Part 8 Deep Mixing Method (Draft Final)

March 2026
Reference

1. Technical Notes

1-1. Characteristics of Deep Mixing Method (DMM)

The Deep Mixing Method (DMM) is a technique used to physically and chemically improve soft ground by utilizing stabilizers. This method can be primarily divided into three approaches:

✓ Slurry Type

In this method, the stabilizer is prepared as a slurry at the plant and hydraulically pumped to the tip of the deep mixing machine. The slurry and soft soil are uniformly mixed by mixing blades across the entire depth of the soft layer that requires improvement, providing columns with predetermined strength.

✓ Dry Type

The stabilizer is not turned into a slurry but transported in powder form through air transport. It fills the space excavated by the mixing blades and is mixed with the soil to provide columns.

✓ High-pressure Jet Type

This method utilizes the impact force of high-pressure jets to fracture the ground, filling the cut sections with cement-based stabilizer (replacement) or mixing some of the cut soil with the stabilizer.

Each approach utilizes chemical reactions to enhance the properties of the soft soil. These reactions include hydration reactions between the stabilizer and soil water, ion exchange reactions between hydration products and clay minerals, and pozzolanic reactions. Through these processes, the improved ground achieves enhanced stability and durability.

The technical standards and guidelines are outlined in TCVN 11820 Part 4-2: 2020 and TCVN 9403. Additionally, the Cement Deep Mixing Method Manual from Japan, along with other international manuals, can be utilized as references.

1-2. Typical Improvement Pattern

In selecting an improvement pattern, it is essential to thoroughly consider stability, cost-effectiveness, and constructability. The different types of improved ground patterns are:

✓ Block Type Improvement

Huge volumes of stabilized body are formed in a ground by overlapping all the stabilized soil columns as shown in Figure 1.1 (a). This improvement can achieve the most stable improvement, but the cost is higher, and the construction period is longer than the other types of improvement. To determine the improvement area using a design method similar to that for gravity structures.

✓ Wall Type Improvement

The wall type improvement consists of long and short walls as shown in Figure 1.1 (b). The basic concept of the design is that the long walls function to transmit the external loads to the foundation ground, while the short walls function to increase the integrity of the improved ground. The improvement area is often influenced by internal stability (material strength).

✓ Grid Type Improvement

The grid type improvement is an intermediate type between the block type improvement and the wall type improvement as shown in Figure 1.1 (c). The stabilized soil columns are installed by overlapping execution so that grid shaped stabilized bodies are produced in the ground. This pattern is highly stable next to the block type improvement and its cost range between the block type and wall type improvement. There are cases where it is necessary to analyze three-dimensional internal stresses.

✓ Pile-type Improvement

In the pile-type improvement, isolated stabilized soil columns or elements are installed in rows with rectangular or triangular arrangements in a ground, as shown in Figure 1.1(d). The execution requires a relatively short period, and the volume of improvement is small. In addition to the overall stability analysis, it is necessary to analyze the stress in the columns.

✓ Other-types Improvement

Other methods include the contact-type improvement, where the improvement columns are installed in direct contact with each other, and the contact-overlap type, where the columns are placed in contact along the normal direction and overlapped along the direction perpendicular to the normal.

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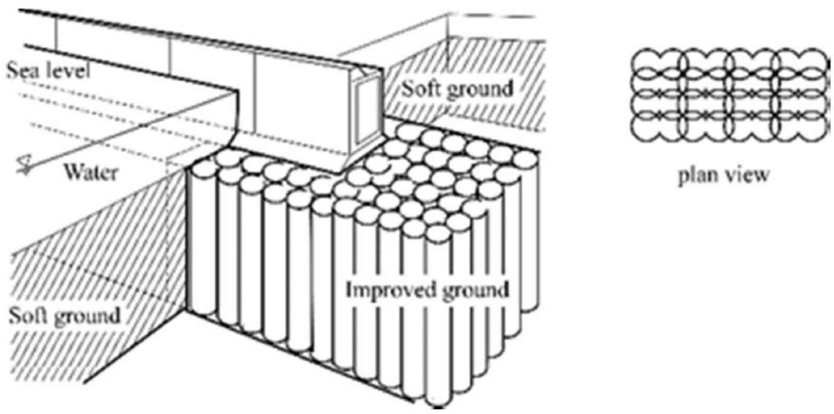
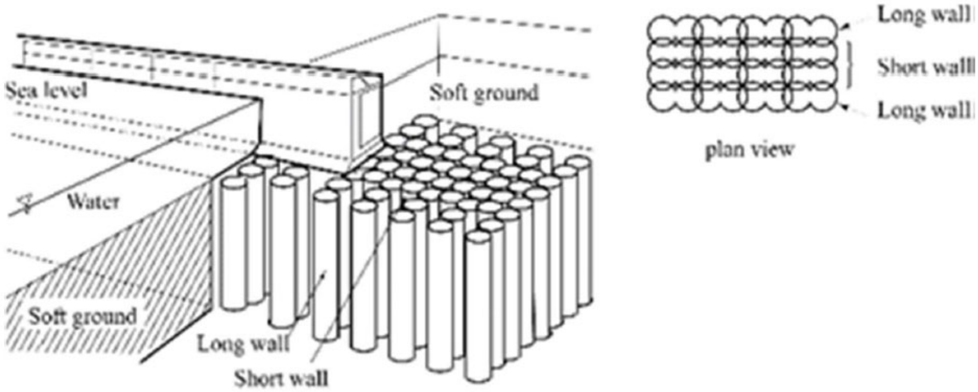
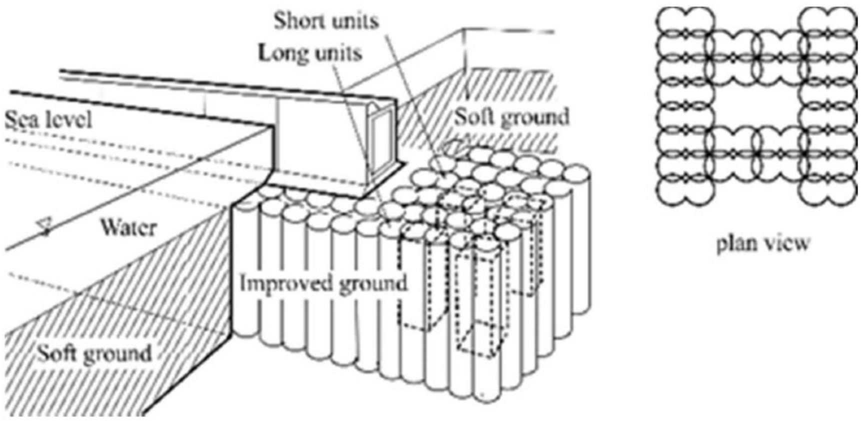


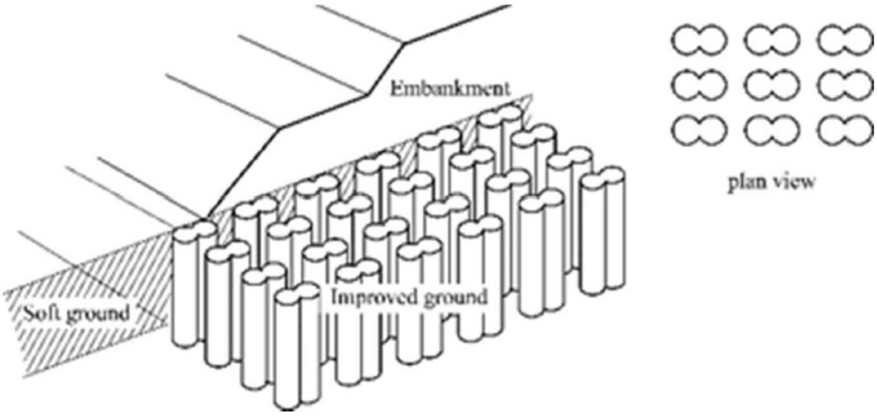
(a) Block Type Improvement



(b) Wall Type Improvement



(c) Grid Type Improvement



(d) Pile-type Improvement

Source: TCVN 11820-4-2-2020

Figure 1.1- Typical Improvement Patterns

Table 1.1- Comparison of Typical Improvement Patterns

Item	(a) Block Type Improvement	(b) Wall Type Improvement	(c) Grid Type Improvement	(d) Pile-type Improvement
Overview	Full overlapping columns forming a block-like mass	Alternating long and short walls forming a wall-like structure	Grid-shaped overlapping columns forming a lattice structure	Isolated columnar improvements arranged in a rectangular or triangular grid
Stability	Very high (most stable)	Moderate (long walls resist load)	High (next to block type)	Low to moderate (composite ground behavior)
Typical Use	Heavy structures (e.g., gravity-type quay walls)	Retaining walls, revetments, slope stabilization	Medium-load structures	Light structures, embankments, ground improvement under soft ground
Construction Cost	High	Moderate	Moderate to high	Low
Construction Duration	Long	Moderate	Moderate	Short
Improvement Ratio	Very high (~100%)	High (mainly at wall locations)	Medium to high (depends on spacing)	Low (typically 30–50%)
Design Approach	Treated as a structural body	Long walls are designed structurally	Requires 3D stress analysis in some cases	Treated as composite ground with stress distribution
Seismic Performance	Excellent (less deformation or cracking)	Moderate (deformation may concentrate in wall areas)	Depends on connectivity (may require verification)	Depends on interaction with untreated soil (more sensitive)
Ground Reaction	Entire structure resists external forces	Reaction occurs beneath long walls	Acts collectively as a grid	Load shared between improved and untreated zones

1-3. Basic Concept

(1) Definitions

- ✓ Stabilized soil

  Improved soil produced by the deep mixing method.
- ✓ Stabilized body

  A kind of structure formed underground with stabilized soil.
- ✓ Improved ground

  A portion of the ground where stabilized bodies and untreated soil are positioned close to each other (including untreated soil between long walls in the case of wall-types).
- ✓ Improved ground system

  A portion of the ground above the bottom face of the improved ground and between the vertical planes passing through the front toe and hind toe of the improved ground.
- ✓ External stability

  An examination of the stability of the process from the integration of improved ground and the superstructure (main construction) into a rigid body to the behavior of the rigid body until its final failure.
- ✓ Internal stability

  The examination of internal failure of a stabilized body which is stable externally.
- ✓ Fixed type

  A type of improved ground constructed by improving soft ground all the way through the bearing layer so that a stabilized body is seated on the bearing layer and transfers actions of the superstructure to the bearing layer.
- ✓ Floating type

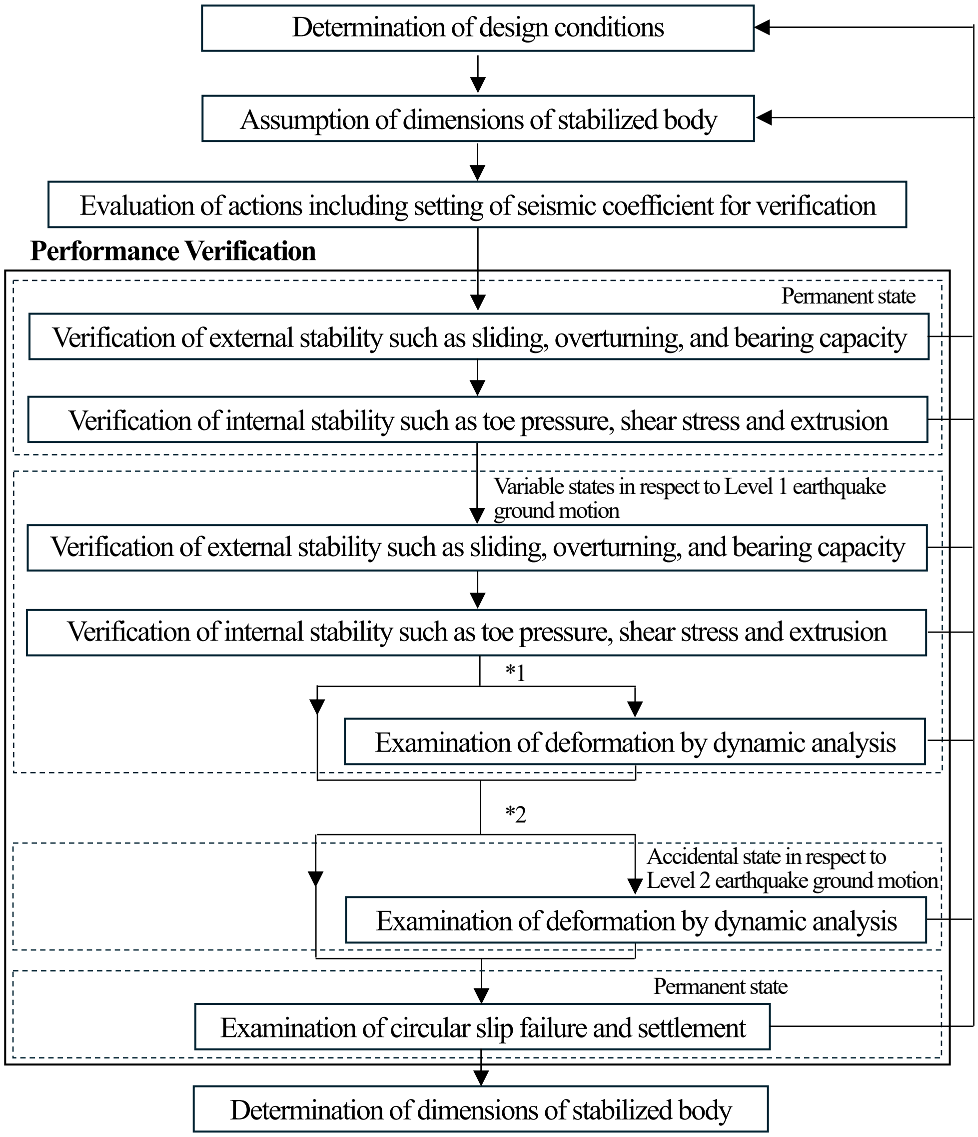
  A type of improved ground constructed by improving soft ground with a portion of soft ground remaining untreated below a stabilized body as if the stabilized body is floating on the soft ground without being seated on a bearing layer.

(2) Performance Verification and Structural Analysis of Stabilized Soil

Stabilized soil using the deep mixing method generally has extremely high strength and a high elastic modulus and extremely small strain at failure in comparison with the original ground soil. Accordingly, stabilized bodies made of stabilized soil are preferably regarded as structures subjected to examination of the stability of the structures as a whole (external stability), examination of the resistance of the structures themselves (internal stability), and, if particularly necessary, the examination of settlement, horizontal displacement and rotation of the stabilized bodies as rigid bodies.

The performance verification of the deep mixing method can be carried out with reference to TCVN 11820 Part 4-2: 2020, OCDI 2020 and Technical Manual for the Deep Mixing Method in Ports and Airports, Japan.

An example of the performance verification procedure for the deep mixing method applied to gravity-type structures is shown in Figure 1.2.



Modified from TCVN 11820 Part 4-2: 2020, Hinh 2

OCDI 2020, Part III Chapter 2 Fig. 5.5.2

*1: Dynamic analysis can be used for the examination of deformation in respect to Level 1 seismic ground motions as needed. It is preferable to use dynamic analyses for examining deformation in cases where the widths of the improved ground are smaller than those of foundation mounds.
*2: Depending on the performance requirements of the main construction, the performance verification shall be carried out in respect to Level 2 seismic ground motions.

Source: OCDI 2020, Modified from TCVN 11820-4-2-2020

Figure 1.2- Example of the Performance Verification Procedure

1-4. Assumption of the Dimensions of Stabilized Bodies

(1) Mix Proportion Design Method for Stabilized Soil

The effectiveness of stabilized soil is primarily influenced by the soil's physical and chemical characteristics, the types of binders used, and the specific mixing and curing conditions. The choice of construction machinery, which varies widely and affects possible water-cement ratios, also plays a critical role. Therefore, determining the optimal mix ratio for stabilized soil typically involves conducting laboratory and field tests that replicate actual usage conditions.

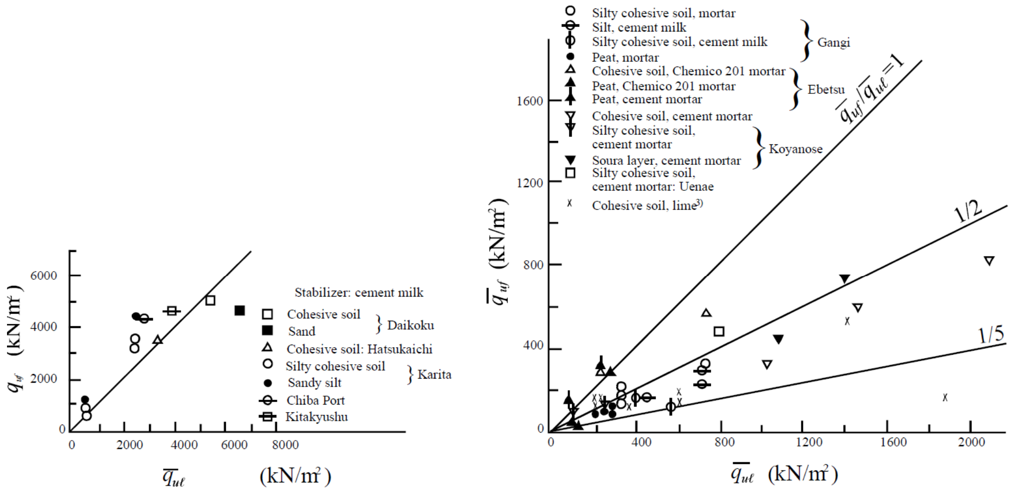
For initial mix design, it is common to base the strength of stabilized soil on data from previous projects. However, laboratory tests, which aim to gauge the soil's strength under controlled conditions, do not directly correlate to field conditions. Accurately predicting field strength from laboratory results requires an in-depth analysis of existing data (example in Figure 1.3) on how laboratory and field strengths relate, especially when using binders like normal Portland cement and lime known for high initial strength.

