1. Technical Notes

1-1. Characteristics of Sheet Pile Quaywall

(1) Outlines

Sheet pile quaywalls are quay walls with sheet piles driven as earth retaining walls. Sheet piles can be made of steel, reinforced concrete, pre-stressed concrete, or wood. Among these materials, steel material has been most frequently used for sheet piles. Given the large yield stress and availability of models with large section moduli, steel sheet piles can be used for quay walls with large depths.

The cross-sectional shapes of normally used steel piles are classified into three types: hat-type, U-shaped, and steel pipes with joints. When using sheet piles with U-shaped cross sections, it is necessary to design their joints in a manner that prevents them from sliding because the configuration of the cross sections of U-shaped sheet piles with joints arranged along the neutral axes of sheet pile walls is subjected to reductions in section moduli obtained from sheet pile walls as integrated structures when sliding occurs between joints.

Steel pipe sheet piles fabricated by connecting large diameter steel pipes with joints can enlarge section moduli without large increments in the weight of steel per unit width, thereby enabling sheet piles with larger section moduli than steel sheet piles to be easily fabricated.

Reinforced concrete and pre-stressed concrete sheet piles are not often used for large-scale quaywalls because it is difficult to drive sheet piles with increased thickness owing to large section moduli. Even if they can be driven, there may be cases wherein sheet piles are damaged while being driven into stiff ground. Therefore, when driving reinforced concrete or pre-stressed concrete sheet piles into stiff ground, possible damage should be inspected by pullout test, or the sheet piles should be driven using water jetting to protect them from being damaged. Furthermore, sheet piles need to have joint materials to prevent the possible washing out of backfill soil through the joints.

(2) Characteristics

Sheet pile type quaywalls can be constructed using relatively simple machines.

In many cases, sheet pile quaywalls do not require underwater construction as foundation; therefore, such quay walls can be rapidly constructed.

In cases wherein the existing seafloor is deep, sheet pile walls are placed into a state that is vulnerable to waves until backfill or anchorage work is constructed.

Sheet pile quaywalls are classified into the following categories on the basis of the method used to resist the earth and water pressure acting on the quaywalls:

- ✓ Sheet pile quaywalls (connected to anchorage work via tie materials).
- ✓ Cantilevered sheet pile quaywalls.
- ✓ Sheet pile quaywalls with batter pile anchorages.
- ✓ Sheet pile quaywalls anchored by forward batter piles.
- ✓ Double sheet pile quaywalls

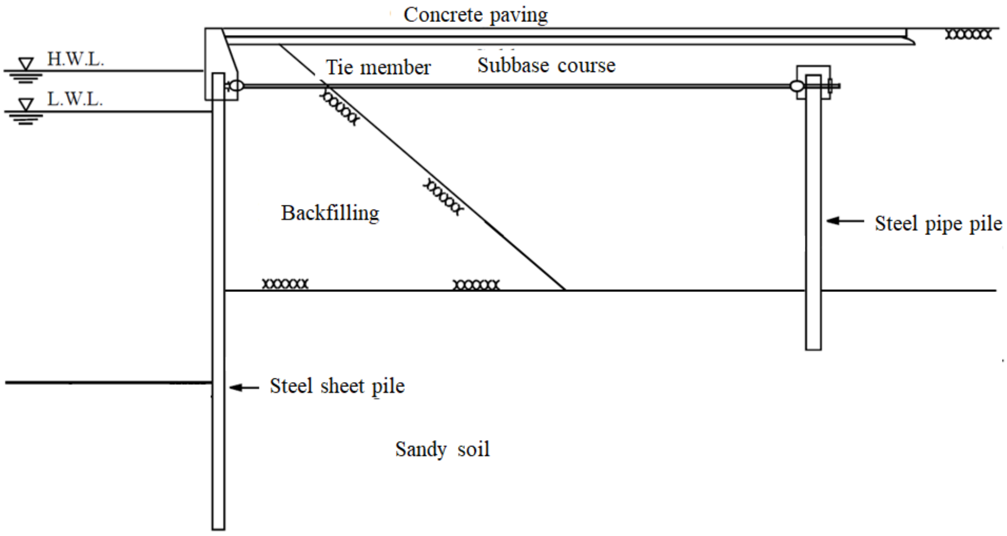
1-2. Basic Policy for Performance Verification

This part introduces the performance review of a sheet pile type quaywall, where the sheet pile wall in front of the quay and the upright pile support structure are tied together using tie materials such as tie rods. The sheet pile type quaywall resists the forces acting on the sheet pile wall such as earth pressure, residual water pressure, mooring forces, and hydrodynamic pressure by utilizing the horizontal resistance of the sheet pile embedded portion and the horizontal resistance of the support structure tied with the tie rods.

Compared to gravity type quaywalls, the sheet pile type is often used in relatively soft ground conditions or in deeper water areas. An example of a cross-section of a sheet pile quaywall is shown in Figure 1.1.

The performance verification of sheet pile quaywalls applies to the reliability design method (partial factor method) and dynamic analysis method. The partial factor method is applied for verifying the permanent state and variable conditions related to earthquake motion, while the dynamic analysis method is used for verifying the amount of deformation and the stress levels of steel materials.

Additionally, the seismic coefficient for verification is estimated using a static coefficient method refer to TCVN 11820 Part 2, 2025.



Modified from TCVN 11820 Part 5: 2021, Hinh10

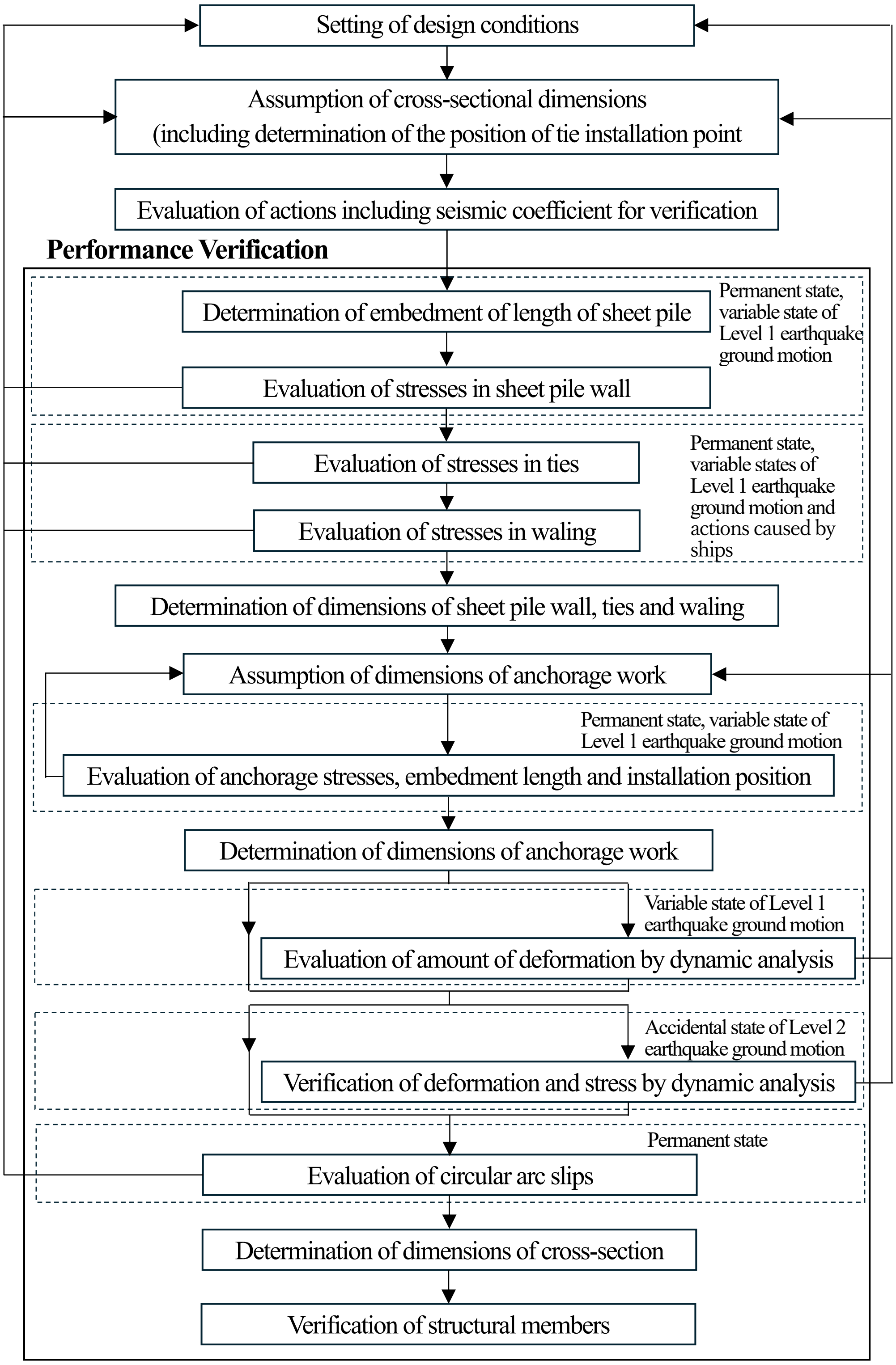
Source: Modified from TCVN 11820-5-2021

Figure 1.1- Example of Cross-section of Sheet Pile Quaywall

An example of the sequence of the performance verification is shown in Figure 1.2.

The performance verification of sheet pile quaywalls on soft ground, such as alluvial cohesive soil on a soft seabed, should ideally be conducted through a comprehensive examination using the performance verification methods outlined in this section for ties and anchorages, among others. Sheet piles constructed on soft ground may experience unexpected large deformations due to lateral flows caused by the settlement of the ground behind the wall. Several methods for predicting lateral flow have been proposed and should be considered during performance verifications.

Caution is advised when using the performance verification methods for sheet-pile quaywalls described in this casebook, as many of these methods are predicated on the assumption that steel sheet pile walls are driven primarily into sandy or hard clayey soil. For soft grounds, soil improvement work is recommended. If soil improvement is not feasible due to site conditions, it is advisable to consider additional performance verification methods, including numerical analysis techniques that can accurately assess the nonlinear characteristics of soil, to enable a comprehensive analysis.



OCDI 2020 Part III, Chapter 5 Fig. 2.3.1

Note: The evaluation of liquefaction and settlement is not shown; therefore, it is necessary to consider these effects separately.

When necessary, an evaluation of amount of deformation by dynamic analysis can be performed for the Level 1 earthquake ground motion. For high earthquake-resistance facilities, it is preferable that an examination of the amount of deformation be performed by dynamic analysis.

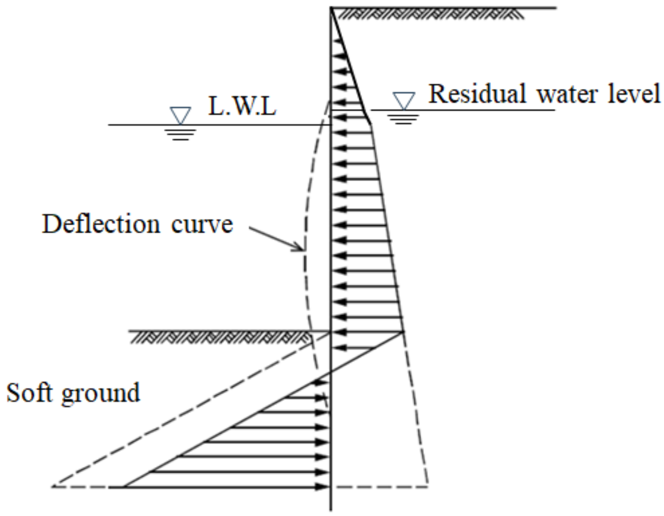
Verification with respect to Level 2 earthquake ground motion is performed for high earthquake-resistance facilities.

Source: OCDI 2020

Figure 1.2- Example of the Sequence of the Performance Verification of Sheet Pile Quaywalls

Reference:

In determining the embedded lengths of sheet piles, the deflection curve method which is a type of fixed earth support method based on the classical earth pressure theory for deep embedded sheet piles may be used alongside the methods introduced in this casebook. The deflection curve method calculates the embedded length by solving an Equation under specific conditions: zero displacement and deflection angle at the lower end of embedment, zero displacement at the tie member installation point, and under the loading conditions depicted in Figure 1.3. This method may be used for soft ground.



