^2 \cdot X = -20.93 \times 3.900^2 \times X = -318.35 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L \cdot Y^2 \cdot Y = -20.93 \times 3.900^2 \times Y = -318.35 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.61- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0389	12.38	-0.0065	2.07
	4	0.0102	-3.25	0.0111	-3.53
	3	0.0095	-3.02	0.0147	-4.68
	2	0.0017	-0.54	0.0069	-2.20
	1	-0.0171	5.44	-0.0029	0.92
II	5	-0.0251	7.99	-0.0042	1.34
	4	0.0061	-1.94	0.0053	-1.69
	3	0.0048	-1.53	0.0054	-1.72
	2	0.0006	-0.19	0.0021	-0.67
	1	-0.0092	2.93	-0.0015	0.48
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0049	1.56	-0.0291	9.26
	3	-0.0055	1.75	-0.0332	10.57
	2	-0.0025	0.80	-0.0151	4.81
	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 B, Permanent State (upward load))

		M_X (kN·m)	M_Y (kN·m)
I	5	-5.42	-0.89
	4	0.54	2.23
	3	3.00	4.71
	2	3.25	3.56
	1	-12.36	-2.04
II	5	-2.93	-0.47
	4	0.19	0.67
	3	1.56	1.72
	2	1.94	1.69
	1	-7.99	-1.33
III	5	0.00	0.00
	4	-0.80	-4.81
	3	-1.78	-10.57
	2	-1.56	-9.26
	1	0.00	0.00

iii) Room B: Permanent state (downward load)

$$P_1 = -30.04 \text{ (kN/m}^2\text{)}$$

$$P_2 = 0.00 \text{ (kN/m}^2\text{)}$$

$$L_X = 5.050 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 5.050/3.900 = 1.29$, The coefficient table for $\lambda = 1.25$ is applied.

- Sectional forces induced by triangularly distributed load

$$P = -30.04 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = -30.04 \times 3.900^2 \times X = -456.91 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = -30.04 \times 3.900^2 \times Y = -456.91 \times Y \text{ (kN}\cdot\text{m)}$$

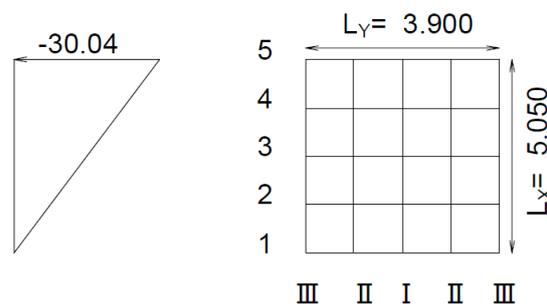


Figure 2.43- Design Model for Sectional Force Estimation

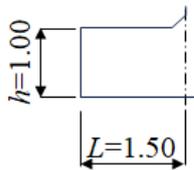
Table 2.63- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	5	-0.0389	17.77	-0.0065	2.97
	4	0.0102	-4.66	0.0111	-5.07
	3	0.0095	-4.34	0.0147	-6.72
	2	0.0017	-0.78	0.0069	-3.15

	1	-0.0171	7.81	-0.0029	1.33
II	5	-0.0251	11.47	-0.0042	1.92
	4	0.0061	-2.79	0.0053	-2.42
	3	0.0048	-2.19	0.0054	-2.47
	2	0.0006	-0.27	0.0021	-0.96
	1	-0.0092	4.20	-0.0015	0.69
III	5	0.0000	0.00	0.0000	0.00
	4	-0.0049	2.24	-0.0291	13.30
	3	-0.0055	2.51	-0.0332	15.17
	2	-0.0025	1.14	-0.0151	6.90
	1	0.0000	0.00	0.0000	0.00

3) Serviceability Verification of Footing

The sectional forces acting on the footing are calculated by modeling it as a cantilever beam fixed at the front face of the front wall.



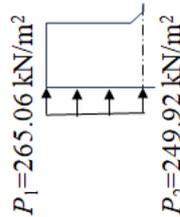
- Bending moment

$$M_d = 1/6 \cdot L^2 \cdot (2 \cdot P_1 + P_2)$$

- Shear force

$$V_d = 1/2 \cdot L \cdot (P_1 + P_2)$$

- i) Crack, Shear force and Compression



$$M_d = 1/6 \times 1.50^2 \times (2 \times 265.06 + 249.92) = 292.52 \text{ (kN} \cdot \text{m/m)}$$

$$V_d = 1/2 \times 1.50 \times (265.06 + 249.92) = 386.24 \text{ (kN/m)}$$

4) Serviceability Verification of Outer Wall

The sectional force of outer wall is estimated by the model of slabs fixed on three sides and free on one side according to constraint conditions and making calculations based on numerical tables.

- i) Floating condition (Side wall)

$$P_1 = 0.00 \text{ (kN/m}^2\text{)}$$

$$P_2 = 57.97 \text{ (kN/m}^2\text{)}$$

$$L_X = 16.200 \text{ (m)}$$

$$L_Y = 5.050 \text{ (m)}$$

$$\lambda = 16.200/5.050 = 3.21, \text{ The coefficient table for } \lambda = 3.25 \text{ is applied.}$$

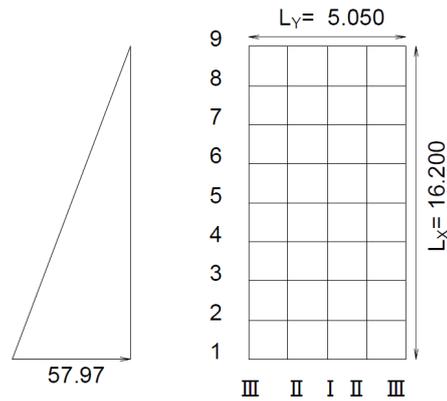


Figure 2.44- Design Model for Sectional Force Estimation

- Sectional forces induced by triangularly distributed load

$$P = 57.97 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 57.97 \times 5.050^2 \times X = 1,478.38 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = 57.97 \times 5.050^2 \times Y = 1,478.38 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.64- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0029	4.29
	8	0.0004	0.59	0.0057	8.43
	7	0.0017	2.51	0.0105	15.52
	6	0.0026	3.84	0.0156	23.06
	5	0.0035	5.17	0.0209	30.90
	4	0.0048	7.10	0.0261	38.59
	3	0.0085	12.57	0.0293	43.32
	2	0.0122	18.04	0.0212	31.34
	1	-0.0496	-73.33	-0.0083	-12.27
II	9	0.0000	0.00	0.0003	0.44
	8	-0.0001	-0.15	0.0013	1.92
	7	0.0004	0.59	0.0026	3.84
	6	0.0006	0.89	0.0039	5.77
	5	0.0008	1.18	0.0052	7.69
	4	0.0014	2.07	0.0066	9.76
	3	0.0034	5.03	0.0082	12.12
	2	0.0064	9.46	0.0077	11.38
	1	-0.0302	-44.65	-0.0050	-7.39
III	9	0.0000	0.00	-0.0008	-1.18
	8	-0.0019	-2.81	-0.0112	-16.56
	7	-0.0037	-5.47	-0.0224	-33.12
	6	-0.0054	-7.98	-0.0325	-48.05
	5	-0.0071	-10.50	-0.0428	-63.27
	4	-0.0086	-12.71	-0.0516	-76.28
	3	-0.0096	-14.19	-0.0578	-85.45
	2	-0.0080	-11.83	-0.0479	-70.81
	1	0.0000	0.00	0.0000	0.00

ii) Floating condition (Front wall)

$$P_1 = 0.00 \text{ (kN/m}^2\text{)}$$

$$P_2 = 57.97 \text{ (kN/m}^2\text{)}$$

$$L_X = 16.200 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 16.200/3.900 = 4.15$, The coefficient table for $\lambda = 4.25$ is applied.

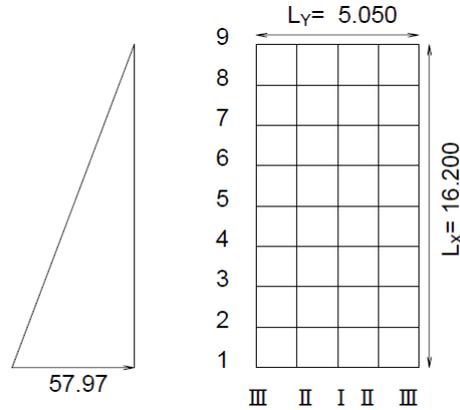


Figure 2.45- Design Model for Sectional Force Estimation

- Sectional forces induced by triangularly distributed load

$$P = 57.97 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 57.97 \times 3.900^2 \times X = 881.72 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = 57.97 \times 3.900^2 \times Y = 881.72 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.65- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0022	1.94
	8	0.0006	0.53	0.0054	4.76
	7	0.0018	1.59	0.0105	9.26
	6	0.0027	2.38	0.0157	13.84
	5	0.0034	3.00	0.0209	18.43
	4	0.0043	3.79	0.0261	23.01
	3	0.0064	5.64	0.0309	27.25
	2	0.0123	10.85	0.0274	24.16
	1	-0.0507	-44.70	-0.0085	-7.49
II	9	0.0000	0.00	0.0004	0.35
	8	0.0000	0.00	0.0012	1.06
	7	0.0003	0.26	0.0027	2.38
	6	0.0005	0.44	0.0039	3.44
	5	0.0007	0.62	0.0052	4.58
	4	0.0011	0.97	0.0065	5.73
	3	0.0023	2.03	0.0080	7.05
	2	0.0059	5.20	0.0088	7.76
	1	-0.0301	-26.54	-0.0050	-4.41
III	9	0.0000	0.00	0.0018	1.59
	8	-0.0018	-1.59	-0.0108	-9.52
	7	-0.0039	-3.44	-0.0234	-20.63
	6	-0.0056	-4.94	-0.0334	-29.45
	5	-0.0072	-6.35	-0.0434	-38.27

	4	-0.0085	-7.49	-0.0507	-44.70
	3	-0.0097	-8.55	-0.0580	-51.14
	2	-0.0097	-8.55	-0.0580	-51.14
	1	0.0000	0.00	0.0000	0.00

iii) Permanent state (Side wall)

$$P_1 = -48.03 \text{ (kN/m}^2\text{)}$$

$$P_2 = -55.79 \text{ (kN/m}^2\text{)}$$

$$L_X = 16.200 \text{ (m)}$$

$$L_Y = 5.050 \text{ (m)}$$

$\lambda = 16.200/5.050 = 3.21$, The coefficient table for $\lambda = 3.25$ is applied.

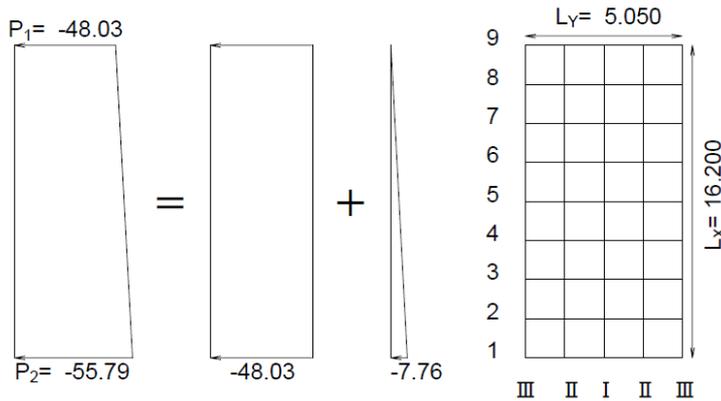


Figure 2.46- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = -48.03 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = -48.03 \times 5.050^2 \times X = -1,224.89 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = -48.03 \times 5.050^2 \times Y = -1,224.89 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.66- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0432	-52.92
	8	0.0064	-7.84	0.0414	-50.71
	7	0.0071	-8.70	0.0416	-50.96
	6	0.0070	-8.57	0.0417	-51.08
	5	0.0070	-8.57	0.0418	-51.20
	4	0.0075	-9.19	0.0417	-51.08
	3	0.0104	-12.74	0.0395	-48.38
	2	0.0128	-15.68	0.0256	-31.36
	1	-0.0565	69.21	-0.0094	11.51
II	9	0.0000	0.00	0.0105	-12.86
	8	0.0014	-1.71	0.0103	-12.62
	7	0.0019	-2.33	0.0104	-12.74
	6	0.0019	-2.33	0.0104	-12.74
	5	0.0017	-2.08	0.0105	-12.86
	4	0.0019	-2.33	0.0106	-12.98
	3	0.0039	-4.78	0.0108	-13.23
	2	0.0067	-8.21	0.0088	-10.78
	1	-0.0338	41.40	-0.0056	6.86

III	9	0.0000	0.00	-0.0872	106.81
	8	-0.0140	17.15	-0.0837	102.52
	7	-0.0136	16.66	-0.0816	99.95
	6	-0.0137	16.78	-0.0823	100.81
	5	-0.0141	17.27	-0.0846	103.63
	4	-0.0141	17.27	-0.0845	103.50
	3	-0.0132	16.17	-0.0791	96.89
	2	-0.0092	11.27	-0.0553	67.74
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$$P = -7.76 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L \cdot Y^2 \cdot X = -7.76 \times 5.050^2 \times X = -197.90 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L^2 \cdot Y = -7.76 \times 5.050^2 \times Y = -197.90 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.67- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0029	-0.57
	8	0.0004	-0.08	0.0057	-1.13
	7	0.0017	-0.34	0.0105	-2.08
	6	0.0026	-0.51	0.0156	-3.09
	5	0.0035	-0.69	0.0209	-4.14
	4	0.0048	-0.95	0.0261	-5.17
	3	0.0085	-1.68	0.0293	-5.80
	2	0.0122	-2.41	0.0212	-4.20
	1	-0.0496	9.82	-0.0083	1.64
II	9	0.0000	0.00	0.0003	-0.06
	8	-0.0001	0.02	0.0013	-0.26
	7	0.0004	-0.08	0.0026	-0.51
	6	0.0006	-0.12	0.0039	-0.77
	5	0.0008	-0.16	0.0052	-1.03
	4	0.0014	-0.28	0.0066	-1.31
	3	0.0034	-0.67	0.0082	-1.62
	2	0.0064	-1.27	0.0077	-1.52
	1	-0.0302	5.98	-0.0050	0.99
III	9	0.0000	0.00	-0.0008	0.16
	8	-0.0019	0.38	-0.0112	2.22
	7	-0.0037	0.73	-0.0224	4.43
	6	-0.0054	1.07	-0.0325	6.43
	5	-0.0071	1.41	-0.0428	8.47
	4	-0.0086	1.70	-0.0516	10.21
	3	-0.0096	1.90	-0.0578	11.44
	2	-0.0080	1.58	-0.0479	9.48
	1	0.0000	0.00	0.0000	0.00

Table 2.68- Summary of Sectional Force (Side wall, Permanent State)

		M_X (kN·m)	M_Y (kN·m)
I	9	0.00	-53.49
	8	-7.92	-51.84
	7	-9.04	-53.04
	6	-9.08	-54.17
	5	-9.26	-55.34
	4	-10.14	-56.25
	3	-14.42	-54.18
	2	-18.09	-35.56
	1	79.03	13.15
II	9	0.00	-12.92
	8	-1.69	-12.88
	7	-2.41	-13.25
	6	-2.45	-13.51
	5	-2.24	-13.89
	4	-2.61	-14.29
	3	-5.45	-14.85
	2	-9.48	-12.30
	1	47.38	7.85
III	9	0.00	106.97
	8	17.53	104.74
	7	17.39	104.38
	6	17.85	107.24
	5	18.68	112.10
	4	18.97	113.71
	3	18.07	108.33
	2	12.85	77.22
	1	0.00	0.00

iv) Permanent state (Front and Rear wall)

$$P_1 = -51.11 \text{ (kN/m}^2\text{)}$$

$$P_2 = -62.69 \text{ (kN/m}^2\text{)}$$

$$L_X = 16.200 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 16.200/3.900 = 4.15$, The coefficient table for $\lambda = 4.25$ is applied.

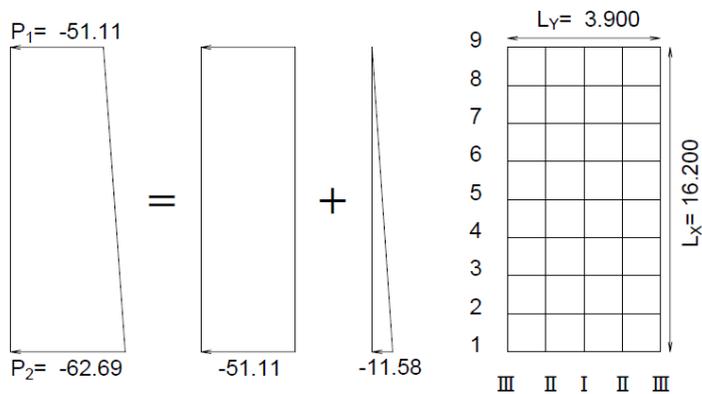


Figure 2.47- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = -51.11 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L^2 \cdot X = -51.11 \times 3.900^2 \times X = -777.38 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L^2 \cdot Y = -51.11 \times 3.900^2 \times Y = -777.38 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.69- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0433	-33.66
	8	0.0068	-5.29	0.0415	-32.26
	7	0.0072	-5.60	0.0416	-32.34
	6	0.0071	-5.52	0.0417	-32.42
	5	0.0069	-5.36	0.0417	-32.42
	4	0.0068	-5.29	0.0418	-32.49
	3	0.0081	-6.30	0.0413	-32.11
	2	0.0134	-10.42	0.0321	-24.95
	1	-0.0560	43.53	-0.0093	7.23
II	9	0.0000	0.00	0.0105	-8.16
	8	0.0016	-1.24	0.0104	-8.08
	7	0.0022	-1.71	0.0105	-8.16
	6	0.0020	-1.55	0.0104	-8.08
	5	0.0015	-1.17	0.0104	-8.08
	4	0.0015	-1.17	0.0104	-8.08
	3	0.0026	-2.02	0.0106	-8.24
	2	0.0065	-5.05	0.0101	-7.85
	1	-0.0325	25.26	-0.0054	4.20
III	9	0.0000	0.00	-0.0891	69.26
	8	-0.0141	10.96	-0.0844	65.61
	7	-0.0135	10.49	-0.0807	62.73
	6	-0.0136	10.57	-0.0816	63.43
	5	-0.0142	11.04	-0.0852	66.23
	4	-0.0141	10.96	-0.0848	65.92
	3	-0.0133	10.34	-0.0801	62.27
	2	-0.0108	8.40	-0.0651	50.61
	1	0.0000	0.00	0.0000	0.00

- Sectional forces induced by triangularly distributed load

$$P = -11.58 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L^2 \cdot X = -11.58 \times 3.900^2 \times X = -176.13 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L^2 \cdot Y = -11.58 \times 3.900^2 \times Y = -176.13 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.70- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0022	-0.39
	8	0.0006	-0.11	0.0054	-0.95
	7	0.0018	-0.32	0.0105	-1.85
	6	0.0027	-0.48	0.0157	-2.77
	5	0.0034	-0.60	0.0209	-3.68

	4	0.0043	-0.76	0.0261	-4.60
	3	0.0064	-1.13	0.0309	-5.44
	2	0.0123	-2.17	0.0274	-4.83
	1	-0.0507	8.93	-0.0085	1.50
II	9	0.0000	0.00	0.0004	-0.07
	8	0.0000	0.00	0.0012	-0.21
	7	0.0003	-0.05	0.0027	-0.48
	6	0.0005	-0.09	0.0039	-0.69
	5	0.0007	-0.12	0.0052	-0.92
	4	0.0011	-0.19	0.0065	-1.14
	3	0.0023	-0.41	0.0080	-1.41
	2	0.0059	-1.04	0.0088	-1.55
	1	-0.0301	5.30	-0.0050	0.88
III	9	0.0000	0.00	0.0018	-0.32
	8	-0.0018	0.32	-0.0108	1.90
	7	-0.0039	0.69	-0.0234	4.12
	6	-0.0056	0.99	-0.0334	5.88
	5	-0.0072	1.27	-0.0434	7.64
	4	-0.0085	1.50	-0.0507	8.93
	3	-0.0097	1.71	-0.0580	10.22
	2	-0.0097	1.71	-0.0580	10.22
	1	0.0000	0.00	0.0000	0.00

Table 2.71- Summary of Sectional Force (Side wall, Permanent State)

		M_x (kN·m)	M_y (kN·m)
I	9	0.00	-34.05
	8	-5.40	-33.21
	7	-5.92	-34.19
	6	-6.00	-35.19
	5	-5.96	-36.10
	4	-6.05	-37.09
	3	-7.43	-37.55
	2	-12.59	-29.78
	1	52.46	8.73
II	9	0.00	-8.23
	8	-1.24	-8.29
	7	-1.76	-8.64
	6	-1.64	-8.77
	5	-1.29	-9.00
	4	-1.36	-9.22
	3	-2.43	-9.65
	2	-6.09	-9.40
	1	30.56	5.08
III	9	0.00	68.94
	8	11.28	67.51
	7	11.18	66.85
	6	11.56	69.31
	5	12.31	73.87
	4	12.46	74.85