Where:

q_{uf} : unconfined compressive strength of field stabilized soil (kN/m^2)

q_{ul} : unconfined compressive strength from laboratory mix tests (kN/m^2)

OCDI 2020, Part III Chapter 2 Fig. 5.5.3



(a) Examples of offshore construction

(b) Examples of onshore construction

Source: OCDI 2020

Figure 1.3- Relationship between Average Strength obtained through Laboratory Mix Tests and Average Field Strength

(2) Material Strength of Stabilized Bodies

The design compressive strength f_c of stabilized bodies can be obtained using the Equation (1.1) with the standard design strength q_{uc} as a basis. In the Equation, the subscript k represents a characteristic value.

$$f_{ck}=\alpha\beta q_{uck} \tag{1.1}$$

TCVN 11820 Part 4-2: 2020, Equation (10)

Where:

f_{ck} : design compressive strength of a stabilized body (kN/m^2)

α : factor of an effective cross-sectional area

β : reliability index of an overlapped section

q_{uck} : standard design strength (kN/m^2)

The design shear strength f_{sh} and design tensile strength f_t of stabilized bodies can be obtained from the Equations (1.2) and (1.3) using the design compressive strength f_c.

$$f_{shk}=\frac{1}{2}f_{ck} \tag{1.2}$$

$$f_{tk}=0.15f_{ck}\leq 200\text{kN/m}^2 \tag{1.3}$$

TCVN 11820 Part 4-2: 2020, Equation (10)

Where:

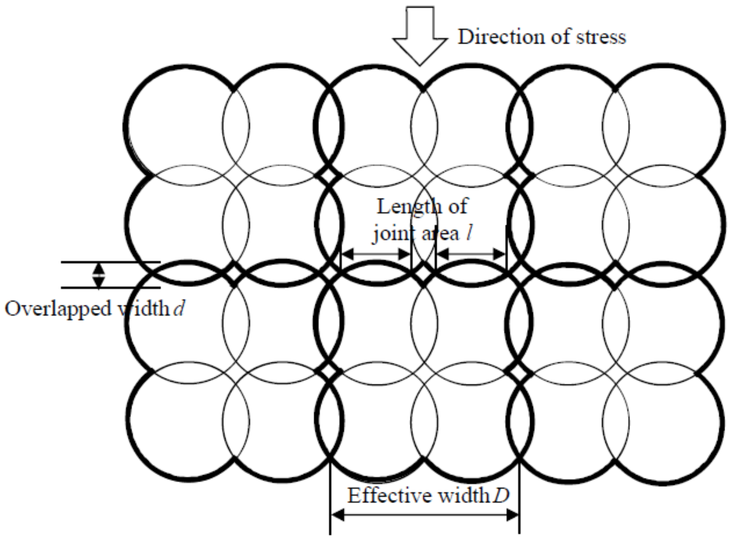
f_{sh} : design shear strength of a stabilized body (kN/m^2)

f_t : design tensile strength of a stabilized body (kN/m^2)

The performance verification of stabilized bodies is based on the assumption that stabilized bodies are made of materials with homogeneous strength. However, in actual construction work, because the stabilized bodies are formed by overlapping columns made of stabilized soil, there are cases where inhomogeneous stabilized soil is constructed underground in the form of residual untreated existing soil or overlapped sections with strength different from other sections depending on the mixing machines used and the methods of overlapping.

1) Factor for Effective Cross-sectional Area *α*

In the block-type and wall-type improved ground, because the stabilized bodies are formed by overlapping columns made of stabilized soil as shown in Figure 1.4, existing soil remains unimproved soil around the overlapped sections, thereby making the widths l of the joint areas shorter than the effective widths D of the stabilized bodies.



Modified from TCVN 11820 Part 4-2: 2020, Hinh 9

Source: Modified from TCVN 11820-4-2-2020

Figure 1.4- Concept of the Factor for Effective Cross-Sectional Area (When Using Four-Axis Construction Machines)

$$\alpha = \frac{Nl}{D} \quad (1.4)$$

Modified from TCVN 11820 Part 4-2: 2020, Equation (11)

Where:

N : Member of axes on a joint area (N=2 in the case of Figure 1.4)

2) Reliability Factor for Effective Cross-sectional Area *β*

The reliability index of overlap β is defined as a ratio of the strength of an overlapped section to that of other sections. Although the values of β differ depending on the elapsed time until new columns are joined to the existing ones, the mixing capacity of construction machines and the methods for discharging binders, the stabilized bodies can be designed with the value of β set at 1 according to the performance records.

When designing stabilized bodies using the characteristic values of standard design strength in the range of 1,500 to 2,500 kN/m^2 with an overlapped width of 30 cm or more, and a factor for effective cross-sectional area α of 0.8 or more, the value of $\alpha\beta$ can be set at 0.8 according to the performance records of the deep mixing method.

3) Relationship between Standard Design Strength, Field Strength and Laboratory Mix Tests

The relationship between the average $\overline{q_{uf}}$ of the unconfined compressive strength q_{uf} of field stabilized soil and the characteristic value q_{uck} of the standard design strength can be expressed by the Equation (1.5).

$$\overline{q_{uf}} = q_{uck} \ /(1\text{-}KV/100) \qquad (1.5)$$

Modified from TCVN 11820 Part 4-2: 2020, Equation (9)

Where:

- K : coefficient showing a normal deviation (a magnification ratio with respect to a standard deviation σ) where the value is generally set at 1.0
- V : coefficient of variation of unconfined compressive strength q_{uf} of field stabilized soil (although the coefficient varies depending on the construction machines and technologies, and is preferably set for individual cases, it can be set at V = 30-40 (%) according to examples of previous construction works)

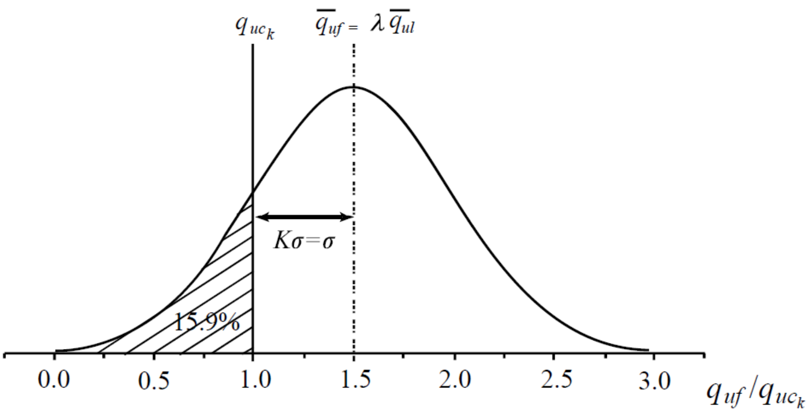
The relationship between the average $\overline{q_{uf}}$ of the unconfined compressive strength q_{uf} of field stabilized soil and the average of the unconfined compressive strength q_{ul} from laboratory mix tests can be expressed by the Equation (1.6).

$$\overline{q_{uf}} = \lambda \overline{q_{ul}} \qquad (1.6)$$

TCVN 11820 Part 4-2: 2020 Equation (9)

The value of is affected by numerous factors including the construction machines and conditions, types of object soil for improvement and binders, curing conditions and material ages. The target values for offshore construction are 0.8 to 1 when using middle to large scale work barges, and 0.5 to 1 when using small work barges; provided, however, that the value of may also be determined based on tests or performance records.

Figure 1.5 show the schematic diagram of the relationships of standard design strength q_{uck} with the average value $\overline{q_{uf}}$ of the unconfined compressive strength of the specimens for laboratory mix tests and the average value $\overline{q_{uf}}$ of the unconfined compressive strength of field stabilized soil.



Modified from TCVN 11820 Part 4-2: 2020, Hinh 8

Source: Modified from TCVN 11820-4-2-2020

Figure 1.5- Relationships of q_{uck} with $\overline{q_{uf}}$ and $\overline{q_{ul}}$

1-5. Performance Verification of Stabilized Bodies

Figure 1.6 shows a schematic diagram of loads acting on a stabilized body in the case of gravity-type revetments and quay walls.

Because untreated existing soil remains in wall-type improved ground, the loading conditions shall be set separately for untreated soil sections and stabilized soil sections for certain performance verification items.

For examinations on the external stability of improved ground systems, P_a or P_p can be determined using Coulomb's earth pressure theory under permanent states and Mononobe and Okabe theory under earthquake motions. When examining internal stability, P_a may be considered as active earth pressure. However, it is preferable that P_p be set appropriately within a range from earth pressure at rest to passive earth pressure while considering the external stability of the improved ground systems.

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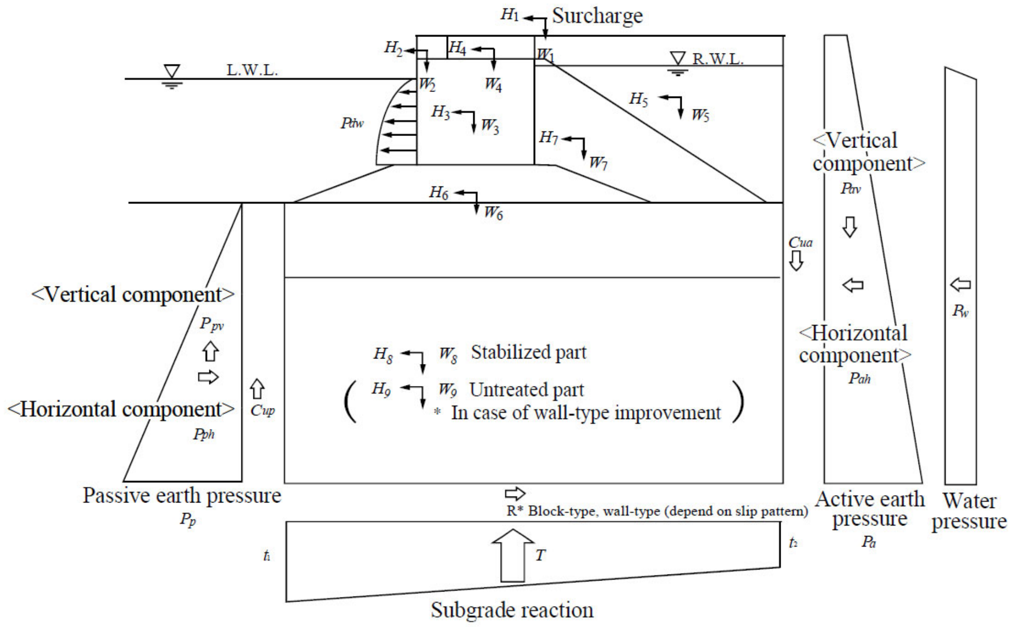


Source: TCVN 11820-4-2-2020

Figure 1.6- Loads Acting on Stabilized Body

Where:

- P_a : resultant earth pressure per unit length acting on the vertical plane of the active side (kN/m)
- P_{ah} : horizontal component of resultant earth pressure per unit length acting on the vertical plane of the active side (kN/m)
- P_{av} : vertical component of resultant earth pressure per unit length acting on the vertical plane of the active side (kN/m)
- P_p : resultant earth pressure per unit length acting on the vertical plane of the passive side (kN/m)
- P_{ph} : horizontal component of resultant earth pressure per unit length acting on the vertical plane of the passive side (kN/m)
- P_{pv} : vertical component of resultant earth pressure per unit length acting on the vertical plane of the passive side (kN/m)
- P_w : resultant residual water pressure per unit length (kN/m)
- P_{dw} : resultant dynamic water pressure per unit length (kN/m)
- W_1 to W_9 : weight per unit of length of each part (kN/m)
- H_1 to H_9 : seismic inertia force per unit of length of each part (kN/m)
- C_{ua} : resultant adhesion on the vertical plane per unit length acting on the vertical plane of the active side (kN/m)

C_{up}	:	resultant adhesion on the vertical plane per unit length acting on the vertical plane of the passive side (kN/m)
R	:	shear resistance per unit length acting on the bottom of the stabilized body (kN/m)
T	:	resultant of subgrade reaction per unit of length acting on the bottom of stabilized body (kN/m)
t_1, t_2	:	subgrade reaction at toes of stabilized body (kN/m^2)

In the performance verification of soil layers subjected to liquefaction during the actions of seismic ground motions, it is necessary to consider the dynamic water pressure on stabilized bodies during the actions of seismic ground motions.

(1) External Stability of Improved Ground

In the performance verification of the external stability of improved ground, the following items shall be examined, assuming that the stabilized bodies and main construction behave integrally. It shall be noted that the following items provide descriptions for the cases of gravity-type revetments and quay walls; however, the same descriptions can also be applied to breakwaters, provided that actions due to waves and other relevant factors are appropriately set.

1) Verification of Sliding Failure

Improved ground shall secure the required stability with respect to sliding failures.

It is necessary to conduct performance verification of wall-type improved ground for two patterns, namely, the sliding failure pattern 1 (refer to Figure 1.7(a)), which considers the frictional resistance of the bottom of the improved ground as a whole as resistance to slip failure, and the sliding failure pattern 2 (refer to Figure 1.7(b)), which considers t the frictional resistance directly under long walls and the shearing resistance of the unimproved ground between walls, while considering the improved ground to be a structure in which the stabilized ground long walls fully demonstrate shear strength. For an examination of stability with respect to sliding failures, the Equation (1.7) can be used. In the Equation, the subscripts k and d denote the characteristic value and design value, respectively.

Modified from TCVN 11820 Part 4-2: 2020, Equation (2a), (2b), (2c), (2d)

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

$$R_k=P_{phk}+R_{1k}+R_{2k}\ \text{(Sliding failure pattern 1)} \tag{1.7}$$

$$R_k=P_{phk}+R_{1k}+R_{3k}\ \text{(Sliding failure pattern 2)}$$

$$S_k=P_{ahk}+P_{Wk}+P_{dwk}+H_{ik}$$

Where:

OCDI 2020, Part III Chapter 2 Equation (5.5.8)

$$R_{1k}=\mu_k\left(\sum W_{ik}+W_{8k}+P_{avk}-P_{pvk}+C_{uak}-C_{upk}\right)$$

$$R_{2k}=\mu_k W_{9k}$$

$$R_{3k}=C_{uk}BL_s/(L_l+L_s)$$

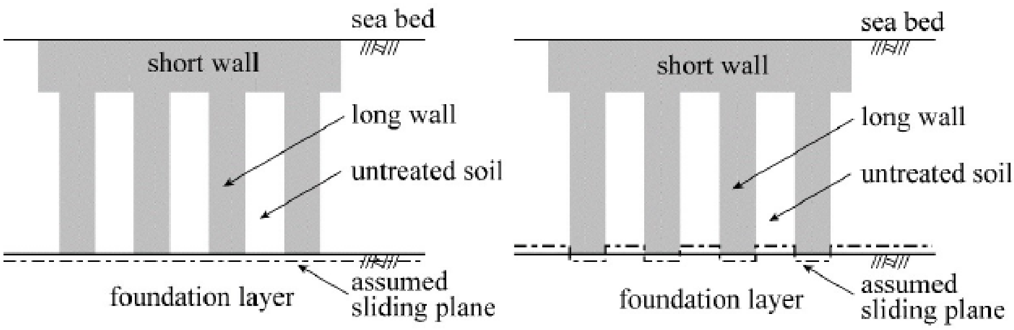
$$P_{wk}=\rho_w g(RWL_k-WL_k)\left\{\frac{1}{2}(RWL_k-WL_k)+h_L+WL_k\right\}$$

$$P_{dwk}=\frac{7}{12}k_{h3}\rho_w g(h_1+WL_k)^2$$

$$H_{ik}=k_{h1}\sum W_{nik}+k_{h2}(W_{n8k}+W_{n9k})$$

R_k	:	characteristic value related to a resistance term (kN/m)
S_k	:	characteristic value related to a load term (kN/m)
R_1	:	frictional resistance of bearing ground per unit length acting on the bottom of a stabilized body (kN/m)
R_2	:	frictional resistance of bearing ground per unit length acting on the bottom of an untreated soil section (kN/m)
R_2	:	shearing resistance per unit length acting on the bottom of an untreated soil section (kN/m)
P_w	:	residual water pressure per unit length (kN/m)
P_{dw}	:	dynamic water pressure during an earthquake per unit length (kN/m)
H_i	:	inertia force per unit length acting on each section (kN/m)
W_i	:	weight per unit length of materials (such as surcharge, superstructures, main construction, foundation mounds, backfill and reclamation) on improved ground constituting an improved ground system (kN/m)
W_8	:	weight of a stabilized body per unit length (kN/m)
W_9	:	weight of untreated soil between long walls per unit length (kN/m)
B	:	width of a stabilized body (m)
R_s	:	ratio of short walls to long walls in a stabilized body $L_s / (L_l + L_s)$ (refer to Figure 1.8)
μ	:	static friction coefficient
C_u	:	shear strength on the bottom of untreated soil (kN/m^2)
P_{ah}	:	horizontal component of the earth pressure per unit length acting on the vertical plane of the active side (kN/m)
P_{av}	:	vertical component of the earth pressure per unit length acting on the vertical plane of the active side (kN/m)
P_{ph}	:	horizontal component of the earth pressure per unit length acting on the vertical plane of the passive side (kN/m)
P_{pv}	:	vertical component of the earth pressure per unit length acting on the vertical plane of the passive side (kN/m)
C_{ua}	:	resultant adhesion on a vertical plane per unit length acting on the vertical plane of the active side (kN/m)
C_{up}	:	resultant adhesion on a vertical plane per unit length acting on the vertical plane of the passive side (kN/m)
$\rho_w g$	:	unit weight of seawater (kN/m^3)
RWL	:	residual water level (m)
WL	:	water level on the offshore side (m)
h_L	:	water depth at the bottom of a stabilized body (m)
h_1	:	water depth on the offshore side of a structure (m)
k_{h1}	:	seismic coefficient for verification when calculating the inertia force acting on materials (surcharge, superstructures, main construction, foundation mounds, backfill and reclamation) over improved ground constituting an improved ground system

k_{h2}	:	seismic coefficient for verification when calculating the inertia force acting on improved ground
k_{h3}	:	seismic coefficient for verification when calculating the earth pressure and dynamic water pressure acting on an improved ground system
W_{n1}	:	weight per unit length of materials (surcharge, superstructures, main construction, foundation mounds, backfill and reclamation) over improved ground constituting an improved ground system (saturated unit weight when submerged) (kN/m)
W_{n8}	:	weight per unit length of a stabilized body (saturated unit weight when submerged) (kN/m)
W_{n9}	:	weight per unit length of untreated soil between long walls (saturated unit weight when submerged) (kN/m)
γ_R	:	partial factor multiplied by the resistance term
γ_S	:	partial factor multiplied by the load term
m	:	adjustment factor



Sliding failure pattern 1 Sliding failure pattern 2

Source: TCVN 11820-4-2-2020

Figure 1.7- Sliding Failure Patterns

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The partial and adjustment factors in the Equation (1.7) can be selected from Table 1.2. If a corresponding column in the table has the symbol "-", the value in parentheses in the column can be used for performance verification for convenience.