Modified from TCVN 11820 Part 5: 2021, Hinh 27

Source: Modified from TCVN 11820-5-2021

Figure 1.3- Earth Pressure and Deflection Curve by Fixed Earth Support Method (Soft Ground)

1-3. Design Conditions

(1) Setting of Design Conditions

Design conditions necessary for performance verification of the sheet pile quaywall are set according to the classification, nature of the facility and the situation in which it is placed. The main design conditions are as follows:

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

(2) Approach to Structural Specifications

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

1) Top Elevation of the Quaywall

The top elevation of quaywall is determined considering factors such as the scale of the quaywall, tidal variations, waves, abnormal tidal levels, and settlement. In cases where the specifications of the target vessel are unclear, Table 1.1 is often used as a reference to determine the elevation. However, the reference tidal level should be H.W.L. (High Water Level).

Table 1.1- Standard Crown Heights of Quaywall

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

Source: OCDI 2020

OCDI 2020 Part III, Chapter 5 Table 2.1.2

The top elevation of the sheet-pile is determined considering the construction conditions, such as the tide level, dimension of cap concrete, as well as economic factors.

2) Design Water Depth and Construction Limits

When the seabed in front of the quaywall is dredged, the design water depth should take into account the extra dredging depth as a construction tolerance.

3) Superstructure

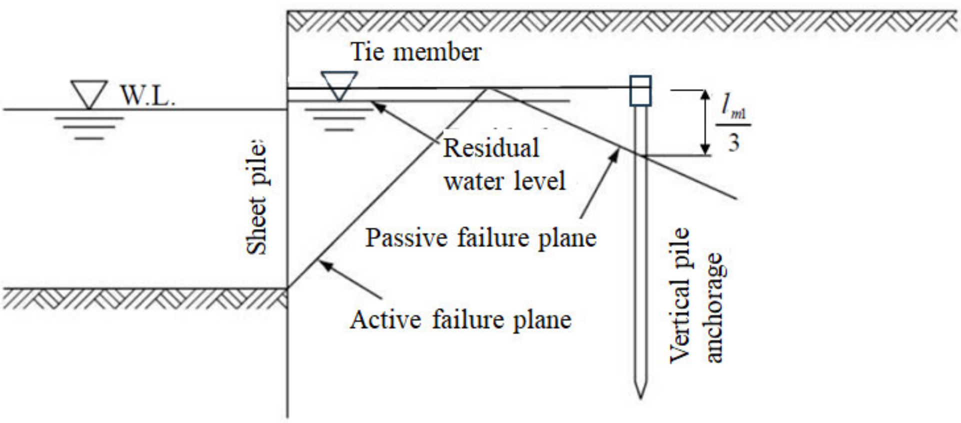
The shape of the superstructure shall be designed to ensure safety against the mooring force of vessels. In particular, the area where mooring bollards are installed is often partially widened and reinforced to secure the anchors.

4) Install Location of Anchorage

In principle, the location of the anchorages needs to be set at an appropriate distance from the sheet pile wall to ensure the structural stability of the main body of the wall and anchorage depending on the characteristics of the anchorages. The location of an anchorage should be determined appropriately in consideration of the structural type of the anchorage because the stability of the anchorage itself is affected by its position, and the location at which the stability is achieved varies depending on the structural type.

i) Vertical Pile Anchorage

The location of a vertical pile anchorage is preferably determined to ensure that the passive failure plane from the point of $l_{m1}/3$ below the tie member installation point of the anchorage, and the active failure plane from the intersection of the sea bottom and sheet piles do not intersect at the level below the horizontal surface containing the tie member installation point at the anchorage (Figure 1.4). The value of l_{m1} is the depth of the first zero point of the bending moment for a free-head pile below the tie member installation point.



Modified from TCVN 11820 Part 5: 2021, Hinh 20

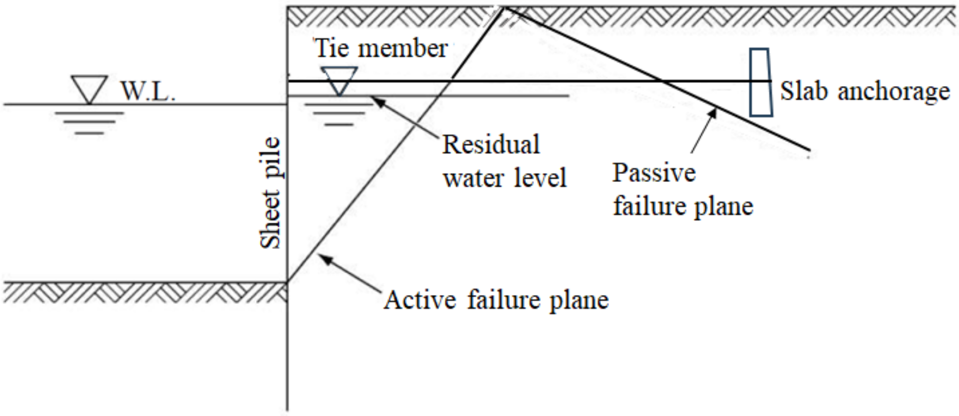
Source: Modified from TCVN 11820-5-2021

Figure 1.4- Location of Vertical Pile Anchorage

ii) Slab Anchorage

The slab anchorage type is often used for quays with shallow water depths.

The location of the slab anchorage is preferably determined to ensure that the active failure plane starting from the intersection of the sea bottom and sheet pile wall and the passive failure plane of the slab anchorage drawn from the bottom of the anchorage do not intersect below the ground surface (Figure 1.5).



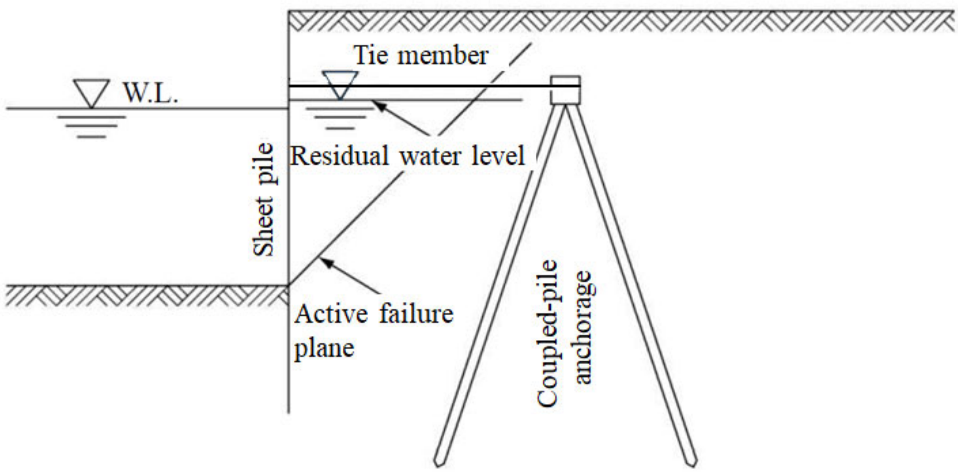
Modified from TCVN 11820 Part 5: 2021, Hinh 20

Source: Modified from TCVN 11820-5-2021

Figure 1.5- Location of Slab Anchorage

iii) Coupled Pile Anchorage

The location of a coupled-pile anchorage should be behind the active failure plane of the sheet pile wall drawn from the sea bottom when it is assumed that the tension of the tie member is resisted only by the axial bearing capacity of the piles (Figure. 1.6).



Modified from TCVN 11820 Part 5: 2021, Hinh 21

Source: Modified from TCVN 11820-5-2021

Figure 1.6- Location of Coupled Pile Anchorage

5) Installation Height of Tie Member

Tie member is a collective term of materials such as tie rods and tie rods connecting sheet piles and anchorages. The position of tie member installation should be determined by considering the difficulty of the work of tie member attachments and the costs.

When the wall height of a sheet pile wall is large, tie members may be provided at two levels to support the wall structure at two points to reduce the bending moments in the sheet pile. The tie member installation position is generally set above the residual water level (RWL).

1-4. Loads and Actions

(1) Seismic Forces

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

The seismic coefficient is determined depending on soil conditions, importance coefficient and ground acceleration according to the earthquake classification criteria of TCVN 9386: 2012, and TCVN 11820 Part 2: 2025.

Assumption of cross-sectional dimensions

↓

Setting of soil condition, importance coefficient

↓

Setting of peak ground acceleration

↓

Seismic coefficient for verification

Figure 1.7- Example Flow for Setting Seismic Coefficient

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

NOTE:

According to the design ground acceleration value $a_g = \gamma_1 \times a_gR$, divided into three earthquake cases:

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

TCVN 11820 Part 2: 2025, Chapter 5.13.4

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

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

The regional seismic coefficient shall be the values listed in Figure 1.8. The soil condition coefficient shall be the value listed in Table 1.2 corresponding to the type of subsoil given in Table 1.3, and the importance coefficient shall be the value listed in Table 1.4 corresponding to the characteristics of structures.

1) Regional Seismic Coefficient

The regional seismic coefficient is calculated according to the Equation (1.1):

TCVN 11820 Part 2: 2025, Equation (72)

$$k_{h1}=a_gR/g \tag{1.1}$$

Where:

a_gR : reference peak ground acceleration (Figure 1.8)
g : gravitational acceleration

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

However, Figure 1.8 presents acceleration for a return period of 500 years, and for a Level 1 performance verification with a return period of 75 years, the acceleration for a may be reduced. Since there are no acceleration maps with a return period of 75 years in Vietnam, Consultants are expected to set the design background acceleration after careful consideration and/or consultation with seismic experts.

2) Soil Condition Coefficient

Table 1.2- Soil Condition Coefficient γ_s

TCVN 11820 Part 2: 2025, Bang 13

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

Source: TCVN 11820-2-2025

Table 1.3- Classification by Type of Subsoil

TCVN 11820 Part 2: 2025, Bang 14

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

Source: TCVN 11820-2-2025

NOTE:

In the above tables, "weak soil" is sandy soil with N value of SPT test less than 4 or cohesive soil with free lateral compressive strength less than 20 kN/m^2. When the ground consists of two or more layers, the soil type must be determined according to which layer has the greatest thickness. If the foundation soil consists of two or more layers of almost equal thickness, the soil type must be determined according to which soil layer has the largest soil condition coefficient value in those layers.