	3	12.05	72.49
	2	10.11	60.83
	1	0.00	0.00

v) Corrected moment at floating condition

- Moment correction at corners (III-axis)

No.	Side wall			Unbalanced moment	Front wall		
	Corrected moment	$\Delta M\alpha$	Original moment		Original moment	$\Delta M\beta$	Corrected moment
9	-1.18	1.21	-1.18	2.77	1.59	-1.56	1.59
8	-16.56	3.07	-16.56	7.04	-9.52	-3.97	-16.56
7	-33.12	5.45	-33.12	12.49	-20.63	-7.04	-33.12
6	-48.05	8.11	-48.05	18.60	-29.45	-10.49	-48.05
5	-63.27	10.90	-63.27	25.00	-38.27	-14.10	-63.27
4	-76.28	13.77	-76.28	31.58	-44.70	-17.81	-76.28
3	-85.45	14.96	-85.45	34.31	-51.14	-19.35	-85.45
2	-70.81	8.58	-70.81	19.67	-51.14	-11.09	-70.81
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	$1/2 \cdot \Delta M\alpha$	Original moment		Original moment	$1/2 \cdot \Delta M\beta$	Corrected moment
9	4.89	0.60	4.29	2.77	1.94	-0.78	1.94
8	9.96	1.53	8.43	7.04	4.76	-1.99	4.76
7	18.24	2.72	15.52	12.49	9.26	-3.52	9.26
6	27.11	4.05	23.06	18.60	13.84	-5.25	13.84
5	36.35	5.45	30.90	25.00	18.43	-7.05	18.43
4	45.47	6.88	38.59	31.58	23.01	-8.91	23.01
3	50.80	7.48	43.32	34.31	27.25	-9.68	27.25
2	35.63	4.29	31.34	19.67	24.16	-5.55	24.16
1	-12.27	0.00	-12.27	0.00	-7.49	0.00	-7.49

vii) Corrected moment at permanent state

- Moment correction at corners (III-axis)

No.	Side wall			Unbalanced moment	Front wall		
	Corrected moment	$\Delta M\alpha$	Original moment		Original moment	$\Delta M\beta$	Corrected moment
9	106.97	-16.58	106.97	38.03	68.94	21.45	90.39
8	104.74	-16.23	104.74	37.23	67.51	21.00	88.51
7	104.38	-16.36	104.38	37.53	66.85	21.17	88.02
6	107.24	-16.54	107.24	37.93	69.31	21.39	90.70
5	112.10	-16.67	112.10	38.23	73.87	21.56	95.43
4	113.71	-16.94	113.71	38.86	74.85	21.92	96.77
3	108.33	-15.63	108.33	35.84	72.49	20.21	92.70
2	77.22	-7.15	77.22	16.39	60.83	9.24	70.07
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	-61.78	-8.29	-53.49	38.03	-34.05	10.72	-34.05
8	-59.96	-8.12	-51.84	37.23	-33.21	10.50	-33.21
7	-61.22	-8.18	-53.04	37.53	-34.19	10.58	-34.19
6	-62.44	-8.27	-54.17	37.93	-35.19	10.70	-35.19
5	-63.67	-8.33	-55.34	38.23	-36.10	10.78	-36.10
4	-64.72	-8.47	-56.25	38.86	-37.09	10.96	-37.09
3	-61.99	-7.81	-54.18	35.84	-37.55	10.11	-37.55
2	-39.13	-3.57	-35.56	16.39	-29.78	4.62	-29.78
1	13.15	0.00	13.15	0.00	8.73	0.00	8.73

5) Serviceability Verification of Partition Wall

The sectional force of partition wall is estimated by the model of slabs fixed on three sides and free on one side according to constraint conditions and making calculations based on numerical tables. The sign convention for bending moments is as follows:

- Positive (+) when tension occurs on the inner side,
- Negative (-) when tension occurs on the outer side.

i) Transverse direction

$$P_1 = 5.05 \text{ (kN/m}^2\text{)}$$

$$P_2 = 5.05 \text{ (kN/m}^2\text{)}$$

$$L_X = 16.200 \text{ (m)}$$

$$L_Y = 5.050 \text{ (m)}$$

$\lambda = 16.200/5.050 = 3.21$, The coefficient table for $\lambda = 3.25$ is applied.

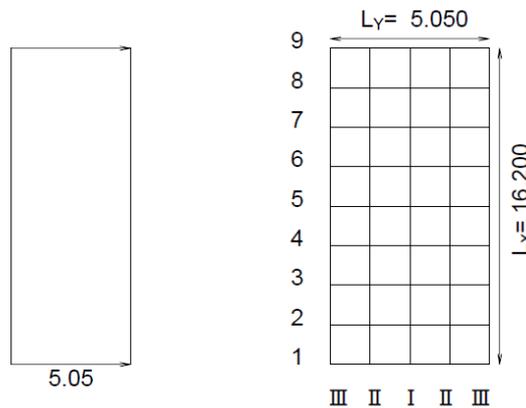


Figure 2.48- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = 5.05 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L_Y^2 \cdot X = 5.05 \times 5.050^2 \times X = 128.79 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L_Y^2 \cdot Y = 5.05 \times 5.050^2 \times Y = 128.79 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.72- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0432	5.56
	8	0.0064	0.82	0.0414	5.33
	7	0.0071	0.91	0.0416	5.36
	6	0.0070	0.90	0.0417	5.37
	5	0.0070	0.90	0.0418	5.38
	4	0.0075	0.97	0.0417	5.37
	3	0.0104	1.34	0.0395	5.09
	2	0.0128	1.65	0.0256	3.30
	1	-0.0565	-7.28	-0.0094	-1.21
II	9	0.0000	0.00	0.0105	1.35
	8	0.0014	0.18	0.0103	1.33
	7	0.0019	0.24	0.0104	1.34
	6	0.0019	0.24	0.0104	1.34
	5	0.0017	0.22	0.0105	1.35
	4	0.0019	0.24	0.0106	1.37
	3	0.0039	0.50	0.0108	1.39
	2	0.0067	0.86	0.0088	1.13
	1	-0.0338	-4.35	-0.0056	-0.72
III	9	0.0000	0.00	-0.0872	-11.23
	8	-0.0140	-1.80	-0.0837	-10.78
	7	-0.0136	-1.75	-0.0816	-10.51
	6	-0.0137	-1.76	-0.0823	-10.60
	5	-0.0141	-1.82	-0.0846	-10.90
	4	-0.0141	-1.82	-0.0845	-10.88
	3	-0.0132	-1.70	-0.0791	-10.19
	2	-0.0092	-1.18	-0.0553	-7.12
	1	0.0000	0.00	0.0000	0.00

ii) Longitudinal direction

$$P_1 = 5.05 \text{ (kN/m}^2\text{)}$$

$$P_2 = 5.05 \text{ (kN/m}^2\text{)}$$

$$L_X = 16.200 \text{ (m)}$$

$$L_Y = 3.900 \text{ (m)}$$

$\lambda = 16.200/3.900 = 4.15$, The coefficient table for $\lambda = 4.25$ is applied.

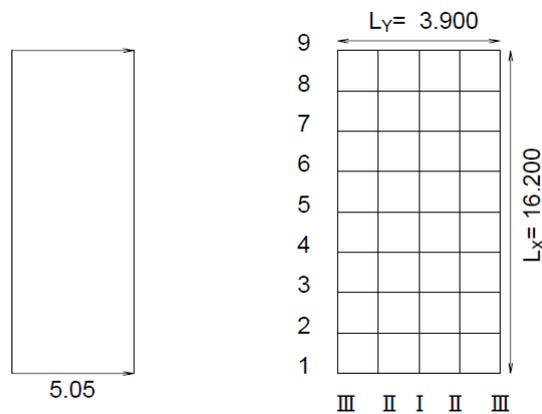


Figure 2.49- Design Model for Sectional Force Estimation

- Sectional forces induced by uniformly distributed load

$$P = 5.05 \text{ (kN/m}^2\text{)}$$

$$M_X = P \cdot L^2 \cdot X = 5.05 \times 3.900^2 \times X = 76.81 \times X \text{ (kN}\cdot\text{m)}$$

$$M_Y = P \cdot L^2 \cdot Y = 5.05 \times 3.900^2 \times Y = 76.81 \times Y \text{ (kN}\cdot\text{m)}$$

Table 2.73- Coefficient and Sectional Force

		Coefficient, X	M_X (kN·m)	Coefficient, Y	M_Y (kN·m)
I	9	0.0000	0.00	0.0433	3.33
	8	0.0068	0.52	0.0415	3.19
	7	0.0072	0.55	0.0416	3.20
	6	0.0071	0.55	0.0417	3.20
	5	0.0069	0.53	0.0417	3.20
	4	0.0068	0.52	0.0418	3.21
	3	0.0081	0.62	0.0413	3.17
	2	0.0134	1.03	0.0321	2.47
	1	-0.0560	-4.30	-0.0093	-0.71
II	9	0.0000	0.00	0.0105	0.81
	8	0.0016	0.12	0.0104	0.80
	7	0.0022	0.17	0.0105	0.81
	6	0.0020	0.15	0.0104	0.80
	5	0.0015	0.12	0.0104	0.80
	4	0.0015	0.12	0.0104	0.80
	3	0.0026	0.20	0.0106	0.81
	2	0.0065	0.50	0.0101	0.78
	1	-0.0325	-2.50	-0.0054	-0.41
III	9	0.0000	0.00	-0.0891	-6.84
	8	-0.0141	-1.08	-0.0844	-6.48
	7	-0.0135	-1.04	-0.0807	-6.20
	6	-0.0136	-1.04	-0.0816	-6.27
	5	-0.0142	-1.09	-0.0852	-6.54
	4	-0.0141	-1.08	-0.0848	-6.51
	3	-0.0133	-1.02	-0.0801	-6.15
	2	-0.0108	-0.83	-0.0651	-5.00
	1	0.0000	0.00	0.0000	0.00

(5) 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 (1.19).

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

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

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

Where:

- A_s : area of tension reinforcement (mm^2)
- p_w : reinforcing bar ratio ($= A_s / (b_w \cdot d)$)
- f'_{cd} : design compressive strength of concrete
 $f'_{cd} = f'_{ck} / \gamma_c = 30.0 / 1.30 = 23.1 \text{ (N/mm}^2\text{)}$
- f_{yd} : design yield strength of tensile reinforcement
 $f_{yd} = f_{yk} / \gamma_s = 345.0 / 1.00 = 345 \text{ (N/mm}^2\text{)}$
- 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.

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

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/mm^2). 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

Table 2.74- 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.

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

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 (Permanent state / Room A / Transverse direction / Upper reinforcement)

$B=100\text{cm}$

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	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	4	16.32	52.0	1.00	D13	20.0	6.34	102.291	1.00	0.16
	3	31.63	52.0	1.95	D13	20.0	6.34	102.291	1.00	0.31
	2	23.57	52.0	1.45	D13	20.0	6.34	102.291	1.00	0.23
	1	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
II	5	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	4	8.89	52.0	0.55	D13	20.0	6.34	102.291	1.00	0.09
	3	16.30	52.0	1.00	D13	20.0	6.34	102.291	1.00	0.16
	2	13.58	52.0	0.83	D13	20.0	6.34	102.291	1.00	0.13
	1	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
III	5	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	4	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	3	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	2	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	1	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00

ii) Safety verification (Level 1 earthquake ground motion / Room A / Transverse direction / Upper reinforcement)

$B=100\text{cm}$

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	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	4	8.85	52.0	0.54	D13	20.0	6.34	102.291	1.00	0.09
	3	33.35	52.0	2.05	D13	20.0	6.34	102.291	1.00	0.33
	2	33.34	52.0	2.05	D13	20.0	6.34	102.291	1.00	0.33
	1	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
II	5	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	4	3.95	52.0	0.24	D13	20.0	6.34	102.291	1.00	0.04
	3	17.34	52.0	1.07	D13	20.0	6.34	102.291	1.00	0.17
	2	19.80	52.0	1.22	D13	20.0	6.34	102.291	1.00	0.19
	1	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
III	5	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	4	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	3	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	2	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	1	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00

iii) Crack width of serviceability verification (Permanent state / Room A / Transverse direction / Upper reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	0.00	52.0	D13	20.0	6.34	0.000	0.0040 × 7.0 = 0.0280	0.16	
	4	5.10	52.0	D13	20.0	6.34	16.132		0.0082	0.29
	3	13.31	52.0	D13	20.0	6.34	42.102		0.0140	0.50
	2	11.69	52.0	D13	20.0	6.34	36.978		0.0129	0.46
	1	0.00	52.0	D13	20.0	6.34	0.000		0.0045	0.16
II	5	0.00	52.0	D13	20.0	6.34	0.000	0.0040 × 7.0 = 0.0280	0.16	
	4	2.60	52.0	D13	20.0	6.34	8.224		0.0064	0.23
	3	6.89	52.0	D13	20.0	6.34	21.794		0.0095	0.34
	2	6.86	52.0	D13	20.0	6.34	21.699		0.0094	0.34
	1	0.00	52.0	D13	20.0	6.34	0.000		0.0045	0.16
III	5	0.00	52.0	D13	20.0	6.34	0.000	0.0040 × 7.0 = 0.0280	0.16	
	4	0.00	52.0	D13	20.0	6.34	0.000		0.0045	0.16
	3	0.00	52.0	D13	20.0	6.34	0.000		0.0045	0.16
	2	0.00	52.0	D13	20.0	6.34	0.000		0.0045	0.16
	1	0.00	52.0	D13	20.0	6.34	0.000		0.0045	0.16

iv) Compression stress of serviceability verification (Permanent state / Room A / Transverse direction / Upper reinforcement)

$\gamma_f=1.00$, $N=7.1$, $B=100\text{cm}$

NO.	M_s (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (N/mm ²)	$0.4 \cdot f_{ck}$ (N/mm ²)
I	5	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	4	5.10	52.0	D13	20.0	6.34	0.959	0.32 ≤ 12.00
	3	13.31	52.0	D13	20.0	6.34	0.959	0.83 ≤ 12.00
	2	11.69	52.0	D13	20.0	6.34	0.959	0.73 ≤ 12.00
	1	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
II	5	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	4	2.60	52.0	D13	20.0	6.34	0.959	0.16 ≤ 12.00
	3	6.89	52.0	D13	20.0	6.34	0.959	0.43 ≤ 12.00
	2	6.86	52.0	D13	20.0	6.34	0.959	0.43 ≤ 12.00
	1	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
III	5	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	4	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	3	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	2	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	1	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00

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

$B=100\text{cm}$

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	84.35	56.6	4.79	D13	20.0	6.34	111.439	1.00	0.76
	4	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	3	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	2	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	1	102.94	56.6	5.85	D16	20.0	9.93	173.565	1.00	0.59
II	5	50.71	56.6	2.87	D13	20.0	6.34	111.439	1.00	0.46
	4	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	3	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	2	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	1	64.27	56.6	3.64	D13	20.0	6.34	111.439	1.00	0.58
III	5	0.00	56.6	0.00	D13	20.0	6.34	111.439	—	0.00
	4	11.38	56.6	0.64	D13	20.0	6.34	111.439	1.00	0.10
	3	18.65	56.6	1.05	D13	20.0	6.34	111.439	1.00	0.17
	2	13.43	56.6	0.76	D13	20.0	6.34	111.439	1.00	0.12
	1	0.00	56.6	0.00	D13	20.0	6.34	111.439	—	0.00

v) Safety verification (Level 1 earthquake ground motion / Room A / Transverse direction / Lower reinforcement)

$B=100\text{cm}$

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	67.51	56.6	3.83	D13	20.0	6.34	111.439	1.00	0.61
	4	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	3	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	2	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	1	130.33	56.6	7.43	D16	20.0	9.93	173.565	1.00	0.75
II	5	37.88	56.6	2.14	D13	20.0	6.34	111.439	1.00	0.34
	4	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	3	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	2	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	1	83.70	56.6	4.75	D13	20.0	6.34	111.439	1.00	0.75
III	5	0.00	56.6	0.00	D13	20.0	6.34	111.439	—	0.00
	4	9.66	56.6	0.54	D13	20.0	6.34	111.439	1.00	0.09
	3	19.82	56.6	1.12	D13	20.0	6.34	111.439	1.00	0.18
	2	16.58	56.6	0.94	D13	20.0	6.34	111.439	1.00	0.15
	1	0.00	56.6	0.00	D13	20.0	6.34	111.439	—	0.00

vi) Crack width of serviceability verification (Permanent state / Room A / Transverse direction / Lower reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	30.97	56.6	D13	20.0	6.34	89.851	0.0297	0.0035 × 9.0 = 0.0315	0.94
	4	0.00	50.0	D13	20.0	6.34	0.000	0.0054		0.17
	3	0.00	50.0	D13	20.0	6.34	0.000	0.0054		0.17
	2	0.00	50.0	D13	20.0	6.34	0.000	0.0054		0.17
	1	47.87	56.6	D16	20.0	9.93	89.520	0.0294		0.93
II	5	18.05	56.6	D13	20.0	6.34	52.367	0.0195	0.0035 × 9.0 = 0.0315	0.62
	4	0.00	50.0	D13	20.0	6.34	0.000	0.0054		0.17
	3	0.00	50.0	D13	20.0	6.34	0.000	0.0054		0.17
	2	0.00	50.0	D13	20.0	6.34	0.000	0.0054		0.17
	1	30.38	56.6	D13	20.0	6.34	88.140	0.0292		0.93
III	5	0.00	56.6	D13	20.0	6.34	0.000	0.0054	0.0035 × 9.0 = 0.0315	0.17
	4	4.29	56.6	D13	20.0	6.34	12.446	0.0088		0.28
	3	7.88	56.6	D13	20.0	6.34	22.862	0.0116		0.37
	2	6.15	56.6	D13	20.0	6.34	17.843	0.0102		0.32
	1	0.00	56.6	D13	20.0	6.34	0.000	0.0054		0.17

vii) Compression stress of serviceability verification (Permanent state / Room A / Transverse direction / Lower reinforcement)

$\gamma_f=1.00$, $N=7.1$, $B=100\text{cm}$

NO.	M_s (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (N/mm ²)	$0.4 \cdot f_{ck}$ (N/mm ²)
I	5	30.97	56.6	D13	20.0	6.34	0.961	1.70 ≤ 12.00
	4	0.00	50.0	D13	20.0	6.34	0.958	0.00 ≤ 12.00
	3	0.00	50.0	D13	20.0	6.34	0.958	0.00 ≤ 12.00
	2	0.00	50.0	D13	20.0	6.34	0.958	0.00 ≤ 12.00
	1	47.87	56.6	D16	20.0	9.93	0.951	2.16 ≤ 12.00
II	5	18.05	56.6	D13	20.0	6.34	0.961	0.99 ≤ 12.00
	4	0.00	50.0	D13	20.0	6.34	0.958	0.00 ≤ 12.00
	3	0.00	50.0	D13	20.0	6.34	0.958	0.00 ≤ 12.00
	2	0.00	50.0	D13	20.0	6.34	0.958	0.00 ≤ 12.00
	1	30.38	56.6	D13	20.0	6.34	0.961	1.67 ≤ 12.00
III	5	0.00	56.6	D13	20.0	6.34	0.961	0.00 ≤ 12.00
	4	4.29	56.6	D13	20.0	6.34	0.961	0.24 ≤ 12.00
	3	7.88	56.6	D13	20.0	6.34	0.961	0.43 ≤ 12.00
	2	6.15	56.6	D13	20.0	6.34	0.961	0.34 ≤ 12.00
	1	0.00	56.6	D13	20.0	6.34	0.961	0.00 ≤ 12.00

viii) Safety verification (Permanent state / Room A / Longitudinal direction / Upper reinforcement)

$B=100\text{cm}$

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	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	4	28.59	54.0	1.69	D13	20.0	6.34	106.273	1.00	0.27
	3	49.49	54.0	2.94	D13	20.0	6.34	106.273	1.00	0.47
	2	32.17	54.0	1.91	D13	20.0	6.34	106.273	1.00	0.30
	1	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
II	5	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	4	11.04	54.0	0.65	D13	20.0	6.34	106.273	1.00	0.10
	3	18.11	54.0	1.07	D13	20.0	6.34	106.273	1.00	0.17
	2	13.77	54.0	0.81	D13	20.0	6.34	106.273	1.00	0.13
	1	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
III	5	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	4	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	3	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	2	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	1	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00

ix) Safety verification (Level 1 earthquake ground motion / Room A / Longitudinal direction / Upper reinforcement)