Table 1.2- Partial Factors to be used in the Verification of Sliding Failure

Mode of failure		Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
External stability of the stabilized body (Sliding failure: permanent state)	Sliding failure pattern 1	0.90	1.09	– (1.00)
	Sliding failure pattern 2	0.91	1.10	– (1.00)
External stability of the stabilized body (Sliding failure: variable state of seismic ground motions)		– (1.00)	– (1.00)	1.00

TCVN 11820 Part 4-2: 2020, Bang 1

Source: TCVN 11820-4-2-2020

2) Verification of Overturning Failure

Improved ground shall secure the required stability with respect to overturning. The Equation (1.8) can be used for the verification of stability with respect to the overturning of wall-type improved ground. In the Equation, the subscripts k and d denote the characteristic value and design value, respectively.

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

$$R_k=P_{phk}y_p+\sum(W_{ik}x_i)+W_{8k}x_8+W_{9k}x_9+P_{avk}x_{av}+C_{uak}x_{Cua}$$

(1.8)

(Permanent situation)

$$S_k=P_{ahk}y_a+P_{Wk}y_w$$

(Variable situation in respect to Level 1 earthquake ground motion)

$$S_k=P_{ahk}y_a+P_{Wk}y_w+P_{dwk}y_{dw}+\sum H_{ik}y_i$$

Where:

$$P_{wk}=\rho_w g(RWL_k-WL_k)\left\{\frac{1}{2}(RWL_k-WL_k)+h_L+WL_k\right\}$$

$$P_{dwk}=\frac{7}{12}k_{h3}\rho_w g(h_1+WL_k)^2$$

$$\sum H_{ik}=k_{h1}\sum W_{nik}+k_{h2}(W_{n8k}+W_{n9k})$$

- R_k : characteristic value related to a resistance term (kN/m·m)
- S_k : characteristic value related to a load term (kN/m·m)
- W_i : weight per unit length of materials (such as surcharge, superstructures, main construction, foundation mounds, backfill and reclamation) on improved ground constituting an improved ground system (kN/m)
- W_{n1} : weight per unit length of materials (surcharge, superstructures, main construction, foundation mounds, backfill and reclamation) over improved ground constituting an improved ground system (saturated unit weight when submerged) (kN/m)
- W_8 : weight of a stabilized body per unit length (kN/m)
- W_9 : weight of untreated soil between long walls per unit length (kN/m)
- W_{n8} : weight per unit length of a stabilized body (saturated unit weight when submerged) (kN/m)
- W_{n9} : weight per unit length of untreated soil between long walls (saturated unit weight when submerged) (kN/m)
- H_i : inertia force per unit length acting on each section of an improved ground system (kN/m)
- P_{ph} : horizontal component of the earth pressure per unit length acting on the vertical plane of the passive side (kN/m)
- P_{av} : vertical component of the earth pressure per unit length acting on the vertical plane of the active side (kN/m)
- P_{ah} : horizontal component of the earth pressure per unit length acting on the vertical plane of the active side (kN/m)
- C_{ua} : cohesion on a vertical plane per unit length acting on the vertical plane of the active side (kN/m)
- P_w : residual water pressure per unit length (kN/m)
- P_{dw} : dynamic water pressure during an earthquake per unit length (kN/m)

Modified from TCVN 11820 Part 4-2: 2020, Equation (3a), (3b)

RWL : residual water level (m)

WL : water level on the offshore side (m)

k_{h1} : seismic coefficient for verification when calculating the inertia force acting on materials (surcharge, superstructures, main construction, foundation mounds, backfill and reclamation) over improved ground constituting an improved ground system

k_{h2} : seismic coefficient for verification when calculating the inertia force acting on improved ground

k_{h3} : seismic coefficient for verification when calculating the earth pressure and dynamic water pressure acting on an improved ground system

x_i, x_{av}, x_{cua} : distances from the action lines of the vertical force acting on improved ground to the front toe of a stabilized body (m)

$y_i, y_p, y_a, y_w, y_{dw}$: heights from the action lines of the horizontal force acting on improved ground to bottom of a stabilized body (m)

γ_R : partial factor multiplied by the resistance term

γ_S : partial factor multiplied by the load term

m : adjustment factor

The partial factors to be used in the verification of the overturning of improved ground can be selected from Table 1.3. If a corresponding column in the table has the symbol "-", the value in parentheses in the column can be used for performance verification for convenience.

Table 1.3- Partial Factors to be used in the Verification of Overturning Failure

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
External stability of the stabilized body (Overturning failure: permanent state)	0.97	1.18	– (1.00)
External stability of the stabilized body (Overturning failure: variable state of seismic ground motions)	– (1.00)	– (1.00)	1.10

Source: TCVN 11820-4-2-2020

TCVN 11820 Part 4-2: 2020, Bang 2

3) Verification of Bearing Capacity Failure

Improved ground shall secure the required stability with respect to the failure of the bearing capacity of the original ground under the bottom of the improved ground. In the verification of the bearing capacity of block-type improved ground refer to Terzaghi's bearing capacity formula for sandy ground.

In the case of wall-type improved ground with sandy ground as the bearing ground, the bearing capacity can be verified by the Equation (1.9) using toe pressure t_1 and t_2 while taking into consideration the effect of the mutual interference of long walls. In the Equation, the subscripts k and d denote the characteristic value and design value, respectively.

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

$$R_k=q_{apk}+q_{ar1k} \text{ (In the case of } \frac{1}{\eta} \geq 3)$$

$$R_k=q_{apk}+q_{ark} \text{ (In the case of } 1 \leq \frac{1}{\eta} < 3) \qquad (1.9)$$

$$S_k=t_{1k}, t_{2k}$$

Modified from TCVN 11820 Part 4-2: 2020, Equation (4)

Where:

$$q_{apk}=\frac{1}{m_B}p_{0k}(N_{qk}\text{-}1)+p_0$$

$$q_{ar1k}=\frac{1}{m_B}w_k\frac{L_l}{2}N_{rk}$$

$$q_{ar2k}=\frac{1}{m_B}w_k\frac{B}{2}N_{rk}$$

$$q_{ark}=q_{ar1k}+\frac{1}{2}(q_{ar2k}\text{-}\,q_{ar1k})(3\text{-}\,\frac{1}{\eta})$$

$$\eta = \frac{L_l}{L_l+L_s}$$

γ_R : partial factor multiplied by a resistance term
γ_S : partial factor multiplied by a load term
m : adjustment factor
t_1, t_2 : toe pressure (kN/m^2)
m_B : adjustment factor with respect to the bearing capacity
N_q, N_r : bearing capacity coefficient (by Terzaghi's formula)
p_0 : effective overburden pressure to a sandy bearing layer (kN/m^2)
w : unit weight of bearing ground (submerged unit weight when submerged) (kN/m^3)
L_l : length of a long wall in the normal direction (m) (refer to Figure 1.10)
L_s : length of a short wall in the normal direction (m) (refer to Figure 1.10)
B : improvement width (m) (refer to Figure 1.10)

The partial factors to be used in the verification of the bearing capacity can be selected from Table 1.4. If a corresponding column in the table has the symbol "-", the value in parentheses in the column can be used for performance verification for convenience.

The partial factors listed in Table 1.4 are set with reference to the safety levels in the past standards.

Table 1.4- Partial Factors to be used in the Verification of Bearing Capacity Failure

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m	Adjustment factor with respect to bearing capacity m_B
External stability of the stabilized body (Failure of bearing capacity: permanent state)	0.49	1.15	– (1.00)	– (1.00)
External stability of the stabilized body (Failure of bearing capacity: variable state of seismic ground motions)	– (1.00)	– (1.00)	– (1.00)	1.50

Source: TCVN 11820-4-2-2020

TCVN 11820 Part 4-2: 2020, Bang 3

4) Verification of Circular Slip Failure

The overall stability of the improved ground, the superstructure and the surrounding soil should be evaluated by circle slip failure analysis with Equation (1.10). As the strength of stabilized soil is very high value, a circular slip failure analysis passing through the improved ground is not necessary in many cases, as shown in Figure 1.8.

In cases where sufficient bearing capacity is assured, circular slip failure analysis is not necessary in many cases. The circular slip failure analysis in variable situation is not specified in the design standard.

TCVN 11820 Part 4-2: 2020, Equation (13)

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

$$R_k=r\cdot(\tau_c\cdot l_c+\tau_f\cdot l_f+\tau_i\cdot l_i) \tag{1.10}$$

$$S_k=W_{sp}\cdot x_{sp}+W_m\cdot x_m+W_{bf}\cdot x_{bf}+W_f\cdot x_f+W_c\cdot x_c+W_i\cdot x_i+W_{su}\cdot x_{su}+W_{Rw}\cdot x_{Rw}$$

Where:

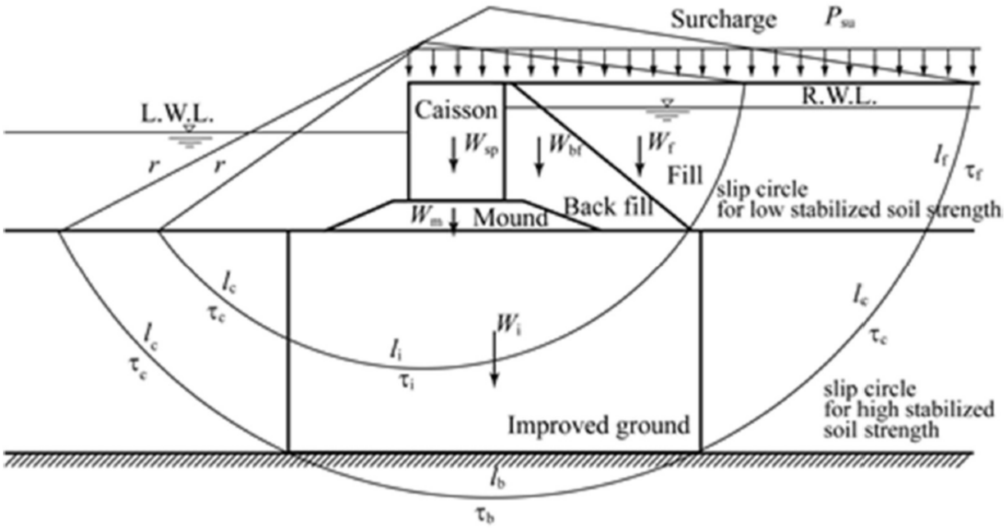
- γ_R : partial factor multiplied by a resistance term
- γ_S : partial factor multiplied by a load term
- m : adjustment factor
- l_c : length of circular arc in surrounding ground (m)
- l_f : length of circular arc in fill (m)
- l_i : length of circular arc in improved ground (m)
- r : radius of slip circle (m)
- x_{bf} : horizontal distance of weight of backfill from center of slip circle (m)
- x_c : horizontal distance of weight of surrounding ground from center of slip circle (m)
- x_f : horizontal distance of weight of fill from center of slip circle (m)
- x_i : horizontal distance of weight of improved ground from center of slip circle (m)
- x_m : horizontal distance of weight of mound from center of slip circle (m)
- x_{sp} : horizontal distance of weight of superstructure from center of slip circle (m)
- x_{su} : horizontal distance of total surcharge force from center of slip circle (m)
- y_{Rw} : vertical distance of total residual water force from center of

slip circle (m)

τ_c : shear strength of surrounding ground (kN/m^2)

τ_i : average shear strength of improved ground (kN/m^2)

τ_f : shear strength of fill (kN/m^2)



TCVN 11820 Part 4-2: 2020, Hinh 12

Source: TCVN 11820-4-2-2020

Figure 1.8- Circular Slip Failure Analysis

Table 1.5- Partial Factors to be used in the Verification of Circular Slip Failure

TCVN 11820 Part 4-2: 2020, Bang 8

State	Partial factor multiplied by resistance term, γ_R	Partial factor multiplied by load term, γ_S	Adjustment factor, m
Permanent state	1.00	1.00	1.30
Seismic ground motion	-	-	-

Source: TCVN 11820-4-2-2020

(2) Internal Stability of Improved Ground

The internal stability of the block-type and wall-type improved grounds can be examined by the method presented below; provided, however, that the examination by FEM analysis is preferable in cases where the shapes of the stabilized bodies are complex, or the depths of the stabilized bodies are large in comparison with their widths.

1) Verification of Toe Pressure

The verification of internal stability with respect to the toe pressure at the bottom of the stabilized bodies can be performed using the Equation (1.11) while considering the effect of the confining pressure acting on the improved ground. In the Equation, the subscripts k and d denote the characteristic value and design value, respectively.

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

$$R_k = f_{ck},\ S_k = t_{1,2k} - K \sum (w_{ik}, h_i) \tag{1.11}$$

TCVN 11820 Part 4-2: 2020, Equation (6)

Where:

R_k	:	characteristic value related to a resistance term (kN/m)
S_k	:	characteristic value related to a load term (kN/m)
f_c	:	design compressive strength (kN/m^2)
$t_{1,2}$	:	toe pressure (kN/m^2)
K	:	coefficient of earth pressure
w_i	:	unit weight of untreated soil (submerged unit weight when submerged) (kN/m^3)
h_i	:	thickness of untreated soil layers (m)
γ_R	:	partial factor multiplied by a resistance term
γ_S	:	partial factor multiplied by a load term
m	:	adjustment factor

However, it is necessary to determine the value of the confining pressure $K\sum(w_{ik},h_i)$of untreated soil acting on the bottom edges of the stabilized bodies while taking into consideration the improvement patterns and the external stability of improved ground.

The partial factors to be used in the verification of the toe pressure can be selected from Table 1.6. If a corresponding column in the table has the symbol "-", the value in parentheses in the column can be used for performance verification for convenience.

Table 1.6- Partial Factors to be used in the Verification of Toe Pressure

TCVN 11820 Part 4-2: 2020, Bang 4

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Internal stability of the stabilized body (Toe pressure: permanent state)	0.72	1.33	– (1.00)
Internal stability of the stabilized body (Toe pressure: variable state of seismic ground motions)	– (1.00)	– (1.00)	1.50

Source: TCVN 11820-4-2-2020

2) Verification of Shear Failures along Vertical Plane

The verification of internal stability with respect to shear stresses along the vertical planes (Figure 1.8) can be performed for the long wall and short wall sections using the Equations (1.12) and (1.13), respectively. In these Equations, the subscripts k and d denote the characteristic value and design value, respectively.

i) Long walls

TCVN 11820 Part 4-2: 2020, Equation (7)

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

$$R_k=\frac{1}{2}\alpha\beta q_{uck},\ S_k=(T_{lk}-W_{lk})/A \tag{1.12}$$

Where:

R_k	:	characteristic value related to a resistance term (kN/m)
S_k	:	characteristic value related to a load term (kN/m)
α	:	factor of an effective cross-sectional area
β	:	reliability index of an overlapped section between improvement piles
T_l		subgrade reaction acting on an area from the front toe of

improved ground to B_l (kN)

q_{uc} : standard design strength (kN/m^2)

W_l : submerged weight of a stabilized body from the front toe of improved ground to B_l (kN)

A : cross-sectional area of a stabilized body; in the case of long walls $A = D_l L_l + D_s L_s$ (m^2) (refer to Figure 1.8)

D_l, D_s : vertical length (improvement depth) of a long wall and the vertical length of a short wall (m)

L_l, L_s : lengths of long and short walls in a normal direction (m)

γ_R : partial factor multiplied by a resistance term

γ_S : partial factor multiplied by a load term

m : adjustment factor

When a foundation mound exists between a stabilized body and the superstructure, the verification of shear stress can be performed with respect to a plane considering the dispersion of loads inside the foundation mound from the position of the face line of the superstructure (refer to Figure 1.8 where is a load dispersion angle inside the foundation mound).

The partial factors to be used in the verification of the vertical shear failure of long wall can be selected from Table 1.7. If a corresponding column in the table has the symbol "-", the value in parentheses in the column can be used for performance verification for convenience.

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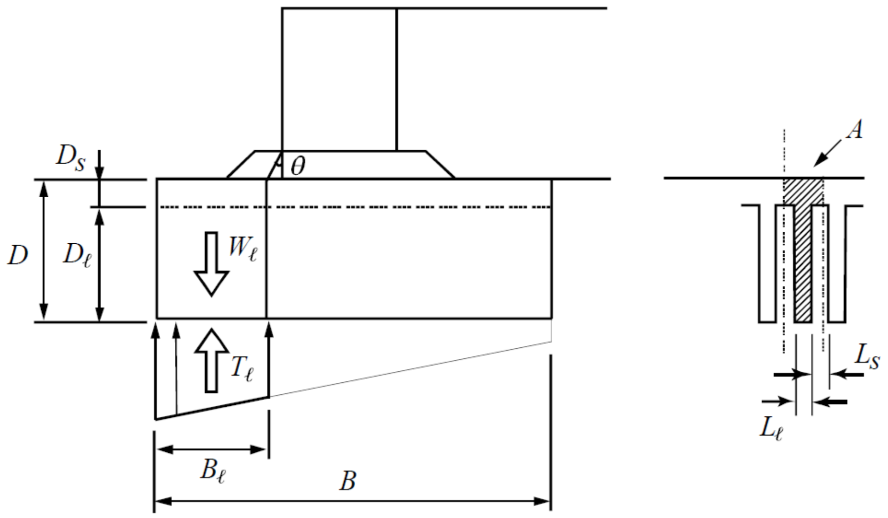


Source: TCVN 11820-4-2-2020

Figure 1.8- Schematic Calculation Diagram of Vertical Shear Stress (Long Wall)

Table 1.7- Partial Factors to be used in the Verification of Vertical Shear Failures of Long Wall Sections

TCVN 11820 Part 4-2: 2020. Bang 5

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Internal stability of the stabilized body (Vertical shear failure (long wall sections) under a permanent state)	– (1.00)	– (1.00)	1.80
Internal stability of the stabilized body (Vertical shear failure (long wall sections) under a variable state of seismic ground motions)	– (1.00)	– (1.00)	1.50

Source: TCVN 11820-4-2-2020

ii) Short walls

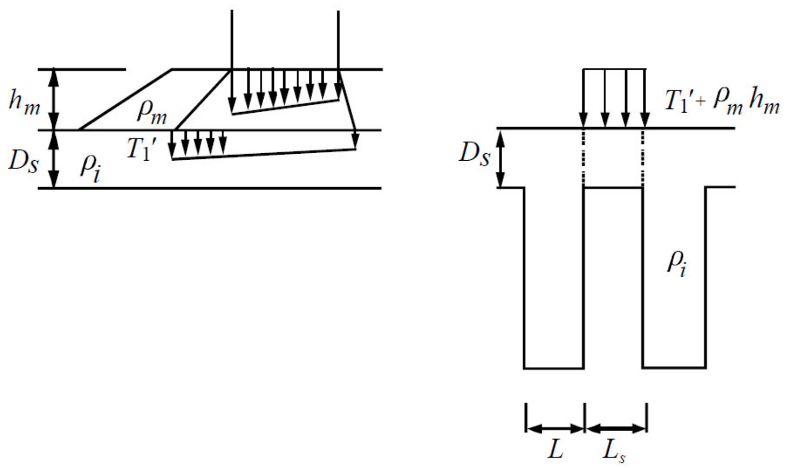
$$m \cdot \frac{S_d}{R_d} \leq 1.0;\ R_d = \gamma_R R_k;\ S_d = \gamma_S S_k$$

$$R_k = \frac{1}{2}\alpha\beta q_{uck},\ S_k = (T'_{1k} + w_{mk}h_m + w_{ik}D_s)L_s/(2D_s) \tag{1.13}$$

TCVN 11820 Part 4-2: 2020, Equation (8)

Where:

R_k	:	characteristic value related to a resistance term (kN/m)
S_k	:	characteristic value related to a load term (kN/m)
α	:	factor of an effective cross-sectional area
β	:	reliability index of an overlapped section between improvement piles
T'_1		toe pressure after dispersion inside a mound (excluding the self-weight of the mound) (kN/m^2) (refer to Figure 1.9)
q_{uc}	:	standard design strength (kN/m^2)
w_m	:	unit weight of a mound (submerged unit weight when submerged) (kN/m^3)
h_m	:	thickness of a mound (m)
w_i	:	unit weight of a stabilized body (submerged unit weight when submerged) (kN/m^3)
D_s	:	vertical length of a short wall (m)
L_s	:	lengths of a short wall in a normal direction (m)
γ_R	:	partial factor multiplied by a resistance term
γ_S	:	partial factor multiplied by a load term
m	:	adjustment factor



TCVN 11820 Part 4-2: 2020, Hinh 7

Source: TCVN 11820-4-2-2020

Figure 1.9- Schematic Calculation Diagram of Vertical Shear Stress (Short Wall)

The partial factors to be used in the verification of the vertical shear failure of short wall can be selected from Table 1.8. If a corresponding column in the table has the symbol "-", the value in parentheses in the column can be used for performance verification for convenience.