3) Importance Coefficient

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

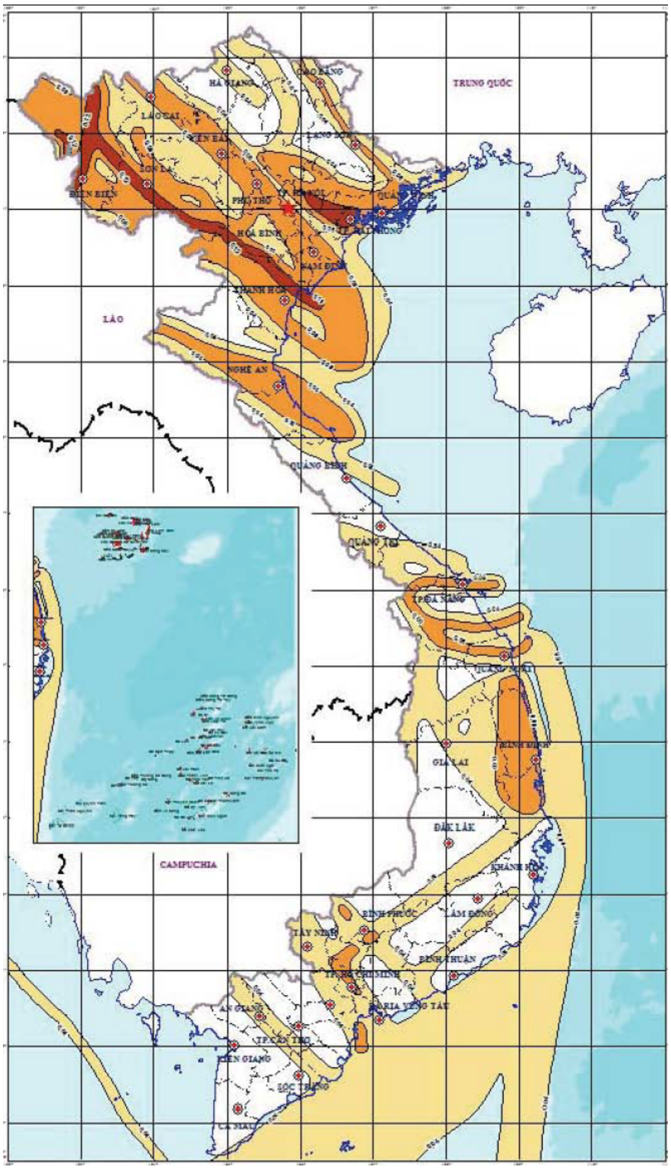
Table 1.4- Level and Importance Coefficient

TCVN 11820 Part 2: 2025, Bang 15

Type of structure	Structural Characteristics	Importance Coefficient γ_i
Special structure	The structure has special importance, and not allow being damaged by earthquake	Designed with the biggest acceleration
Type I	The structure has vital importance with community protection, function cannot be discontinued during earthquake	1.25
Type II	The structure has importance in preventing seismic consequence, if it is collapse causing big damage of person and property	1.0
Type III	The structure not belong to special category, Type I, II, IV	0.75
Type IV	The structure has secondary importance for safety of person life	No seismic calculation required

Source: TCVN 11820-2-2025

TCVN 9386 Part 1: 2012, Hinh G1



Source: TCVN 9386-1-2012

Figure 1.8- Zoning maps of Background Acceleration on Viet Nam Territory (Return period: 500 years)

(2) Surcharge Loads

The surcharge loads are applied by considering the utilization pattern of the quay. Both live and dead loads are considered. For general cargo piers, a load of about 10 to 30 kN/m² is commonly used. Regarding the variable conditions for earthquake motions, the loads are often considered as half of the permanent load conditions.

(3) Residual Water Level

The residual water level to be used in the determination of the residual water pressure needs to be estimated appropriately in consideration of the structure of the sheet pile wall and the soil conditions. The residual water level varies depending on the characteristics of the subsoil and the conditions of sheet pile joints. However, in many cases, the elevation with the height equivalent to 2/3 of the tidal range above the mean monthly LWL is used for sheet pile walls. However, in the case of a steel sheet pile wall driven into cohesive soil ground, care should be exercised in the determination of the residual water level because it is sometimes nearly the same as the high water level. When sheet piles made of other materials are used, it is preferable to determine the residual water level on the basis of the result of investigations of similar structures.

(4) Dynamic Water Pressure

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

$$p_{dwk} = \pm\frac{7}{8} c k_{hk} \rho_w g \sqrt{Hy} \qquad (1.2)$$

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 $L/H \leq 1.5$, $c = L/1.5H$; when $L/H \leq 1.5$, $c = 1.0$)
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 (1.3).

$$P_{dw} = \pm\frac{7}{12} k_{hk} \rho_w g h^2, \quad h_{dw} = \frac{3}{5} h \qquad (1.3)$$

TCVN 11820 Part 5: 2021, Equation (15)

Where:

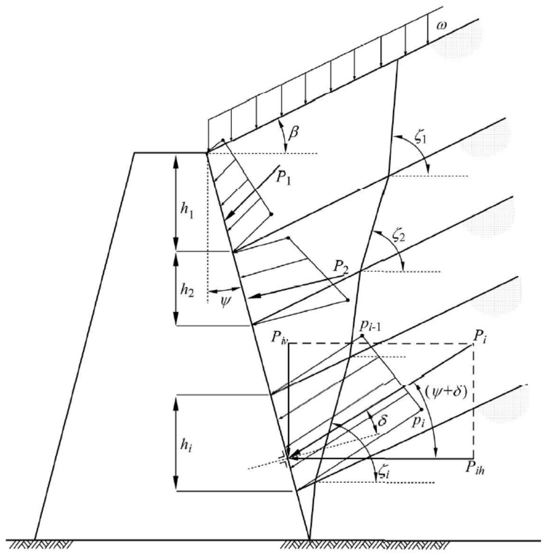
P_{dwk}	:	resultant force of dynamic water pressure (kN/m)
h_{dw}	:	depth of the acting point of the dynamic water pressure resultant force from the still water level (m)

(5) Earth Pressure

The active earth pressure acting on the quaywall is calculated based on both permanent and variable states associated with earthquake motions.

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

The earth pressure below the water surface is evaluated using the apparent seismic coefficient according to Equation (1.21).



TCVN 11820 Part 4-1: 2020, Hinh 1

Source: TCVN 11920-4-1-2020

Figure 1.9- Earth Pressure

1) Earth Pressure in Permanent State

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

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

$$p_{ai}=K_{ai}\left[\sum \gamma_i h_i+\frac{\omega\cos\psi}{\cos(\psi-\beta)}\right]\cos\psi \qquad (1.4)$$

$$K_{ai}=\frac{\cos^2(\varphi_i-\psi)}{\cos^2\psi\cos(\delta+\psi)\left[1+\sqrt{\frac{\sin(\varphi_i+\delta)\sin(\varphi_i-\beta)}{\cos(\delta+\psi)\cos(\psi-\beta)}}\right]^2} \qquad (1.5)$$

$$\cot(\zeta_i-\beta)=-\tan(\varphi_i+\delta+\psi-\beta)+\sec(\varphi_i+\delta+\psi-\beta)\sqrt{\frac{\cos(\delta+\psi)\sin(\varphi_i+\delta)}{\cos(\psi-\beta)\cos(\varphi_i-\beta)}} \qquad (1.6)$$

✓ Passive earth pressure

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

$$p_{pi}=K_{pi}\left[\sum \gamma_i h_i+\frac{\omega\cos\psi}{\cos(\psi-\beta)}\right]\cos\psi \qquad (1.7)$$

$$K_{pi}=\frac{\cos^2(\varphi_i+\psi)}{\cos^2\psi\cos(\delta+\psi)\left[1-\sqrt{\frac{\sin(\varphi_i-\delta)\sin(\varphi_i+\beta)}{\cos(\delta+\psi)\cos(\psi-\beta)}}\right]^2} \qquad (1.8)$$

$$\cot(\zeta_i\text{-}\beta)=\tan(\varphi_i\text{-}\delta\text{-}\psi+\beta)+\sec(\varphi_i\text{-}\delta\text{-}\psi+\beta)\sqrt{\frac{\cos(\delta+\psi)\sin(\varphi_i\text{-}\delta)}{\cos(\psi\text{-}\beta)\cos(\varphi_i+\beta)}} \quad (1.9)$$

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

$$p_{ai}=\sum\gamma_i h_i+\omega\text{-}2c_i \quad (1.10)$$

- ✓ Passive earth pressure

$$p_{pi}=\sum\gamma_i h_i+\omega+2c_i \quad (1.11)$$

TCVN 11820 Part 4-1: 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 State (Earthquake Ground Motion)

i) Earth Pressure for Sandy Soil

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

- ✓ Active earth pressure

$$p_{ai}=K_{ai}\left[\sum\gamma_i h_i+\frac{\omega\cos\psi}{\cos(\psi\text{-}\beta)}\right]\cos\psi \quad (1.12)$$

$$K_{ai}=\frac{\cos^2(\varphi_i\text{-}\psi\text{-}\theta)}{\cos\theta\cos^2\psi\cos(\delta+\psi+\theta)\left[1+\sqrt{\frac{\sin(\varphi_i+\delta)\sin(\varphi_i\text{-}\beta\text{-}\theta)}{\cos(\delta+\psi+\theta)\cos(\psi\text{-}\beta)}}\right]^2} \quad (1.13)$$

$$\cot(\zeta_i\text{-}\beta)=\text{-}\tan(\varphi_i+\delta+\psi\text{-}\beta)+\sec(\varphi_i+\delta+\psi\text{-}\beta)\sqrt{\frac{\cos(\delta+\psi+\theta)\sin(\varphi_i+\delta)}{\cos(\psi\text{-}\beta)\cos(\varphi_i\text{-}\beta\text{-}\theta)}} \quad (1.14)$$

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

✓ Passive earth pressure

$$p_{pi}=K_{pi}\left[\sum \gamma_i h_i+\frac{\omega\cos\psi}{\cos(\psi-\beta)}\right]\cos\psi \quad (1.15)$$

$$K_{pi}=\frac{\cos^2(\varphi_i+\psi-\theta)}{\cos\theta\cos^2\psi\cos(\delta+\psi-\theta)\left[1-\sqrt{\frac{\sin(\varphi_i-\delta)\sin(\varphi_i+\beta-\theta)}{\cos(\delta+\psi-\theta)\cos(\psi-\beta)}}\right]^2} \quad (1.16)$$

$$\cot(\zeta_i-\beta)=\tan(\varphi_i-\delta-\psi+\beta)+\sec(\varphi_i-\delta-\psi+\beta)\sqrt{\frac{\cos(\delta+\psi-\theta)\sin(\varphi_i-\delta)}{\cos(\psi-\beta)\cos(\varphi_i+\beta-\theta)}} \quad (1.17)$$

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

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 states (earthquake motion) can be performed using the following Equations:

✓ Active Pressure

$$p_{ai}=\frac{\left(\sum \gamma_i h_i+\omega\right)\sin(\zeta_i+\theta)}{\cos\theta\sin\zeta_i}-\frac{c_i}{\cos\zeta_i\sin\zeta_i} \quad (1.18)$$

$$\zeta_i=\tan^{-1}\sqrt{1-\frac{\sum \gamma_i h_i+2\omega}{2c}\tan\theta} \quad (1.19)$$

✓ Passive Pressure

$$p_{pi}=\sum \gamma_i h_i+\omega+2c_i \quad (1.20)$$

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

iii) Calculation Formula for Apparent Seismic Coefficient

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

$$k'=\frac{2\left(\sum \gamma_{ti} h_i+\sum \gamma_{sat\,j} h_j+\omega\right)+\gamma_{sat} h}{2\left\{\sum \gamma_{ti} h_i+\sum (\gamma_{sat\,j}-10) h_j+\omega\right\}+(\gamma_{sat}-10)h}k \quad (1.21)$$

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)

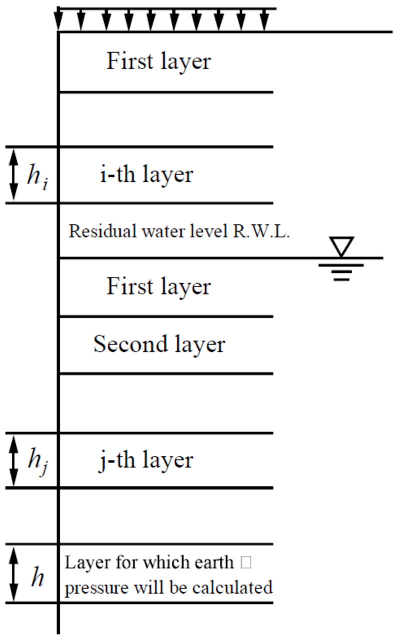
γ_{ti} : unit weight of soil in the i-th layer above the residual water level (kN/m^3)

$\gamma_{sat\,j}$: 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



TCVN 11820 Part 4-1: 2020, Hinh 3

Source: TCVN 11820-4-2020

Figure 1.10- Symbols for Apparent Seismic Coefficient

1-5. Performance Criteria

(1) Sheet Pile Quaywalls

The required performance of sheet pile quaywalls under a permanent state in which the dominant action is earth pressure and a variable state in which the dominant actions are Level 1 earthquake ground motions shall be serviceability. The performance verification items and standard indexes for determining the limit values with respect to the actions shall be shown in Table 1.5, below respectively.