$B=100\text{cm}$

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	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	4	26.17	54.0	1.55	D13	20.0	6.34	106.273	1.00	0.25
	3	52.43	54.0	3.11	D13	20.0	6.34	106.273	1.00	0.49
	2	38.28	54.0	2.27	D13	20.0	6.34	106.273	1.00	0.36
	1	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
II	5	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	4	8.51	54.0	0.50	D13	20.0	6.34	106.273	1.00	0.08
	3	19.14	54.0	1.13	D13	20.0	6.34	106.273	1.00	0.18
	2	17.73	54.0	1.05	D13	20.0	6.34	106.273	1.00	0.17
	1	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
III	5	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	4	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	3	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	2	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	1	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00

x) Crack width of serviceability verification (Permanent state / Room A / Longitudinal direction / Upper reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	0.00	54.0	D13	20.0	6.34	0.000	0.0040 × 5.0 = 0.0200	0.18	
	4	11.19	54.0	D13	20.0	6.34	34.057		0.0098	0.49
	3	20.87	54.0	D13	20.0	6.34	63.518		0.0152	0.76
	2	14.44	54.0	D13	20.0	6.34	43.949		0.0116	0.58
	1	0.00	54.0	D13	20.0	6.34	0.000		0.0036	0.18
II	5	0.00	54.0	D13	20.0	6.34	0.000	0.0040 × 5.0 = 0.0200	0.18	
	4	3.98	54.0	D13	20.0	6.34	12.113		0.0058	0.29
	3	7.62	54.0	D13	20.0	6.34	23.192		0.0079	0.40
	2	6.46	54.0	D13	20.0	6.34	19.661		0.0072	0.36
	1	0.00	54.0	D13	20.0	6.34	0.000		0.0036	0.18
III	5	0.00	54.0	D13	20.0	6.34	0.000	0.0040 × 5.0 = 0.0200	0.18	
	4	0.00	54.0	D13	20.0	6.34	0.000		0.0036	0.18
	3	0.00	54.0	D13	20.0	6.34	0.000		0.0036	0.18
	2	0.00	54.0	D13	20.0	6.34	0.000		0.0036	0.18
	1	0.00	54.0	D13	20.0	6.34	0.000		0.0036	0.18

xi) Compression stress of serviceability verification (Permanent state / Room A / Longitudinal direction / Upper reinforcement)

$\gamma_f=1.00$, $N=7.1$, $B=100\text{cm}$

NO.	M_s (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (N/mm ²)	$0.4 \cdot f_{ck}$ (N/mm ²)
I	5	0.00	54.0	D13	20.0	6.34	0.960	0.00 ≤ 12.00
	4	11.19	54.0	D13	20.0	6.34	0.960	0.66 ≤ 12.00
	3	20.87	54.0	D13	20.0	6.34	0.960	1.23 ≤ 12.00
	2	14.44	54.0	D13	20.0	6.34	0.960	0.85 ≤ 12.00
	1	0.00	54.0	D13	20.0	6.34	0.960	0.00 ≤ 12.00
II	5	0.00	54.0	D13	20.0	6.34	0.960	0.00 ≤ 12.00
	4	3.98	54.0	D13	20.0	6.34	0.960	0.24 ≤ 12.00
	3	7.62	54.0	D13	20.0	6.34	0.960	0.45 ≤ 12.00
	2	6.46	54.0	D13	20.0	6.34	0.960	0.38 ≤ 12.00
	1	0.00	54.0	D13	20.0	6.34	0.960	0.00 ≤ 12.00
III	5	0.00	54.0	D13	20.0	6.34	0.960	0.00 ≤ 12.00
	4	0.00	54.0	D13	20.0	6.34	0.960	0.00 ≤ 12.00
	3	0.00	54.0	D13	20.0	6.34	0.960	0.00 ≤ 12.00
	2	0.00	54.0	D13	20.0	6.34	0.960	0.00 ≤ 12.00
	1	0.00	54.0	D13	20.0	6.34	0.960	0.00 ≤ 12.00

xii) Safety verification (Permanent state / Room A / Longitudinal direction / Lower reinforcement)

$B=100\text{cm}$

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	14.01	58.6	0.76	D13	20.0	6.34	115.418	1.00	0.12
	4	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	3	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	2	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	1	17.08	58.6	0.93	D13	20.0	6.34	115.418	1.00	0.15
II	5	8.40	58.6	0.46	D13	20.0	6.34	115.418	1.00	0.07
	4	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	3	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	2	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	1	10.70	58.6	0.58	D13	20.0	6.34	115.418	1.00	0.09
III	5	0.00	58.6	0.00	D13	20.0	6.34	115.418	—	0.00
	4	68.11	58.6	3.73	D13	20.0	6.34	115.418	1.00	0.59
	3	111.28	58.6	6.11	D16	20.0	9.93	179.795	1.00	0.62
	2	80.04	58.6	4.38	D16	20.0	9.93	179.795	1.00	0.45*
	1	0.00	58.6	0.00	D13	20.0	6.34	115.418	—	0.00

*: reinforcement is determined by serviceability verification

xiii) Safety verification (Level 1 earthquake ground motion / Room A / Longitudinal direction / Lower reinforcement)

$B=100\text{cm}$

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	11.15	58.6	0.61	D13	20.0	6.34	115.418	1.00	0.10
	4	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	3	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	2	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	1	21.52	58.6	1.17	D13	20.0	6.34	115.418	1.00	0.19
II	5	6.21	58.6	0.34	D13	20.0	6.34	115.418	1.00	0.05
	4	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	3	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	2	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	1	13.99	58.6	0.76	D13	20.0	6.34	115.418	1.00	0.12
III	5	0.00	58.6	0.00	D13	20.0	6.34	115.418	—	0.00
	4	58.16	58.6	3.18	D13	20.0	6.34	115.418	1.00	0.50
	3	117.68	58.6	6.47	D16	20.0	9.93	179.795	1.00	0.65
	2	98.51	58.6	5.40	D16	20.0	9.93	179.795	1.00	0.55*
	1	0.00	58.6	0.00	D13	20.0	6.34	115.418	—	0.00

*: reinforcement is determined by serviceability verification

xiv) Crack width of serviceability verification (Permanent state / Room A / Longitudinal direction / Lower reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	5.13	58.6	D13	20.0	6.34	14.365	0.0035 × 7.0 = 0.0245	0.32	
	4	0.00	52.0	D13	20.0	6.34	0.000		0.0045	0.18
	3	0.00	52.0	D13	20.0	6.34	0.000		0.0045	0.18
	2	0.00	52.0	D13	20.0	6.34	0.000		0.0045	0.18
	1	7.92	58.6	D13	20.0	6.34	22.178		0.0095	0.39
II	5	2.97	58.6	D13	20.0	6.34	8.317	0.0035 × 7.0 = 0.0245	0.26	
	4	0.00	52.0	D13	20.0	6.34	0.000		0.0045	0.18
	3	0.00	52.0	D13	20.0	6.34	0.000		0.0045	0.18
	2	0.00	52.0	D13	20.0	6.34	0.000		0.0045	0.18
	1	5.07	58.6	D13	20.0	6.34	14.197		0.0077	0.31
III	5	0.00	58.6	D13	20.0	6.34	0.000	0.0035 × 7.0 = 0.0245	0.18	
	4	25.77	58.6	D13	20.0	6.34	72.163		0.0208	0.85
	3	46.88	58.6	D16	20.0	9.93	84.607		0.0235	0.96
	2	36.63	58.6	D16	20.0	9.93	66.109		0.0194	0.79
	1	0.00	58.6	D13	20.0	6.34	0.000		0.0045	0.18

xv) Compression stress of serviceability verification (Permanent state / Room A / Longitudinal direction / Lower reinforcement)

$\gamma_f=1.00$, $N=7.1$, $B=100\text{cm}$

NO.	M_s (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (N/mm ²)	$0.4 \cdot f_{ck}$ (N/mm ²)
I	5	5.13	58.6	D13	20.0	6.34	0.961	0.27 ≤ 12.00
	4	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	3	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	2	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	1	7.92	58.6	D13	20.0	6.34	0.961	0.41 ≤ 12.00
II	5	2.97	58.6	D13	20.0	6.34	0.961	0.15 ≤ 12.00
	4	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	3	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	2	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	1	5.07	58.6	D13	20.0	6.34	0.961	0.26 ≤ 12.00
III	5	0.00	58.6	D13	20.0	6.34	0.961	0.00 ≤ 12.00
	4	25.77	58.6	D13	20.0	6.34	0.961	1.34 ≤ 12.00
	3	46.88	58.6	D16	20.0	9.93	0.952	2.00 ≤ 12.00
	2	36.63	58.6	D16	20.0	9.93	0.952	1.56 ≤ 12.00
	1	0.00	58.6	D13	20.0	6.34	0.961	0.00 ≤ 12.00

4) Bottom Slab (Room B)

i) Safety verification (Permanent state / Room B / Transverse direction / Upper reinforcement)

$B=100\text{cm}$

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	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	4	16.91	52.0	1.14	D13	20.0	6.34	102.291	1.10	0.18
	3	26.86	52.0	1.82	D13	20.0	6.34	102.291	1.10	0.29
	2	16.91	52.0	1.14	D13	20.0	6.34	102.291	1.10	0.18
	1	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
II	5	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	4	9.52	52.0	0.64	D13	20.0	6.34	102.291	1.10	0.10
	3	13.78	52.0	0.93	D13	20.0	6.34	102.291	1.10	0.15
	2	9.52	52.0	0.64	D13	20.0	6.34	102.291	1.10	0.10
	1	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
III	5	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	4	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	3	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	2	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	1	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00

ii) Safety verification (Level 1 earthquake ground motion / Room B / Transverse direction / Upper reinforcement)

$B=100\text{cm}$

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	99.20	52.0	6.15	D13	20.0	6.34	102.291	1.00	0.97
	4	0.56	52.0	0.03	D13	20.0	6.34	102.291	1.00	0.01
	3	3.11	52.0	0.19	D13	20.0	6.34	102.291	1.00	0.03
	2	3.38	52.0	0.21	D13	20.0	6.34	102.291	1.00	0.03
	1	43.61	52.0	2.69	D13	20.0	6.34	102.291	1.00	0.43
II	5	64.01	52.0	3.95	D13	20.0	6.34	102.291	1.00	0.63
	4	0.20	52.0	0.01	D13	20.0	6.34	102.291	1.00	0.00
	3	1.62	52.0	0.10	D13	20.0	6.34	102.291	1.00	0.02
	2	2.02	52.0	0.12	D13	20.0	6.34	102.291	1.00	0.02
	1	23.46	52.0	1.44	D13	20.0	6.34	102.291	1.00	0.23
III	5	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	4	12.50	52.0	0.77	D13	20.0	6.34	102.291	1.00	0.12
	3	14.03	52.0	0.86	D13	20.0	6.34	102.291	1.00	0.14
	2	6.38	52.0	0.39	D13	20.0	6.34	102.291	1.00	0.06
	1	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00

iii) Crack width of serviceability verification (Permanent state / Room B / Transverse direction / Upper reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	17.77	52.0	D13	20.0	6.34	56.210	0.0040 × 7.0 = 0.0280	0.61
	4	7.00	52.0	D13	20.0	6.34	22.142		0.34
	3	11.12	52.0	D13	20.0	6.34	35.175		0.45
	2	7.00	52.0	D13	20.0	6.34	22.142		0.34
	1	7.81	52.0	D13	20.0	6.34	24.704		0.36
II	5	11.47	52.0	D13	20.0	6.34	36.282	0.0040 × 7.0 = 0.0280	0.45
	4	3.94	52.0	D13	20.0	6.34	12.463		0.26
	3	5.71	52.0	D13	20.0	6.34	18.062		0.31
	2	3.94	52.0	D13	20.0	6.34	12.463		0.26
	1	4.20	52.0	D13	20.0	6.34	13.285		0.27
III	5	0.00	52.0	D13	20.0	6.34	0.000	0.0040 × 7.0 = 0.0280	0.16
	4	2.24	52.0	D13	20.0	6.34	7.086		0.22
	3	2.51	52.0	D13	20.0	6.34	7.940		0.23
	2	1.14	52.0	D13	20.0	6.34	3.606		0.19
	1	0.00	52.0	D13	20.0	6.34	0.000		0.16

iv) Compression stress of serviceability verification (Permanent state / Room B / Transverse direction / Upper reinforcement)

$\gamma_f=1.00$, $N=7.1$, $B=100\text{cm}$

NO.	M_s (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (N/mm ²)	$0.4 \cdot f_{ck}$ (N/mm ²)
I	5	17.77	52.0	D13	20.0	6.34	0.959	1.11 ≤ 12.00
	4	0.54	52.0	D13	20.0	6.34	0.959	0.03 ≤ 12.00
	3	3.00	52.0	D13	20.0	6.34	0.959	0.19 ≤ 12.00
	2	3.25	52.0	D13	20.0	6.34	0.959	0.20 ≤ 12.00
	1	7.81	52.0	D13	20.0	6.34	0.959	0.49 ≤ 12.00
II	5	11.47	52.0	D13	20.0	6.34	0.959	0.72 ≤ 12.00
	4	0.19	52.0	D13	20.0	6.34	0.959	0.01 ≤ 12.00
	3	1.56	52.0	D13	20.0	6.34	0.959	0.10 ≤ 12.00
	2	1.94	52.0	D13	20.0	6.34	0.959	0.12 ≤ 12.00
	1	4.20	52.0	D13	20.0	6.34	0.959	0.26 ≤ 12.00
III	5	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00
	4	2.24	52.0	D13	20.0	6.34	0.959	0.14 ≤ 12.00
	3	2.51	52.0	D13	20.0	6.34	0.959	0.16 ≤ 12.00
	2	1.14	52.0	D13	20.0	6.34	0.959	0.07 ≤ 12.00
	1	0.00	52.0	D13	20.0	6.34	0.959	0.00 ≤ 12.00

iv) Safety verification (Permanent state / Room B / Transverse direction / Lower reinforcement)

$B=100\text{cm}$

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	79.44	56.6	4.96	D13	20.0	6.34	111.439	1.10	0.78
	4	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	3	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	2	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	1	79.44	56.6	4.96	D13	20.0	6.34	111.439	1.10	0.78
II	5	48.74	56.6	3.03	D13	20.0	6.34	111.439	1.10	0.48
	4	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	3	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	2	0.00	50.0	0.00	D13	20.0	6.34	98.313	—	0.00
	1	48.74	56.6	3.03	D13	20.0	6.34	111.439	1.10	0.48
III	5	0.00	56.6	0.00	D13	20.0	6.34	111.439	—	0.00
	4	10.52	56.6	0.65	D13	20.0	6.34	111.439	1.10	0.10
	3	15.77	56.6	0.98	D13	20.0	6.34	111.439	1.10	0.16
	2	10.52	56.6	0.65	D13	20.0	6.34	111.439	1.10	0.10
	1	0.00	56.6	0.00	D13	20.0	6.34	111.439	—	0.00

v) Safety verification (Level 1 earthquake ground motion / Room B / Transverse direction / Lower reinforcement)

$B=100\text{cm}$

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	5.64	56.6	0.32	D13	20.0	6.34	111.439	1.00	0.05
	4	26.01	50.0	1.66	D13	20.0	6.34	98.313	1.00	0.26
	3	24.23	50.0	1.55	D13	20.0	6.34	98.313	1.00	0.25
	2	4.34	50.0	0.28	D13	20.0	6.34	98.313	1.00	0.04
	1	12.86	56.6	0.73	D13	20.0	6.34	111.439	1.00	0.12
II	5	3.05	56.6	0.17	D13	20.0	6.34	111.439	1.00	0.03
	4	15.56	50.0	0.99	D13	20.0	6.34	98.313	1.00	0.16
	3	12.24	50.0	0.78	D13	20.0	6.34	98.313	1.00	0.12
	2	1.53	50.0	0.10	D13	20.0	6.34	98.313	1.00	0.02
	1	8.32	56.6	0.47	D13	20.0	6.34	111.439	1.00	0.07
III	5	0.00	56.6	0.00	D13	20.0	6.34	111.439	—	0.00
	4	0.83	56.6	0.05	D13	20.0	6.34	111.439	1.00	0.01
	3	1.86	56.6	0.10	D13	20.0	6.34	111.439	1.00	0.02
	2	1.62	56.6	0.09	D13	20.0	6.34	111.439	1.00	0.01
	1	0.00	56.6	0.00	D13	20.0	6.34	111.439	—	0.00

vi) Crack width of serviceability verification (Permanent state / Room B / Transverse direction / Lower reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	32.90	56.6	D13	20.0	6.34	95.451	0.0312	0.0035 × 9.0 = 0.0315	0.99
	4	4.66	50.0	D13	20.0	6.34	15.343	0.0095		0.30
	3	4.34	50.0	D13	20.0	6.34	14.289	0.0093		0.30
	2	0.78	50.0	D13	20.0	6.34	2.568	0.0061		0.19
	1	32.90	56.6	D13	20.0	6.34	95.451	0.0312		0.99
II	5	20.18	56.6	D13	20.0	6.34	58.547	0.0212	0.0035 × 9.0 = 0.0315	0.67
	4	2.79	50.0	D13	20.0	6.34	9.186	0.0079		0.25
	3	2.19	50.0	D13	20.0	6.34	7.210	0.0073		0.23
	2	0.27	50.0	D13	20.0	6.34	0.889	0.0056		0.18
	1	20.18	56.6	D13	20.0	6.34	58.547	0.0212		0.67
III	5	0.00	56.6	D13	20.0	6.34	0.000	0.0054	0.0035 × 9.0 = 0.0315	0.17
	4	4.35	56.6	D13	20.0	6.34	12.620	0.0088		0.28
	3	6.53	56.6	D13	20.0	6.34	18.945	0.0105		0.33
	2	4.35	56.6	D13	20.0	6.34	12.620	0.0088		0.28
	1	0.00	56.6	D13	20.0	6.34	0.000	0.0054		0.17

vii) Compression stress of serviceability verification (Permanent state / Room B / Transverse direction / Lower reinforcement)

$\gamma_f=1.00$, $N=7.1$, $B=100\text{cm}$

NO.	M_s (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (N/mm ²)	$0.4 \cdot f_{ck}$ (N/mm ²)
I	5	5.42	56.6	D13	20.0	6.34	0.961	0.30 ≤ 12.00
	4	4.66	50.0	D13	20.0	6.34	0.958	0.31 ≤ 12.00
	3	4.34	50.0	D13	20.0	6.34	0.958	0.29 ≤ 12.00
	2	0.78	50.0	D13	20.0	6.34	0.958	0.05 ≤ 12.00
	1	12.36	56.6	D13	20.0	6.34	0.961	0.68 ≤ 12.00
II	5	2.93	56.6	D13	20.0	6.34	0.961	0.16 ≤ 12.00
	4	2.79	50.0	D13	20.0	6.34	0.958	0.19 ≤ 12.00
	3	2.19	50.0	D13	20.0	6.34	0.958	0.15 ≤ 12.00
	2	0.27	50.0	D13	20.0	6.34	0.958	0.02 ≤ 12.00
	1	7.99	56.6	D13	20.0	6.34	0.961	0.44 ≤ 12.00
III	5	0.00	56.6	D13	20.0	6.34	0.961	0.00 ≤ 12.00
	4	0.80	56.6	D13	20.0	6.34	0.961	0.04 ≤ 12.00
	3	1.78	56.6	D13	20.0	6.34	0.961	0.10 ≤ 12.00
	2	1.56	56.6	D13	20.0	6.34	0.961	0.09 ≤ 12.00
	1	0.00	56.6	D13	20.0	6.34	0.961	0.00 ≤ 12.00

viii) Safety verification (Permanent state / Room B / Longitudinal direction / Upper reinforcement)

$B=100\text{cm}$

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	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	4	25.72	54.0	1.68	D13	20.0	6.34	106.273	1.10	0.27
	3	41.92	54.0	2.73	D13	20.0	6.34	106.273	1.10	0.43
	2	25.72	54.0	1.68	D13	20.0	6.34	106.273	1.10	0.27
	1	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
II	5	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	4	10.52	54.0	0.68	D13	20.0	6.34	106.273	1.10	0.11
	3	15.35	54.0	1.00	D13	20.0	6.34	106.273	1.10	0.16
	2	10.52	54.0	0.68	D13	20.0	6.34	106.273	1.10	0.11
	1	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
III	5	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	4	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	3	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	2	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	1	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00

ix) Safety verification (Level 1 earthquake ground motion / Room B / Longitudinal direction / Upper reinforcement)