Table 1.8- Partial Factors to be used in the Verification of Vertical Shear Failures of Short Wall Sections

TCVN 11820 Part 4-2: 2020, Bang 6

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Internal stability of the stabilized body (Vertical shear failure (short wall sections) under a permanent state)	– (1.00)	– (1.00)	1.80
Internal stability of stabilized body (Vertical shear failure (short wall sections) under a variable state of seismic ground motions)	– (1.00)	– (1.00)	1.50

Source: TCVN 11820-4-2-2020

3) Verification of Extrusion Failure

Because wall-type improved grounds have a large number of long walls which are connected to each other through short walls with soil remaining untreated between the long walls, there may be a risk of extrusion failures of the untreated soil between the long walls depending on the intervals of the walls, the strength of the untreated soil and the thicknesses of the backfill layers. Thus, it is necessary to verify the possibility of extrusion failure of the untreated soil between long walls.

Figure 1.10 is a schematic diagram of the extrusion failure of untreated soil from a wall-type improvement body.

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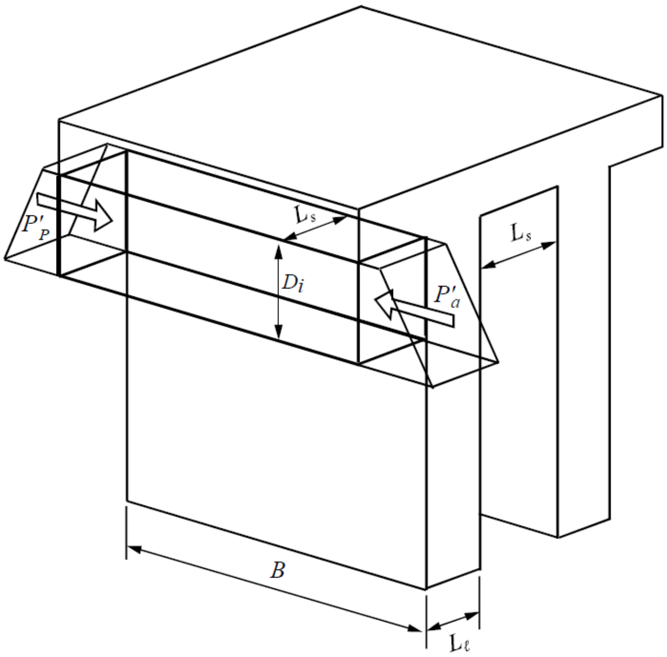


Source: TCVN 11820-4-2-2020

Figure 1.10- Schematic Diagram of the Extrusion of Untreated Soil

The extrusion of untreated soil between long walls can be verified through repeated calculations using the Equation (1.14) while changing the values of D_i.

$$m \cdot \frac{S_d}{R_d} \leq 1.0;\ R_d = \gamma_R R_k;\ S_d = \gamma_S S_k \tag{1.14}$$

$$R_k = 2(L_s + D_i) C_{uk} B + P'_{phk}$$

TCVN 11820 Part 4-2: 2020, Equation (12)

$$S_k = P'_{ahk} + k_{h2k} w_{ik} B D_i L_s + h_w \rho_w g D_i L_s$$

Where:

R_k	:	characteristic value related to a resistance term (kN/m)
S_k	:	characteristic value related to a load term (kN/m)
L_s	:	length of a short wall in the normal direction (m)
D_i	:	depth from the lower edge of a short wall to an object cross section (m)
C_u	:	average shear strength of untreated soil (at the intermediary depth between the lower edge of a short wall and the object cross section) (kN/m^2)
B	:	improvement width (m)
P'_{ah} , P'_{ph}	:	horizontal components of active and passive earth pressure acting on the untreated soil between long walls (from the lower edge of a short wall to D_i) (kN)
k_{h2}	:	seismic coefficient for verification when calculating the inertia force acting on improved ground
h_w	:	difference between the residual water level and the water level on the offshore side (m)
w_i	:	saturated unit weight of untreated soil (kN/m^3)
$\rho_w g$	:	unit weight of seawater (kN/m^3)
γ_R	:	partial factor multiplied by a resistance term
γ_S	:	partial factor multiplied by a load term
m	:	adjustment factor

The partial factors to be used in the verification of the extrusion of untreated soil between long walls can be selected from Table 1.9. If a corresponding column in the table has the symbol "-", the value in parentheses in the column can be used for performance verification for convenience.

The partial factors listed in Table 1.9 are set with reference to the safety levels in the past standards.

Table 1.9- Standard Values of the Partial Factors to be used in the Verification of Extrusion Failure of Untreated Soil

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Internal stability of the stabilized body (Extrusion under a permanent state)	0.81	1.04	– (1.00)
Internal stability of the stabilized body (Extrusion under a variable state of seismic ground motions)	– (1.00)	– (1.00)	1.00

Source: TCVN 11820-4-2-2020

TCVN 11820 Part 4-2: 2020, Bang 7

1-6. Verification of Pile-type Improvement (Composite Ground)

When the foundation ground of relatively lightweight structures such as embankments and retaining walls is soft ground, pile-type improvement (composite ground) is commonly used to prevent circular slip failure, reduce settlement, increase bearing capacity, and reduce lateral displacement. Current design methods for the improved ground involve stability analysis using circular slip failure calculations and settlement analysis considering stress distribution ratios. However, experimental and analytical studies on the bearing capacity of the improved grounds are limited. The current design methods do not fully reflect the actual phenomena, and there are ongoing issues with reliability and precision improvement in design methods that are not yet established.

Centrifuge model loading experiments investigating the failure behavior when fixed type improvement (where the improved piles reach the bearing bed layer) is applied to the foundation of caisson breakwaters report three types of failure:

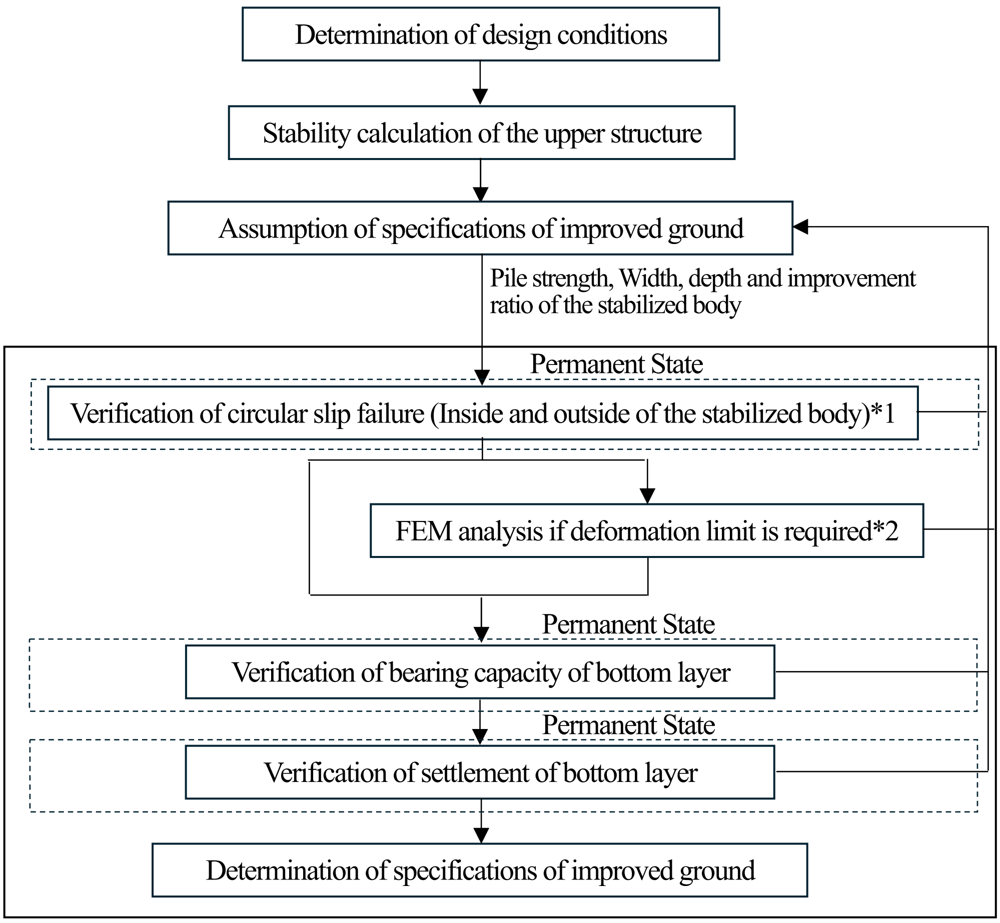
- Improved pile collapsing (bending failure depending on the location of the piles)
- Improved piles collapsing like dominoes
- Caissons sliding failure

The occurrence of these failure modes depends on ground conditions and operational conditions, though many aspects remain unclear. The current design methods for pile-type improved ground, as referenced here, do not consider cases where improved piles collapse like dominoes or bending failures; hence, careful handling is required when applying these methods. Previous studies can serve as references for cases of domino-like collapses or bending failures.

(1) Verification Procedure

The standard procedure for verification is shown in Figure 1.11. However, some of the investigation items listed may be omitted depending on the improvement objectives.

CDM Manual: 2018, Figure 3.8.1



*1: If the width of the stabilized body is small relative to its length, verification of sliding failure is required.

*2: If a deformation limit is specified or required, deformation verification must be conducted.

Source: CDM Manual: 2018, Japan

Figure 1.11- Example of the Performance Verification Procedure

(2) Design Conditions

Design conditions should fully consider: 1) the purpose and function of the structure, 2) importance, 3) load conditions, 4) construction conditions (including the surrounding environment), and 5) soil conditions.

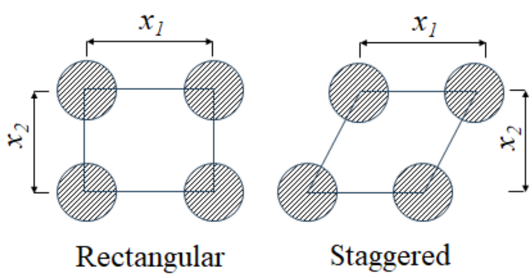
In the design process, it is crucial to set conditions such as allowable deformation or benchmark deformation and allowable settlement amounts based on the above aspects to determine the specifications that fit the purpose of ground improvement.

(3) Assumptions of Specifications of the Pile-type Improved Ground

The assumptions of specifications are implemented regarding the width, depth, and ratio of improvement, and the strength of the improved body (standard design strength). These specifications should be determined based on construction performance.

The mechanical interaction between the improved bodies and the untreated soil between them, especially the behavior under horizontal forces such as active earth pressure on the improved ground, still has unclear aspects, leaving unresolved design challenges. Therefore, it is desirable to pay close attention to past construction performance when deciding on the specifications of the improved ground.

The improvement ratio (a_p) is represented as the percentage of the improved area relative to the target area for improvement. Generally, the improved bodies are arranged in a rectangular or staggered pattern (see Figure 1.12).



CDM Manual: 2018, Figure 3.8.2

Source: CDM Manual: 2018, Japan

Figure 1.12- Arrangement of Pile-type Improvement

$$a_p = \frac{A_p}{x_1 \times x_2} \times 100 \tag{1.15}$$

CDM Manual: 2018, Equation (3.8.1)

Where:

a_p : improvement ratio (%)

A_p : improvement area per pile (m^2)

$x_1 x_2$: share area per pile (m^2)

Since the improvement body is not a uniform high-strength pile, the improvement width of the entire improvement body (B) is set appropriately according to the performance relative to the improvement depth (D) so that the entire improved ground can resist external forces and does not generate excessive bending stress.

The design strength of the improved bodies (standard design strength) and the improvement ratio vary depending on the purpose of the improvement and the target soil condition. However, based on past performance, design strengths of 100-600 kN/m^2 and improvement ratios of 30-50% or more are commonly used.

The assumption for the standard design strength of the improved bodies should be based on performance and the required compression strength of the improved bodies. That is, the design strength is assumed based on the concentrated load on the improved bodies, and approximately 1.0-1.2 of safety factor is considered in the calculations by the following Equation:

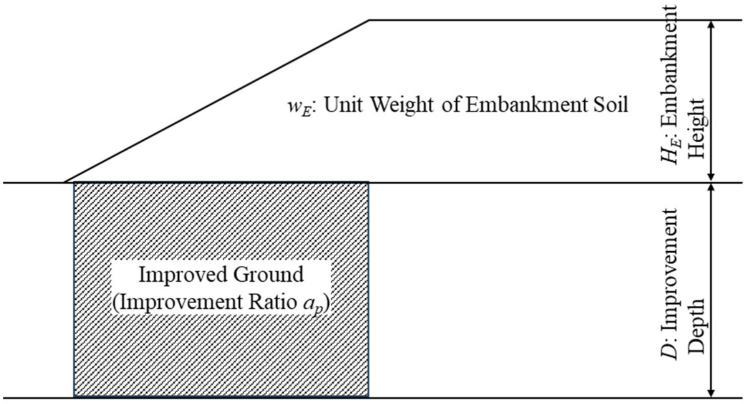
$$q_{uck}=F_s W/a_p \tag{1.16}$$

CDM Manual: 2018, Equation (3.8.2)

Where:

- q_{uck} : characteristic value of the standard design strength of the improved body
- a_p : improvement ratio (%)
- W : surcharge load = $H_E \times w_E$ (see Figure 1.13)
- F_s : safety factor (1.0-1.2)

CDM Manual: 2018, Figure 3.8.3



Source: CDM Manual: 2018, Japan

Figure 1.13- Relationship between Standard Design Strength and Embankment Height

(4) Verification of Circular Slip Failure

The verification of circular slip failure is considered for sliding that passes through the improved ground and outside the improved ground. The method of verification is indicated in Equation (1.17) and Table 1.10.

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 1.10. For parts where a "-" is indicated in Table 1.10, values in parentheses can be used for convenience when performing the verification.

Modified From TCVN 11820 Part 1: 2025, Equation (I.102) (I.103)

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

$$R_k=\sum\left[\left\{c'_k.s+\left(w'_k+q_k\right)\cos^2\theta\tan\varphi'_k\right\}\sec\theta\right] \tag{1.17}$$

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

Where:

- c' : undrained shear strength for cohesive soil or apparent adhesion under a drained condition for sandy soil ground (kN/m²)
- s : width of a segment (m)
- w' : effective weight of a segment (kN/m)
- w : total weight of a segment (kN/m)

Modified from TCVN 11820

q	:	surcharge acting on a segment (kN/m)
φ'	:	apparent shear resistance angle based on effective stress (°)
θ	:	angle between the bottom face of a segment and the horizontal plane (°)
R	:	resistance term(kN/m)
S	:	load term (kN/m)
γ_R	:	partial factor multiplied by resistance term
γ_S	:	partial factor multiplied by load term
m	:	adjustment factor

Part 4-1: 2020, Equation (F.1)

Table 1.10- Partial Factors Used for Performance Verification of Circular Slip Failure

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

Source: Modified from TCVN 11820-6-2023

TCVN 11820 Part 6: 2023, Bang 6

The average shear strength of the improved ground is indicated by Equation (1.18). The second term on the right side of the Equation, which represents the strength of the untreated soil, is generally very small and is often ignored. Here, the reduction factor κ is a coefficient introduced because the failure strains of the improved body and the original ground differ.

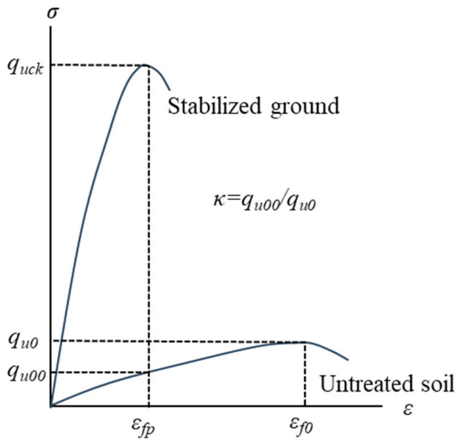
This factor is determined based on the results of uniaxial compression tests of the improved body and the original ground, but it may also correspond to values for 1% to 2% axial strain from past records.

$$\tau_{avg}= C_p a_p+\kappa C_0(1-a_p) \quad (1.18)$$

CDM Manual: 2018, Equation (3.8.3)

Where:

τ_{avg}	:	average shear strength of improved ground (kN/m^2)
C_p	:	shear strength of improved body $=q_{uck}/2$ (kN/m^2)
a_p	:	improvement ratio (%)
κ	:	reduction factor
C_0	:	shear strength of untreated soil (kN/m^2)



Source: CDM Manual: 2018, Japan

Figure 1.14- Relationship σ and ε between Stabilized Ground and Untreated Soil

CDM Manual: 2018, Figure 3.8.4

In cases of composite ground improvement specifications, deformations or failure patterns like slip failure are usually not expected. However, verifications for slip failure are also conducted and the stability of the improved ground is confirmed especially in the following cases:

- The bearing ground is soft.
- The lateral earth pressure on the improved ground is significant.
- The width of improvement relative to the depth is small and the embedded into the bearing ground is insufficient.