Table 1.5- Performance Verification Items and Standard Indexes for Determining Limit Values under the Respective Design Situations of Sheet Pile Quaywalls

OCDI 2020 Part III, Chapter 5 Attached Table 11-6

Performance requirements	Design situation			Verification item	Standard index to provide limit value
	Situation	Dominating action	Non-dominating action		
Serviceability	Permanent	Earth pressure	Water pressure, surcharges	Necessary embedded length	Embedded length required for structural stability
				Yielding of the sheet pile	Design yield stress of sheet pile
	Variable	Level 1 earthquake ground	Earth pressure, water	Necessary embedded length	Embedded length required for structural stability

		motion	pressure, surcharges	Yielding of the sheet pile	Design yield stress of sheet pile

Source: Modified from OCDI 2020

(2) Ties and Waling

For the permanent state in which the dominant action is earth pressure and the variable state in which the dominant actions are Level 1 earthquake ground motions and traction by ships, the performance verification items and the standard indexes for determining the limit values with respect to ties and waling shall be those shown in Table 1.6.

OCDI 2020 Part III, Chapter 5 Attached Table 11-8

Table 1.6- Performance Verification Items and Standard Indexes for Determining the Limit Values with respect to Ties and Waling

Performance requirements	Design situation			Verification item	Standard index to provide limit value
	Situation	Dominating action	Non-dominating action		
Serviceability	Permanent	Earth pressure	Water pressure, surcharges	Yielding of the tie	Design yield stress
				Yielding of the waling	
	Variable	Level 1 earthquake ground motion [traction force of ships]	Earth pressure, water pressure, surcharges	Yielding of the tie	
				Yielding of the waling	

* [ ] indicates an alternative dominant action to be studied as design situations.

Source: Modified from OCDI 2020

(3) Anchorage

For the permanent state in which the dominant action is earth pressure and the variable state in which the dominant actions are Level 1 earthquake ground motions and traction by ships, the performance verification items and standard indexes for determining the limit values with respect to anchorages shall be those shown in Table 1.7.

OCDI 2020 Part III, Chapter 5 Attached Table 11-7

Table 1.7- Performance Verification Items and Standard Indexes for Determining the Limit Values with respect to Anchorages

Performance requirements	Design situation			Verification item	Standard index to provide limit value
	Situation	Dominating action	Non-dominating action		
Serviceability	Permanent	Earth pressure	Water pressure, surcharges	Necessary embedded length	Embedded length required for structural stability
				Yielding of the anchorage*1)	Design yield stress

				Axial force in the anchorage*2)	Action–resistance ratio with respect to the bearing force of an anchorage (pushing force and pullout force)
				Stability of the anchor wall*3)	Design cross-section resistance Passive earth pressure on the front face of anchor plate
	Variable	Level 1 earthquake ground motion [traction force of ships]	Earth pressure, water pressure, surcharges	Necessary embedded length	Embedded length required for structural stability
				Yielding of the anchorage*1)	Design yield stress
				Axial force in the anchorage*2)	Action–resistance ratio with respect to the bearing force of an anchorage (pushing force and pullout force)
				Stability of the anchor wall*3)	Design cross-section resistance Passive earth pressure on the front face of anchor plate

* [ ] indicates an alternative dominant action to be studied as design situations.
*1): Only when the structural type of the anchorage is a vertical pile anchor, a coupled-pile anchor, or sheet pile anchor
*2): Only when the structural type of the anchorage is a coupled-pile anchor
*3): Only when the structural type of the anchorage is a slab anchor

Source: Modified from OCDI 2020

(4) Copings of Sheet Pile

For the permanent state in which the dominant action is earth pressure and the variable state in which the dominant actions are Level 1 earthquake ground motions and traction by ships, the performance verification items and standard indexes for determining the limit values with respect to the copings of sheet pile quaywalls shall be those shown in Table 1.8.

Table 1.8- Performance Verification Items and Standard Indexes for Determining the Limit Values with respect to Copings of Sheet Pil

OCDI 2020 Part III, Chapter 5 Attached Table 11-9

Performance requirements	Design situation			Verification item	Standard index to provide limit value
	Situation	Dominating action	Non-dominating action		
Serviceability	Permanent	Earth pressure	Surcharges	Cross-section stress of the coping	Bending compression stress

	Variable	Level 1 earthquake ground motion [traction force of ships] [berthing force of ships]	Earth pressure, surcharges	Failure of the cross section of the coping	Design cross-section resistance

* [ ] indicates an alternative dominant action to be studied as design situations.

Source: Modified from OCDI 2020

(5) Self-weight (Circular Slip Failure)

For the permanent situation in which the dominant action is the self-weight of sheet pile quaywalls, the performance verification items and standard indexes to determine the limit values of sheet pile quaywalls shall be those shown in Table 1.9.

Table 1.9- Performance Verification Items and Standard Indexes for Determining the Limit Values with respect to Copings of Sheet Pil

OCDI 2020 Part III, Chapter 5 Attached Table 11-10

Performance requirements	Design situation			Verification item	Standard index to provide limit value
	Situation	Dominating action	Non-dominating action		
Serviceability	Permanent	Self-weight	Water pressure, surcharges	Circular slip failure of the ground	Action–resistance ratio with respect to circular slip

Source: Modified from OCDI 2020

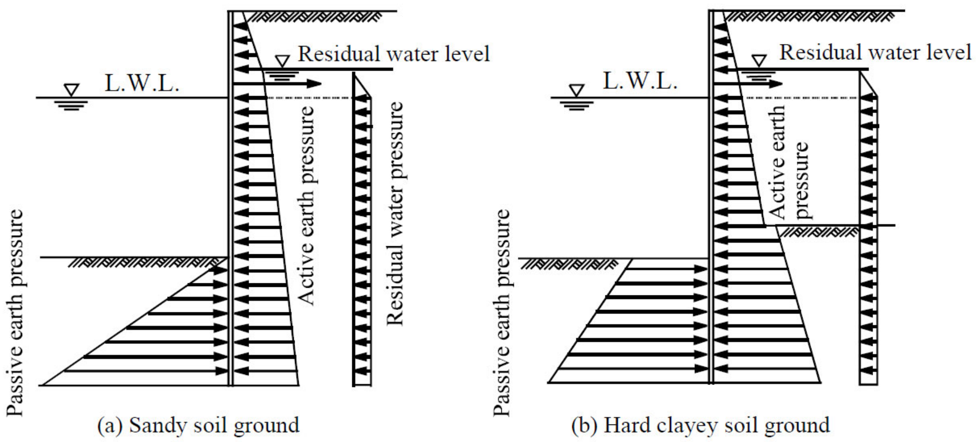
1-6. Performance Verification Method

(1) Model of Earth Pressure and Residual Water Pressure

The active earth pressure is normally used as the earth pressure that acts on the sheet pile wall from the backside. For the front-side reaction that acts on the embedded part of the sheet pile, it is necessary to use an appropriate value such as passive earth pressure or a subgrade reaction that corresponds to modulus of subgrade reaction.

When the free earth support method and the equivalent beam method described in this section are used in the performance verification for a sheet pile wall, the earth pressure and residual water pressure should be assumed to act as those shown in Figure 1.11.

TCVN 11820 Part 5: 2021, Hinh 9



Source: TCVN 11820-5-2021

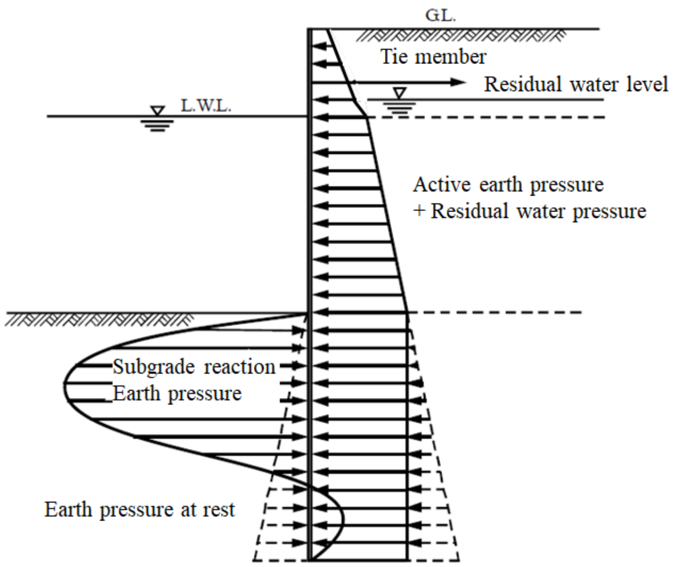
Figure 1.11- Earth Pressure and Residual Water Pressure to be Considered for the Performance Verification of Sheet Pile Quaywalls

Considering that the earth pressure changes in response to the displacement of the sheet pile wall, the actual earth pressure that acts on the sheet pile wall varies depending on the following:

- ✓ The construction method, i.e., whether backfill is executed or the ground in front of the sheet piles is dredged to the required depth after the sheet piles have been driven in
- ✓ The lateral displacement of the sheet pile at the tie member setting point
- ✓ The length of the embedded part of the sheet pile
- ✓ The relationship between the rigidity of the sheet pile and the characteristics of the sea-bottom ground.

When P.W. Rowe's method, i.e., the elastic beam analysis method, is used in a sheet pile stability calculation, it is assumed that the earth pressure and residual water pressure act as those shown in Figure 1.12, and a reaction earth pressure that corresponds to the modulus of subgrade reaction and the earth pressure at rest act on the front surface of the sheet pile.

Modified from TCVN 11820 Part 5: 2021, Hinh 13



Source: Modified from TCVN 11820-5-2021

Figure 1.12- Earth Pressure and Residual Water Pressure to be Considered for the Performance Verification of Sheet Pile Quaywalls Using P.W. Rowe's Method

(2) Performance Verification of Embedded Length of Sheet Pile Wall

1) Free Earth Support Method

When obtaining the embedded length of sheet piles by using the free earth support method, the analysis of the embedded length of the sheet pile wall can be performed using Equation (1.22) on the basis of the equilibrium of moments of the earth pressure and residual water pressure on the point of installation of the tie members (Figure. 1.13). In the following Equation, subscripts k and d indicate the characteristic value and the design value, respectively. Furthermore, the partial factors in the Equation can be selected from Table 1.10. 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.

OCDI 2020 Part III, Chapter 5 Equation (2.3.8)

$$m \cdot S_d / R_d \leq 1.0,\ R_d = \gamma_R \cdot R_k,\ S_d = \gamma_S \cdot S_k$$
$$R_k = a \cdot P_{pk} \qquad (1.22)$$
$$S_k = b \cdot P_{ak} + c \cdot P_{wk} + d \cdot P_{dwk}$$

Where:

R : resistance term(kN/m)

S : load term (kN/m)

P_p : resultant passive earth pressure acting on the sheet-pile wall (kN/m)

P_a : resultant active earth pressure acting on the sheet-pile wall (kN/m)

P_w : resultant residual water pressure acting on the wall structure (kN/m)

P_{dw} : resultant dynamic water pressure acting on the wall body (kN/m) (only during earthquake)

a to d : distance between the position of installation of the tie member and the point of action of resultant force (m)

γ_R : partial factor multiplied by resistance term

γ_S : partial factor multiplied by load term

m : adjustment factor

The soil layer compositions refer to the compositions of the soil layers from the ground surface to the lower end of embedment. If all soil layers are sandy ones, the partial factors for the sandy ground can be used. If cohesive soil is included even partially, the partial factors for the soil layer with the inclusion of cohesive soil can be used.