$B=100\text{cm}$

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	16.58	54.0	0.98	D13	20.0	6.34	106.273	1.00	0.16
	4	2.32	54.0	0.14	D13	20.0	6.34	106.273	1.00	0.02
	3	4.91	54.0	0.29	D13	20.0	6.34	106.273	1.00	0.05
	2	3.71	54.0	0.22	D13	20.0	6.34	106.273	1.00	0.03
	1	7.40	54.0	0.44	D13	20.0	6.34	106.273	1.00	0.07
II	5	10.71	54.0	0.63	D13	20.0	6.34	106.273	1.00	0.10
	4	0.69	54.0	0.04	D13	20.0	6.34	106.273	1.00	0.01
	3	1.79	54.0	0.11	D13	20.0	6.34	106.273	1.00	0.02
	2	1.75	54.0	0.10	D13	20.0	6.34	106.273	1.00	0.02
	1	3.83	54.0	0.23	D13	20.0	6.34	106.273	1.00	0.04
III	5	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00
	4	74.21	54.0	4.41	D13	20.0	6.34	106.273	1.00	0.70
	3	84.66	54.0	5.04	D13	20.0	6.34	106.273	1.00	0.80
	2	38.51	54.0	2.28	D13	20.0	6.34	106.273	1.00	0.36
	1	0.00	54.0	0.00	D13	20.0	6.34	106.273	—	0.00

x) Crack width of serviceability verification (Permanent state / Room B / Longitudinal direction / Upper reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	2.97	54.0	D13	20.0	6.34	9.039	0.0053	0.0040 × 5.0 = 0.0200	0.27
	4	10.65	54.0	D13	20.0	6.34	32.414	0.0095		0.48
	3	17.36	54.0	D13	20.0	6.34	52.836	0.0133		0.67
	2	10.65	54.0	D13	20.0	6.34	32.414	0.0095		0.48
	1	1.33	54.0	D13	20.0	6.34	4.048	0.0044		0.22
II	5	1.92	54.0	D13	20.0	6.34	5.844	0.0047	0.0040 × 5.0 = 0.0200	0.24
	4	4.35	54.0	D13	20.0	6.34	13.239	0.0061		0.31
	3	6.36	54.0	D13	20.0	6.34	19.357	0.0072		0.36
	2	4.35	54.0	D13	20.0	6.34	13.239	0.0061		0.31
	1	0.69	54.0	D13	20.0	6.34	2.100	0.0040		0.20
III	5	0.00	54.0	D13	20.0	6.34	0.000	0.0036	0.0040 × 5.0 = 0.0200	0.18
	4	13.30	54.0	D13	20.0	6.34	40.479	0.0110		0.55
	3	15.17	54.0	D13	20.0	6.34	46.170	0.0121		0.61
	2	6.90	54.0	D13	20.0	6.34	21.000	0.0075		0.38
	1	0.00	54.0	D13	20.0	6.34	0.000	0.0036		0.18

xi) Compression stress of serviceability verification (Permanent state / Room B / Longitudinal direction / Upper reinforcement)

$\gamma_f=1.00$, $N=7.1$, $B=100\text{cm}$

NO.	M_s (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (N/mm ²)	$0.4 \cdot f_{ck}$ (N/mm ²)
I	5	2.97	54.0	D13	20.0	6.34	0.960	0.18 ≤ 12.00
	4	2.23	54.0	D13	20.0	6.34	0.960	0.13 ≤ 12.00
	3	4.71	54.0	D13	20.0	6.34	0.960	0.28 ≤ 12.00
	2	3.56	54.0	D13	20.0	6.34	0.960	0.21 ≤ 12.00
	1	1.33	54.0	D13	20.0	6.34	0.960	0.08 ≤ 12.00
II	5	1.92	54.0	D13	20.0	6.34	0.960	0.11 ≤ 12.00
	4	0.67	54.0	D13	20.0	6.34	0.960	0.04 ≤ 12.00
	3	1.72	54.0	D13	20.0	6.34	0.960	0.10 ≤ 12.00
	2	1.69	54.0	D13	20.0	6.34	0.960	0.10 ≤ 12.00
	1	0.69	54.0	D13	20.0	6.34	0.960	0.04 ≤ 12.00
III	5	0.00	54.0	D13	20.0	6.34	0.960	0.00 ≤ 12.00
	4	13.30	54.0	D13	20.0	6.34	0.960	0.79 ≤ 12.00
	3	15.17	54.0	D13	20.0	6.34	0.960	0.90 ≤ 12.00
	2	6.90	54.0	D13	20.0	6.34	0.960	0.41 ≤ 12.00
	1	0.00	54.0	D13	20.0	6.34	0.960	0.00 ≤ 12.00

xii) Safety verification (Permanent state / Room B / Longitudinal direction / Lower reinforcement)

$B=100\text{cm}$

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	13.22	58.6	0.79	D13	20.0	6.34	115.418	1.10	0.13
	4	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	3	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	2	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	1	13.22	58.6	0.79	D13	20.0	6.34	115.418	1.10	0.13
II	5	8.10	58.6	0.49	D13	20.0	6.34	115.418	1.10	0.08
	4	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	3	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	2	0.00	52.0	0.00	D13	20.0	6.34	102.291	—	0.00
	1	8.10	58.6	0.49	D13	20.0	6.34	115.418	1.10	0.08
III	5	0.00	58.6	0.00	D13	20.0	6.34	115.418	—	0.00
	4	62.81	58.6	3.78	D13	20.0	6.34	115.418	1.10	0.60
	3	94.36	58.6	5.70	D16	20.0	9.93	179.795	1.10	0.58*
	2	62.81	58.6	3.78	D13	20.0	6.34	115.418	1.10	0.60
	1	0.00	58.6	0.00	D13	20.0	6.34	115.418	—	0.00

*: reinforcement is determined by serviceability verification

xiii) Safety verification (Level 1 earthquake ground motion / Room B / Longitudinal direction / Lower reinforcement)

$B=100\text{cm}$

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	0.93	58.6	0.05	D13	20.0	6.34	115.418	1.00	0.01
	4	28.31	52.0	1.74	D13	20.0	6.34	102.291	1.00	0.28
	3	37.49	52.0	2.31	D13	20.0	6.34	102.291	1.00	0.37
	2	17.60	52.0	1.08	D13	20.0	6.34	102.291	1.00	0.17
	1	2.12	58.6	0.12	D13	20.0	6.34	115.418	1.00	0.02
II	5	0.50	58.6	0.03	D13	20.0	6.34	115.418	1.00	0.00
	4	13.52	52.0	0.83	D13	20.0	6.34	102.291	1.00	0.13
	3	13.77	52.0	0.85	D13	20.0	6.34	102.291	1.00	0.13
	2	5.36	52.0	0.33	D13	20.0	6.34	102.291	1.00	0.05
	1	1.39	58.6	0.08	D13	20.0	6.34	115.418	1.00	0.01
III	5	0.00	58.6	0.00	D13	20.0	6.34	115.418	—	0.00
	4	5.01	58.6	0.27	D13	20.0	6.34	115.418	1.00	0.04
	3	11.01	58.6	0.60	D16	20.0	9.93	179.795	1.00	0.06*
	2	9.65	58.6	0.53	D13	20.0	6.34	115.418	1.00	0.08
	1	0.00	58.6	0.00	D13	20.0	6.34	115.418	—	0.00

*: reinforcement is determined by serviceability verification

xiv) Crack width of serviceability verification (Permanent state / Room B/ Longitudinal direction / Lower reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	5.47	58.6	D13	20.0	6.34	15.317	0.0035 × 7.0 = 0.0245	0.33
	4	5.07	52.0	D13	20.0	6.34	16.037		0.33
	3	6.72	52.0	D13	20.0	6.34	21.257		0.38
	2	3.15	52.0	D13	20.0	6.34	9.964		0.28
	1	5.47	58.6	D13	20.0	6.34	15.317		0.33
II	5	3.35	58.6	D13	20.0	6.34	9.381	0.0035 × 7.0 = 0.0245	0.27
	4	2.42	52.0	D13	20.0	6.34	7.655		0.26
	3	2.47	52.0	D13	20.0	6.34	7.813		0.26
	2	0.96	52.0	D13	20.0	6.34	3.037		0.21
	1	3.35	58.6	D13	20.0	6.34	9.381		0.27
III	5	0.00	58.6	D13	20.0	6.34	0.000	0.0035 × 7.0 = 0.0245	0.18
	4	26.01	58.6	D13	20.0	6.34	72.835		0.86
	3	39.07	58.6	D16	20.0	9.93	70.512		0.83
	2	26.01	58.6	D13	20.0	6.34	72.835		0.86
	1	0.00	58.6	D13	20.0	6.34	0.000		0.18

xv) Compression stress of serviceability verification (Permanent state / Room B / Longitudinal direction / Lower reinforcement)

$\gamma_f=1.00$, $N=7.1$, $B=100\text{cm}$

NO.	M_s (kN·m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (N/mm ²)	$0.4 \cdot f_{ck}$ (N/mm ²)
I	5	0.89	58.6	D13	20.0	6.34	0.961	0.05 ≤ 12.00
	4	5.07	52.0	D13	20.0	6.34	0.959	0.32 ≤ 12.00
	3	6.72	52.0	D13	20.0	6.34	0.959	0.42 ≤ 12.00
	2	3.15	52.0	D13	20.0	6.34	0.959	0.20 ≤ 12.00
	1	2.04	58.6	D13	20.0	6.34	0.961	0.11 ≤ 12.00
II	5	0.47	58.6	D13	20.0	6.34	0.961	0.02 ≤ 12.00
	4	2.42	52.0	D13	20.0	6.34	0.959	0.15 ≤ 12.00
	3	2.47	52.0	D13	20.0	6.34	0.959	0.15 ≤ 12.00
	2	0.96	52.0	D13	20.0	6.34	0.959	0.06 ≤ 12.00
	1	1.33	58.6	D13	20.0	6.34	0.961	0.07 ≤ 12.00
III	5	0.00	58.6	D13	20.0	6.34	0.961	0.00 ≤ 12.00
	4	4.81	58.6	D13	20.0	6.34	0.961	0.25 ≤ 12.00
	3	10.57	58.6	D16	20.0	9.93	0.952	0.45 ≤ 12.00
	2	9.26	58.6	D13	20.0	6.34	0.961	0.48 ≤ 12.00
	1	0.00	58.6	D13	20.0	6.34	0.961	0.00 ≤ 12.00

5) Footing (Seaside)

i) Safety verification on bending moment (Permanent state)

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	$\gamma_i \cdot M_d$ $/M_{ud}$
Upper	0.00	92.0	0.00	D13	20.0	6.34	181.829	—	0.00
Lower	324.90	98.6	11.68	D19, D19	10.0	28.65	863.338	1.10	0.41*

$B=100\text{cm}$, *: reinforcement is determined by serviceability verification

ii) Safety verification on bending moment (Level 1 earthquake ground motion)

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	$\gamma_i \cdot M_d$ $/M_{ud}$
Upper	0.00	92.0	0.00	D13	20.0	6.34	181.829	—	0.00
Lower	486.31	98.6	15.95	D19, D19	10.0	28.65	863.338	1.10	0.56*

$B=100\text{cm}$, *: reinforcement is determined by serviceability verification

iii) Crack width of serviceability verification (Permanent state)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

NO.	M_s (kN·m)	d c (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
Upper	0.00	92.0 7.0	D13	20.0	6.34	0.00	0.0045	$0.0035 \times$ 7.0	0.18
Lower	292.52	98.6 7.0	D19, D19	10.0	28.65	110.304	0.0241	= 0.0245	0.98

iv) Compression stress of serviceability verification (Permanent state)

$\gamma_r=1.00$, $N=7.1$, $B=100\text{cm}$

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)
Upper	0.00	92.0	D13	20.0	6.34	0.969	0.00	≤ 12.00
Lower	292.52	98.6	D19, D19	10.0	28.65	0.939	3.49	≤ 12.00

v) Safety verification on shear force (Permanent state)

NO.	V_d (kN/m)	d (cm)	γ_i	γ_b	β_d	β_p	β_a	f_{cd} (N/mm ²)	V_{cd} (kN/m)	$\gamma_i \cdot V_d$ $/V_{cd}$
Upper	0.00	92.0	1.10	1.30	1.02106	0.63134	5.00000	0.913	2082.57	0.00
Lower	429.03	98.6	1.10	1.30	1.00353	0.76972	3.14487	0.913	1682.17	0.28

v) Safety verification on shear force (Level 1 earthquake ground motion)

NO.	V_d (kN/m)	d (cm)	γ_i	γ_b	β_d	β_p	β_a	f_{cd} (N/mm ²)	V_{cd} (kN/m)	$\gamma_i \cdot V_d$ $/V_{cd}$
Upper	0.00	92.0	1.00	1.30	1.02106	0.63134	5.00000	0.913	2082.57	0.00
Lower	634.34	98.6	1.00	1.30	1.00353	0.76972	3.11615	0.913	1666.81	0.38

vi) Serviceability verification on shear force (Permanent state)

NO.	V_d (kN/m)	d (cm)	γ_i	γ_b	β_d	β_p	β_a	f_{cd} (N/mm ²)	V_{cd} (kN/m)	$\gamma_i \cdot V_d$ $/V_{cd}$
Upper	0.00	92.0	—	1.00	1.02106	0.63134	5.00000	1.041	3086.90	0.00
Lower	386.24	98.6	—	1.00	1.00353	0.76972	3.14468	1.041	2493.26	0.15

6) Outer Wall (Side Wall)

i) Safety verification (Permanent state / Side wall / Room A / Lateral reinforcement / Inner reinforcement)

$B=100\text{cm}$

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	10.76	34.0	1.11	D13	20.0	6.34	66.503	1.10	0.18
	8	21.91	34.0	2.27	D13	20.0	6.34	66.503	1.10	0.36
	7	40.14	34.0	4.19	D13	20.0	6.34	66.503	1.10	0.66
	6	59.65	34.0	6.25	D16	20.0	9.93	103.173	1.10	0.64*
	5	79.96	34.0	8.43	D13, D13	10.0	12.67	130.681	1.10	0.67*
	4	100.02	34.0	10.61	D13, D13	10.0	12.67	130.681	1.10	0.84
	3	111.74	34.0	11.89	D13, D16	10.0	16.27	166.196	1.10	0.74*
	2	78.37	34.0	8.26	D13, D13	10.0	12.67	130.681	1.10	0.66*
	1	14.47	34.0	1.36	D13	20.0	6.34	66.503	1.00	0.22
II	9	0.98	34.0	0.10	D13	20.0	6.34	66.503	1.10	0.02
	8	4.23	34.0	0.44	D13	20.0	6.34	66.503	1.10	0.07
	7	8.46	34.0	0.87	D13	20.0	6.34	66.503	1.10	0.14
	6	12.68	34.0	1.31	D13	20.0	6.34	66.503	1.10	0.21
	5	16.91	34.0	1.75	D13	20.0	6.34	66.503	1.10	0.28
	4	21.46	34.0	2.23	D13	20.0	6.34	66.503	1.10	0.35
	3	26.67	34.0	2.77	D13	20.0	6.34	66.503	1.10	0.44
	2	25.04	34.0	2.60	D13	20.0	6.34	66.503	1.10	0.41
	1	8.63	34.0	0.81	D13	20.0	6.34	66.503	1.00	0.13
III	9	117.65	34.0	11.37	D19, D19	10.0	28.65	282.887	1.00	0.42*
	8	115.21	34.0	11.12	D19, D19	10.0	28.65	282.887	1.00	0.41*
	7	114.82	34.0	11.08	D19, D19	10.0	28.65	282.887	1.00	0.41*
	6	117.96	34.0	11.40	D16, D22	10.0	29.29	288.712	1.00	0.41*
	5	123.30	34.0	11.93	D13, D25	10.0	31.67	310.095	1.00	0.40*
	4	125.09	34.0	12.11	D13, D25	10.0	31.67	310.095	1.00	0.40*
	3	119.16	34.0	11.52	D13, D25	10.0	31.67	310.095	1.00	0.38*
	2	84.94	34.0	8.14	D16, D19	10.0	24.26	242.472	1.00	0.35*
	1	0.00	34.0	0.00	D13	20.0	6.34	66.503	-	0.00
III'	9	117.65	34.0	11.37	D19, D19	10.0	28.65	282.887	1.00	0.42*
	8	115.21	34.0	11.12	D19, D19	10.0	28.65	282.887	1.00	0.41*
	7	114.82	34.0	11.08	D19, D19	10.0	28.65	282.887	1.00	0.41*
	6	117.96	34.0	11.40	D16, D22	10.0	29.29	288.712	1.00	0.41*
	5	123.30	34.0	11.93	D13, D25	10.0	31.67	310.095	1.00	0.40*
	4	125.09	34.0	12.11	D13, D25	10.0	31.67	310.095	1.00	0.40*
	3	119.16	34.0	11.52	D13, D25	10.0	31.67	310.095	1.00	0.38*
	2	84.94	34.0	8.14	D16, D19	10.0	24.26	242.472	1.00	0.35*
	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 (Permanent state / Side wall / Room A / Lateral reinforcement / Inner reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	4.89	34.0	D13	20.0	6.34	23.878	0.0080	0.0040 × 5.0 = 0.0200	0.40
	8	9.96	34.0	D13	20.0	6.34	48.635	0.0125		0.63
	7	18.24	34.0	D13	20.0	6.34	89.066	0.0199		1.00
	6	27.11	34.0	D16	20.0	9.93	85.543	0.0191		0.96
	5	36.35	34.0	D13, D13	10.0	12.67	90.576	0.0159		0.80
	4	45.47	34.0	D13, D13	10.0	12.67	113.301	0.0191		0.96
	3	50.80	34.0	D13, D16	10.0	16.27	99.422	0.0172		0.86
	2	35.63	34.0	D13, D13	10.0	12.67	88.782	0.0156		0.78
	1	13.15	34.0	D13	20.0	6.34	64.211	0.0153		0.77
II	9	0.44	34.0	D13	20.0	6.34	2.149	0.0040	0.0040 × 5.0 = 0.0200	0.20
	8	1.92	34.0	D13	20.0	6.34	9.375	0.0053		0.27
	7	3.84	34.0	D13	20.0	6.34	18.751	0.0071		0.36
	6	5.77	34.0	D13	20.0	6.34	28.175	0.0088		0.44
	5	7.69	34.0	D13	20.0	6.34	37.550	0.0105		0.53
	4	9.76	34.0	D13	20.0	6.34	47.658	0.0123		0.62
	3	12.12	34.0	D13	20.0	6.34	59.182	0.0144		0.72
	2	11.38	34.0	D13	20.0	6.34	55.568	0.0138		0.69
	1	7.85	34.0	D13	20.0	6.34	38.331	0.0106		0.53
III	9	106.97	34.0	D19, D19	10.0	28.65	121.623	0.0200	0.0040 × 5.0 = 0.0200	1.00
	8	104.74	34.0	D19, D19	10.0	28.65	119.087	0.0196		0.98
	7	104.38	34.0	D19, D19	10.0	28.65	118.678	0.0196		0.98
	6	107.24	34.0	D16, D22	10.0	29.29	119.378	0.0198		0.99
	5	112.10	34.0	D13, D25	10.0	31.67	115.822	0.0195		0.98
	4	113.71	34.0	D13, D25	10.0	31.67	117.486	0.0197		0.99
	3	108.33	34.0	D13, D25	10.0	31.67	111.927	0.0189		0.95
	2	77.22	34.0	D16, D19	10.0	24.26	102.943	0.0175		0.88
	1	0.00	34.0	D13	20.0	6.34	0.000	0.0036		0.18
III'	9	106.97	34.0	D19, D19	10.0	28.65	121.623	0.0200	0.0040 × 5.0 = 0.0200	1.00
	8	104.74	34.0	D19, D19	10.0	28.65	119.087	0.0196		0.98
	7	104.38	34.0	D19, D19	10.0	28.65	118.678	0.0196		0.98
	6	107.24	34.0	D16, D22	10.0	29.29	119.378	0.0198		0.99
	5	112.10	34.0	D13, D25	10.0	31.67	115.822	0.0195		0.98
	4	113.71	34.0	D13, D25	10.0	31.67	117.486	0.0197		0.99
	3	108.33	34.0	D13, D25	10.0	31.67	111.927	0.0189		0.95
	2	77.22	34.0	D16, D19	10.0	24.26	102.943	0.0175		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 (Permanent state / Side wall / Room A / Lateral reinforcement / Inner reinforcement)