(5) Verification of Bearing Capacity Failure

In the case of pile-type improvements, as the surcharge load concentrates on the improved body, the bearing capacity of the bearing ground is verified. If the bearing ground is not very stiff, settlement is also expected. Hence, assuming that the surcharge load concentrates on the improved body, the allowable bearing capacity of the bearing ground is calculated and verified.

(6) Calculation of Consolidation Settlement of Improved Ground

The calculation of the consolidation settlement is estimated based on the settlement of untreated ground (S_0), improvement ratio (a_p) and stress sharing ratio ($n=\sigma_p/\sigma_c$).

$$S=\beta' S_0$$

$$\beta'=\frac{\sigma_c}{\sigma}=\frac{1}{(n\text{-}1)a_p+1} \qquad (1.19)$$

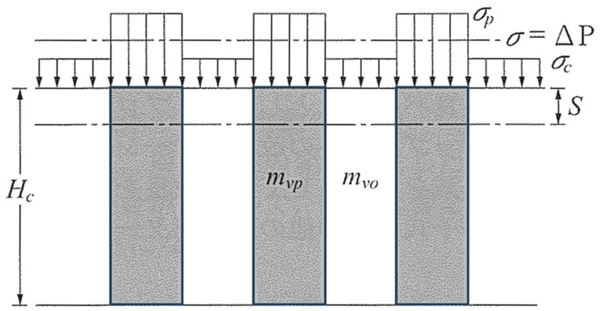
$$S_0=m_{v0}H_c\sigma$$

CDM Manual: 2018, Equation (3.8.4)

Where:

S	:	settlement of improved ground, composite ground (m)
S_0	:	settlement of untreated ground (m)
β'	:	settlement reduction ratio (ratio of the settlement of composite ground to the settlement of untreated ground)
a_p	:	improvement ratio (%)
σ	:	=ΔP, increased stress (kN/m^2)
H_c	:	thickness of improved ground, consolidation layer (m)
n	:	stress sharing ratio $=\sigma_p/\sigma_c\ (=m_{vo}/m_{vp})$

σ_p	:	stress acting on improved body (kN/m^2)
σ_c	:	stress acting on untreated soil (kN/m^2)
m_{vp}	:	coefficient of volume compressibility of improved ground (m^2/kN)
m_{v0}	:	coefficient of volume compressibility of untreated ground (m^2/kN)



CDM Manual: 2018, Figure 3.8.5

Source: CDM Manual: 2018, Japan

Figure 1.15- Settlement of Improved Ground (Composite Ground)

When the improved body has reached the bearing ground, although depending on the design strength and embankment load of the improved body, consolidation settlement caused by the surcharge load on the improved body is generally less likely. However, if a consolidated ground remains below the improved ground, settlement should be considered.

If the improvement rate is small and the consolidation settlement due to embankment and other surcharge loads is being examined, the stress sharing ratio n is usually set within the range of 10 to 20.

These calculations assume uniform settlement between the improved body and the untreated ground parts. Note that if the improvement layer is thick and the improvement ratio is high, the actual settlement might be overestimated.

(7) Examination of Lateral Displacement

When conducting pile-type improvements aimed at suppressing lateral displacement, the improvement specifications determined from the viewpoint of circle slip failure analysis as a composite ground model might be insufficient. Therefore, it is necessary to estimate the amount of lateral displacement caused by eccentric loads and surcharge loads and set improvement specifications to ensure it stays below the allowable values (target values).

Estimations of displacement often use FEM analysis. Also, during construction, lateral displacements may be regulated (allowable amounts or target values set) based on the condition of the construction area.

1-7. Performance Verification of Improved Ground for Reducing Earth Pressure

This section discusses the application methods for reducing seismic earth pressure by improving the ground behind existing quay wall structures, thereby enhancing earthquake resistance. The improved ground, enhanced using the deep mixing method, is considered a material that exhibits superior adhesive strength (c) compared to its internal friction angle (φ), and thus it is assessed as a "c" material in designs verification to reduce seismic earth pressures.

(1) Improvement Patterns of the Improved Ground

Improvement patterns for improved ground using the DMM method include block-type, wall-type, and grid-type, etc. However, for improvement patterns other than block type and overlapping column-type, there are unclear points in the calculation method of earth pressure, etc. Therefore, when performing ground improvement for the purpose of reducing earth pressure, the principle should be to use block-type or overlapping column-type. For other improvement patterns such as grid-type and pile-type, caution is required as there are hardly any practical examples.

The arrangement of the overlapping column-type improved ground must be as shown in Figure 1.16, in the direction of the main active earth pressure from the improved ground to the structure.

(2) Design for Reducing Earth Pressure

While there are still unresolved issues concerning the main active earth pressure from ground improved by the DMM method, it is considered that block-type and overlapping column-type improved ground are nearly 100% improved due to their shape.

Thus, for the design of DMM improved ground aimed at reducing earth pressure, it is preferable to refer to related design methods found in the "Pre-mixing Treatment Technique Manual" or the "Lightweight Mixed Treatment Soil Method Technical Manual for Ports and Airports, Japan."

When calculating the main active earth pressure exerted by the improved ground on structures, the improved ground should be treated as a '*c*' material, ignoring other components such as an internal friction angle (φ). The shear strength c of the block-type improved ground is set according to Equation (1.20).

In the case of overlapping column-type improved ground, there are unknowns in the mechanism of earth pressure action between untreated soil and improved ground, hence design should be carried out under sufficient technical judgment by referencing 1-5 Pile-type Improved Ground Design (Composite Ground Design Method), among others.

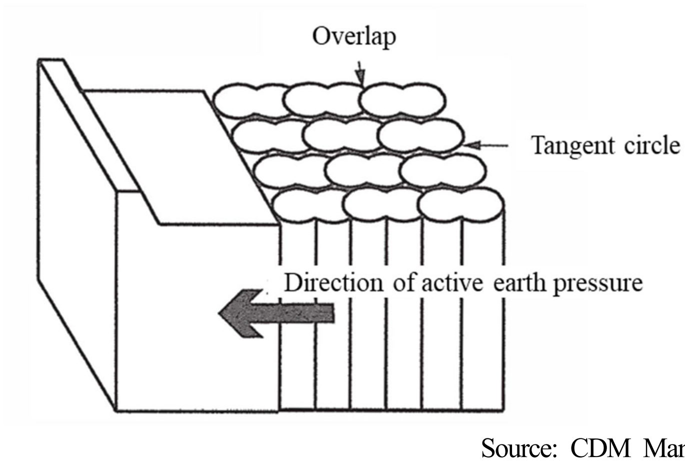
$$c = \frac{1}{2} q_{uck} \tag{1.20}$$

CDM Manual: 2018, Equation (3.8.5)

Where:

c : shear strength of improved ground (kN/m^2)

q_{uck} : standard design strength (kN/m^2)



CDM Manual: 2018, Figure 3.8.6

Source: CDM Manual: 2018, Japan

Figure 1.16- Overlapping Column-type Improved Ground for Reducing Earth Pressure

The width of improvement for DMM ground aimed at reducing earth pressure should be determined according to the following steps:

1) Focus on existing structures and verify the stability of the quay wall structure (sliding, overturning, bearing capacity) due to the earth pressure and hydrostatic pressure exerted by the improved body.
2) Confirm the main active failure surface entering the improved body.
3) Verify the overall stability (including sliding failure) including structures and the improved body.

In cases such as when seismic forces are involved, or when deformation of the surrounding ground occurs, the improved ground may not be able to follow the deformation and cracks may develop, thus it is necessary to thoroughly investigate these impacts.

(3) Verification of Earth Retaining Structure

The structural verification of the sheet pile wall shall be carried out in three distinct construction stages:

- Before DMM installation (i.e., based on the earth pressure from the original ground before mixing),
- During DMM installation (considering the hydraulic pressure exerted by the slurry state of the mixed soil from the land side), and
- Immediately after DMM installation (taking into account the earth pressure from semi-hardened DMM material).

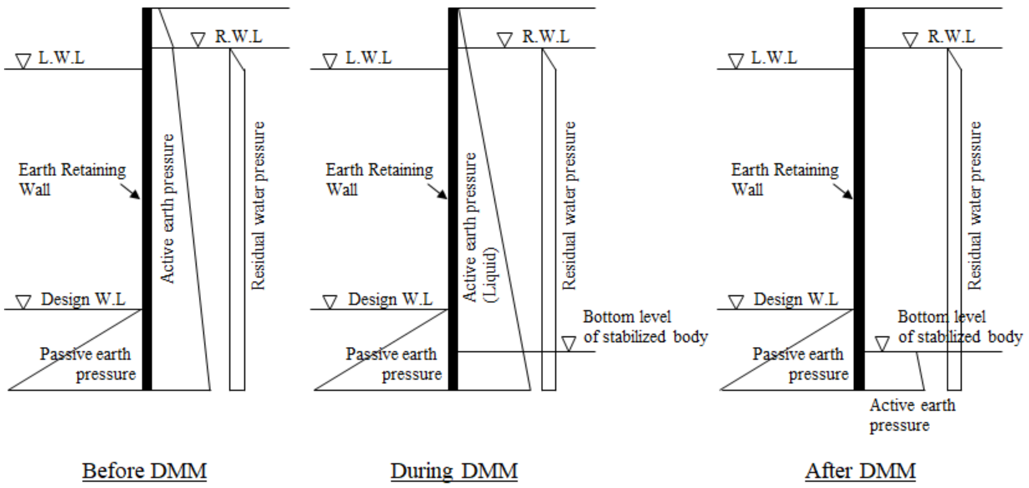


Figure 1.17- External Forces to Sheet Pile Wall

In the post-hardening phase, it is assumed that no earth pressure acts on the sheet pile wall from the side of the DMM-improved body. Therefore, only residual water pressure is considered as an external force.

The embedment length of the sheet pile is determined based on the design method for anchored sheet pile walls, and the structural section of the wall is designed according to the stress generated in the sheet pile.

1-8. Lightweight Treated Soil Method (Super Geo-Material: SGM)

(1) Overview and Features

This method is used to produce the very light foundation or back-filling material by recycling dredged material or surplus soil from construction work as a source soil, mixed with water, air foam or EPS (expanded polystyrene) beads and other cementing compounds.

The SGM forms an improved body similar to CDM, but has the following characteristics as shown in Table 1.11.

(2) Applicability and Expected Benefits

Use in backfill and landfill materials behind new quay walls and revetments:

- Reduced earth pressure allows for the reduction of retaining walls cross-sections and sheet pile walls, thereby reducing construction costs and time.
- Small unit weight reduces consolidation settlement, enabling the efficient construction of landfill sites on soft ground.

Use for reinforcing and improving the functionality of existing quay walls and revetments:

- Because it reduces earth pressure, it allows for seismic reinforcement, revetment heightening, increased overburden loads, and frontal depth increases with minor structural modifications.
- Because it is sufficiently rigid and lightweight, it can suppress lateral displacement and improve the usability of quay walls.

Use in adjacent construction embankments:

- Lightweight embankments minimize impact on existing embankments, structures, buried pipes, etc.
- The effective overburden load can be reduced to about half that of ordinary soil in air and by some extent in submerged portion, making it suitable for use as an embankment material to prevent steps in areas where the load changes suddenly.

Use in submerged backfilling and embankments on soft ground:

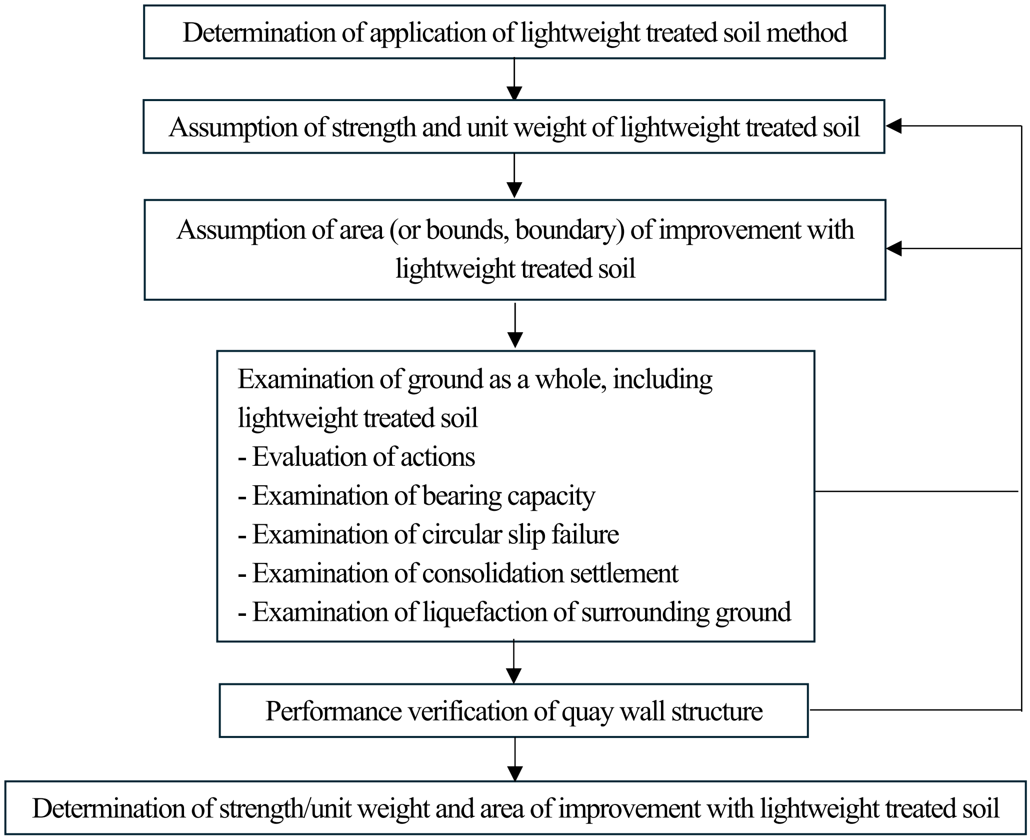
- Because it is resistant to segregation underwater, it can be used as submerged backfilling material.
- Because it is sufficiently rigid and lightweight, its use in embankments on soft ground can help prevent subsidence and improve safety.

Table 1.11- Features of Lightweight Treated Soil Method / SGM

<table>
<tr><th>Characteristics</th><th>Air foam treated soil</th><th>EPS beads treated soil</th></tr>
<tr><td rowspan="3">Unit volume weight</td><td colspan="2">Adjust by mixing amount of lightweight material</td></tr>
<tr><td>The unit weight tends to increase due to defoaming before hardening and water pressure during underwater curing, but this can be addressed by increasing the amount of air bubbles.</td><td>In general application ranges, the unit volume weight when mixed and after underwater curing is approximately the same.</td></tr>
<tr><td>In the long term, there will be some absorption of water from the surface of the treated soil.</td><td>There is almost no change due to water absorption.</td></tr>
<tr><td rowspan="4">Strength</td><td colspan="2">The more solidifying agent added, the greater the unconfined compressive strength (q_u).</td></tr>
<tr><td>When8., pressure such as water pressure is applied during curing, the unit volume weight increases, and accordingly q_u increases slightly.</td><td>In general, even if pressure such as water pressure is applied during curing, it has little (negligible) effect on q_u.</td></tr>
<tr><td>In general, it has been confirmed that a structure can withstand repeated loading if the load is 60 to 70% or less of q_u.</td><td>In general, it has been confirmed that a structure can withstand repeated loading if the load is 60% or less of q_u.</td></tr>
<tr><td colspan="2">The deformation modulus E_{50} tends to decrease as the total confining pressure increases. It exhibits strength characteristics similar to those of hard clay, but the coefficient of earth pressure at rest is small, at around $K_0 = 0.2$.</td></tr>
<tr><td rowspan="2">Permeability</td><td colspan="2">This is almost the same as the permeability coefficient of dredged soil hardened with solidification material.</td></tr>
<tr><td>When the volume fraction of air bubbles exceeds 30%, the permeability tends to increase.</td><td>It is thought that the permeability will not change even if the foam bead mixing ratio increases.</td></tr>
<tr><td>Durability</td><td colspan="2">There is a risk that the strength will decrease if it dries out, so if it is used in the air, measures such as covering it with soil will be necessary.</td></tr>
<tr><td rowspan="2">Flow characteristics</td><td colspan="2">The fluidity is influenced by the moisture content of the adjusted mud, and the higher the moisture content, the greater the fluidity.</td></tr>
<tr><td colspan="2">Fluidity in water is less than in air due to the influence of buoyancy and water pressure.</td></tr>
<tr><td rowspan="2">Anti-washout characteristics</td><td colspan="2">The resistance to separation is affected by the moisture content of the adjusted mud, with the lower the moisture content, the greater the resistance to separation. It is also affected by the soil quality, the amount of lightweight and solidifying material added, and the method of driving.</td></tr>
<tr><td>It is affected by the type of foaming agent and the type of dilution water.</td><td>The larger the particle size of the foam beads, the greater the effect.</td></tr>
<tr><td>Workability</td><td>Consideration must be given to preventing foam from breaking down during pouring and foam shrinkage due to water pressure.</td><td>Care must be taken to prevent foam beads from scattering and to recover materials that have separated onto the water surface.</td></tr>
</table>

(3) Performance Verification

OCDI 2020, Part III Chapter 2 Figure 5.6.1



Source: OCDI 2020

Figure 1.18- Example of the Performance Verification Procedure

The Lightweight Treated Soil Method / SGM method is a soil improvement method in which raw soil, such as dredged soil or construction waste soil, is adjusted to a water content above the liquid limit and turned into a slurry, then mixed with solidification material and lightweighting materials and used as a ground material for landfill or backfill, creating a lightweight and stable foundation. Treated soil that uses air bubbles as the lightweighting material is called air-foam treated soil, and treated soil that uses EPS (expanded polystyrene) beads as the lightweighting material is called foam bead treated soil. Lightweight treated soil method has the following characteristics:

- Compared to regular sand, it is about half the weight in air and about 1/5 in seawater, which helps prevent ground subsidence during landfill and backfilling.
- Because it is lightweight and has strong properties, it has little earth pressure both normally and during earthquakes, making it possible to create highly earthquake-resistant facilities or landfill sites.
- Since the raw material is dredged sand and construction soil generated at ports and disposed of as waste, the amount of waste disposed of can be reduced.