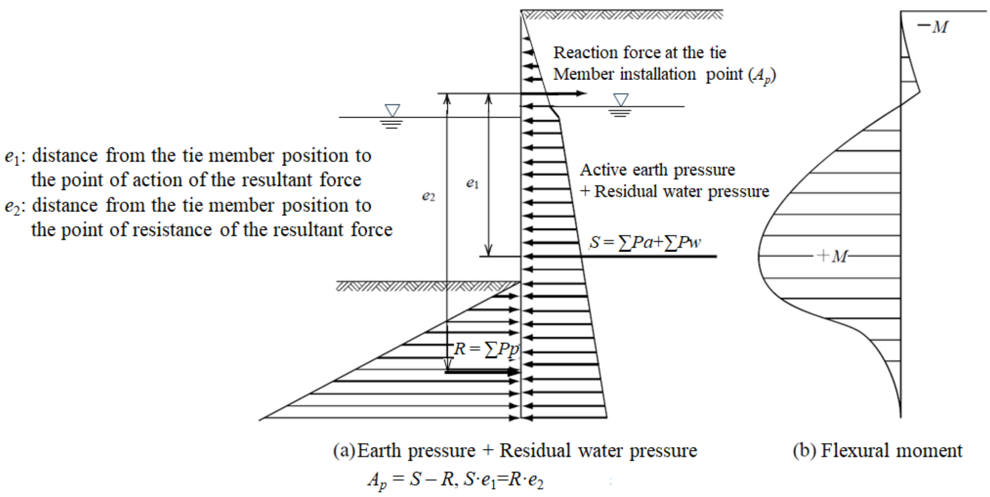
Table 1.10- Partial Factors Used for Performance Verification of Embedded Length of Sheet Pile Wall

TCVN 11820 Part 5: 2021, Bang 4

Verification object	Soil Layer Compositions	Partial factor γ_R	Partial factor γ_S	Adjustment factor m
Embedment length of sheet piles by Free Earth Support Method (Permanent State)	Sandy ground	0.72	1.09	- (1.00)
	Soil layer with the inclusion of cohesive soil	0.77	1.11	
Embedment length of sheet piles by Free Earth Support Method (Level 1 Earthquake motion)	All soil layer	- (1.00)	- (1.00)	1.20

Source: Modified from TCVN 11820-5-2021

Modified From TCVN 11820 Part 5: 2021, Hinh 11



Source: Modified from TCVN 11820-5-2021

Figure 1.13- Free Earth Support Method

2) P.W. Rowe's Method

The characteristic values of the embedded lengths of sheet pile walls using P.W. Rowe's method can be calculated to satisfy Equation (1.23). Considering that Equation (1.23) considers the rigidity of sheet piles without earth pressure, attention is required to the possibility that earth pressure reduction effects do not necessarily contribute to the reduction in the embedded lengths of sheet piles when planning ground improvement methods for alleviating earth pressure on existing steel sheet pile quaywalls. Therefore, when expecting the earth pressure reduction effects, it is advisable to use the methods to above in combination with P.W. Rowe's method.

$$\delta_N = \frac{D_F}{H_T} \geq 4.951\omega^{-0.2} - 0.2486$$

$$\delta_s = \frac{D_F}{H_T} \geq 5.0916\omega^{-0.2} - 0.2591 \quad (1.23)$$

TCVN 11820 Part 5: 2021, Equation (24)

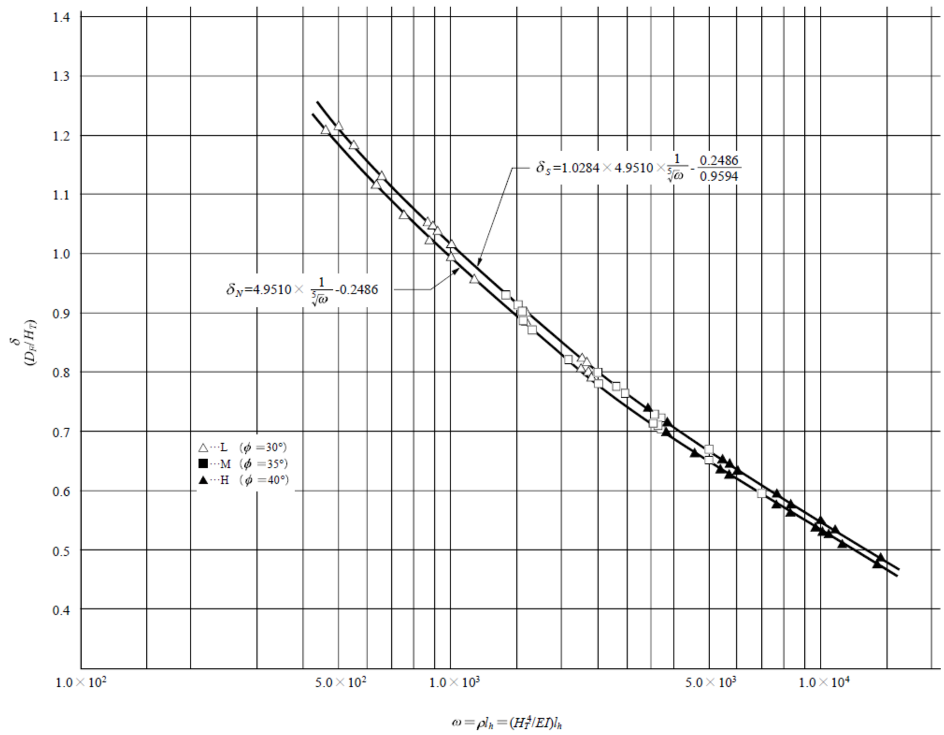
Where:

- δ_N : ratio of the embedded length of a sheet pile wall to the height from a tie member installation position to a seafloor surface (Permanent state)
- δ_s : ratio of the embedded length of a sheet pile wall to the height from a tie member installation position to a seafloor surface (Level 1 earthquake ground motion)
- D_F : embedded length of a sheet pile wall (m)
- H_T : height from a tie member installation position to a seafloor surface (m)
- ω : similarity number (pl_h)
- ρ : flexibility number (H_T^4/EI) (m^3/MN)
- E : Young's modulus of a sheet pile wall (MN/m^2)
- I : geometrical moment of the inertia per unit width of the cross section of the sheet pile (m^4/m)
- l_h : coefficient of subgrade reaction of a sheet pile wall (MN/m^3)

i) Similarity Number and Flexibility Number

Equation (1.23) formulates the relationship between the ratio of the convergent embedded length D_F to the virtual wall height H_T, i.e., $\delta = (D_F / H_T)$, and the similarity number ω as shown in Figure 1.14. This is based on the analysis performed by Takahashi and Kikuchi et al. by using a simulation model for 72 cases with a combination of conditions for the water depth of the quay (-4 to -14 m), soil conditions, seismic conditions (k_h = 0.20), and material conditions of steel sheet piles.

In Figure. 1.14, δ for permanent situations and earthquake motions are obtained as δ_N and δ_S, respectively; however, in Equation (1.23), δ_S is used for the action of earthquakes because it indicates large values.



Source: TCVN 11820-5-2021

Figure 1.14- Relationship between *δ* and *ω*

TCVN 11820 Part 5: 2021, Hinh 16

ii) Modulus of Subgrade Reaction of Sheet Piles

Little reference data provide the measured or suggested values of the modulus of the subgrade reaction of the sheet pile (l_h). Therefore, it is preferable to obtain these values by means of model tests and/or field measurements. The proposed values that have traditionally been used include the values proposed by Terzaghi (Table 1.11) and the ones proposed by Takahashi and Kikuchi et al. (Figure 1.15, 1.16), which have been obtained by modifying Terzaghi's values.

Table 1.11- Modulus of Subgrade Reaction for Sheet Pile Wall in Sandy Ground

OCDI 2020 Part III, Chapter 5 Table 2.3.1

Unit: MN/m^3

Relative density of sand	Loose	Medium	Dense
Modulus of subgrade reaction (l_h)	24	38	58

Source: OCDI 2020

They related the modulus of subgrade reaction with the angle of shearing resistance (Figure 1.16) by using one Equation from Dunham's Equations for calculating the smaller angle of shearing resistance for a given N value:

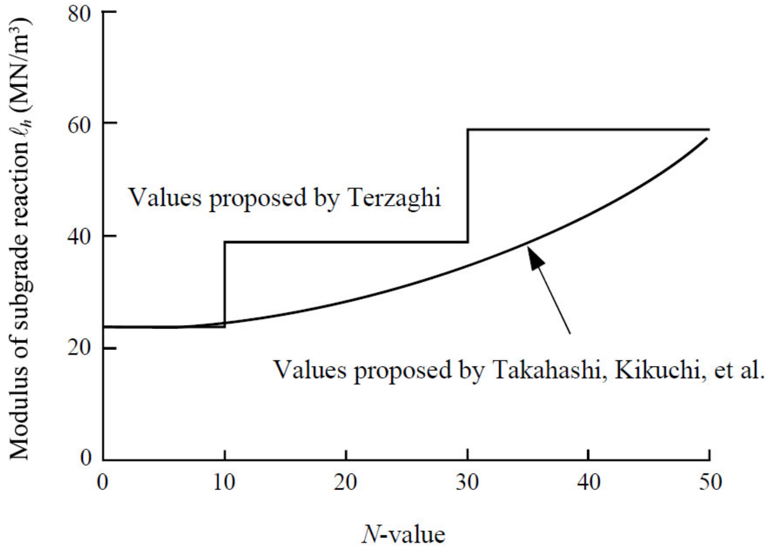
$$\varphi = \sqrt{12N}+15 \tag{1.24}$$

TCVN 11820 Part 5: 2021, Equation (23)

Where:

φ : angle of shearing resistance (°)

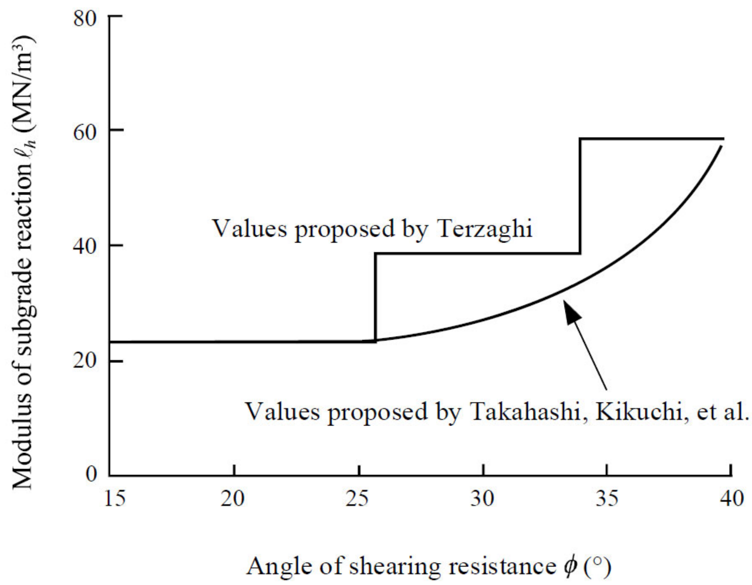
N : N value



TCVN 11820 Part 5: 2021, Hinh 14

Source: TCVN 11820-5-2021

Figure 1.15- Relationship between the Modulus of Subgrade Reaction (l_h) and the N value



TCVN 11820 Part 5: 2021, Hinh 15

Source: TCVN 11820-5-2021

Figure 1.16- Relationship between the Modulus of Subgrade Reaction (l_h) and Angle of Shear Resistance (φ)

(3) Performance Verification of Stress of Sheet Pile Wall

1) Characteristics Value of Maximum bending Moment

The characteristic values of maximum bending moment in the sheet pile wall and the reaction force at the tie member installation point can normally be calculated using the Equations (1.25) and (1.26). In the following Equation, subscript *k* indicates the characteristic value.

i) The reaction force at the tie member installation point

$$A_{pk}=P_{ak}+P_{wk}+P_{dwk}-\frac{(aP_{ak}+bP_{wk}+cP_{dwk})}{L} \qquad (1.25)$$

TCVN 11820 Part 5: 2021,

Where:

A_p : resistance force at the tie member installation point (kN/m)
P_a : resultant active earth pressure from the top of the sheet piling to the seabed surface (kN/m)
P_w : resultant residual water pressure from the top of the sheet piling to the seabed surface (kN/m)
P_{dw} : resultant dynamic water pressure acting on the sheet pile wall (kN/m) (only during earthquakes)
a to c : distance from the installation position of the tie member to the point of action of the resultant force (m)
L : distance from the installation position of the tie member to the seabed surface (m)

Equation (26)

ii) Maximum bending moment

$$M_{maxk}=aA_{pk}+bP'_{ak}- cP'_{wk}- dP'_{dwk} \qquad (1.26)$$

TCVN 11820 Part 5: 2021, Equation (25)

Where:

A_p : reaction at the tie member installation point (kN/m)
P'_a : resultant active earth pressure from the top of the sheet pile to the position where the shear force S becomes 0 (kN/m)
P'_w : resultant residual water pressure from the top of the sheet pile to the position where the shear force S becomes 0 (kN/m)
P'_{dw} : resultant dynamic water pressure from the top of the sheet pile to the position where the shear force S becomes 0 (kN/m) (during an earthquake only)
a : distance from the position where the shear force S becomes 0 to the tie member installation position (m)
b to d : distance from the position where the shear force S becomes 0 to the point of action of the resultant force (m)