$$\gamma_f=1.00, N=7.1, B=100\text{cm}$$

NO.	M_s (kN.m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (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	D16	20.0	9.93	0.939	0.00 ≤ 12.00
	5	0.00	34.0	D13, D13	10.0	12.67	0.932	0.00 ≤ 12.00
	4	0.00	34.0	D13, D13	10.0	12.67	0.932	0.00 ≤ 12.00
	3	0.00	34.0	D13, D16	10.0	16.27	0.924	0.00 ≤ 12.00
	2	0.00	34.0	D13, D13	10.0	12.67	0.932	0.00 ≤ 12.00
	1	13.15	34.0	D13	20.0	6.34	0.950	1.60 ≤ 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	7.85	34.0	D13	20.0	6.34	0.950	0.95 ≤ 12.00
III	9	106.97	34.0	D19, D19	10.0	28.65	0.903	7.04 ≤ 12.00
	8	104.74	34.0	D19, D19	10.0	28.65	0.903	6.89 ≤ 12.00
	7	104.38	34.0	D19, D19	10.0	28.65	0.903	6.87 ≤ 12.00
	6	107.24	34.0	D16, D22	10.0	29.29	0.902	7.00 ≤ 12.00
	5	112.10	34.0	D13, D25	10.0	31.67	0.899	7.11 ≤ 12.00
	4	113.71	34.0	D13, D25	10.0	31.67	0.899	7.21 ≤ 12.00
	3	108.33	34.0	D13, D25	10.0	31.67	0.899	6.87 ≤ 12.00
	2	77.22	34.0	D16, D19	10.0	24.26	0.909	5.41 ≤ 12.00
	1	0.00	34.0	D13	20.0	6.34	0.950	0.00 ≤ 12.00
III'	9	106.97	34.0	D19, D19	10.0	28.65	0.903	7.04 ≤ 12.00
	8	104.74	34.0	D19, D19	10.0	28.65	0.903	6.89 ≤ 12.00
	7	104.38	34.0	D19, D19	10.0	28.65	0.903	6.87 ≤ 12.00
	6	107.24	34.0	D16, D22	10.0	29.29	0.902	7.00 ≤ 12.00
	5	112.10	34.0	D13, D25	10.0	31.67	0.899	7.11 ≤ 12.00
	4	113.71	34.0	D13, D25	10.0	31.67	0.899	7.21 ≤ 12.00
	3	108.33	34.0	D13, D25	10.0	31.67	0.899	6.87 ≤ 12.00
	2	77.22	34.0	D16, D19	10.0	24.26	0.909	5.41 ≤ 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 (Permanent state / Side wall / Room A / 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	67.94	32.0	6.90	D16, D16	10.0	19.86	188.448	1.00	0.36*
	8	65.94	32.0	6.69	D16, D16	10.0	19.86	188.448	1.00	0.35*
	7	67.34	32.0	6.84	D16, D16	10.0	19.86	188.448	1.00	0.36*
	6	68.67	32.0	6.98	D16, D16	10.0	19.86	188.448	1.00	0.36*
	5	70.03	32.0	7.12	D16, D16	10.0	19.86	188.448	1.00	0.37*
	4	71.18	32.0	7.24	D16, D16	10.0	19.86	188.448	1.00	0.38*
	3	68.19	32.0	6.93	D16, D16	10.0	19.86	188.448	1.00	0.36*
	2	43.04	32.0	4.34	D13, D13	10.0	12.67	122.737	1.00	0.35*
	1	26.99	38.6	2.47	D13	20.0	6.34	75.648	1.10	0.39
II	9	14.22	32.0	1.42	D13	20.0	6.34	62.524	1.10	0.23
	8	14.16	32.0	1.42	D13	20.0	6.34	62.524	1.10	0.23
	7	14.58	32.0	1.46	D13	20.0	6.34	62.524	1.10	0.23
	6	14.86	32.0	1.49	D13	20.0	6.34	62.524	1.10	0.24
	5	15.28	32.0	1.53	D13	20.0	6.34	62.524	1.10	0.24
	4	15.72	32.0	1.57	D13	20.0	6.34	62.524	1.10	0.25
	3	16.34	32.0	1.64	D13	20.0	6.34	62.524	1.10	0.26
	2	13.54	32.0	1.35	D13	20.0	6.34	62.524	1.10	0.22
	1	16.26	38.6	1.48	D13	20.0	6.34	75.648	1.00	0.24
III	9	2.60	38.6	0.24	D13	20.0	6.34	75.648	1.00	0.04
	8	36.42	38.6	3.33	D13	20.0	6.34	75.648	1.00	0.53
	7	72.85	38.6	6.72	D13, D13	10.0	12.67	148.968	1.00	0.54*
	6	105.69	38.6	9.82	D13, D13	10.0	12.67	148.968	1.00	0.78*
	5	139.19	38.6	13.03	D13, D16	10.0	16.27	189.668	1.00	0.81*
	4	167.81	38.6	15.82	D16, D16	10.0	19.86	229.554	1.00	0.80*
	3	187.97	38.6	17.80	D16, D19	10.0	24.26	277.496	1.00	0.75*
	2	155.77	38.6	14.64	D16, D16	10.0	19.86	229.554	1.00	0.75*
	1	0.00	38.6	0.00	D13	20.0	6.34	75.648	-	0.00
III'	9	2.60	38.6	0.24	D13	20.0	6.34	75.648	1.00	0.04
	8	36.42	38.6	3.33	D13	20.0	6.34	75.648	1.00	0.53
	7	72.85	38.6	6.72	D13, D13	10.0	12.67	148.968	1.00	0.54*
	6	105.69	38.6	9.82	D13, D13	10.0	12.67	148.968	1.00	0.78*
	5	139.19	38.6	13.03	D13, D16	10.0	16.27	189.668	1.00	0.81*
	4	167.81	38.6	15.82	D16, D16	10.0	19.86	229.554	1.00	0.80*
	3	187.97	38.6	17.80	D16, D19	10.0	24.26	277.496	1.00	0.75*
	2	155.77	38.6	14.64	D16, D16	10.0	19.86	229.554	1.00	0.75*
	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 (Permanent state / Side wall / Room A / Lateral reinforcement / Outer reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	61.78	32.0	D16, D16	10.0	19.86	106.285	0.0235	0.0035 × 7.0 = 0.0245	0.96
	8	59.96	32.0	D16, D16	10.0	19.86	103.154	0.0230		0.94
	7	61.22	32.0	D16, D16	10.0	19.86	105.322	0.0234		0.96
	6	62.44	32.0	D16, D16	10.0	19.86	107.421	0.0237		0.97
	5	63.67	32.0	D16, D16	10.0	19.86	109.537	0.0241		0.98
	4	64.72	32.0	D16, D16	10.0	19.86	111.343	0.0245		1.00
	3	61.99	32.0	D16, D16	10.0	19.86	106.647	0.0236		0.96
	2	39.13	32.0	D13, D13	10.0	12.67	103.802	0.0232		0.95
	1	12.27	38.6	D13	20.0	6.34	52.618	0.0164		0.67
II	9	12.92	32.0	D13	20.0	6.34	67.134	0.0197	0.0035 × 7.0 = 0.0245	0.80
	8	12.88	32.0	D13	20.0	6.34	66.926	0.0197		0.80
	7	13.25	32.0	D13	20.0	6.34	68.848	0.0201		0.82
	6	13.51	32.0	D13	20.0	6.34	70.199	0.0204		0.83
	5	13.89	32.0	D13	20.0	6.34	72.174	0.0208		0.85
	4	14.29	32.0	D13	20.0	6.34	74.252	0.0213		0.87
	3	14.85	32.0	D13	20.0	6.34	77.162	0.0220		0.90
	2	12.30	32.0	D13	20.0	6.34	63.912	0.0190		0.78
	1	7.39	38.6	D13	20.0	6.34	31.691	0.0117		0.48
III	9	1.18	38.6	D13	20.0	6.34	5.060	0.0057	0.0035 × 7.0 = 0.0245	0.23
	8	16.56	38.6	D13	20.0	6.34	71.015	0.0206		0.84
	7	33.12	38.6	D13, D13	10.0	12.67	72.398	0.0173		0.71
	6	48.05	38.6	D13, D13	10.0	12.67	105.033	0.0235		0.96
	5	63.27	38.6	D13, D16	10.0	16.27	108.587	0.0241		0.98
	4	76.28	38.6	D16, D16	10.0	19.86	108.013	0.0239		0.98
	3	85.45	38.6	D16, D19	10.0	24.26	99.812	0.0223		0.91
	2	70.81	38.6	D16, D16	10.0	19.86	100.267	0.0224		0.91
	1	0.00	38.6	D13	20.0	6.34	0.000	0.0045		0.18
III'	9	1.18	38.6	D13	20.0	6.34	5.060	0.0057	0.0035 × 7.0 = 0.0245	0.23
	8	16.56	38.6	D13	20.0	6.34	71.015	0.0206		0.84
	7	33.12	38.6	D13, D13	10.0	12.67	72.398	0.0173		0.71
	6	48.05	38.6	D13, D13	10.0	12.67	105.033	0.0235		0.96
	5	63.27	38.6	D13, D16	10.0	16.27	108.587	0.0241		0.98
	4	76.28	38.6	D16, D16	10.0	19.86	108.013	0.0239		0.98
	3	85.45	38.6	D16, D19	10.0	24.26	99.812	0.0223		0.91
	2	70.81	38.6	D16, D16	10.0	19.86	100.267	0.0224		0.91
	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 (Permanent state / Side wall / Room A / Lateral reinforcement / Outer reinforcement)

$\gamma_f=1.00, N=7.1, B=100\text{cm}$

NO.	M_s (kN.m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (N/mm ²)	$0.4 \cdot f_{ck}$ (N/mm ²)
I'	9	61.78	32.0	D16, D16	10.0	19.86	0.915	5.15 ≤ 12.00
	8	59.96	32.0	D16, D16	10.0	19.86	0.915	5.00 ≤ 12.00
	7	61.22	32.0	D16, D16	10.0	19.86	0.915	5.10 ≤ 12.00
	6	62.44	32.0	D16, D16	10.0	19.86	0.915	5.20 ≤ 12.00
	5	63.67	32.0	D16, D16	10.0	19.86	0.915	5.31 ≤ 12.00
	4	64.72	32.0	D16, D16	10.0	19.86	0.915	5.39 ≤ 12.00
	3	61.99	32.0	D16, D16	10.0	19.86	0.915	5.17 ≤ 12.00
	2	39.13	32.0	D13, D13	10.0	12.67	0.930	3.90 ≤ 12.00
	1	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
II	9	12.92	32.0	D13	20.0	6.34	0.949	1.72 ≤ 12.00
	8	12.88	32.0	D13	20.0	6.34	0.949	1.72 ≤ 12.00
	7	13.25	32.0	D13	20.0	6.34	0.949	1.77 ≤ 12.00
	6	13.51	32.0	D13	20.0	6.34	0.949	1.80 ≤ 12.00
	5	13.89	32.0	D13	20.0	6.34	0.949	1.85 ≤ 12.00
	4	14.29	32.0	D13	20.0	6.34	0.949	1.91 ≤ 12.00
	3	14.85	32.0	D13	20.0	6.34	0.949	1.98 ≤ 12.00
	2	12.30	32.0	D13	20.0	6.34	0.949	1.64 ≤ 12.00
	1	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
III	9	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
	8	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
	7	0.00	38.6	D13, D13	10.0	12.67	0.935	0.00 ≤ 12.00
	6	0.00	38.6	D13, D13	10.0	12.67	0.935	0.00 ≤ 12.00
	5	0.00	38.6	D13, D16	10.0	16.27	0.928	0.00 ≤ 12.00
	4	0.00	38.6	D16, D16	10.0	19.86	0.921	0.00 ≤ 12.00
	3	0.00	38.6	D16, D19	10.0	24.26	0.914	0.00 ≤ 12.00
	2	0.00	38.6	D16, D16	10.0	19.86	0.921	0.00 ≤ 12.00
	1	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
III'	9	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
	8	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
	7	0.00	38.6	D13, D13	10.0	12.67	0.935	0.00 ≤ 12.00
	6	0.00	38.6	D13, D13	10.0	12.67	0.935	0.00 ≤ 12.00
	5	0.00	38.6	D13, D16	10.0	16.27	0.928	0.00 ≤ 12.00
	4	0.00	38.6	D16, D16	10.0	19.86	0.921	0.00 ≤ 12.00
	3	0.00	38.6	D16, D19	10.0	24.26	0.914	0.00 ≤ 12.00
	2	0.00	38.6	D16, D16	10.0	19.86	0.921	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 (Permanent state / Side wall / Room A / Vertical reinforcement / Inner reinforcement)

$B=100\text{cm}$

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	1.30	32.0	0.14	D13	20.0	6.34	62.524	1.10	0.02
	7	5.53	32.0	0.61	D13	20.0	6.34	62.524	1.10	0.10
	6	8.46	32.0	0.93	D13	20.0	6.34	62.524	1.10	0.15
	5	11.38	32.0	1.25	D13	20.0	6.34	62.524	1.10	0.20
	4	15.61	32.0	1.72	D13	20.0	6.34	62.524	1.10	0.27
	3	27.64	32.0	3.06	D13	20.0	6.34	62.524	1.10	0.49
	2	39.68	32.0	4.40	D13	20.0	6.34	62.524	1.10	0.70
	1	86.92	32.0	8.88	D16, D19	10.0	24.26	227.268	1.00	0.38*
II	9	0.00	32.0	0.00	D13	20.0	6.34	62.524	—	0.00
	8	0.00	32.0	0.00	D13	20.0	6.34	62.524	—	0.00
	7	1.30	32.0	0.14	D13	20.0	6.34	62.524	1.10	0.02
	6	1.95	32.0	0.21	D13	20.0	6.34	62.524	1.10	0.03
	5	2.60	32.0	0.29	D13	20.0	6.34	62.524	1.10	0.05
	4	4.55	32.0	0.50	D13	20.0	6.34	62.524	1.10	0.08
	3	11.06	32.0	1.22	D13	20.0	6.34	62.524	1.10	0.19
	2	20.81	32.0	2.30	D13	20.0	6.34	62.524	1.10	0.37
	1	52.12	32.0	5.27	D13, D13	10.0	12.67	122.737	1.00	0.42*
III	9	0.00	32.0	0.00	D13	20.0	6.34	62.524	—	0.00
	8	19.27	32.0	1.93	D13	20.0	6.34	62.524	1.00	0.31
	7	19.13	32.0	1.92	D13	20.0	6.34	62.524	1.00	0.31
	6	19.64	32.0	1.97	D13	20.0	6.34	62.524	1.00	0.31
	5	20.55	32.0	2.06	D13	20.0	6.34	62.524	1.00	0.33
	4	20.87	32.0	2.09	D13	20.0	6.34	62.524	1.00	0.33
	3	19.87	32.0	1.99	D13	20.0	6.34	62.524	1.00	0.32
	2	14.14	32.0	1.41	D13	20.0	6.34	62.524	1.00	0.23
	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 (Permanent state / Side wall / Room A / Vertical reinforcement / Inner reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	0.949	0.0040 × 7.0 = 0.0280	0.16	
	8	0.00	32.0	D13	20.0	6.34	0.949		0.19	
	7	0.00	32.0	D13	20.0	6.34	0.949		0.27	
	6	0.00	32.0	D13	20.0	6.34	0.949		0.32	
	5	0.00	32.0	D13	20.0	6.34	0.949		0.38	
	4	0.00	32.0	D13	20.0	6.34	0.949		0.46	
	3	0.00	32.0	D13	20.0	6.34	0.949		0.69	
	2	0.00	32.0	D13	20.0	6.34	0.949		0.92	
	1	79.03	32.0	D16,D19	10.0	24.26	0.907		6.11	0.88
II	9	0.00	32.0	D13	20.0	6.34	0.949	0.0040 × 7.0 = 0.0280	0.16	
	8	0.00	32.0	D13	20.0	6.34	0.949		0.16	
	7	0.00	32.0	D13	20.0	6.34	0.949		0.19	
	6	0.00	32.0	D13	20.0	6.34	0.949		0.20	
	5	0.00	32.0	D13	20.0	6.34	0.949		0.21	
	4	0.00	32.0	D13	20.0	6.34	0.949		0.25	
	3	0.00	32.0	D13	20.0	6.34	0.949		0.37	
	2	0.00	32.0	D13	20.0	6.34	0.949		0.56	
	1	47.38	32.0	D13,D13	10.0	12.67	0.930		4.72	0.98
III	9	0.00	32.0	D13	20.0	6.34	0.949	0.0040 × 7.0 = 0.0280	0.16	
	8	17.53	32.0	D13	20.0	6.34	0.949		2.34	0.90
	7	17.39	32.0	D13	20.0	6.34	0.949		2.32	0.89
	6	17.85	32.0	D13	20.0	6.34	0.949		2.38	0.91
	5	18.68	32.0	D13	20.0	6.34	0.949		2.49	0.95
	4	18.97	32.0	D13	20.0	6.34	0.949		2.53	0.96
	3	18.07	32.0	D13	20.0	6.34	0.949		2.41	0.92
	2	12.85	32.0	D13	20.0	6.34	0.949		1.71	0.70
	1	0.00	32.0	D13	20.0	6.34	0.949		0.00	0.16

ix) Compression stress of serviceability verification (Permanent state / Side wall / Room A / Vertical reinforcement / Inner reinforcement)

$\gamma_f=1.00, N=7.1, B=100\text{cm}$

NO.		M_s (kN.m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (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	D16	20.0	9.93	0.939	0.00	≤ 12.00
	5	0.00	34.0	D13, D13	10.0	12.67	0.932	0.00	≤ 12.00
	4	0.00	34.0	D13, D13	10.0	12.67	0.932	0.00	≤ 12.00
	3	0.00	34.0	D13, D16	10.0	16.27	0.924	0.00	≤ 12.00
	2	0.00	34.0	D13, D13	10.0	12.67	0.932	0.00	≤ 12.00
	1	13.15	34.0	D13	20.0	6.34	0.950	1.60	≤ 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	7.85	34.0	D13	20.0	6.34	0.950	0.95	≤ 12.00
III	9	106.97	34.0	D19, D19	10.0	28.65	0.903	7.04	≤ 12.00
	8	104.74	34.0	D19, D19	10.0	28.65	0.903	6.89	≤ 12.00
	7	104.38	34.0	D19, D19	10.0	28.65	0.903	6.87	≤ 12.00
	6	107.24	34.0	D16, D22	10.0	29.29	0.902	7.00	≤ 12.00
	5	112.10	34.0	D13, D25	10.0	31.67	0.899	7.11	≤ 12.00
	4	113.71	34.0	D13, D25	10.0	31.67	0.899	7.21	≤ 12.00
	3	108.33	34.0	D13, D25	10.0	31.67	0.899	6.87	≤ 12.00
	2	77.22	34.0	D16, D19	10.0	24.26	0.909	5.41	≤ 12.00
	1	0.00	34.0	D13	20.0	6.34	0.950	0.00	≤ 12.00

x) Safety verification (Permanent state / Side wall / Room A / Vertical reinforcement / Outer reinforcement)

$B=100\text{cm}$

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	30.0	0.00	D13	20.0	6.34	58.548	—	0.00
	8	8.71	30.0	0.93	D13	20.0	6.34	58.548	1.00	0.15
	7	9.94	30.0	1.06	D13	20.0	6.34	58.548	1.00	0.17
	6	10.00	30.0	1.07	D13	20.0	6.34	58.548	1.00	0.17
	5	10.19	30.0	1.09	D13	20.0	6.34	58.548	1.00	0.17
	4	11.15	30.0	1.19	D13	20.0	6.34	58.548	1.00	0.19
	3	15.86	30.0	1.69	D13	20.0	6.34	58.548	1.00	0.27
	2	19.91	30.0	2.13	D16	20.0	9.93	90.715	1.00	0.22*
	1	161.30	36.6	16.08	D16, D16	10.0	19.86	217.099	1.10	0.82*
II	9	0.00	30.0	0.00	D13	20.0	6.34	58.548	—	0.00
	8	1.87	30.0	0.20	D13	20.0	6.34	58.548	1.00	0.03
	7	2.65	30.0	0.28	D13	20.0	6.34	58.548	1.00	0.05
	6	2.69	30.0	0.29	D13	20.0	6.34	58.548	1.00	0.05
	5	2.46	30.0	0.26	D13	20.0	6.34	58.548	1.00	0.04
	4	2.86	30.0	0.30	D13	20.0	6.34	58.548	1.00	0.05
	3	5.99	30.0	0.64	D13	20.0	6.34	58.548	1.00	0.10
	2	10.42	30.0	1.11	D13	20.0	6.34	58.548	1.00	0.18
	1	98.21	36.6	9.63	D13, D13	10.0	12.67	141.019	1.10	0.77*
III	9	0.00	36.6	0.00	D13	20.0	6.34	71.671	—	0.00
	8	6.18	36.6	0.59	D13	20.0	6.34	71.671	1.10	0.09
	7	12.03	36.6	1.16	D13	20.0	6.34	71.671	1.10	0.18
	6	17.56	36.6	1.69	D13	20.0	6.34	71.671	1.10	0.27
	5	23.09	36.6	2.22	D13	20.0	6.34	71.671	1.10	0.35
	4	27.97	36.6	2.70	D13	20.0	6.34	71.671	1.10	0.43
	3	31.22	36.6	3.01	D13	20.0	6.34	71.671	1.10	0.48
	2	26.02	36.6	2.51	D13	20.0	6.34	71.671	1.10	0.40
	1	0.00	36.6	0.00	D13	20.0	6.34	71.671	—	0.00