The properties of Lightweight Treated Soil Method / SGM are similar to those of hard clayey soil, except for its light weight. Therefore, its design can be treated as if it were a clayey soil.

Lightweight Treated Soil Method / SGM is a lightweight geomaterial, and its performance verification methods are essentially the same as those for general earth structures, except for mix testing.

Generally, the same design procedures of CDM can be used, but the characteristics of Lightweight Treated Soil Method / SGM described below should be taken into consideration.

(4) Properties of Treated Soil

1) Unit Weight

The unit weight can be set within the range of γ_t = 8 to 13 kN/m^3 by adjusting the amount of lightweight material (air bubble or EPS beads) and added water. When used in port facilities, if the unit weight is lighter than that of seawater, there is a risk of the treated body floating up when the water level rises. In particular, attention must be paid to the rise in sea levels due to global warming.

Therefore, the following values are usually used as the characteristic values for the unit weight:

- Below the groundwater level/submerged area: γ_{tk} = 11.5 – 12.0 kN/m^3
- Submerged unit volume weight: γ_k' = 1.5 – 2.0 kN/m^3
- Above the ground water level: γ_{tk} = 10.0 kN/m^3

In addition, since the unit volume weight of treated soil changes depending on the environmental conditions during and after pouring, particularly the amount of water (confining) pressure, the following points must be taken into consideration when mixing.

2) Effect of Water Pressure during Pouring (Installation)

The volume of air bubbles in the treated soil is compressed by water pressure during filling. This amount can be calculated according to Boyle's law. Since the volume change of EPS beads varies depending on the expansion ratio, the relationship between water pressure and volume change must be confirmed in a rational manner and the unit weight corrected.

3) Effect of Time Elapsed since Construction

When the treated soil is used underwater, its unit weight may increase over time due to water absorption. Therefore, an increase of approximately 0.5 kN/m^3 shall be considered in the mix design. This increase in unit weight is primarily due to the gradual displacement of water from the outside of the treated soil block to the air bubbles within the soil. The rate of change in unit weight varies depending on the air void content of the treated soil and the environmental conditions in which it is placed. In either case, a conservative estimate of an increase in unit weight of 0.5 kN/m^3 is considered. However, if the soil is used above the water level, it is not necessary to consider changes in unit weight over time.

4) Effect of Shrinkage during Solidification

The Lightweight Treated Soil Method / SGM solidifies through coagulation and hydration reactions due to the solidification materials mixed in, causing shrinkage. If it is necessary to consider the change in unit weight due to shrinkage, the shrinkage strains ε_h and ε_r can be determined through laboratory tests, and the volumetric strain ε_v can be calculated using the following equation:

$$\varepsilon_v = \varepsilon_h + 2\varepsilon_r \qquad (1.21)$$

Where:

ε_v : volumetric strain

ε_h : axial shrinkage strain

ε_r : transverse shrinkage strain

The overall concept of volume change is shown in following figure.

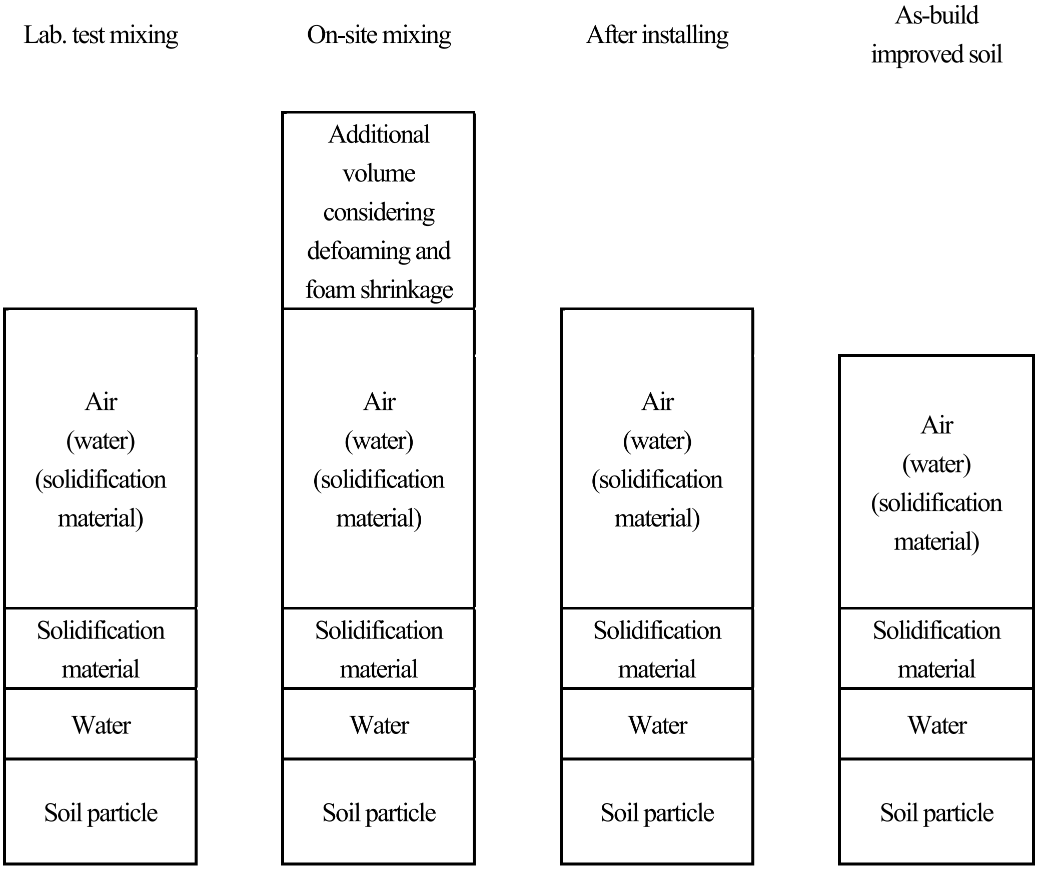


Figure 1.19- Conceptual Diagram of SGM Volumetric Change

y (CDIT): Super Geo-Material Technical Manual Equation (3.3) Table 4.1

2. Design Example

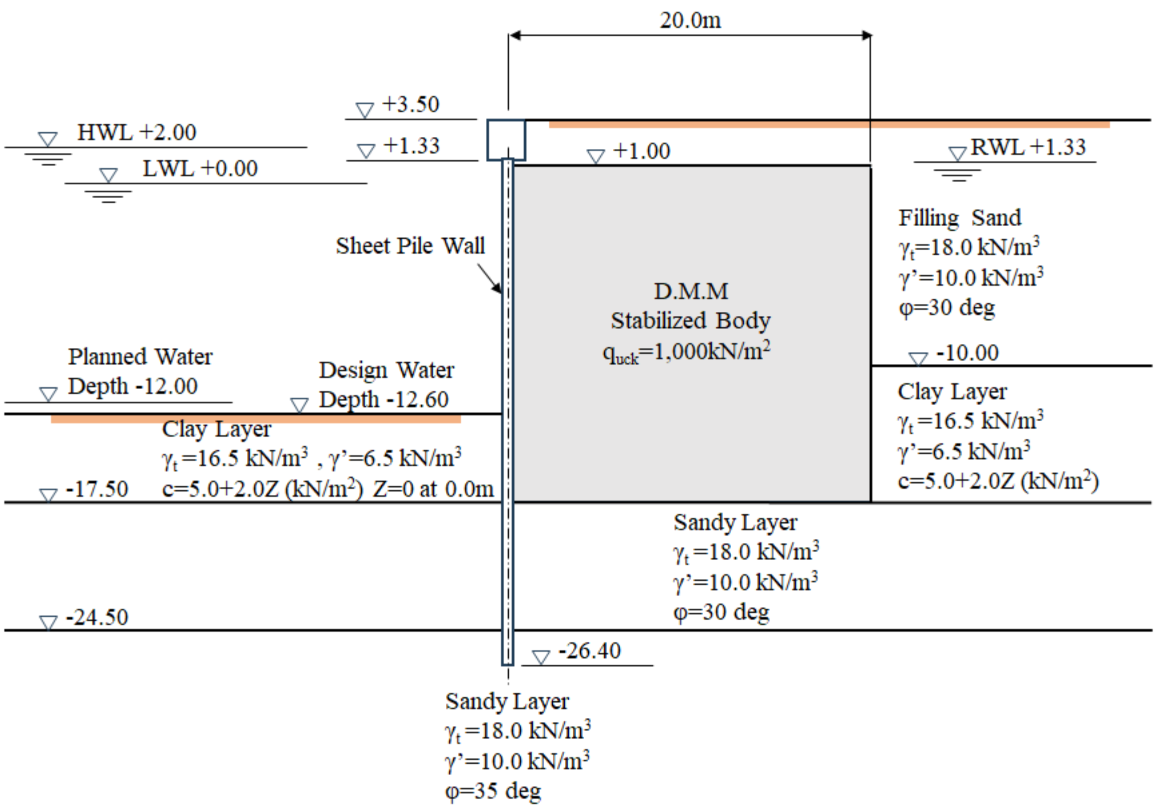


Figure 2.1- Typical Cross Section for Verification

2-1. Design Conditions

(1) Tide levels

H.W.L. + 2.00 m

L.W.L. ±0.00 m

Residual water level (R.W.L.): +1.33 m, [2/3 (H.W.L. –L.W.L.)] + L.W.L.

(2) Ground conditions

Ground surface +3.50m

Backfilling soil φ=30° γt = 18 .0 kN/m^3, γ = 20.0 kN/m^3, γ'= 10.0 kN/m^3 Original ground -10.00m
Cohesive (Clay) layer c=5.0+2.0Z (kN/m2) Z=0 at 0.0m γt = 16.5 kN/m^3, γ = 16.5 kN/m^3, γ'=6.5 kN/m^3 -17.50m
Sand layer φ=30° γt = 18.0 kN/m^3, γ = 20.0 kN/m^3, γ'= 10.0 kN/m^3 -24.50m
Sand layer φ=35° γt = 18.0 kN/m^3, γ = 20.0 kN/m^3, γ'= 10.0 kN/m^3

(3) Water Depths of Quay Wall

Planned water depth: -12.00 m

Design water depth: -12.60 m

(4) Surcharge Loads

Permanent state: 30 kN/m^2

Earthquake motion: 15 kN/m^2

(5) Seismic Coefficient for Verification

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

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

0.10 of the seismic coefficients is applied to the verification of stabilization of quay wall.

According to TCVN 11820-4.2-2020, the seismic coefficient for dynamic forces acting on improved ground can be applied as 0.65 times of seismic coefficient for superstructure. However, for the characteristic value of the seismic coefficient for calculating earth pressure during earthquakes in improved ground systems, the maximum value of corrected acceleration shall not be multiplied by the reduction coefficient.

(6) Assumption of Stabilized Body

1) Standard Design Strength

The average $\overline{q_{uf}}$ of the unconfined compressive strength q_{uf} of field stabilized soil is assumed as 1,667 (kN/m^2). This strength is a typical target strength for marine DMM.

$$q_{uck} = \overline{q_{uf}} \times (1\text{-}KV/100) = 1{,}667 \times (1\text{-}1.0\times40/100) = 1{,}000 \text{ kN/m}^2 \qquad (2.1)$$

Modified from TCVN 11820 Part 4-2: 2020, Equation (9)

Where:

q_{uck} : standard design strength (kN/m^2)

$\overline{q_{uf}}$: average unconfined compressive strength of field stabilized soil (kN/m^2)

K : coefficient (=1.0)

V : coefficient of variation of unconfined compressive strength q_{uf} of field stabilized soil (In general, V = 40 (%).

2) Improvement Pattern

In this design example, since the DMM serves as a retaining structure for the backfill soil, block-type improvement is assumed as a basis.

2-1. Performance Verification

(1) Weight and Moment

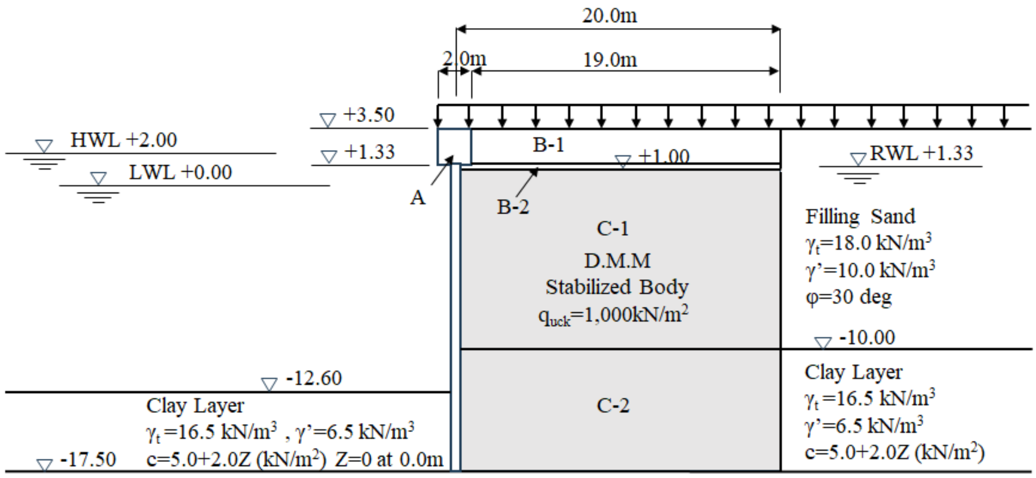


Figure 2.2- Calculation Model

1) Self-weight and Moment

The characteristic values of the self-weight and moment are shown in Table 2.1.

Table 2.1- Characteristic Values of Self-weight and Moment

No	Name	Width×Height×Unit weight	W (kN)	x (m)	$W{\cdot}x$ (kN·m)
A	Superstructure	2.000×2.170×24.000	104.160	0.000	0.000
B-1	Fill material Above RWL	19.000×2.170×18.000	742.140	10.500	7,792.470
B-2	Fill material Below RWL	20.000× 0.330×10.000	66.000	10.000	660.000
C-1	DMM body	20.000×11.000×10.000	2,200.000	10.000	22,000.000
C-2	DMM body	20.000×7.500×6.500	975.000	10.000	9,750.000
	Total		4,087.300		40,202.470

2) Inertial Force and Moment at Earthquake

The characteristic values of the inertial force and moment are shown in Table 2.2.

Table 2.2- Characteristic Values of Inertial Force and Moment at Earthquake

No	Name	W (kN/m)	k_h	P_E (kN/m)	y (m)	$P_E{\cdot}y$ (kN·m/m)
A	Superstructure	104.160	0.100	10.416	19.915	207.435
B-1	Fill material	742.140	0.100	74.214	19.915	1,477.972
B-2	Fill material	66.000	0.100	6.600	18.665	123.189
C-1	DMM body	2,200.000	0.100	220.000	13.000	2,860.000
C-2	DMM body	975.000	0.100	97.500	3.750	365.625
Point a (-17.500m)				408.730		5,034.221

3) Surcharge Load and Moment

The characteristic values of surcharge load and moment are shown in Table 2.3.

Table 2.3- Characteristic Values of Surcharge Load and Moment

Permanent state

	No	Load × Acting width w (kN/m^2) × b (m)	Vj (kN/m)	x (m)	$Vj{\cdot}x$ (kN·m/m)
Point a (-17.500m)	1	30.000 × 21.000	630.000	10.000	6,300.000

Variable situation: Earthquake ground motion - Vertical force

	No	Load × Acting width w (kN/m^2) × b (m)	Vj (kN/m)	x (m)	$Vj{\cdot}x$ (kN·m/m)
Point a (-17.500m)	1	15.000 × 21.000	315.000	10.000	3,150.000

Variable situation: Earthquake ground motion - Inertial force

	No	Vj (kN/m) kh	Hj (kN/m)	y (m)	$Hj{\cdot}y$ (kN·m/m)
Point a (-17.500m)	1	315.000 × 0.100	31.500	21.000	661.500

(2) Earth Pressure and Moment

1) Earth Pressure in Permanent State

Earth pressure is calculated by the following equation.

i) Earth Pressure for Sandy Soil

The earth pressure for sandy soil under permanent conditions can be calculated using the following Equations:

✓ Active earth pressure

$$p_{ai}=K_{ai}\left[\sum \gamma_i h_i+\frac{\omega\cos\psi}{\cos(\psi\text{-}\beta)}\right]\cos\psi \qquad (2.2)$$

$$K_{ai}=\frac{\cos^2(\varphi_i\text{-}\psi)}{\cos^2\psi\cos(\delta+\psi)\left[1+\sqrt{\frac{\sin(\varphi_i+\delta)\sin(\varphi_i\text{-}\beta)}{\cos(\delta+\psi)\cos(\psi\text{-}\beta)}}\right]^2} \qquad (2.3)$$

$$\cot(\zeta_i\text{-}\beta)=-\tan(\varphi_i+\delta+\psi\text{-}\beta)+\sec(\varphi_i+\delta+\psi\text{-}\beta)\sqrt{\frac{\cos(\delta+\psi)\sin(\varphi_i+\delta)}{\cos(\psi\text{-}\beta)\cos(\varphi_i\text{-}\beta)}} \qquad (2.4)$$

TCVN 11820 Part 4-1: 2020, Equation (18)(19)

✓ Passive earth pressure

$$p_{pi}=K_{pi}\left[\sum \gamma_i h_i+\frac{\omega\cos\psi}{\cos(\psi\text{-}\beta)}\right]\cos\psi \qquad (2.5)$$

$$K_{pi}=\frac{\cos^2(\varphi_i+\psi)}{\cos^2\psi\cos(\delta+\psi)\left[1\text{-}\sqrt{\frac{\sin(\varphi_i\text{-}\delta)\sin(\varphi_i+\beta)}{\cos(\delta+\psi)\cos(\psi\text{-}\beta)}}\right]^2} \qquad (2.6)$$

$$\cot(\zeta_i\text{-}\beta)=\tan(\varphi_i\text{-}\delta\text{-}\psi+\beta)+\sec(\varphi_i\text{-}\delta\text{-}\psi+\beta)\sqrt{\frac{\cos(\delta+\psi)\sin(\varphi_i\text{-}\delta)}{\cos(\psi\text{-}\beta)\cos(\varphi_i+\beta)}} \qquad (2.7)$$