The analysis of stresses in the sheet pile wall may be performed using Equation (1.26). In this Equation, subscripts k and d indicate the characteristic value and the design value, respectively. The partial factor in the Equation can be selected from the values listed in Table 1.12 in which the symbol "-" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot S_d/R_d \leq 1.0,\ R_d = \gamma_R \cdot R_{k,},\ S_d = \gamma_s \cdot S_k$$
$$R_k = \sigma_{yk} \qquad (1.27)$$
$$S_k = M_{max\,k}/Z$$

OCDI 2020 Part III, Chapter 5 Equation (2.3.12)

Where:

σ_y : bending yield stress of the steel material (N/mm^2)
M_{max} : maximum bending moment in the sheet-pile wall (N·mm/m)
Z : section modulus of steel material (mm^3/m)
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

Table 1.12- Partial Factors for Performance Verification of Stress of Sheet Pile

Verification object	Partial factor γ_R	Partial factor γ_S	Adjustment factor m
Stress in the sheet pile wall (Permanent State)	0.84	1.18	- (1.00)
Stress in the sheet pile wall (Variable state with respect to Level 1 earthquake ground motions)	- (1.00)	- (1.00)	1.12

Source: OCDI 2020

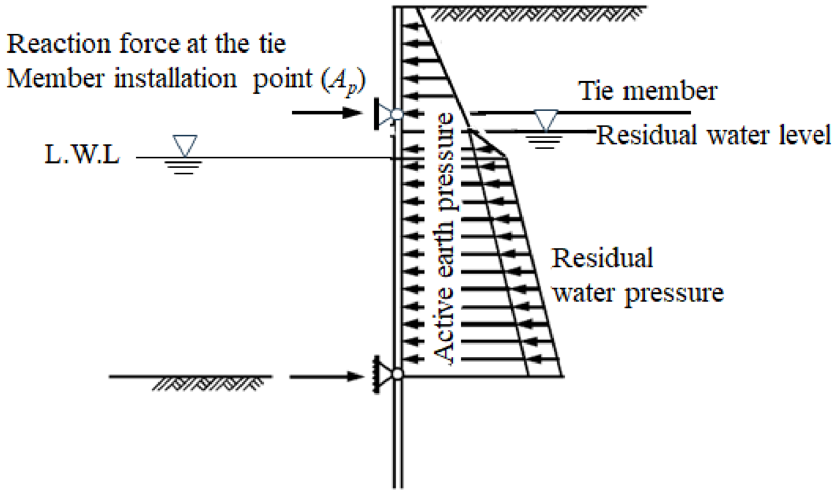
OCDI 2020 Part III, Chapter 5 Table 2.3.3

The maximum bending moment and reaction force at the tie member installation points on sheet piles may be determined using the equivalent beam method or P.W. Rowe's method. However, care should be exercised when using the equivalent beam method for sheet piles with high rigidity because the method causes the point of contraflexure of the bending moment to be deeper than a seafloor surface and may underestimate the sectional force in the sheet piles.

2) Equivalent Beam Method

The equivalent beam method calculates the maximum bending moment and reaction force at the tie member installation point of the sheet piles by assuming a simple beam supported at the tie member installation point and the sea bottom, with the earth pressure and residual water pressure acting as the load above the sea bottom as shown in Figure 1.17.

Modified from TCVN 11820 Part 5: 2021, Hinh 12

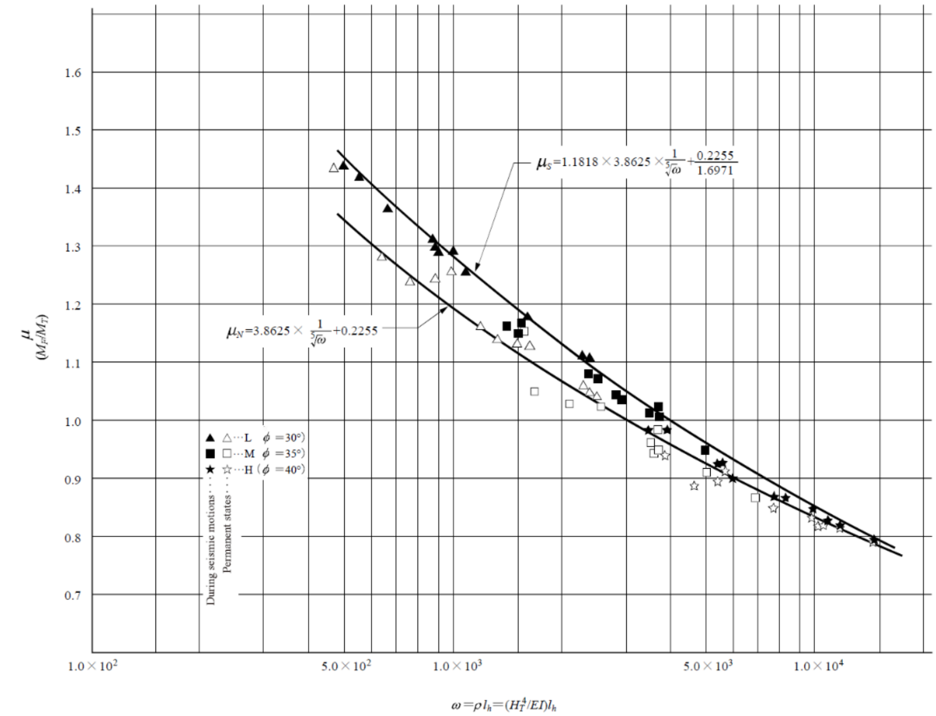


Source: Modified from TCVN 11820-5-2021

Figure 1.17- Equivalent Beam Method for Obtaining Bending Moment

3) P.W. Rowe's Method

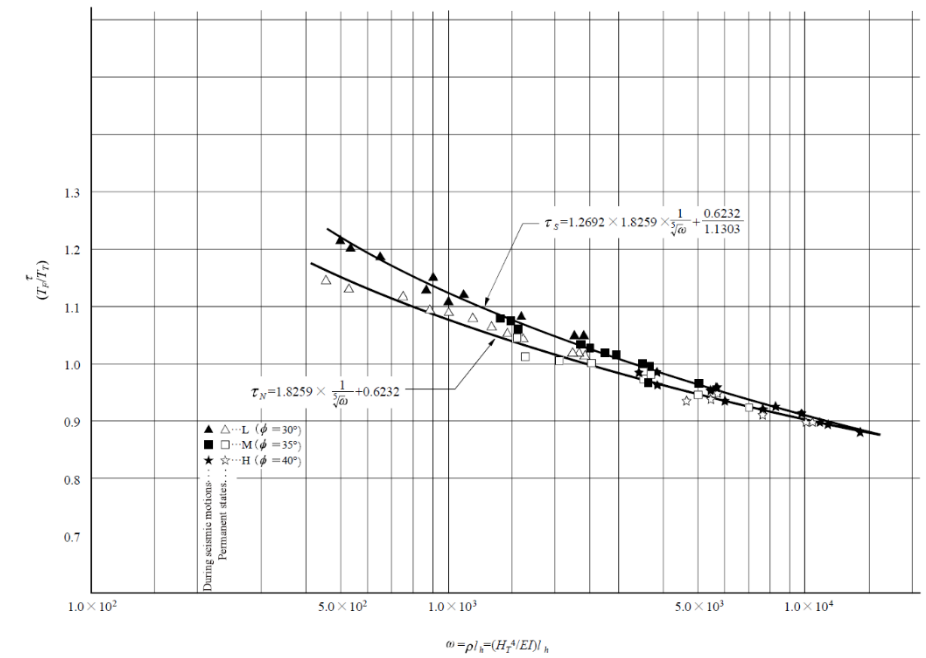
In the analysis by Takahashi and Kikuchi et al. the relationship between the similarity number ω, ratio μ (M_F/M_T), and ratio τ (T_F/T_T) were studied. The ratio μ is the ratio of the maximum bending moment M_F when there is convergent embedded length D_F in the bending curve analysis to the maximum bending moment M_T calculated by the equivalent beam method by assuming the tie installation point and the seabed surface as the support points. The ratio is the ratio τ of tie tension force T_F when there is convergent embedded length D_F in the bending curve analysis to the tie tension force T_T calculated from the equivalent beam method. These relationships are shown in Figure 1.18 to 1.19.



TCVN 11820 Part 5: 2021, Hinh 17

Source: TCVN 11820-5-2021

Figure 1.18- Relationship between μ and ω



TCVN 11820 Part 5: 2021, Hinh 18

Source: TCVN 11820-5-2021

Figure 1.19- Relationship between τ and ω

(4) Performance Verification of Stress on Tie Members

The analysis of stresses in tie members may be performed using Equation (1.28). In this Equation, subscripts k and d indicate the characteristic value and the design value, respectively. The partial factor in the Equation can be selected from the values listed in Table 1.13 in which the symbol "-" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot S_d / R_d \leq 1.0,\ R_d = \gamma_R \cdot R_k,\ S_d = \gamma_s \cdot S_k$$
$$R_k = \sigma_{yk} \tag{1.28}$$
$$S_k = T_k / A$$

OCDI 2020 Part III, Chapter 5 Equation (2.3.13)

Where:

σ_y : tensile yield stress of a tie member (N/mm^2)
T : tension force on a tie member (N)
A : cross section area of a tie member (mm^2)
R : resistance term (N/mm^2)
S : the load term (N/mm^2)
γ_R : partial factor multiplied by resistance term
γ_s : partial factor multiplied by load term
m : adjustment factor

Table 1.13- Partial Factors for Stress Verification of Tie Members

OCDI 2020 Part III, Chapter 5 Table 2.3.4

Verification object	Partial factor γ_R	Partial factor γ_S	Adjustment factor m
Stress in the tie member (Permanent State)	0.64	1.29	- (1.00)
Stress in the tie member (Variable state with respect to Level 1 earthquake motions)	- (1.00)	- (1.00)	1.67

Source: OCDI 2020

1) Tension Force acting on Tie Members

The tension force that acts on a tie member is generally calculated by Equation (1.29). In the Equation below, subscript k stands for the characteristic value.

$$T_k = A_{pk}\, l \sec\theta \tag{1.29}$$

TCVN 11820 Part 5: 2021, Equation (27)

Where:

T : tension force of the tie member (kN)
A_p : reaction at the tie member installation point (kN)
l : tie member installation interval (m)
θ : inclination angle of the tie member to the line perpendicular to the sheet pile wall (°)

2) Tension Force acting on Tie Members

In some cases, bollards are installed on the coping of a sheet pile wall, and the tractive forces of ships acting on the bollards are transmitted to the tie members. Usually, the coping is assumed to be a beam with the tie members as elastic supports, and the tie member tension force may be calculated using Equation (1.30) by assuming that the tractive force is evenly shared by four tie members near a bollard. In the Equation below, subscript k stands for the characteristic value.

$$T_k = (A_{pk}\, l + P_k / 4\ ) \sec\theta \tag{1.30}$$

TCVN 11820 Part 5: 2021, Equation (28)

Where:

T : tension force acting in the tie member (kN)
A_p : reaction force at the installation point of the tie member (kN)
l : spacing of installation of tie member (m)

θ : inclination angle of the tie member in perpendicular to the sheet pile wall and the tie member (°)

P : horizontal component of the tractive force of a ship acting on a bollard (kN)

(5) Performance Verification of Stress in Waling

The analysis of stress in waling may be performed using Equation (1.31). In this Equation, subscripts *k* and *d* indicate the characteristic value and the design value, respectively. The partial factor in the Equation can be selected from the values listed in Table 1.14 in which the symbol "-" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

OCDI 2020 Part III, Chapter 5 Equation (2.3.16)

$$m\cdot S_d/R_d \leq 1.0,\ R_d = \gamma_R \cdot R_k,\ S_d = \gamma_S \cdot S_k$$
$$R_k = \sigma_{yk} \qquad (1.31)$$
$$S_k = M_{max\,k}/Z$$

Where:

σ_y : bending yield stress in the waling (N/mm^2)

M_{max} : maximum bending moment in the waling (N·mm/m)