*: reinforcement is determined by serviceability verification

xi) Crack width of serviceability verification (Permanent state / Side wall / Room A / Vertical reinforcement / Outer reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	0.0035 × 9.0 = 0.0315	0.17	
	8	7.92	30.0	D13	20.0	6.34	43.967		0.0173	0.55
	7	9.04	30.0	D13	20.0	6.34	50.185		0.0190	0.60
	6	9.08	30.0	D13	20.0	6.34	50.407		0.0190	0.60
	5	9.26	30.0	D13	20.0	6.34	51.406		0.0193	0.61
	4	10.14	30.0	D13	20.0	6.34	56.291		0.0206	0.65
	3	14.42	30.0	D13	20.0	6.34	80.051		0.0270	0.86
	2	18.09	30.0	D16	20.0	9.93	64.937		0.0228	0.72
	1	73.33	36.6	D16, D16	10.0	19.86	109.728		0.0299	0.95
II	9	0.00	30.0	D13	20.0	6.34	0.000	0.0035 × 9.0 = 0.0315	0.17	
	8	1.69	30.0	D13	20.0	6.34	9.382		0.0079	0.25
	7	2.41	30.0	D13	20.0	6.34	13.379		0.0090	0.29
	6	2.45	30.0	D13	20.0	6.34	13.601		0.0091	0.29
	5	2.24	30.0	D13	20.0	6.34	12.435		0.0088	0.28
	4	2.61	30.0	D13	20.0	6.34	14.489		0.0093	0.30
	3	5.45	30.0	D13	20.0	6.34	30.255		0.0136	0.43
	2	9.48	30.0	D13	20.0	6.34	52.627		0.0196	0.62
	1	44.65	36.6	D13, D13	10.0	12.67	103.107		0.0285	0.90
III	9	0.00	36.6	D13	20.0	6.34	0.000	0.0035 × 9.0 = 0.0315	0.17	
	8	2.81	36.6	D13	20.0	6.34	12.724		0.0088	0.28
	7	5.47	36.6	D13	20.0	6.34	24.770		0.0121	0.38
	6	7.98	36.6	D13	20.0	6.34	36.136		0.0152	0.48
	5	10.50	36.6	D13	20.0	6.34	47.547		0.0182	0.58
	4	12.71	36.6	D13	20.0	6.34	57.555		0.0209	0.66
	3	14.19	36.6	D13	20.0	6.34	64.256		0.0228	0.72
	2	11.83	36.6	D13	20.0	6.34	53.570		0.0199	0.63
	1	0.00	36.6	D13	20.0	6.34	0.000		0.0054	0.17

xii) Compression stress of serviceability verification (Permanent state / Side wall / Room A / Vertical reinforcement / Outer reinforcement)

$$\gamma_f=1.00, N=7.1, B=100\text{cm}$$

NO.		M_s (kN.m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (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	7.92	30.0	D13	20.0	6.34	0.947	1.17	≤ 12.00
	7	9.04	30.0	D13	20.0	6.34	0.947	1.34	≤ 12.00
	6	9.08	30.0	D13	20.0	6.34	0.947	1.34	≤ 12.00
	5	9.26	30.0	D13	20.0	6.34	0.947	1.37	≤ 12.00
	4	10.14	30.0	D13	20.0	6.34	0.947	1.50	≤ 12.00
	3	14.42	30.0	D13	20.0	6.34	0.947	2.13	≤ 12.00
	2	18.09	30.0	D16	20.0	9.93	0.935	2.21	≤ 12.00
	1	0.00	36.6	D16, D16	10.0	19.86	0.919	0.00	≤ 12.00
II	9	0.00	30.0	D13	20.0	6.34	0.947	0.00	≤ 12.00
	8	1.69	30.0	D13	20.0	6.34	0.947	0.25	≤ 12.00
	7	2.41	30.0	D13	20.0	6.34	0.947	0.36	≤ 12.00
	6	2.45	30.0	D13	20.0	6.34	0.947	0.36	≤ 12.00
	5	2.24	30.0	D13	20.0	6.34	0.947	0.33	≤ 12.00
	4	2.61	30.0	D13	20.0	6.34	0.947	0.39	≤ 12.00
	3	5.45	30.0	D13	20.0	6.34	0.947	0.81	≤ 12.00
	2	9.48	30.0	D13	20.0	6.34	0.947	1.40	≤ 12.00
	1	0.00	36.6	D13, D13	10.0	12.67	0.934	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

7) Outer Wall (Front Wall)

i) Safety verification (Permanent state / Lateral reinforcement / Inner reinforcement)

$B=100\text{cm}$

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	4.27	34.0	0.44	D13	20.0	6.34	66.503	1.10	0.07
	8	10.47	34.0	1.08	D13	20.0	6.34	66.503	1.10	0.17
	7	20.37	34.0	2.11	D13	20.0	6.34	66.503	1.10	0.34
	6	30.45	34.0	3.17	D13	20.0	6.34	66.503	1.10	0.50
	5	40.54	34.0	4.23	D16	20.0	9.93	103.173	1.10	0.43*
	4	50.62	34.0	5.29	D16	20.0	9.93	103.173	1.10	0.54*
	3	59.93	34.0	6.28	D16	20.0	9.93	103.173	1.10	0.64*
	2	53.14	34.0	5.56	D16	20.0	9.93	103.173	1.10	0.57*
	1	9.60	34.0	0.90	D13	20.0	6.34	66.503	1.00	0.14
II	9	0.78	34.0	0.08	D13	20.0	6.34	66.503	1.10	0.01
	8	2.33	34.0	0.24	D13	20.0	6.34	66.503	1.10	0.04
	7	5.24	34.0	0.54	D13	20.0	6.34	66.503	1.10	0.09
	6	7.56	34.0	0.78	D13	20.0	6.34	66.503	1.10	0.13
	5	10.09	34.0	1.04	D13	20.0	6.34	66.503	1.10	0.17
	4	12.61	34.0	1.31	D13	20.0	6.34	66.503	1.10	0.21
	3	15.52	34.0	1.61	D13	20.0	6.34	66.503	1.10	0.26
	2	17.07	34.0	1.77	D13	20.0	6.34	66.503	1.10	0.28
	1	5.59	34.0	0.52	D13	20.0	6.34	66.503	1.00	0.08
III	9	75.85	34.0	7.25	D16, D16	10.0	19.86	200.914	1.00	0.38*
	8	74.27	34.0	7.09	D16, D16	10.0	19.86	200.914	1.00	0.37*
	7	73.55	34.0	7.02	D16, D16	10.0	19.86	200.914	1.00	0.37*
	6	76.26	34.0	7.29	D16, D16	10.0	19.86	200.914	1.00	0.38*
	5	81.27	34.0	7.78	D16, D16	10.0	19.86	200.914	1.00	0.40*
	4	82.35	34.0	7.88	D13, D19	10.0	20.66	208.543	1.00	0.39*
	3	79.74	34.0	7.63	D16, D16	10.0	19.86	200.914	1.00	0.40*
	2	66.91	34.0	6.38	D13, D16	10.0	16.27	166.196	1.00	0.40*
	1	0.00	34.0	0.00	D13	20.0	6.34	66.503	—	0.00
I'	9	4.27	34.0	0.44	D13	20.0	6.34	66.503	1.10	0.07
	8	10.47	34.0	1.08	D13	20.0	6.34	66.503	1.10	0.17
	7	20.37	34.0	2.11	D13	20.0	6.34	66.503	1.10	0.34
	6	30.45	34.0	3.17	D13	20.0	6.34	66.503	1.10	0.50
	5	40.54	34.0	4.23	D16	20.0	9.93	103.173	1.10	0.43*
	4	50.62	34.0	5.29	D16	20.0	9.93	103.173	1.10	0.54*
	3	59.93	34.0	6.28	D16	20.0	9.93	103.173	1.10	0.64*
	2	53.14	34.0	5.56	D16	20.0	9.93	103.173	1.10	0.57*
	1	9.60	34.0	0.90	D13	20.0	6.34	66.503	1.00	0.14
III'	9	99.43	34.0	9.56	D13, D22	10.0	25.69	255.754	1.00	0.39*
	8	97.36	34.0	9.36	D16, D19	10.0	24.26	242.472	1.00	0.40*
	7	96.83	34.0	9.30	D16, D19	10.0	24.26	242.472	1.00	0.40*
	6	99.78	34.0	9.59	D13, D22	10.0	25.69	255.754	1.00	0.39*
	5	104.97	34.0	10.11	D19, D19	10.0	28.65	282.887	1.00	0.37*
	4	106.46	34.0	10.26	D19, D19	10.0	28.65	282.887	1.00	0.38*
	3	101.97	34.0	9.81	D13, D22	10.0	25.69	255.754	1.00	0.40*
	2	77.08	34.0	7.37	D16, D16	10.0	19.86	200.914	1.00	0.38*
	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 (Permanent state / Lateral reinforcement / Inner reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	1.94	34.0	D13	20.0	6.34	9.473	0.0040 × 5.0 = 0.0200	0.27
	8	4.76	34.0	D13	20.0	6.34	23.243		0.40
	7	9.26	34.0	D13	20.0	6.34	45.216		0.60
	6	13.84	34.0	D13	20.0	6.34	67.581		0.80
	5	18.43	34.0	D16	20.0	9.93	58.154		0.71
	4	23.01	34.0	D16	20.0	9.93	72.606		0.84
	3	27.25	34.0	D16	20.0	9.93	85.985		0.96
	2	24.16	34.0	D16	20.0	9.93	76.234		0.87
	1	8.73	34.0	D13	20.0	6.34	42.628		0.57
II	9	0.35	34.0	D13	20.0	6.34	1.709	0.0040 × 5.0 = 0.0200	0.20
	8	1.06	34.0	D13	20.0	6.34	5.176		0.23
	7	2.38	34.0	D13	20.0	6.34	11.622		0.29
	6	3.44	34.0	D13	20.0	6.34	16.797		0.34
	5	4.58	34.0	D13	20.0	6.34	22.364		0.39
	4	5.73	34.0	D13	20.0	6.34	27.980		0.44
	3	7.05	34.0	D13	20.0	6.34	34.425		0.50
	2	7.76	34.0	D13	20.0	6.34	37.892		0.53
	1	5.08	34.0	D13	20.0	6.34	24.806		0.41
III	9	68.94	34.0	D16, D16	10.0	19.86	111.358	0.0040 × 5.0 = 0.0200	0.94
	8	67.51	34.0	D16, D16	10.0	19.86	109.048		0.92
	7	66.85	34.0	D16, D16	10.0	19.86	107.982		0.91
	6	69.31	34.0	D16, D16	10.0	19.86	111.955		0.94
	5	73.87	34.0	D16, D16	10.0	19.86	119.321		0.99
	4	74.85	34.0	D13, D19	10.0	20.66	116.406		0.98
	3	72.49	34.0	D16, D16	10.0	19.86	117.092		0.98
	2	60.83	34.0	D13, D16	10.0	16.27	119.051		1.00
1	0.00	34.0	D13	20.0	6.34	0.000	0.0036	0.18	
I'	9	1.94	34.0	D13	20.0	6.34	9.473	0.0040 × 5.0 = 0.0200	0.27
	8	4.76	34.0	D13	20.0	6.34	23.243		0.40
	7	9.26	34.0	D13	20.0	6.34	45.216		0.60
	6	13.84	34.0	D13	20.0	6.34	67.581		0.80
	5	18.43	34.0	D16	20.0	9.93	58.154		0.71
	4	23.01	34.0	D16	20.0	9.93	72.606		0.84
	3	27.25	34.0	D16	20.0	9.93	85.985		0.96
	2	24.16	34.0	D16	20.0	9.93	76.234		0.87
	1	8.73	34.0	D13	20.0	6.34	42.628		0.57
III'	9	90.39	34.0	D13, D22	10.0	25.69	114.068	0.0040 × 5.0 = 0.0200	0.97
	8	88.51	34.0	D16, D19	10.0	24.26	117.994		0.98
	7	88.02	34.0	D16, D19	10.0	24.26	117.340		0.98
	6	90.70	34.0	D13, D22	10.0	25.69	114.459		0.97
	5	95.43	34.0	D19, D19	10.0	28.65	108.502		0.91
	4	96.77	34.0	D19, D19	10.0	28.65	110.025		0.92
	3	92.70	34.0	D13, D22	10.0	25.69	116.983		0.99
	2	70.07	34.0	D16, D16	10.0	19.86	113.183		0.95
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 (Permanent state / Lateral reinforcement / Inner reinforcement)

$$\gamma_f=1.00, N=7.1, B=100\text{cm}$$

NO.	M_s (kN.m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (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	D16	20.0	9.93	0.939	0.00 ≤ 12.00
	4	0.00	34.0	D16	20.0	9.93	0.939	0.00 ≤ 12.00
	3	0.00	34.0	D16	20.0	9.93	0.939	0.00 ≤ 12.00
	2	0.00	34.0	D16	20.0	9.93	0.939	0.00 ≤ 12.00
	1	8.73	34.0	D13	20.0	6.34	0.950	1.06 ≤ 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	5.08	34.0	D13	20.0	6.34	0.950	0.62 ≤ 12.00
III	9	68.94	34.0	D16, D16	10.0	19.86	0.917	5.21 ≤ 12.00
	8	67.51	34.0	D16, D16	10.0	19.86	0.917	5.11 ≤ 12.00
	7	66.85	34.0	D16, D16	10.0	19.86	0.917	5.06 ≤ 12.00
	6	69.31	34.0	D16, D16	10.0	19.86	0.917	5.24 ≤ 12.00
	5	73.87	34.0	D16, D16	10.0	19.86	0.917	5.59 ≤ 12.00
	4	74.85	34.0	D13, D19	10.0	20.66	0.915	5.58 ≤ 12.00
	3	72.49	34.0	D16, D16	10.0	19.86	0.917	5.48 ≤ 12.00
	2	60.83	34.0	D13, D16	10.0	16.27	0.924	4.97 ≤ 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	D16	20.0	9.93	0.939	0.00 ≤ 12.00
	4	0.00	34.0	D16	20.0	9.93	0.939	0.00 ≤ 12.00
	3	0.00	34.0	D16	20.0	9.93	0.939	0.00 ≤ 12.00
	2	0.00	34.0	D16	20.0	9.93	0.939	0.00 ≤ 12.00
	1	8.73	34.0	D13	20.0	6.34	0.950	1.06 ≤ 12.00
III'	9	90.39	34.0	D13, D22	10.0	25.69	0.907	6.19 ≤ 12.00
	8	88.51	34.0	D16, D19	10.0	24.26	0.909	6.20 ≤ 12.00
	7	88.02	34.0	D16, D19	10.0	24.26	0.909	6.17 ≤ 12.00
	6	90.70	34.0	D13, D22	10.0	25.69	0.907	6.22 ≤ 12.00
	5	95.43	34.0	D19, D19	10.0	28.65	0.903	6.28 ≤ 12.00
	4	96.77	34.0	D19, D19	10.0	28.65	0.903	6.37 ≤ 12.00
	3	92.70	34.0	D13, D22	10.0	25.69	0.907	6.35 ≤ 12.00
	2	70.07	34.0	D16, D16	10.0	19.86	0.917	5.30 ≤ 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 (Permanent state / Lateral reinforcement / Outer reinforcement)

$B=100\text{cm}$

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	37.46	32.0	3.77	D13, D13	10.0	12.67	122.737	1.00	0.31*
	8	36.54	32.0	3.68	D13, D13	10.0	12.67	122.737	1.00	0.30*
	7	37.61	32.0	3.79	D13, D13	10.0	12.67	122.737	1.00	0.31*
	6	38.70	32.0	3.90	D13, D13	10.0	12.67	122.737	1.00	0.32*
	5	39.71	32.0	4.00	D13, D13	10.0	12.67	122.737	1.00	0.32*
	4	40.80	32.0	4.11	D13, D13	10.0	12.67	122.737	1.00	0.33*
	3	41.30	32.0	4.16	D13, D13	10.0	12.67	122.737	1.00	0.34*
	2	32.76	32.0	3.29	D13, D13	10.0	12.67	122.737	1.00	0.27*
	1	16.49	38.6	1.50	D13	20.0	6.34	75.648	1.10	0.24
II	9	9.06	32.0	0.90	D13	20.0	6.34	62.524	1.00	0.14
	8	9.12	32.0	0.91	D13	20.0	6.34	62.524	1.00	0.15
	7	9.50	32.0	0.95	D13	20.0	6.34	62.524	1.00	0.15
	6	9.65	32.0	0.96	D13	20.0	6.34	62.524	1.00	0.15
	5	9.90	32.0	0.99	D13	20.0	6.34	62.524	1.00	0.16
	4	10.15	32.0	1.01	D13	20.0	6.34	62.524	1.00	0.16
	3	10.62	32.0	1.06	D13	20.0	6.34	62.524	1.00	0.17
	2	10.34	32.0	1.03	D13	20.0	6.34	62.524	1.00	0.17
	1	9.70	38.6	0.88	D13	20.0	6.34	75.648	1.10	0.14
III	9	0.00	38.6	0.00	D13	20.0	6.34	75.648	—	0.00
	8	20.95	38.6	1.91	D13	20.0	6.34	75.648	1.10	0.30
	7	45.39	38.6	4.16	D16	20.0	9.93	117.502	1.10	0.42*
	6	64.78	38.6	5.97	D16	20.0	9.93	117.502	1.10	0.61*
	5	84.18	38.6	7.79	D13, D13	10.0	12.67	148.968	1.10	0.62*
	4	98.34	38.6	9.12	D13, D13	10.0	12.67	148.968	1.10	0.73*
	3	112.50	38.6	10.47	D13, D16	10.0	16.27	189.668	1.10	0.65*
	2	112.50	38.6	10.47	D13, D16	10.0	16.27	189.668	1.10	0.65*
	1	0.00	38.6	0.00	D13	20.0	6.34	75.648	—	0.00
I'	9	37.46	32.0	3.77	D13, D13	10.0	12.67	122.737	1.00	0.31*
	8	36.54	32.0	3.68	D13, D13	10.0	12.67	122.737	1.00	0.30*
	7	37.61	32.0	3.79	D13, D13	10.0	12.67	122.737	1.00	0.31*
	6	38.70	32.0	3.90	D13, D13	10.0	12.67	122.737	1.00	0.32*
	5	39.71	32.0	4.00	D13, D13	10.0	12.67	122.737	1.00	0.32*
	4	40.80	32.0	4.11	D13, D13	10.0	12.67	122.737	1.00	0.33*
	3	41.30	32.0	4.16	D13, D13	10.0	12.67	122.737	1.00	0.34*
	2	32.76	32.0	3.29	D13, D13	10.0	12.67	122.737	1.00	0.27*
	1	16.49	38.6	1.50	D13	20.0	6.34	75.648	1.10	0.24
III'	9	0.00	38.6	0.00	D13	20.0	6.34	75.648	—	0.00
	8	36.42	38.6	3.33	D13	20.0	6.34	75.648	1.10	0.53
	7	72.85	38.6	6.72	D13, D13	10.0	12.67	148.968	1.10	0.54*
	6	105.69	38.6	9.82	D13, D13	10.0	12.67	148.968	1.10	0.78*
	5	139.19	38.6	13.03	D13, D16	10.0	16.27	189.668	1.10	0.81*
	4	167.81	38.6	15.82	D16, D16	10.0	19.86	229.554	1.10	0.80*
	3	187.97	38.6	17.80	D16, D19	10.0	24.26	277.496	1.10	0.75*
	2	155.77	38.6	14.64	D16, D16	10.0	19.86	229.554	1.10	0.75*
	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 (Permanent state / Lateral reinforcement / Outer reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	34.05	32.0	D13,D13	10.0	12.67	90.326	0.0207	0.0035 × 7.0 = 0.0245	0.84
	8	33.21	32.0	D13,D13	10.0	12.67	88.098	0.0203		0.83
	7	34.19	32.0	D13,D13	10.0	12.67	90.698	0.0208		0.85
	6	35.19	32.0	D13,D13	10.0	12.67	93.351	0.0213		0.87
	5	36.10	32.0	D13,D13	10.0	12.67	95.765	0.0217		0.89
	4	37.09	32.0	D13,D13	10.0	12.67	98.391	0.0222		0.91
	3	37.55	32.0	D13,D13	10.0	12.67	99.611	0.0224		0.91
	2	29.78	32.0	D13,D13	10.0	12.67	78.999	0.0186		0.76
	1	7.49	38.6	D13	20.0	6.34	32.120	0.0118		0.48
II	9	8.23	32.0	D13	20.0	6.34	42.764	0.0142	0.0035 × 7.0 = 0.0245	0.58
	8	8.29	32.0	D13	20.0	6.34	43.076	0.0143		0.58
	7	8.64	32.0	D13	20.0	6.34	44.894	0.0147		0.60
	6	8.77	32.0	D13	20.0	6.34	45.570	0.0148		0.60
	5	9.00	32.0	D13	20.0	6.34	46.765	0.0151		0.62
	4	9.22	32.0	D13	20.0	6.34	47.908	0.0154		0.63
	3	9.65	32.0	D13	20.0	6.34	50.142	0.0159		0.65
	2	9.40	32.0	D13	20.0	6.34	48.843	0.0156		0.64
	1	4.41	38.6	D13	20.0	6.34	18.912	0.0088		0.36
III	9	0.00	38.6	D13	20.0	6.34	0.000	0.0045	0.0035 × 7.0 = 0.0245	0.18
	8	9.52	38.6	D13	20.0	6.34	40.825	0.0138		0.56
	7	20.63	38.6	D16	20.0	9.93	57.130	0.0173		0.71
	6	29.45	38.6	D16	20.0	9.93	81.554	0.0228		0.93
	5	38.27	38.6	D13,D13	10.0	12.67	83.655	0.0194		0.79
	4	44.70	38.6	D13,D13	10.0	12.67	97.711	0.0221		0.90
	3	51.14	38.6	D13,D16	10.0	16.27	87.769	0.0202		0.82
	2	51.14	38.6	D13,D16	10.0	16.27	87.769	0.0202		0.82
	1	0.00	38.6	D13	20.0	6.34	0.000	0.0045		0.18
I'	9	34.05	32.0	D13,D13	10.0	12.67	90.326	0.0207	0.0035 × 7.0 = 0.0245	0.84
	8	33.21	32.0	D13,D13	10.0	12.67	88.098	0.0203		0.83
	7	34.19	32.0	D13,D13	10.0	12.67	90.698	0.0208		0.85
	6	35.19	32.0	D13,D13	10.0	12.67	93.351	0.0213		0.87
	5	36.10	32.0	D13,D13	10.0	12.67	95.765	0.0217		0.89
	4	37.09	32.0	D13,D13	10.0	12.67	98.391	0.0222		0.91
	3	37.55	32.0	D13,D13	10.0	12.67	99.611	0.0224		0.91
	2	29.78	32.0	D13,D13	10.0	12.67	78.999	0.0186		0.76
	1	7.49	38.6	D13	20.0	6.34	32.120	0.0118		0.48
III'	9	0.00	38.6	D13	20.0	6.34	0.000	0.0045	0.0040 × 5.0 = 0.0200	0.18
	8	16.56	38.6	D13	20.0	6.34	71.015	0.0206		0.84
	7	33.12	38.6	D13,D13	10.0	12.67	72.398	0.0173		0.71
	6	48.05	38.6	D13,D13	10.0	12.67	105.033	0.0235		0.96
	5	63.27	38.6	D13,D16	10.0	16.27	108.587	0.0241		0.98
	4	76.28	38.6	D16,D16	10.0	19.86	108.013	0.0239		0.98
	3	85.45	38.6	D16,D19	10.0	24.26	99.812	0.0223		0.91
	2	70.81	38.6	D16,D16	10.0	19.86	100.267	0.0224		0.91
	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 (Permanent state / Lateral reinforcement / Outer reinforcement)