TCVN 11820 Part 4-1: 2020, Equation (20)(21)

ii) Earth Pressure for Cohesive Soil

The calculation of earth pressure for cohesive soil under permanent conditions can be performed using the following Equations:

✓ Active earth pressure

$$p_{ai}=\sum \gamma_i h_i+\omega\text{-}2c_i \qquad (2.8)$$

TCVN 11820 Part 4-1:

✓ Passive earth pressure

$$p_{pi}=\sum \gamma_i h_i+\omega+2c_i \qquad (2.9)$$

2020, Equation (27)(28)

Where:

p_{ai}, p_{pi}	:	active (or passive) earth pressure acting on the retaining wall of the *i*-th layer (kN/m²)
φ_i	:	angle of internal friction of the *i*-th layer of soil (degrees)
γ_i	:	unit weight of the *i*-th layer of soil (kN/m³)
h_i	:	thickness of the *i*-th layer of soil (m)
K_{ai}, K_{pi}	:	active (or passive) earth pressure coefficient for the *i*-th layer
ψ	:	angle of batter of the retaining wall from the vertical plane (degrees)
β	:	angle of the ground surface from the horizontal plane (degrees)
δ	:	angle of wall friction (degrees). (Active = +15°, Passive = -15°)
ζ_i	:	angle that the failure surface of the *i*-th layer makes with the horizontal (degrees)
ω	:	surcharge load per unit area on the ground surface (kN/m²)
c_i	:	undrained shear strength of the *i*-th layer of cohesive soil (kN/m²)

2) Earth Pressure in Variable Situation (Earthquake Ground Motion)

i) Earth Pressure for Sandy Soil

The calculation of earth pressure for sandy soil under variable situation (earthquake ground motion) can be performed using the following Equation:

✓ Active earth pressure

TCVN 11820 Part 4-1: 2020, Equation (30)(31)

$$p_{ai}=K_{ai}\left[\sum \gamma_i h_i+\frac{\omega\cos\psi}{\cos(\psi-\beta)}\right]\cos\psi \qquad (2.10)$$

$$K_{ai}=\frac{\cos^2(\varphi_i-\psi-\theta)}{\cos\theta\cos^2\psi\cos(\delta+\psi+\theta)\left[1+\sqrt{\dfrac{\sin(\varphi_i+\delta)\sin(\varphi_i-\beta-\theta)}{\cos(\delta+\psi+\theta)\cos(\psi-\beta)}}\right]^2} \qquad (2.11)$$

$$\cot(\zeta_i-\beta)=-\tan(\varphi_i+\delta+\psi-\beta)+\sec(\varphi_i+\delta+\psi-\beta)\sqrt{\frac{\cos(\delta+\psi+\theta)\sin(\varphi_i+\delta)}{\cos(\psi-\beta)\cos(\varphi_i-\beta-\theta)}} \qquad (2.12)$$

✓ Passive earth pressure

TCVN 11820 Part 4-1: 2020, Equation (32)(33)

$$p_{pi}=K_{pi}\left[\sum \gamma_i h_i+\frac{\omega\cos\psi}{\cos(\psi-\beta)}\right]\cos\psi \qquad (2.13)$$

$$K_{pi}=\frac{\cos^2(\varphi_i+\psi-\theta)}{\cos\theta\cos^2\psi\cos(\delta+\psi-\theta)\left[1-\sqrt{\dfrac{\sin(\varphi_i-\delta)\sin(\varphi_i+\beta-\theta)}{\cos(\delta+\psi-\theta)\cos(\psi-\beta)}}\right]^2} \qquad (2.14)$$

$$\cot\left(\zeta_i-\beta\right)=\tan\left(\varphi_i-\delta-\psi+\beta\right)+\sec\left(\varphi_i-\delta-\psi+\beta\right)\sqrt{\frac{\cos(\delta+\psi-\theta)\sin(\varphi_i-\delta)}{\cos(\psi-\beta)\cos(\varphi_i+\beta-\theta)}} \quad (2.15)$$

Where:

θ : composite seismic angle (°)
 $\theta = \tan^{-1}(k)$ (Above water)
 $\theta = \tan^{-1}(k')$ (Under water)
k : seismic coefficient above water
k' : apparent seismic coefficient underwater

ii) Earth Pressure for Cohesive Soil

The calculation of earth pressure for cohesive soil under variable situation (earthquake ground motion) can be performed using the following Equations:

✓ Active Pressure

$$p_{ai}=\frac{\left(\sum\gamma_i h_i+\omega\right)\sin(\zeta_i+\theta)}{\cos\theta\sin\zeta_i}-\frac{c_i}{\cos\zeta_i\sin\zeta_i} \quad (2.16)$$

$$\zeta_i=\tan^{-1}\sqrt{1-\frac{\sum\gamma_i h_i+2\omega}{2c}\tan\theta} \quad (2.17)$$

TCVN 11820 Part 4-1: 2020, Equation (34)(35)

✓ Passive Pressure

$$p_{pi}=\sum\gamma_i h_i+\omega+2c_i \quad (2.18)$$

iii) Calculation Formula for Apparent Seismic Coefficient

In the variable situation concerning earthquake ground motion for soil under water, the earth pressure is generally calculated using the apparent seismic coefficient obtained from the following Equation:

$$k'=\frac{2\left(\sum\gamma_{ti}h_i+\sum\gamma_{sat\,j}h_j+\omega\right)+\gamma_{sat}h}{2\left\{\sum\gamma_{ti}h_i+\sum(\gamma_{sat\,j}-10)h_j+\omega\right\}+(\gamma_{sat}-10)h}k \quad (2.19)$$

TCVN 11820 Part 4-1: 2020, Equation (36)

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 calsulated below the residual water level (m)
h : thickness of soil layer for which the earth pressure is being calsulated below the residual water level (m)
γ_{ti} : unit weight of soil in the i-th layer above the residual water level (kN/m^3)
$\gamma_{sat\,j}$: saturated unit weight of soil in the j-th layer above the layer for which the earth pressure is being calsulated below the residual water level (kN/m^3)
ω : surcharge per unit area of ground surface (kN/m^2)
k : seismic coefficient
k' : apparent seismic coefficient

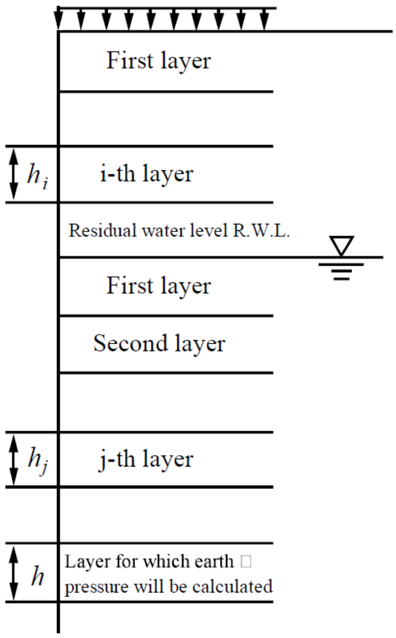


Figure 2.3- Symbols for Apparent Seismic Coefficient

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

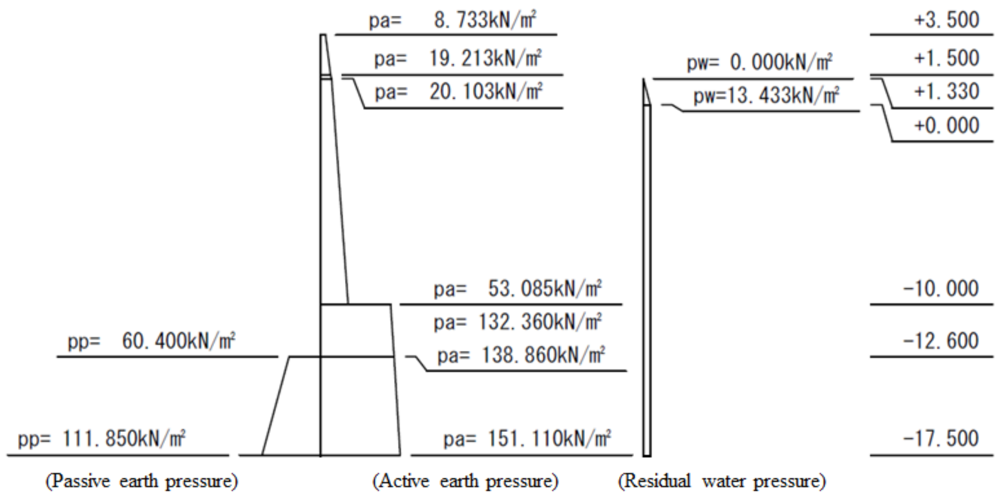


Figure 2.4- Earth Pressure (Permanent state)

Table 2.4- Characteristic Values of Active Earth Pressure (Permanent state)

Level (m)	*h* (m)	*γ* (kN/m³)	*γh* (kN/m²)	*Σγh* (kN/m²)	*c* (kN/m²)	*ω*cos*ψ*/ cos(*ψ-β*)	*Ka*cos (*δ*+*ψ*)	*Pa* (kN/m²)
3.500	0.000	18.000	0.000	0.000	0.0	30.000	0.2911	8.733
1.330	2.170	18.000	39.060	39.060	0.0	30.000	0.2911	20.103
1.330				39.060	0.0	30.000	0.2911	20.103
1.000	0.330	10.000	3.300	42.360	0.0	30.000	0.2911	21.064
1.000				42.360	0.0	30.000	0.2911	21.064
-10.000	11.000	10.000	110.000	152.360	0.0	30.000	0.2911	53.085
-10.000				152.360	25.000	30.000	-	132.360
-12.600	2.600	6.500	16.900	169.260	30.200	30.000	-	138.860
-12.600				169.260	30.200	30.000	-	138.860
-17.500	4.900	6.500	31.850	201.110	40.000	30.000	-	151.110

* Ψ=0

Table 2.5- Characteristic Values of Total Active Earth Pressure and Moment (Permanent state)

Horizontal forces and moments

No.	Calculation formula Pa (kN/m^2) h(m)	P_H (kN/m)	Arm length y (m)	Moment M_{PH} (kN·m/m)
1	1/2 × 8.733 × 2.170	9.475	20.277	192.125
2	1/2 × 20.103 × 2.170	21.812	19.553	426.490
3	1/2 × 20.103 × 0.330	3.317	18.720	62.094
4	1/2 × 21.064 × 0.330	3.476	18.610	64.688
5	1/2 × 21.064 × 11.000	115.852	14.833	1,718.433
6	1/2 × 53.085 × 11.000	291.968	11.167	3,260.407
7	1/2 × 132.360 × 2.600	172.068	6.633	1,141.327
8	1/2 × 138.860 × 2.600	180.518	5.767	1,041.047
9	1/2 × 138.860 × 4.900	340.207	3.267	1,111.456
10	1/2 × 151.110 × 4.900	370.220	1.633	604.569
total		1,508.913		9,622.636

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	9.475	0.268	2.540	20.000	50.800
2	21.812	0.268	5.846	20.000	116.920
3	3.317	0.268	0.889	20.000	17.780
4	3.476	0.268	0.932	20.000	18.640
5	115.852	0.268	31.048	20.000	620.960
6	291.968	0.268	78.247	20.000	1,564.940
7	172.068	-	-	-	-
8	180.518	-	-	-	-
9	340.207	-	-	-	-
10	370.220	-	-	-	-
total			119.502		2,390.040

Table 2.6- Characteristic Values of Passive Earth Pressure (Permanent state)

Level (m)	h (m)	γ (kN/m^3)	γh (kN/m^2)	$\Sigma\gamma h$ (kN/m^2)	c (kN/m^2)	$\omega\cos\psi/\cos(\psi-\beta)$	$Kp\cos(\delta+\psi)$	Pp (kN/m^2)
-12.600	0.000	6.500	0.000	0.000	30.200	0.000	-	60.400
-17.500	4.900	6.500	31.850	31.850	40.000	0.000	-	111.850

* Ψ=0

Table 2.7- Characteristic Values of Total Passive Earth Pressure and Moment (Permanent state)

Horizontal forces and moments

No.	Calculation formula Pp (kN/m^2) h(m)	P_H (kN/m)	Arm length y (m)	Moment M_{PH} (kN·m/m)
1	1/2 × 60.400 × 4.900	147.980	3.267	483.451
2	1/2 × 111.850 × 4.900	274.033	1.633	447.496
total		422.013		930.947

4) Characteristic Values of Earth Pressure and Moment (Variable Situation: Earthquake Ground Motion)

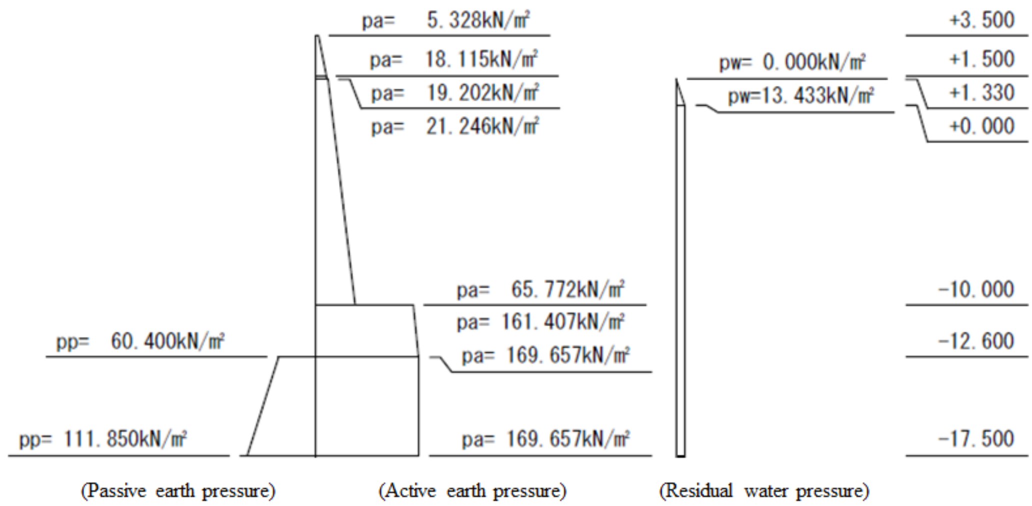


Figure 2.5- Active Earth Pressure (Variable Situation: Earthquake Gorund Motion)

Table 2.8- Characteristic Values of Active Earth Pressure (Variable Situation: Earthquake Ground Motion)

Earth pressure coefficient

Earth Pressure Level (m)	β (degrees)	φ (degrees)	δ (degrees)	ψ (degrees)	k or k'	θ (degrees)	Ka
3.500 to 1.000	0.0	30.0	15.0	0.0	0.10	5.7	0.3677
1.000 to -10.000	0.0	30.0	15.0	0.0	0.15	8.5	0.4069
-10.000 to -17.500	-	-	-	-	0.18	10.2	-

Earth pressure

Level (m)	h (m)	γ (kN/m^3)	$\Sigma\gamma h$ (kN/m	ζ (degrees)	c (kN/m^2)	$\omega\cos\psi/\cos(\psi-\beta)$	$Ka\cos(\delta+\psi)$	Pa (kN/m^2)
3.500	0.000	18.000	0.000	51.6	0.0	15.000	0.3552	5.328
1.330	2.170	18.000	39.060	51.6	0.0	15.000	0.3552	19.202
1.330			39.060	48.6	0.0	15.000	0.3930	21.246
1.000	0.330	10.000	42.360	48.6	0.0	15.000	0.3930	22.542
1.000			42.360	48.6	0.0	15.000	0.3930	22.542
-10.000	11.000	10.000	152.360	48.6	0.0	15.000	0.3930	65.772
-10.000			152.360	30.4	25.000	15.000	-	161.407
-12.600	2.600	6.500	169.260	32.5	30.200	15.000	-	169.657
-12.600			169.260	32.5	30.200	15.000	-	169.657*[1]
-17.500	4.900	6.500	201.110	45.0	40.000	15.000	-	169.657*[1]

* Ψ=0

Note *[1]: Application of Earth Pressure Formulas for Cohesive Soils below the Seabed

When evaluating seismic earth pressure acting on cohesive (clay) soil layers below the seabed, the following procedures shall be applied:

1) Case where cohesive soil layer extends deeper than 10 m below the seabed

- At the seabed level and above:

The seismic earth pressure shall be calculated using the apparent seismic coefficient by applying Equation (2.16).

- At 10 m below the seabed:

The seismic coefficient shall be set to zero, and earth pressure shall be calculated using Equation

(2.16). If the calculated earth pressure at this depth is less than that at the seabed surface, then the value at the seabed shall be applied.

- Between the seabed and 10 m depth:

The earth pressure intensity shall be assumed to vary linearly between the seabed and the 10 m depth point.

2) Case where a sand layer exists within 10 m below the seabed

- For the cohesive soil layer above the sand layer:

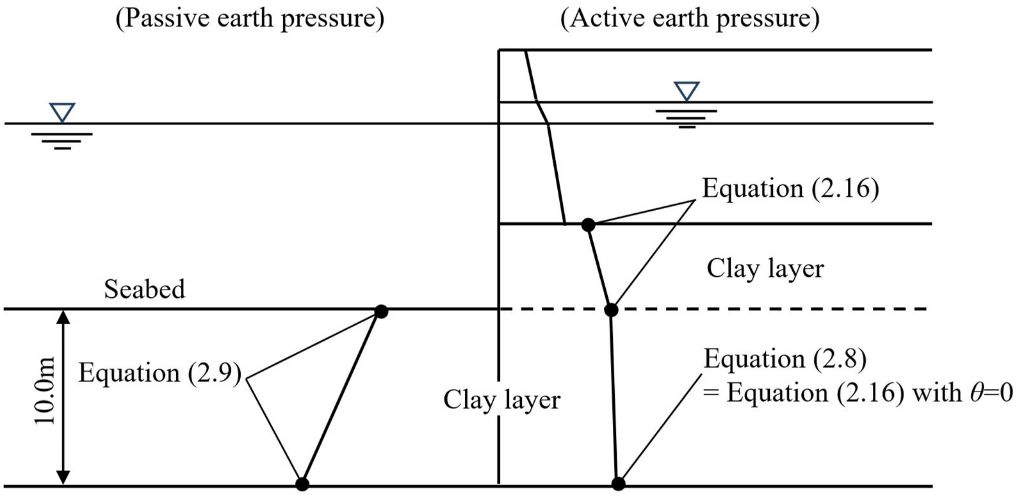
The earth pressure shall be calculated using Equation (2.16), as in Case 1).