Z : section modulus of the waling (mm^3)

R : resistance term (N/mm^2)

S : the load term (N/mm^2)

γ_R : partial factor multiplied by resistance term

γ_S : partial factor multiplied by load term

m : adjustment factor

Table 1.14- Partial Factors for the Stress Verification of Waling

OCDI 2020 Part III, Chapter 5 Table 2.3.6

Verification object	Partial factor γ_R	Partial factor γ_S	Adjustment factor m
Stress in waling (Permanent State)	- (1.00)	- (1.00)	1.67
Stress in waling (Variable state with respect to Level 1 earthquake motions)	- (1.00)	- (1.00)	1.12

Source: OCDI 2020

In general, the maximum bending moment of the waling may be calculated using Equation (1.32). In the Equation below, subscript k stands for the characteristic value.

$$M_{max} = T_k\, l/10 \qquad (1.32)$$

TCVN 11820 Part 5: 2021, Equation (29)

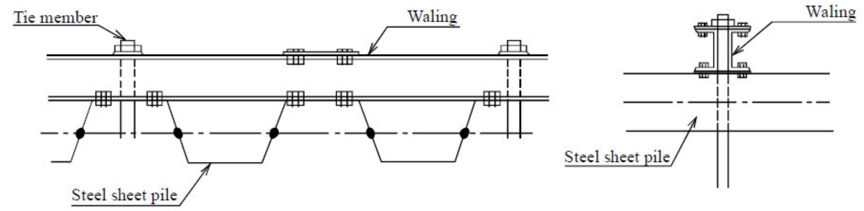
Where:

M_{max} : maximum bending moment in the waling (N·mm/m)

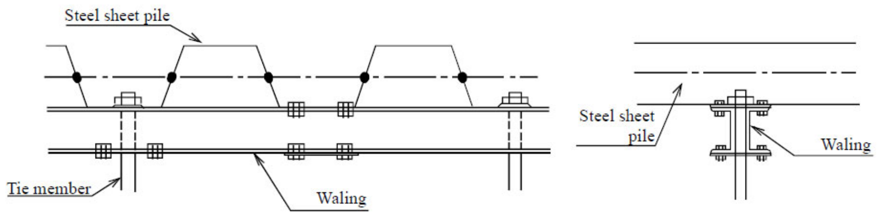
T : tension force of a tie member (kN)

l : tie member installation interval (m)

Sheet piles and tie members need to be connected via waling materials horizontally installed on the upper portions of the sheet piles. The waling materials are generally fabricated by assembling channel steel (Figure 1.24) in general, but angle steel or H-section steel can also be used in place of channel steel.



(a) Case of the installation at the seaside of a sheet pile



(b) Case of the installation at the landside of a sheet pile

Source: TCVN 11820-5-2021

TCVN 11820 Part 5: 2021, Hinh 19

Figure 1.24- Examples of Waling Installation

(6) Performance Verification of Anchorage

For the following performance verification of the vertical pile anchorages, refer to TCVN 11820 Part 4-1: 2020.

The analysis of stresses in vertical pile anchorages may be performed using Equation (1.33). In this Equation, subscripts *k* and *d* indicate the characteristic value and the design value, respectively. The partial factor in the Equation can be selected from the values listed in Table 1.15 in which the symbol "-" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot S_d / R_d \leq 1.0,\ R_d = \gamma_R \cdot R_k,\ S_d = \gamma_s \cdot S_k$$
$$R_k = \sigma_{yk} \qquad (1.33)$$
$$S_k = M_{max\,k} / Z$$

OCDI 2020 Part III, Chapter 5 Equation (2.3.18)

Where:

- σ_y : bending yield stress of a pile anchorage (N/mm^2)
- M_{max} : maximum bending moment in a pile anchorage (N·mm/m)
- Z : section modulus of a pile anchorage (mm^3/m)
- R : resistance term (N/mm^2)
- S : the load term (N/mm^2)
- γ_R : partial factor multiplied by resistance term
- γ_s : partial factor multiplied by load term
- m : adjustment factor

Table 1.15- Partial Factors for Assessing the Stresses in the Anchor Piles

OCDI 2020 Part III, Chapter 5 Table 2.3.7

Verification object	Partial factor γ_R	Partial factor γ_S	Adjustment factor m
Stress in vertical pile anchorage (Permanent State)	- (1.00)	- (1.00)	1.67
Stress in vertical pile anchorage (Variable state with respect to level 1 earthquake motions)	- (1.00)	- (1.00)	1.12

Source: OCDI 2020

1) Subgrade Reaction

The basic Equation that represents the behavior of a beam on elastic foundation is expressed by Equation (1.34).

$$EI\frac{d^4y}{dx^4} = -P = -pB \tag{1.34}$$

TCVN 11820 Part 4-1: 2020, Equation (C.27)

Where:

EI	:	bending stiffness of a pile (kN/m^2)
x	:	depth below the ground level (m)
y	:	displacement of a pile at depth x (m)
P	:	subgrade reaction per unit length of a pile at depth x (kN/m)
p	:	subgrade reaction per unit area of a pile at depth x (kN/m^2), $p=P/B$
B	:	width of a pile (m)

How to express the subgrade reaction in Equation (1.34) has been largely discussed. Typical ways include Chang's method and the Port and Harbour Research Institute (PHRI) method. Chang's method is simple to use as an analysis method, whereas the PHRI method is said to be able to express the behavior of a pile more accurately. Therefore, analysis by the PHRI method should be generally used. If the PHRI method is difficult to apply in cases such as an analysis coupling the pile foundation and superstructures, Chang's method may be used.

2) Displacement of Pile Head and Maximum Bending Moment

The PHRI method classifies ground into the S-type ground and the C-type ground. S-type ground is the ground the SPT-N value of which increases linearly with the depth such as sandy ground of uniform density and clayey ground in normal consolidation condition. C-type ground is the ground the SPT-N value of which is constant regardless of depth such as sandy ground with compacted surface and clayey ground subjected to large pre-consolidation.

For piles under relatively simple conditions in which only forces perpendicular to the axis act on the pile head, the displacement of the pile head, maximum bending moment below the ground surface, and deflection are calculated based on the PHRI method. Several values for a ground level loading model pile the head of which coincides with the ground level (pile protrusion length = 0) can be calculated by Equation (1.35) and (1.36).

- S-type ground, free head pile

$$\begin{aligned}
\log y_0 &= 0.38958 - \frac{4}{7}\log EI - \frac{6}{7}\log Bk_s + \frac{10}{7}\log T \\
\log M_{max} &= -0.05825 + \frac{1}{7}\log EI - \frac{2}{7}\log Bk_s + \frac{8}{7}\log T \\
\log i_0 &= 0.22539 - \frac{5}{7}\log EI - \frac{4}{7}\log Bk_s + \frac{9}{7}\log T \\
\log l_{m1} &= 0.53473 + \frac{1}{7}\log EI - \frac{2}{7}\log Bk_s + \frac{1}{7}\log T
\end{aligned} \tag{1.35}$$

TCVN 11820 Part 1: 2025, Equation (I.67)

- C-type ground, free head pile

$$\log y_0 = 0.11328 - \frac{2}{5}\log EI - \frac{6}{5}\log Bk_s + \frac{8}{5}\log T$$
$$\log M_{max} = -0.28846 + \frac{1}{5}\log EI - \frac{2}{5}\log Bk_s + \frac{6}{5}\log T$$
$$\log i_0 = -0.00634 - \frac{3}{5}\log EI - \frac{4}{5}\log Bk_s + \frac{7}{5}\log T \qquad (1.36)$$
$$\log l_{m1} = 0.55205 + \frac{1}{5}\log EI - \frac{2}{5}\log Bk_s + \frac{1}{5}\log T$$

TCVN 11820 Part 1: 2025, Equation (I.69)

Where:

y_0	:	displacement of a pile on the ground level (m)
M_0	:	bending moment caused in the pile body on the ground level (kN·m)
i_0	:	angle of deflection of a pile on the ground level (rad)
M_{max}	:	maximum bending moment below the ground level (kN·m)
l_{m1}	:	depth of bending moment first zero point of a free head pile or depth of bending moment second zero point of a fixed head pile (m)
EI	:	bending stiffness of a pile (kN·m^2)
B	:	width of a pile (m)
k_s	:	lateral resistance coefficient in S-type ground (kN/m$^{3.5}$)
k_c	:	lateral resistance coefficient in C-type ground (kN/m$^{2.5}$)
T	:	force in the direction perpendicular to the axis acting on the pile head (kN)

3) Relation between Lateral Resistance Coefficient and Increment of *N*-value

i) S-type ground

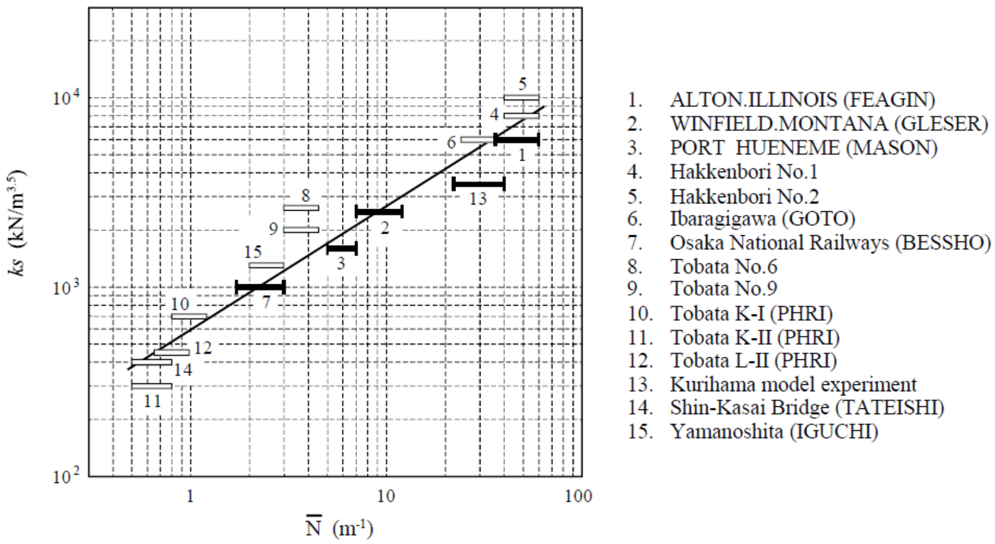
Relation between the lateral resistance coefficient in the S-type ground and the increment of SPT-*N* values per unit depth as in Figure 1.25. Here, the increment of SPT-*N* values per unit depth means the inclination of a line approximating the distribution of SPT-*N* values in depth direction obtained from a ground exploration. The increment of SPT-*N* values in the range from the ground level to (0.5 to 1.0) l_{m1} which greatly influences the lateral resistance of piles is generally used. Even when the distribution of SPT-*N* values in depth direction is not 0 on the ground level, the inclination approximating with a line passing 0 on the ground level may be used.

$$k_s = 592\,\bar{N}^{0.654} \qquad (1.37)$$

TCVN 11820 Part 1: 2025, Equation (I.71)

Where:

$\bar{N}$	:	increment of SPT-N value per unit depth (m^{-1})



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

Figure 1.25- Relation between lateral resistance coefficient in S-type ground and increment of SPT-N values