$$\gamma_f=1.00, N=7.1, B=100\text{cm}$$

NO.	M_s (kN.m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (N/mm ²)	$0.4 \cdot f_{ck}$ (N/mm ²)
I	9	34.05	32.0	D13, D13	10.0	12.67	0.930	3.39 ≤ 12.00
	8	33.21	32.0	D13, D13	10.0	12.67	0.930	3.31 ≤ 12.00
	7	34.19	32.0	D13, D13	10.0	12.67	0.930	3.41 ≤ 12.00
	6	35.19	32.0	D13, D13	10.0	12.67	0.930	3.51 ≤ 12.00
	5	36.10	32.0	D13, D13	10.0	12.67	0.930	3.60 ≤ 12.00
	4	37.09	32.0	D13, D13	10.0	12.67	0.930	3.70 ≤ 12.00
	3	37.55	32.0	D13, D13	10.0	12.67	0.930	3.74 ≤ 12.00
	2	29.78	32.0	D13, D13	10.0	12.67	0.930	2.97 ≤ 12.00
	1	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
II	9	8.23	32.0	D13	20.0	6.34	0.949	1.10 ≤ 12.00
	8	8.29	32.0	D13	20.0	6.34	0.949	1.11 ≤ 12.00
	7	8.64	32.0	D13	20.0	6.34	0.949	1.15 ≤ 12.00
	6	8.77	32.0	D13	20.0	6.34	0.949	1.17 ≤ 12.00
	5	9.00	32.0	D13	20.0	6.34	0.949	1.20 ≤ 12.00
	4	9.22	32.0	D13	20.0	6.34	0.949	1.23 ≤ 12.00
	3	9.65	32.0	D13	20.0	6.34	0.949	1.29 ≤ 12.00
	2	9.40	32.0	D13	20.0	6.34	0.949	1.25 ≤ 12.00
	1	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
III	9	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
	8	0.00	38.6	D13	20.0	6.34	0.953	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	D13, D13	10.0	12.67	0.935	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, D16	10.0	16.27	0.928	0.00 ≤ 12.00
	2	0.00	38.6	D13, D16	10.0	16.27	0.928	0.00 ≤ 12.00
	1	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
I'	9	34.05	32.0	D13, D13	10.0	12.67	0.930	3.39 ≤ 12.00
	8	33.21	32.0	D13, D13	10.0	12.67	0.930	3.31 ≤ 12.00
	7	34.19	32.0	D13, D13	10.0	12.67	0.930	3.41 ≤ 12.00
	6	35.19	32.0	D13, D13	10.0	12.67	0.930	3.51 ≤ 12.00
	5	36.10	32.0	D13, D13	10.0	12.67	0.930	3.60 ≤ 12.00
	4	37.09	32.0	D13, D13	10.0	12.67	0.930	3.70 ≤ 12.00
	3	37.55	32.0	D13, D13	10.0	12.67	0.930	3.74 ≤ 12.00
	2	29.78	32.0	D13, D13	10.0	12.67	0.930	2.97 ≤ 12.00
	1	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
III'	9	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
	8	0.00	38.6	D13	20.0	6.34	0.953	0.00 ≤ 12.00
	7	0.00	38.6	D13, D13	10.0	12.67	0.935	0.00 ≤ 12.00
	6	0.00	38.6	D13, D13	10.0	12.67	0.935	0.00 ≤ 12.00
	5	0.00	38.6	D13, D16	10.0	16.27	0.928	0.00 ≤ 12.00
	4	0.00	38.6	D16, D16	10.0	19.86	0.921	0.00 ≤ 12.00
	3	0.00	38.6	D16, D19	10.0	24.26	0.914	0.00 ≤ 12.00
	2	0.00	38.6	D16, D16	10.0	19.86	0.921	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 (Permanent state / Vertical reinforcement / Inner reinforcement)

$B=100\text{cm}$

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	1.16	32.0	0.13	D13	20.0	6.34	62.524	1.10	0.02
	7	3.49	32.0	0.38	D13	20.0	6.34	62.524	1.10	0.06
	6	5.24	32.0	0.58	D13	20.0	6.34	62.524	1.10	0.09
	5	6.59	32.0	0.72	D13	20.0	6.34	62.524	1.10	0.12
	4	8.34	32.0	0.92	D13	20.0	6.34	62.524	1.10	0.15
	3	12.41	32.0	1.37	D13	20.0	6.34	62.524	1.10	0.22
	2	23.86	32.0	2.63	D13	20.0	6.34	62.524	1.10	0.42
	1	57.71	32.0	5.84	D13, D16	10.0	16.27	156.004	1.00	0.37*
II	9	0.00	32.0	0.00	D13	20.0	6.34	62.524	—	0.00
	8	0.00	32.0	0.00	D13	20.0	6.34	62.524	—	0.00
	7	0.58	32.0	0.06	D13	20.0	6.34	62.524	1.10	0.01
	6	0.97	32.0	0.11	D13	20.0	6.34	62.524	1.10	0.02
	5	1.36	32.0	0.15	D13	20.0	6.34	62.524	1.10	0.02
	4	2.13	32.0	0.23	D13	20.0	6.34	62.524	1.10	0.04
	3	4.46	32.0	0.49	D13	20.0	6.34	62.524	1.10	0.08
	2	11.44	32.0	1.26	D13	20.0	6.34	62.524	1.10	0.20
	1	33.63	32.0	3.38	D16	20.0	9.93	96.947	1.00	0.35*
III	9	0.00	32.0	0.00	D13	20.0	6.34	62.524	—	0.00
	8	12.41	32.0	1.24	D13	20.0	6.34	62.524	1.00	0.20
	7	12.31	32.0	1.23	D13	20.0	6.34	62.524	1.00	0.20
	6	12.71	32.0	1.27	D13	20.0	6.34	62.524	1.00	0.20
	5	13.53	32.0	1.35	D13	20.0	6.34	62.524	1.00	0.22
	4	13.71	32.0	1.37	D13	20.0	6.34	62.524	1.00	0.22
	3	13.25	32.0	1.33	D13	20.0	6.34	62.524	1.00	0.21
	2	11.12	32.0	1.11	D13	20.0	6.34	62.524	1.00	0.18
	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 (Permanent state / Vertical reinforcement / Inner reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	0.0040 × 7.0 = 0.0280	0.16	
	8	0.53	32.0	D13	20.0	6.34	2.754		0.0051	0.18
	7	1.59	32.0	D13	20.0	6.34	8.262		0.0064	0.23
	6	2.38	32.0	D13	20.0	6.34	12.367		0.0073	0.26
	5	3.00	32.0	D13	20.0	6.34	15.588		0.0080	0.29
	4	3.79	32.0	D13	20.0	6.34	19.693		0.0090	0.32
	3	5.64	32.0	D13	20.0	6.34	29.306		0.0111	0.40
	2	10.85	32.0	D13	20.0	6.34	56.378		0.0173	0.62
	1	52.46	32.0	D13, D16	10.0	16.27	109.322		0.0243	0.87
II	9	0.00	32.0	D13	20.0	6.34	0.000	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	0.26	32.0	D13	20.0	6.34	1.351		0.0048	0.17
	6	0.44	32.0	D13	20.0	6.34	2.286		0.0050	0.18
	5	0.62	32.0	D13	20.0	6.34	3.222		0.0053	0.19
	4	0.97	32.0	D13	20.0	6.34	5.040		0.0057	0.20
	3	2.03	32.0	D13	20.0	6.34	10.548		0.0069	0.25
	2	5.20	32.0	D13	20.0	6.34	27.020		0.0106	0.38
	1	30.56	32.0	D16	20.0	9.93	102.637		0.0276	0.99
III	9	0.00	32.0	D13	20.0	6.34	0.000	0.0040 × 7.0 = 0.0280	0.16	
	8	11.28	32.0	D13	20.0	6.34	58.612		0.0178	0.64
	7	11.18	32.0	D13	20.0	6.34	58.092		0.0177	0.63
	6	11.56	32.0	D13	20.0	6.34	60.067		0.0181	0.65
	5	12.31	32.0	D13	20.0	6.34	63.964		0.0190	0.68
	4	12.46	32.0	D13	20.0	6.34	64.743		0.0192	0.69
	3	12.05	32.0	D13	20.0	6.34	62.613		0.0187	0.67
	2	10.11	32.0	D13	20.0	6.34	52.533		0.0164	0.59
	1	0.00	32.0	D13	20.0	6.34	0.000		0.0045	0.16

ix) Compression stress of serviceability verification (Permanent state / Vertical reinforcement / Inner reinforcement)

$$\gamma_f=1.00, N=7.1, B=100\text{cm}$$

NO.		M_s (kN.m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (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	52.46	32.0	D13,D16	10.0	16.27	0.922	4.73	≤ 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	30.56	32.0	D16	20.0	9.93	0.937	3.37	≤ 12.00
III	9	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤ 12.00
	8	11.28	32.0	D13	20.0	6.34	0.949	1.51	≤ 12.00
	7	11.18	32.0	D13	20.0	6.34	0.949	1.49	≤ 12.00
	6	11.56	32.0	D13	20.0	6.34	0.949	1.54	≤ 12.00
	5	12.31	32.0	D13	20.0	6.34	0.949	1.64	≤ 12.00
	4	12.46	32.0	D13	20.0	6.34	0.949	1.66	≤ 12.00
	3	12.05	32.0	D13	20.0	6.34	0.949	1.61	≤ 12.00
	2	10.11	32.0	D13	20.0	6.34	0.949	1.35	≤ 12.00
	1	0.00	32.0	D13	20.0	6.34	0.949	0.00	≤ 12.00

x) Safety verification (Permanent state / Vertical reinforcement / Outer reinforcement)

$B=100\text{cm}$

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	30.0	0.00	D13	20.0	6.34	58.548	—	0.00
	8	5.94	30.0	0.63	D13	20.0	6.34	58.548	1.00	0.10
	7	6.51	30.0	0.69	D13	20.0	6.34	58.548	1.00	0.11
	6	6.59	30.0	0.70	D13	20.0	6.34	58.548	1.00	0.11
	5	6.56	30.0	0.70	D13	20.0	6.34	58.548	1.00	0.11
	4	6.65	30.0	0.71	D13	20.0	6.34	58.548	1.00	0.11
	3	8.17	30.0	0.87	D13	20.0	6.34	58.548	1.00	0.14
	2	13.84	30.0	1.48	D13	20.0	6.34	58.548	1.00	0.24
	1	98.34	36.6	9.65	D13, D13	10.0	12.67	141.019	1.10	0.77*
II	9	0.00	30.0	0.00	D13	20.0	6.34	58.548	—	0.00
	8	1.37	30.0	0.15	D13	20.0	6.34	58.548	1.00	0.02
	7	1.94	30.0	0.21	D13	20.0	6.34	58.548	1.00	0.03
	6	1.81	30.0	0.19	D13	20.0	6.34	58.548	1.00	0.03
	5	1.42	30.0	0.15	D13	20.0	6.34	58.548	1.00	0.02
	4	1.49	30.0	0.16	D13	20.0	6.34	58.548	1.00	0.03
	3	2.67	30.0	0.28	D13	20.0	6.34	58.548	1.00	0.05
	2	6.70	30.0	0.71	D13	20.0	6.34	58.548	1.00	0.11
	1	58.38	36.6	5.67	D16	20.0	9.93	111.274	1.10	0.58*
III	9	0.00	36.6	0.00	D13	20.0	6.34	71.671	—	0.00
	8	3.49	36.6	0.33	D13	20.0	6.34	71.671	1.10	0.05
	7	7.56	36.6	0.73	D13	20.0	6.34	71.671	1.10	0.12
	6	10.86	36.6	1.04	D13	20.0	6.34	71.671	1.10	0.17
	5	13.96	36.6	1.34	D13	20.0	6.34	71.671	1.10	0.21
	4	16.49	36.6	1.59	D13	20.0	6.34	71.671	1.10	0.25
	3	18.81	36.6	1.81	D13	20.0	6.34	71.671	1.10	0.29
	2	18.81	36.6	1.81	D13	20.0	6.34	71.671	1.10	0.29
	1	0.00	36.6	0.00	D13	20.0	6.34	71.671	—	0.00

*: reinforcement is determined by serviceability verification

xi) Crack width of serviceability verification (Permanent state / Vertical reinforcement / Outer reinforcement)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	0.0035 × 9.0 = 0.0315	0.17	
	8	5.40	30.0	D13	20.0	6.34	29.978		0.0135	0.43
	7	5.92	30.0	D13	20.0	6.34	32.864		0.0143	0.45
	6	6.00	30.0	D13	20.0	6.34	33.308		0.0144	0.46
	5	5.96	30.0	D13	20.0	6.34	33.086		0.0143	0.45
	4	6.05	30.0	D13	20.0	6.34	33.586		0.0145	0.46
	3	7.43	30.0	D13	20.0	6.34	41.247		0.0165	0.52
	2	12.59	30.0	D13	20.0	6.34	69.892		0.0243	0.77
	1	44.70	36.6	D13, D13	10.0	12.67	103.222		0.0285	0.90
II	9	0.00	30.0	D13	20.0	6.34	0.000	0.0035 × 9.0 = 0.0315	0.17	
	8	1.24	30.0	D13	20.0	6.34	6.884		0.0073	0.23
	7	1.76	30.0	D13	20.0	6.34	9.770		0.0080	0.25
	6	1.64	30.0	D13	20.0	6.34	9.104		0.0079	0.25
	5	1.29	30.0	D13	20.0	6.34	7.161		0.0073	0.23
	4	1.36	30.0	D13	20.0	6.34	7.550		0.0074	0.23
	3	2.43	30.0	D13	20.0	6.34	13.490		0.0090	0.29
	2	6.09	30.0	D13	20.0	6.34	33.808		0.0145	0.46
	1	26.54	36.6	D16	20.0	9.93	77.627		0.0262	0.83
III	9	0.00	36.6	D13	20.0	6.34	0.000	0.0035 × 9.0 = 0.0315	0.17	
	8	1.59	36.6	D13	20.0	6.34	7.200		0.0073	0.23
	7	3.44	36.6	D13	20.0	6.34	15.577		0.0096	0.30
	6	4.94	36.6	D13	20.0	6.34	22.370		0.0114	0.36
	5	6.35	36.6	D13	20.0	6.34	28.755		0.0132	0.42
	4	7.49	36.6	D13	20.0	6.34	33.917		0.0146	0.46
	3	8.55	36.6	D13	20.0	6.34	38.717		0.0159	0.50
	2	8.55	36.6	D13	20.0	6.34	38.717		0.0159	0.50
	1	0.00	36.6	D13	20.0	6.34	0.000		0.0054	0.17

xii) Compression stress of serviceability verification (Permanent state / Vertical reinforcement / Outer reinforcement)

$$\gamma_f=1.00, N=7.1, B=100\text{cm}$$

NO.		M_s (kN.m)	d (cm)	Dia. (mm)	Pitch (cm)	A_s (cm ²)	J	$\gamma_f \cdot \sigma_c$ (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	5.40	30.0	D13	20.0	6.34	0.947	0.80	≤ 12.00
	7	5.92	30.0	D13	20.0	6.34	0.947	0.88	≤ 12.00
	6	6.00	30.0	D13	20.0	6.34	0.947	0.89	≤ 12.00
	5	5.96	30.0	D13	20.0	6.34	0.947	0.88	≤ 12.00
	4	6.05	30.0	D13	20.0	6.34	0.947	0.89	≤ 12.00
	3	7.43	30.0	D13	20.0	6.34	0.947	1.10	≤ 12.00
	2	12.59	30.0	D13	20.0	6.34	0.947	1.86	≤ 12.00
	1	0.00	36.6	D13, D13	10.0	12.67	0.934	0.00	≤ 12.00
II	9	0.00	30.0	D13	20.0	6.34	0.947	0.00	≤ 12.00
	8	1.24	30.0	D13	20.0	6.34	0.947	0.18	≤ 12.00
	7	1.76	30.0	D13	20.0	6.34	0.947	0.26	≤ 12.00
	6	1.64	30.0	D13	20.0	6.34	0.947	0.24	≤ 12.00
	5	1.29	30.0	D13	20.0	6.34	0.947	0.19	≤ 12.00
	4	1.36	30.0	D13	20.0	6.34	0.947	0.20	≤ 12.00
	3	2.43	30.0	D13	20.0	6.34	0.947	0.36	≤ 12.00
	2	6.09	30.0	D13	20.0	6.34	0.947	0.90	≤ 12.00
	1	0.00	36.6	D16	20.0	9.93	0.941	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

8) Partition Wall

i) Pull-out force of partition wall and side wall in the transverse direction

$B=100\text{cm}$

Direction	T_d (kN/m)	γ_i	γ_i	A_{sn} (cm^2)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	N_{ud} (kN/m)	$\gamma_i \cdot T_d$ / N_{ud}
Lateral	268.94	1.10	1.10	9.43	D13, D13	10.0	12.67	397.377	0.74
Vertical	261.94	1.10	1.10	8.35	D16	20.0	9.93	311.441	0.84

ii) Pull-out force of partition wall and side wall in the longitudinal direction

$B=100\text{cm}$

Direction	T_d (kN/m)	γ_i	γ_i	A_{sn} (cm^2)	Dia. (mm)	Pitch (cm)	A_s (cm^2)	N_{ud} (kN/m)	$\gamma_i \cdot T_d$ / N_{ud}
Lateral	309.92	1.10	1.10	10.87	D13, D13	10.0	12.67	397.377	0.86
Vertical	217.96	1.10	1.10	6.95	D16	20.0	9.93	311.441	0.70

iii) Safety verification (Permanent state / Lateral reinforcement / Transverse direction)