- For the Sand Layer:

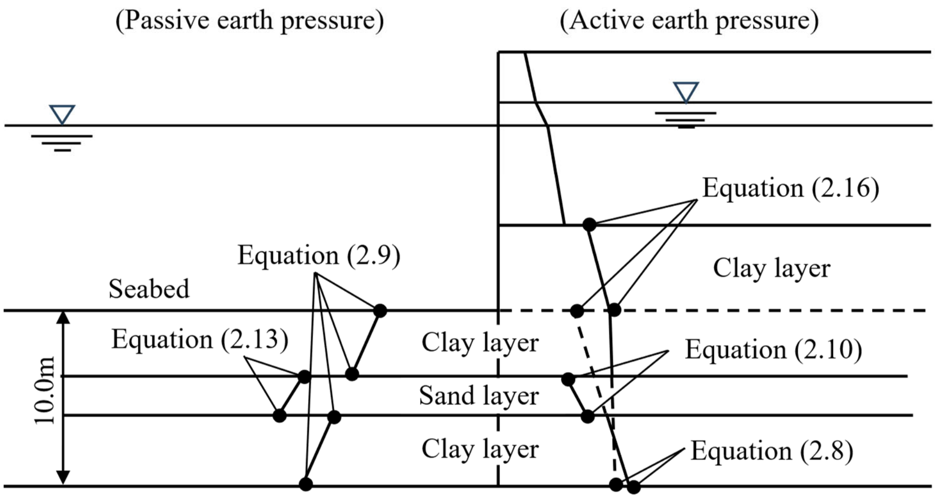
The earth pressure at the top and bottom boundaries of the sand layer shall be calculated using Equations (2.10) and (2.13), respectively. The variation of earth pressure within the sand layer shall be represented by a straight line connecting these two boundary values.

- For the Cohesive Soil Beneath the Sand Layer:

The earth pressure shall be computed using the earth pressure equation for permanent states, Equation (2.8), assuming that cohesive soil uniformly continues beneath the sand layer.



Case where cohesive soil layer extends deeper than 10 m below the seabed



Case where a sand layer exists within 10 m below the seabed

Figure 2.6- Earth Pressure for Cohesive Soil (Variable Situation: Earthquake Gorund Motion)

Table 2.9- Characteristic Values of Total Active Earth Pressure and Moment (Variable Situation: Earthquake Ground Motion)

Horizontal forces and moments

No.	Calculation formula Pa (kN/m^2) h(m)	P_H (kN/m)	Arm length y (m)	Moment M_{PH} (kN·m/m)
1	1/2 × 5.328 × 2.170	5.781	20.277	117.221
2	1/2 × 19.202 × 2.170	20.834	19.553	407.367
3	1/2 × 21.246 × 0.330	3.506	18.720	65.632
4	1/2 × 22.542 × 0.330	3.719	18.610	69.211
5	1/2 × 22.542 × 11.000	123.981	14.833	1,839.010
6	1/2 × 65.772 × 11.000	361.746	11.167	4,039.618
7	1/2 × 161.407 × 2.600	209.829	6.633	1,391.796
8	1/2 × 169.657 × 2.600	220.554	5.767	1,271.935
9	1/2 × 169.657 × 4.900	415.660	3.267	1,357.961
10	1/2 × 169.657 × 4.900	415.660	1.633	678.773
total		1,781.270	7.458	11,238.524

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	5.781	0.268	1.549	20.000	30.980
2	20.834	0.268	5.584	20.000	111.680
3	3.506	0.268	0.940	20.000	18.800
4	3.719	0.268	0.997	20.000	19.940
5	123.981	0.268	33.227	20.000	664.540
6	361.746	0.268	96.948	20.000	1,938.960
7	1,391.796	-	-	-	-
8	1,271.935	-	-	-	-
9	1,357.961	-	-	-	-
10	678.773	-	-	-	-
total			139.245		2,784.900

Table 2.10- Characteristic Values of Passive Earth Pressure (Variable Situation: Earthquake Ground Motion)

Level (m)	h (m)	γ (kN/m^3)	γh (kN/m^2)	$\Sigma\gamma h$ (kN/m^2)	c (kN/m^2)	$\omega\cos\psi/\cos(\psi-\beta)$	$Kp\cos(\delta+\psi)$	Pp (kN/m^2)
-12.600	0.000	6.500	0.000	0.000	30.200	0.000	-	60.400
-17.500	4.900	6.500	31.850	31.850	40.000	0.000	-	111.850

Table 2.11- Characteristic Values of Total Passive Earth Pressure and Moment (Variable Situation: Earthquake Ground Motion)

Horizontal forces and moments

No.	Calculation formula Pp (kN/m^2) h(m)	P_H (kN/m)	Arm length y (m)	Moment M_{PH} (kN·m/m)
1	1/2 × 60.400 × 4.900	147.980	3.267	483.451
2	1/2 × 111.850 × 4.900	274.033	1.633	447.496
total		422.013		930.947

5) Residual Water Pressure and Moment

Residual water pressure considers the water pressure due to the difference of the water levels between the frontal 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/m)

h_w : residual water level (m); 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

γ_w : unit weight of water (kN/m^3)
$\gamma_w = \rho_0 g = 10.1$ (kN/m^3)

Water level difference = 1.330 - 0.000 = 1.330 (m)
Residual water pressure p_w = 10.100×1.330 = 13.433 (kN/m^2)

Table 2.12- 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 × 13.433 × 1.330	8.933	18.165	162.268
2	13.433 x 17.500	235.078	8.750	2,056.933
Point a (-17.500m)		244.011		2,219.201

6) Dynamic Water Pressure and Moment

Under variable situation associated with earthquake ground 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).

$$p_{dwk} = \pm\frac{7}{8} c k_{hk} \rho_w g \sqrt{Hy} \quad (2.20)$$

Where:

p_{dwk} : dynamic water pressure (kN/m²)
k_{hk} : design seismic coefficient
ρ_w : density of water (kg/m³)
g : gravitational acceleration (m/s²)
y : depth from the still water level to the point where dynamic water pressure is calculated (m)
h : water depth (m)
c : correction coefficient (when $L/H \le 1.5$, $c = L/1.5H$; when $L/H \le 1.5$, $c = 1.0$)
L : length of the space occupied by water in the direction of vibration (m)

For the quaywall, $c = 1.0$, and the resultant of the dynamic water pressure and the point of action can be calculated using Equation (2.21).

$$P_{dw} = \pm\frac{7}{12} k_{hk} \rho_w g h^2, \ h_{dw} = \frac{3}{5} h \quad (2.21)$$

TCVN 11820 Part 5: 2021, Equation (15)

Where:

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 × 0.10 × 10.1 × 12.60^2 = 93.536 (kN/m)
h_{dw} = 3 /5 × 12.60 = 7.56 (m) (elevation -7.560 m)
$P_{dwk} \cdot y$ = 93.536 × (12.60 – 7.560+5.50) = 985.869 (kN·m/m)

2-3. Verification of Stability of Stabilized Body

(1) Verification of Sliding Failure

Mode of failure		Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
External stability of the stabilized body (Sliding failure: permanent state)	Sliding failure pattern 1	0.90	1.09	– (1.00)
	Sliding failure pattern 2	0.91	1.10	– (1.00)
External stability of the stabilized body (Sliding failure: variable state of seismic ground motions)		– (1.00)	– (1.00)	1.00

1) Permanent state

Resistance term R_d	Load term S_d
P_{phk}= 422.013 (kN/m) $R_{1k}= \mu_k\times(\sum W_{ik}+P_{avk}-P_{pvk}+C_{uak}-C_{upk})$ = 0.70×(4,087.30+630.0+119.502-0+0-0) = 3,385.761 (kN/m) $R_d= \gamma_R\times(P_{phk}+R_{1k})$ = 0.90×(422.013+3,385.761) = 3,426.997 (kN/m)	P_{ahk}=1,508.913 +244.011 = 1,752.924 (kN/m) $S_d= \gamma_S\times P_{ahk}$ = 1.09×1,752.924 = 1,910.687 (kN/m)
$m\cdot S_d/R_d$ = 1.0× (1,910.687/3,426.997) = 0.558 ≤ 1.0 O.K	

2) Variable situation (Earthquake ground motion)

Resistance term R_d	Load term S_d
P_{phk}= 422.013 (kN/m) $R_{1k}= \mu_k\times(\sum W_{ik}+P_{avk}-P_{pvk}+C_{uak}-C_{upk})$ =0.70×(4,087.300+315.0+139.245+0-0) = 3,179.082 (kN/m) $R_d= \gamma_R\times(P_{phk}+R_{1k})$ = 1.0×(422.013+3,179.082) = 3,601.095 (kN/m)	P_{ahk}= 1,781.270 +244.011+93.536 = 2,118.817 (kN/m) $\sum H_{ik}$= 408.730+31.500 = 440.230 (kN/m) $S_d= \gamma_S\times(P_{ahk}+\sum H_{ik})$ = 1.0×(2,118.817+440.230) = 2,559.047 (kN/m)
$m\cdot S_d/R_d$ = 1.0× (2,559.047/3,601.095) = 0.711 ≤ 1.0 O.K	

(2) Verification of Overturning Failure

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
External stability of the stabilized body (Overturning failure: permanent state)	0.97	1.18	– (1.00)
External stability of the stabilized body (Overturning failure: variable state of seismic ground motions)	– (1.00)	– (1.00)	1.10

1) Permanent state

Resistance term R_d	Load term S_d
$P_{phk} \cdot y_p = 930.947$ (kN·m/m) $\sum(W_{ik} \cdot x_i) = 40{,}202.470 + 6{,}300.0$ $= 46{,}502.470$ (kN·m/m) $P_{avk} \cdot x_{av} = 2{,}390.040$ (kN·m/m) $R_d = \gamma_R \times (P_{phk} \cdot y_p + \sum(W_{ik} \cdot x_i) + P_{avk} \cdot x_{av})$ $= 0.97 \times (930.947 + 46{,}502.470 + 2{,}390.040)$ $= 48{,}328.753$ (kN·m/m)	$P_{ahk} \cdot y_a = 9{,}622.636 + 2{,}219.201$ $= 11{,}841{,}837$ (kN·m/m) $S_d = \gamma_S \times P_{ahk}$ $= 1.18 \times 11{,}841.837$ $= 13{,}973.368$ (kN·m/m)
$m \cdot S_d / R_d = 1.0 \times (13{,}973.368/48{,}328.753) = 0.289 \leq 1.0$ O.K	

2) Variable situation (Earthquake ground motion)

Resistance term R_d	Load term S_d
$P_{phk} \cdot y_p = 930.947$ (kN·m/m) $\sum(W_{ik} \cdot x_i) = 40{,}202.470 + 3{,}150.000$ $= 43{,}352.470$ (kN·m/m) $P_{avk} \cdot x_{av} = 2{,}784.900$ (kN·m/m) $R_d = \gamma_R \times (P_{phk} \cdot y_p + \sum(W_{ik} \cdot x_i) + P_{avk} \cdot x_{av})$ $= 1.0 \times (930.947 + 43{,}352.470 + 2{,}784.900)$ $= 47{,}068.317$ (kN·m/m)	$P_{ahk} \cdot y_a = 11{,}238.524 + 2{,}219.201 + 985.869$ $= 14{,}443.594$ (kN·m/m) $\sum(H_{ik} \cdot y_i) = 5{,}034.221 + 661.500$ $= 5{,}695.721$ $S_d = \gamma_S \times P_{ahk}$ $= 1.0 \times (14{,}443.594 + 5{,}695.721)$ $= 20{,}139.315$ (kN·m/m)
$m \cdot S_d / R_d = 1.10 \times (20{,}139.315/47{,}068.317) = 0.471 \leq 1.0$ O.K	

(3) Verification of Stability against Bearing Capacity Failure of Bearing Ground

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

1) Permanent state

$$x=\frac{\Sigma M}{\Sigma V}=\frac{\Sigma M_V\text{-}\Sigma M_H}{\Sigma V}=\frac{49{,}823.457\ \text{-}\ 11{,}841{,}837}{4{,}836.802}=7.853\ \text{(m)}$$

$e = B/2 - x = 20.000/2 - 7.853 = 2.147$ (m)

$e < B/6 = 20.000 / 6 = 3.333$ (m)

$$t_{1k},t_{2k}=\frac{\Sigma V}{B}(1\pm\frac{6\times e}{B})=\frac{4{,}836.802}{20.0}(1\pm\frac{6\times 2.147}{20.0})$$

$$= 397.609,\ 86.071\ \text{(kN/m}^2\text{)}$$

2) Variable situation (Earthquake ground motion)

$$x=\frac{\Sigma M}{\Sigma V}=\frac{\Sigma M_V\text{-}\Sigma M_H}{\Sigma V}=\frac{47{,}068.317\ \text{-}\ 20{,}139.315}{4{,}541.545}=5.929\ \text{(m)}$$

$e = B/2 - x = 20.000/2 - 5.929 = 4.071$ (m)

$e > B/6 = 20.000 / 6 = 3.333$ (m)

$$t_1=\frac{2\Sigma V}{3(B/2\text{-}e)}=\frac{2\times 4{,}541.545}{3\times(20.0/2\ \text{-}\ 4.071)}$$

$$= 510.659\ \text{(kN/m}^2\text{)}$$

3) Terzaghi's Bearing Capacity Formula

$$q_d=\frac{1}{m_B}(\beta\gamma_{1k}\frac{B}{2}N_{rk}+\gamma_{2k}D\left(N_{qk}\text{-}1\right))+\gamma_{2k}D \qquad (2.22)$$

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

Where:

- q_d : design value of foundation bearing capacity considering buoyancy of submerged part (kN/m^2)
- m_B : adjustment factor for bearing capacity of sandy ground
- β : shape factor of a foundation (refer to Table 2.13)
- γ_{1k} : characteristic value of unit volume weight of soil of ground below the foundation bottom, or unit volume weight in water, if submerged (kN/m^3)
- N_{rk}, N_{qk}, : characteristic value of bearing capacity coefficients
- B : minimum width of foundation (m)
- γ_{2k} : characteristic value of unit volume weight of soil of ground above the foundation bottom, or unit volume weight in water, if submerged (kN/m^3)
- D : embedded depth of foundation (m)

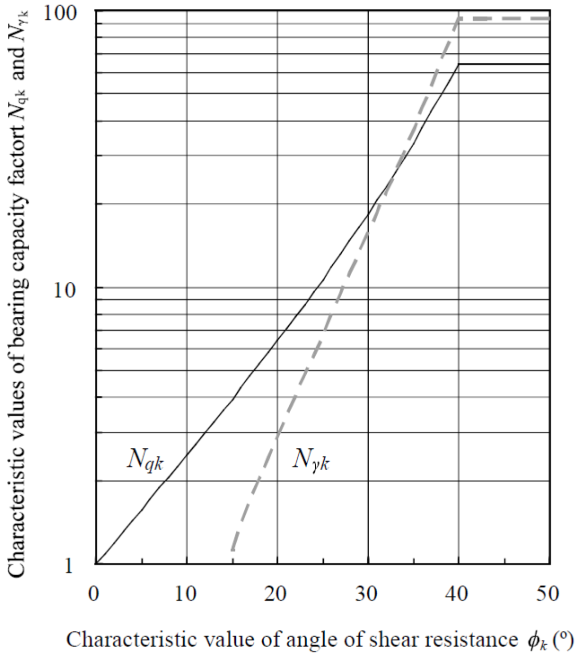
Table 2.13- Shape Factors (β)

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

Shape of foundation	Continuous	Square	Round	Rectangular
β	1	0.8	0.6	1–0.2 (B/L)

B: length of short side of rectangle, L: length of long side of rectangle

Source: TCVN 11820-4-1-2020



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

Source: TCVN 11820-4-1-2020

Figure 2.7- Relationship between Bearing Capacity Coefficients *Nr* and *Nq* and Angle of Shear Resistance *φ*

Since the front of the stabilized body is a sheet pile wall, the embedment effect is disregarded. In the verification of the bearing capacity of block-type improved ground refer to Terzaghi's bearing capacity formula for sandy ground.

4) Verification of bearing capacity at Permanent state

$$q_d = \frac{1}{m_B}\left(\beta\gamma_{1k}\frac{B}{2}N_{rk} + \gamma_{2k}D\left(N_{qk} - 1\right)\right) + \gamma_{2k}D$$

$$= \frac{1}{2.5}(1.0\times10.0\times\frac{20.0}{2}\times14+0)+0 = 560.00\ (\text{kN/m}^2)$$

$$> 397.609\ (\text{kN/m}^2)\ \text{O.K}$$

5) Verification of bearing capacity at Variable Situation

$$q_d = \frac{1}{m_B}\left(\beta\gamma_{1k}\frac{B}{2}N_{rk} + \gamma_{2k}D\left(N_{qk} - 1\right)\right) + \gamma_{2k}D$$

$$= \frac{1}{1.5}(1.0\times10.0\times\frac{20.0}{2}\times14+0)+0 = 933.33\ (\text{kN/m}^2)$$

$$> 510.659\ (\text{kN/m}^2)\ \text{O.K}$$

2-5. Internal Stability of Stabilized Body

(1) Verification of Toe Pressure

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Internal stability of the stabilized body (Toe pressure: permanent state)	0.72	1.33	– (1.00)
Internal stability of the stabilized body (Toe pressure: variable state of seismic ground motions)	– (1.00)	– (1.00)	1.50

1) Permanent state

Resistance term R_d	Load term S_d
$R_k = f_{ck}$ $= \alpha\beta q_{uck}$ $= 0.80\times 1{,}000$ $= 800$ (kN/m^2) $R_d = \gamma_R \times R_k$ $= 0.72\times 800$ $= 576.0$ (kN/m^2)	$S_k = t_1, t_2$ $t_{1k} = 397.609$ (kN/m^2) $t_{2k} = 86.071$ (kN/m^2) $S_d = \gamma_S \times S_k$ $= 1.33\times 397.609$ $= 528.820$ (kN/m)
$m \cdot S_d / R_d = 1.0\times (528.820/576.0) = 0.918 \leq 1.0$ O.K	

2) Variable situation (Earthquake ground motion)

Resistance term R_d	Load term S_d
$R_k = f_{ck}$ $= \alpha\beta q_{uck}$ $= 0.80\times 1{,}000$ $= 800$ (kN/m^2) $R_d = \gamma_R \times R_k$ $= 1.0\times 800$ $= 800.0$ (kN/m^2)	$S_k = t_1$ $t_{1k} = 510.659$ (kN/m^2) $S_d = \gamma_S \times S_k$ $= 1.0\times 510.659$ $= 510.659$ (kN/m)
$m \cdot S_d / R_d = 1.5\times (510.659/800.0) = 0.957 \leq 1.0$ O.K	

\- End -