$B=100\text{cm}$

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	12.24	10.0	4.47	D16	20.0	9.93	28.427	1.10	0.47
	8	11.73	10.0	4.27	D16	20.0	9.93	28.427	1.10	0.45
	7	11.79	10.0	4.30	D16	20.0	9.93	28.427	1.10	0.46
	6	11.81	10.0	4.30	D16	20.0	9.93	28.427	1.10	0.46
	5	11.84	10.0	4.32	D16	20.0	9.93	28.427	1.10	0.46
	4	11.81	10.0	4.30	D16	20.0	9.93	28.427	1.10	0.46
	3	11.19	10.0	4.07	D16	20.0	9.93	28.427	1.10	0.43
	2	7.25	10.0	2.60	D16	20.0	9.93	28.427	1.10	0.28
	1	2.66	10.0	0.94	D16	20.0	9.93	28.427	1.10	0.10
II	9	2.97	10.0	1.05	D16	20.0	9.93	28.427	1.10	0.11
	8	2.92	10.0	1.03	D16	20.0	9.93	28.427	1.10	0.11
	7	2.95	10.0	1.04	D16	20.0	9.93	28.427	1.10	0.11
	6	2.95	10.0	1.04	D16	20.0	9.93	28.427	1.10	0.11
	5	2.97	10.0	1.05	D16	20.0	9.93	28.427	1.10	0.11
	4	3.00	10.0	1.06	D16	20.0	9.93	28.427	1.10	0.12
	3	3.06	10.0	1.08	D16	20.0	9.93	28.427	1.10	0.12
	2	2.49	10.0	0.88	D16	20.0	9.93	28.427	1.10	0.10
	1	1.59	10.0	0.56	D16	20.0	9.93	28.427	1.10	0.06
III	9	24.71	10.0	9.45	D13, D13	10.0	12.67	35.315	1.10	0.77*
	8	23.71	10.0	9.03	D13, D13	10.0	12.67	35.315	1.10	0.74*
	7	23.12	10.0	8.79	D13, D13	10.0	12.67	35.315	1.10	0.72*
	6	23.32	10.0	8.87	D13, D13	10.0	12.67	35.315	1.10	0.73*
	5	23.97	10.0	9.14	D13, D13	10.0	12.67	35.315	1.10	0.75*
	4	23.94	10.0	9.13	D13, D13	10.0	12.67	35.315	1.10	0.75*
	3	22.41	10.0	8.49	D16	20.0	9.93	28.427	1.10	0.87
	2	15.67	10.0	5.79	D16	20.0	9.93	28.427	1.10	0.61
	1	0.00	10.0	0.00	D16	20.0	9.93	28.427	—	0.00

*: reinforcement is determined by serviceability verification

iv) Crack width of serviceability verification (Permanent state / Lateral reinforcement / Transverse direction)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	5.56	10.0	D16	20.0	9.93	62.481	0.0040 × 10.0 = 0.0400	0.60
	8	5.33	10.0	D16	20.0	9.93	59.897		0.58
	7	5.36	10.0	D16	20.0	9.93	60.234		0.58
	6	5.37	10.0	D16	20.0	9.93	60.346		0.59
	5	5.38	10.0	D16	20.0	9.93	60.458		0.59
	4	5.37	10.0	D16	20.0	9.93	60.346		0.59
	3	5.09	10.0	D16	20.0	9.93	57.200		0.56
	2	3.30	10.0	D16	20.0	9.93	37.084		0.42
	1	1.21	10.0	D16	20.0	9.93	13.598		0.25
II	9	1.35	10.0	D16	20.0	9.93	15.171	0.0040 × 10.0 = 0.0400	0.26
	8	1.33	10.0	D16	20.0	9.93	14.946		0.26
	7	1.34	10.0	D16	20.0	9.93	15.058		0.26
	6	1.34	10.0	D16	20.0	9.93	15.058		0.26
	5	1.35	10.0	D16	20.0	9.93	15.171		0.26
	4	1.37	10.0	D16	20.0	9.93	15.396		0.26
	3	1.39	10.0	D16	20.0	9.93	15.620		0.26
	2	1.13	10.0	D16	20.0	9.93	12.699		0.24
	1	0.72	10.0	D16	20.0	9.93	8.091		0.21
III	9	11.23	10.0	D13,D13	10.0	12.67	100.101	0.0040 × 10.0 = 0.0400	0.76
	8	10.78	10.0	D13,D13	10.0	12.67	96.090		0.74
	7	10.51	10.0	D13,D13	10.0	12.67	93.683		0.72
	6	10.60	10.0	D13,D13	10.0	12.67	94.486		0.73
	5	10.90	10.0	D13,D13	10.0	12.67	97.160		0.74
	4	10.88	10.0	D13,D13	10.0	12.67	96.981		0.74
	3	10.19	10.0	D16	20.0	9.93	114.511		0.98
	2	7.12	10.0	D16	20.0	9.93	80.012		0.73
	1	0.00	10.0	D16	20.0	9.93	0.000		0.15

v) Safety verification (Permanent state / Vertical reinforcement / Transverse direction)

$B=100\text{cm}$

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	10.0	0.00	D13	20.0	6.34	18.777	—	0.00
	8	1.81	10.0	0.64	D13	20.0	6.34	18.777	1.10	0.11
	7	2.01	10.0	0.71	D13	20.0	6.34	18.777	1.10	0.12
	6	1.98	10.0	0.70	D13	20.0	6.34	18.777	1.10	0.12
	5	1.98	10.0	0.70	D13	20.0	6.34	18.777	1.10	0.12
	4	2.12	10.0	0.75	D13	20.0	6.34	18.777	1.10	0.12
	3	2.95	10.0	1.04	D13	20.0	6.34	18.777	1.10	0.17
	2	3.63	10.0	1.29	D13	20.0	6.34	18.777	1.10	0.21
	1	16.01	10.0	5.92	D16	20.0	9.93	28.427	1.10	0.62
II	9	0.00	10.0	0.00	D13	20.0	6.34	18.777	—	0.00
	8	0.40	10.0	0.14	D13	20.0	6.34	18.777	1.10	0.02
	7	0.54	10.0	0.19	D13	20.0	6.34	18.777	1.10	0.03
	6	0.54	10.0	0.19	D13	20.0	6.34	18.777	1.10	0.03
	5	0.48	10.0	0.17	D13	20.0	6.34	18.777	1.10	0.03
	4	0.54	10.0	0.19	D13	20.0	6.34	18.777	1.10	0.03
	3	1.10	10.0	0.39	D13	20.0	6.34	18.777	1.10	0.06
	2	1.90	10.0	0.67	D13	20.0	6.34	18.777	1.10	0.11
	1	9.58	10.0	3.47	D16	20.0	9.93	28.427	1.10	0.37
III	9	0.00	10.0	0.00	D13	20.0	6.34	18.777	—	0.00
	8	3.97	10.0	1.41	D13	20.0	6.34	18.777	1.10	0.23
	7	3.85	10.0	1.37	D13	20.0	6.34	18.777	1.10	0.23
	6	3.88	10.0	1.38	D13	20.0	6.34	18.777	1.10	0.23
	5	3.99	10.0	1.42	D13	20.0	6.34	18.777	1.10	0.23
	4	3.99	10.0	1.42	D13	20.0	6.34	18.777	1.10	0.23
	3	3.74	10.0	1.33	D13	20.0	6.34	18.777	1.10	0.22
	2	2.61	10.0	0.92	D13	20.0	6.34	18.777	1.10	0.15
	1	0.00	10.0	0.00	D16	20.0	9.93	28.427	—	0.00

*: reinforcement is determined by serviceability verification

vi) Crack width of serviceability verification (Permanent state / Vertical reinforcement / Transverse direction)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	10.0	D13	20.0	6.34	0.000	0.0040 × 10.0 = 0.0400	0.15	
	8	0.82	10.0	D13	20.0	6.34	14.153		0.0100	0.25
	7	0.91	10.0	D13	20.0	6.34	15.706		0.0104	0.26
	6	0.90	10.0	D13	20.0	6.34	15.533		0.0104	0.26
	5	0.90	10.0	D13	20.0	6.34	15.533		0.0104	0.26
	4	0.97	10.0	D13	20.0	6.34	16.742		0.0107	0.27
	3	1.34	10.0	D13	20.0	6.34	23.128		0.0126	0.32
	2	1.65	10.0	D13	20.0	6.34	28.478		0.0142	0.36
	1	7.28	10.0	D16	20.0	9.93	81.810		0.0296	0.74
II	9	0.00	10.0	D13	20.0	6.34	0.000	0.0040 × 10.0 = 0.0400	0.15	
	8	0.18	10.0	D13	20.0	6.34	3.107		0.0067	0.17
	7	0.24	10.0	D13	20.0	6.34	4.142		0.0071	0.18
	6	0.24	10.0	D13	20.0	6.34	4.142		0.0071	0.18
	5	0.22	10.0	D13	20.0	6.34	3.797		0.0070	0.18
	4	0.24	10.0	D13	20.0	6.34	4.142		0.0071	0.18
	3	0.50	10.0	D13	20.0	6.34	8.630		0.0084	0.21
	2	0.86	10.0	D13	20.0	6.34	14.843		0.0102	0.26
	1	4.35	10.0	D16	20.0	9.93	48.884		0.0200	0.50
III	9	0.00	10.0	D13	20.0	6.34	0.000	0.0040 × 10.0 = 0.0400	0.15	
	8	1.80	10.0	D13	20.0	6.34	31.067		0.0149	0.37
	7	1.75	10.0	D13	20.0	6.34	30.204		0.0147	0.37
	6	1.76	10.0	D13	20.0	6.34	30.377		0.0147	0.37
	5	1.82	10.0	D13	20.0	6.34	31.412		0.0150	0.38
	4	1.82	10.0	D13	20.0	6.34	31.412		0.0150	0.38
	3	1.70	10.0	D13	20.0	6.34	29.341		0.0144	0.36
	2	1.18	10.0	D13	20.0	6.34	20.366		0.0118	0.30
	1	0.00	10.0	D16	20.0	9.93	0.000		0.0058	0.15

vii) Safety verification (Permanent state / Lateral reinforcement / Longitudinal direction)

 $B=100\text{cm}$

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	7.32	10.0	2.63	D13, D13	10.0	12.67	35.315	1.10	0.23
	8	7.01	10.0	2.51	D13, D13	10.0	12.67	35.315	1.10	0.22
	7	7.03	10.0	2.52	D13, D13	10.0	12.67	35.315	1.10	0.22
	6	7.05	10.0	2.53	D13, D13	10.0	12.67	35.315	1.10	0.22
	5	7.05	10.0	2.53	D13, D13	10.0	12.67	35.315	1.10	0.22
	4	7.06	10.0	2.53	D13, D13	10.0	12.67	35.315	1.10	0.22
	3	6.98	10.0	2.50	D13, D13	10.0	12.67	35.315	1.10	0.22
	2	5.42	10.0	1.93	D13, D13	10.0	12.67	35.315	1.10	0.17
	1	1.57	10.0	0.55	D13, D13	10.0	12.67	35.315	1.10	0.05
II	9	1.77	10.0	0.62	D13, D13	10.0	12.67	35.315	1.10	0.06
	8	1.76	10.0	0.62	D13, D13	10.0	12.67	35.315	1.10	0.05
	7	1.77	10.0	0.62	D13, D13	10.0	12.67	35.315	1.10	0.06
	6	1.76	10.0	0.62	D13, D13	10.0	12.67	35.315	1.10	0.05
	5	1.76	10.0	0.62	D13, D13	10.0	12.67	35.315	1.10	0.05
	4	1.76	10.0	0.62	D13, D13	10.0	12.67	35.315	1.10	0.05
	3	1.79	10.0	0.63	D13, D13	10.0	12.67	35.315	1.10	0.06
	2	1.71	10.0	0.60	D13, D13	10.0	12.67	35.315	1.10	0.05
	1	0.91	10.0	0.32	D13, D13	10.0	12.67	35.315	1.10	0.03
III	9	15.06	10.0	5.55	D13, D13	10.0	12.67	35.315	1.10	0.47
	8	14.26	10.0	5.24	D13, D13	10.0	12.67	35.315	1.10	0.44
	7	13.64	10.0	5.00	D13, D13	10.0	12.67	35.315	1.10	0.42
	6	13.79	10.0	5.06	D13, D13	10.0	12.67	35.315	1.10	0.43
	5	14.40	10.0	5.30	D13, D13	10.0	12.67	35.315	1.10	0.45
	4	14.33	10.0	5.27	D13, D13	10.0	12.67	35.315	1.10	0.45
	3	13.54	10.0	4.97	D13, D13	10.0	12.67	35.315	1.10	0.42
	2	11.00	10.0	4.00	D13, D13	10.0	12.67	35.315	1.10	0.34
	1	0.00	10.0	0.00	D13, D13	10.0	12.67	35.315	—	0.00

*: reinforcement is determined by serviceability verification

viii) Crack width of serviceability verification (Permanent state / Lateral reinforcement / Longitudinal direction)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	3.33	10.0	D13, D13	10.0	12.67	29.683	0.0126	0.0040 × 10.0 = 0.0400	0.32
	8	3.19	10.0	D13, D13	10.0	12.67	28.435	0.0123		0.31
	7	3.20	10.0	D13, D13	10.0	12.67	28.524	0.0123		0.31
	6	3.20	10.0	D13, D13	10.0	12.67	28.524	0.0123		0.31
	5	3.20	10.0	D13, D13	10.0	12.67	28.524	0.0123		0.31
	4	3.21	10.0	D13, D13	10.0	12.67	28.613	0.0123		0.31
	3	3.17	10.0	D13, D13	10.0	12.67	28.257	0.0122		0.31
	2	2.47	10.0	D13, D13	10.0	12.67	22.017	0.0107		0.27
	1	0.71	10.0	D13, D13	10.0	12.67	6.329	0.0067		0.17
II	9	0.81	10.0	D13, D13	10.0	12.67	7.220	0.0069	0.0040 × 10.0 = 0.0400	0.17
	8	0.80	10.0	D13, D13	10.0	12.67	7.131	0.0069		0.17
	7	0.81	10.0	D13, D13	10.0	12.67	7.220	0.0069		0.17
	6	0.80	10.0	D13, D13	10.0	12.67	7.131	0.0069		0.17
	5	0.80	10.0	D13, D13	10.0	12.67	7.131	0.0069		0.17
	4	0.80	10.0	D13, D13	10.0	12.67	7.131	0.0069		0.17
	3	0.81	10.0	D13, D13	10.0	12.67	7.220	0.0069		0.17
	2	0.78	10.0	D13, D13	10.0	12.67	6.953	0.0068		0.17
	1	0.41	10.0	D13, D13	10.0	12.67	3.655	0.0060		0.15
III	9	6.84	10.0	D13, D13	10.0	12.67	60.970	0.0205	0.0040 × 10.0 = 0.0400	0.51
	8	6.48	10.0	D13, D13	10.0	12.67	57.761	0.0197		0.49
	7	6.20	10.0	D13, D13	10.0	12.67	55.265	0.0191		0.48
	6	6.27	10.0	D13, D13	10.0	12.67	55.889	0.0192		0.48
	5	6.54	10.0	D13, D13	10.0	12.67	58.296	0.0199		0.50
	4	6.51	10.0	D13, D13	10.0	12.67	58.028	0.0198		0.50
	3	6.15	10.0	D13, D13	10.0	12.67	54.819	0.0190		0.48
	2	5.00	10.0	D13, D13	10.0	12.67	44.569	0.0164		0.41
	1	0.00	10.0	D13, D13	10.0	12.67	0.000	0.0051		0.13

ix) Safety verification (Permanent state / Vertical reinforcement / Longitudinal direction)

$B=100\text{cm}$

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	10.0	0.00	D13	20.0	6.34	18.777	—	0.00
	8	1.15	10.0	0.40	D13	20.0	6.34	18.777	1.10	0.07
	7	1.22	10.0	0.43	D13	20.0	6.34	18.777	1.10	0.07
	6	1.20	10.0	0.42	D13	20.0	6.34	18.777	1.10	0.07
	5	1.17	10.0	0.41	D13	20.0	6.34	18.777	1.10	0.07
	4	1.15	10.0	0.40	D13	20.0	6.34	18.777	1.10	0.07
	3	1.37	10.0	0.48	D13	20.0	6.34	18.777	1.10	0.08
	2	2.26	10.0	0.80	D13	20.0	6.34	18.777	1.10	0.13
	1	9.46	10.0	3.42	D16	20.0	9.93	28.427	1.10	0.37
II	9	0.00	10.0	0.00	D13	20.0	6.34	18.777	—	0.00
	8	0.27	10.0	0.09	D13	20.0	6.34	18.777	1.10	0.02
	7	0.37	10.0	0.13	D13	20.0	6.34	18.777	1.10	0.02
	6	0.34	10.0	0.12	D13	20.0	6.34	18.777	1.10	0.02
	5	0.25	10.0	0.09	D13	20.0	6.34	18.777	1.10	0.01
	4	0.25	10.0	0.09	D13	20.0	6.34	18.777	1.10	0.01
	3	0.44	10.0	0.15	D13	20.0	6.34	18.777	1.10	0.03
	2	1.10	10.0	0.39	D13	20.0	6.34	18.777	1.10	0.06
	1	5.49	10.0	1.96	D16	20.0	9.93	28.427	1.10	0.21
III	9	0.00	10.0	0.00	D13	20.0	6.34	18.777	—	0.00
	8	2.38	10.0	0.84	D13	20.0	6.34	18.777	1.10	0.14
	7	2.28	10.0	0.81	D13	20.0	6.34	18.777	1.10	0.13
	6	2.30	10.0	0.81	D13	20.0	6.34	18.777	1.10	0.13
	5	2.40	10.0	0.85	D13	20.0	6.34	18.777	1.10	0.14
	4	2.38	10.0	0.84	D13	20.0	6.34	18.777	1.10	0.14
	3	2.25	10.0	0.79	D13	20.0	6.34	18.777	1.10	0.13
	2	1.82	10.0	0.64	D13	20.0	6.34	18.777	1.10	0.11
	1	0.00	10.0	0.00	D16	20.0	9.93	28.427	—	0.00

*: reinforcement is determined by serviceability verification

x) Crack width of serviceability verification (Permanent state / Vertical reinforcement / Longitudinal direction)

Young's Modulus Ratio $N=7.1$, $B=100\text{cm}$

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	10.0	D13	20.0	6.34	0.000	0.0040 × 10.0 = 0.0400	0.15	
	8	0.52	10.0	D13	20.0	6.34	8.975		0.0085	0.21
	7	0.55	10.0	D13	20.0	6.34	9.493		0.0086	0.22
	6	0.55	10.0	D13	20.0	6.34	9.493		0.0086	0.22
	5	0.53	10.0	D13	20.0	6.34	9.147		0.0085	0.21
	4	0.52	10.0	D13	20.0	6.34	8.975		0.0085	0.21
	3	0.62	10.0	D13	20.0	6.34	10.701		0.0090	0.23
	2	1.03	10.0	D13	20.0	6.34	17.777		0.0110	0.28
	1	4.30	10.0	D16	20.0	9.93	48.322		0.0199	0.50
II	9	0.00	10.0	D13	20.0	6.34	0.000	0.0040 × 10.0 = 0.0400	0.15	
	8	0.12	10.0	D13	20.0	6.34	2.071		0.0064	0.16
	7	0.17	10.0	D13	20.0	6.34	2.934		0.0067	0.17
	6	0.15	10.0	D13	20.0	6.34	2.589		0.0066	0.17
	5	0.12	10.0	D13	20.0	6.34	2.071		0.0064	0.16
	4	0.12	10.0	D13	20.0	6.34	2.071		0.0064	0.16
	3	0.20	10.0	D13	20.0	6.34	3.452		0.0069	0.17
	2	0.50	10.0	D13	20.0	6.34	8.630		0.0084	0.21
	1	2.50	10.0	D16	20.0	9.93	28.094		0.0140	0.35
III	9	0.00	10.0	D13	20.0	6.34	0.000	0.0040 × 10.0 = 0.0400	0.15	
	8	1.08	10.0	D13	20.0	6.34	18.640		0.0113	0.28
	7	1.04	10.0	D13	20.0	6.34	17.950		0.0111	0.28
	6	1.04	10.0	D13	20.0	6.34	17.950		0.0111	0.28
	5	1.09	10.0	D13	20.0	6.34	18.813		0.0113	0.28
	4	1.08	10.0	D13	20.0	6.34	18.640		0.0113	0.28
	3	1.02	10.0	D13	20.0	6.34	17.605		0.0110	0.28
	2	0.83	10.0	D13	20.0	6.34	14.325		0.0100	0.25
	1	0.00	10.0	D16	20.0	9.93	0.000		0.0058	0.15

-End -