TCVN 11820 Part 4-1: 2020, Hinh C.15

TCVN 11820 Part 1: 2025, Hinh I.17

ii) C-type ground

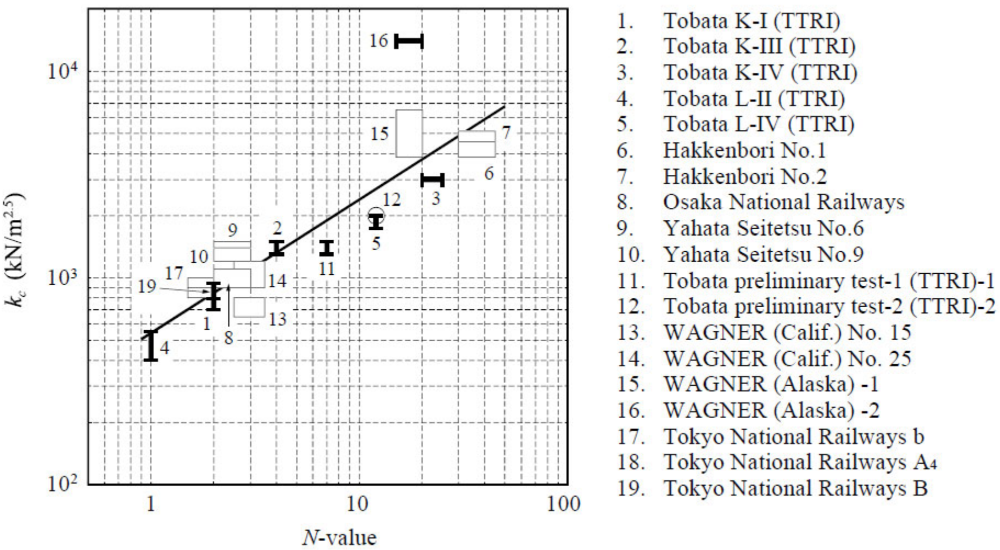
Relation between the lateral resistance coefficient in the C-type ground and the increment of SPT-*N* values per unit depth as in Figure 1.26. Mean SPT-*N* values in the range from the ground level to (0.5 to 1.0) l_{m1} which greatly influences the lateral resistance of piles is generally used for SPT-*N* values.

$$k_c=540\ N^{0.648} \tag{1.38}$$

TCVN 11820 Part 1: 2025, Equation (I.72)

Where:

N : mean SPT-N value in a dominant range of lateral resistance of piles



TCVN 11820 Part 4-1: 2020, Hinh C.16

TCVN 11820 Part 1: 2025, Hinh I.18

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

Figure 1.26- Relation between lateral resistance coefficient in C-type ground and SPT-N values

4) Other types of Anchorage

The verification methods for other types of anchorage, such as coupled-pile anchorage and sheet pile anchorage, are referenced in OCDI 2020.

In Vietnam, the concrete slab type is often used for anchorage, and its verification can be referred to the followings:

The height and placing depth of slab anchorage may be determined to satisfy Equation (1.39) on the assumption that the tie member tension force and the active earth pressure behind the slab anchorage are resisted by the passive earth pressure in front of the slab anchorage as shown in Figure 1.27. In this equation, subscripts *k* and *d* indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in Table 1.16. The value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot S_d / R_d \leq 1.0,\ R_d = \gamma_R \cdot R_{k},\ S_d = \gamma_S \cdot S_k$$
$$R_k = E_{Pk} \qquad (1.39)$$
$$S_k = (A_{Pk} + E_{Ak})$$

TCVN 11820 Part 5: 2021, Equation (37)

Where:

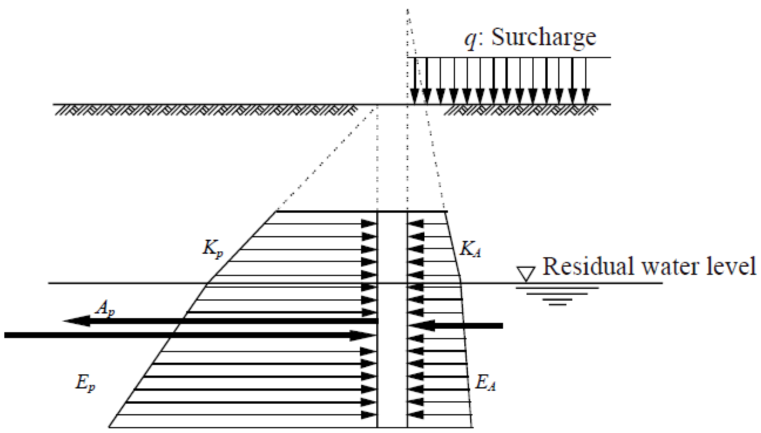
- E_P : passive earth pressure acting on slab anchorage (kN/m)
- A_P : reaction at the tie member installation point under a permanent state and variable state with respect to Level 1 earthquake ground motion and the tractive forces of ships (kN/m);
- E_A : active earth pressure acting on slab anchorage (kN/m)

Table 1.16- Partial Factors for the Stability Verification of Slab Anchorages

OCDI 2020 Part III, Chapter 5 Table 2.3.10

Verification object	Partial factor γ_R	Partial factor γ_S	Adjustment factor *m*
Stability of slab anchorage (Permanent State)	- (1.00)	- (1.00)	2.50
Stability of slab anchorage (Variable state with respect to Level 1 earthquake motions)	- (1.00)	- (1.00)	2.00

Source: OCDI 2020



Source: TCVN 11820-5-2021

Figure 1.27- Force Acting on Slab Anchorage

TCVN 11820 Part 5: 2021, Hinh 22

The wall surface friction angle used in calculating the earth pressure is normally assumed to be 15° in the case of active earth pressure and 0° in the case of passive earth pressure. However, in the case of a dead man anchor, an upward acting tension force acts on the slab anchorage; therefore, the wall surface friction force acts upwards, which is the opposite of the normal case of passive earth pressure. Furthermore, the passive earth pressure will be reduced. In this case the wall surface friction angle is normally assumed to be 15°.

When the active failure plane of the sheet pile and the passive failure plane of the slab anchorage drawn in accordance with 1-3. Design Conditions (1) Setting of Design Conditions 4) Installation Locations of Anchorages intersect below the ground surface level, it is preferable to consider the fact that the passive earth pressure acting on the vertical surface above the intersection point does not function as a resistance force (Figure 1.28); it should be subtracted from the design value of E_P of Equation (1.39). When the intersection point is located above the residual water level, the passive earth pressure to be subtracted may be calculated using Equation (1.40). In the following equation, subscript k indicates the characteristic value.

$$\Delta E_{Pk} = K_{Pk} \cdot w_k \cdot h_f^2 / 2 \qquad (1.40)$$

TCVN 11820 Part 5: 2021, Equation (32)

Where:

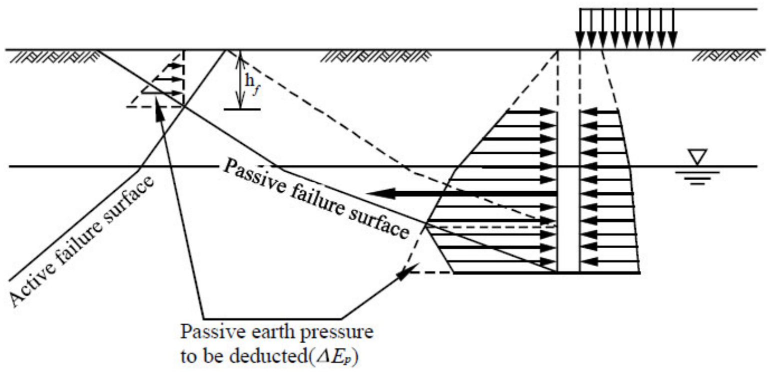
w : weight of soil (kN/m^2)

h_f : depth from the ground surface to the intersection of the failure planes (m)

K_P : coefficient of passive earth pressure

The characteristic value w_k for the weight of soil is expressed as the product of the characteristic value for the unit weight of the soil layer under review and the depth hf from the ground surface to the intersection of the failure planes.

TCVN 11820 Part 5: 2021, Hinh 23



Source: TCVN 11820-5-2021

Figure 1.28- Earth Pressure to be Subtracted from the Passive Earth Pressure that Acts on Anchorage Wall when the Active Failure Plane of the Sheet Pile Wall and the Passive Failure Plane of the Slab Anchorage Intersect

When a soft cohesive soil layer exists below the area around the bottom of a slab anchorage, there is a risk that the slab anchorage does not have sufficient resistance owing to the generation of a slip surface below the lower edge of the slab anchorage. Therefore, in such a case, it is advisable to examine the stability of a slab anchorage by assuming circular or linear slip surfaces in general.

When examining circular slips, it is generally considered that a slab anchorage is unstable if the slab anchorage is positioned within a slip circle drawn on the basis of an action–resistance ratio of 1.0 or more without considering a sheet pile wall.

When examining linear slips, a slab anchorage can be determined to be stable if an action–resistance ratio with respect to the tension force of a tie member is 1.5 or lower by taking into consideration the resistance to slip of a soil mass obtainable by balancing the force acting on the soil mass defined by a slip plane passing through the lower edge of the slab anchorage, a vertical plane passing through the slab anchorage, and an active failure plane of a sheet pile wall.

A slab anchorage should have stability against the bending moment caused by the earth pressure and the tension force of tie members. In general, the maximum bending moment may be calculated by Equation (1.41) on the assumption that the earth pressure is approximated to an equally distributed load, and the slab anchorage is a continuous slab in the horizontal direction and a cantilever slab fixed at the tie member installation point in the vertical direction. In the following equation, the subscript *k* indicates the characteristic value

$$M_{Hk} = T_k \cdot l / 12$$

$$M_{Vk} = T_k \cdot h / (8 \cdot l) \qquad (1.41)$$

TCVN 11820 Part 5: 2021, Equation (33)(34)

Where:

M_H : horizontal maximum bending moment (kN·m)

M_V : vertical maximum bending moment per meter in length (kN·m /m)

T : tie member tension force (kN)

l : tie member interval (m)

h : height of slab anchorage (m)

The layout of the reinforcing bars for M_H may be determined on the assumption that the effective width of the slab anchorage is 2*b* with the tie member installation point as the center, where *b* is the thickness of the slab anchorage at the tie member installation point. Slab anchorages are constructed from reinforced concrete or prestressed concrete. For the performance verification of reinforced concrete and prestressed concrete slab anchorages. In many cases, the installation position of a tie member on a slab anchorage is the point of resultant earth pressure or the center of the heights of slab anchorages.

(7) Performance Verification for Overall Stability in Permanent State

In the permanent state, where the primary force is the self-weight of the quaywall structure and backfilling, performance verification for overall stability typically involves verifying for circular slip failure. The verification of circular slip failure can be carried out using the following Equation (Equation 1.42).

The modified Fellenius method assumes that the direction of the resultant force acting on vertical planes between slice segments is parallel to the base of the slice segments. This method is also referred to as the simplified method or Tschebotarioff method. When a circular arc and a slice segment are as shown in Figure 1.30, according to the modified Fellenius method is applicable.

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 1.17. For parts where a "-" is indicated in Table 1.17, values in parentheses can be used for convenience when performing the verification.

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

$$S_k=\Sigma\left\{\left(W_k+q_k\right)\sin\theta+\frac{1}{R}aP_{Hk}\right\} \tag{1.42}$$

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

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

Where:

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

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

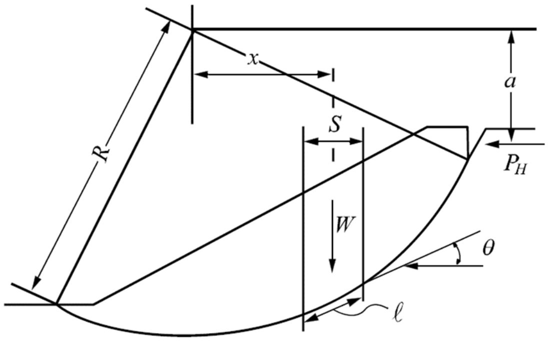
Modified from TCVN 11820 Part 6: 2023, Equation (7)

Table 1.17- Partial Factors for Performance Verification of Circular Slip Failure

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

TCVN 11820 Part 6: 2023, Bang 6

Source: TCVN 11820-6-2023



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

Figure 1.30- Circular Slip Failure Analysis Using Modified Fellenius Method

TCVN 11820 Part 1: 2025 Hinh I.37

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

(8) Reference on Allowable Horizontal Displacement

There are no specific requirements regarding allowable horizontal displacement in TCVN.

While OCDI 2020 provides allowable horizontal displacements during earthquakes, it adopts a different approach from TCVN by applying seismic coefficients derived from time history response analysis. Therefore, the allowable horizontal displacements during earthquakes as presented in PIANC WG34 are introduced in Table 1.18 to 1.20.

Table 1.18- Allowable Horizontal Displacement at Earthquake Ground Motion

Modified from PIANC Guideline WG34 Table 4.2

Extent of damage	Degree I	Degree II	Degree III	Degree IV
Normalized residual horizontal displacement (*d*/*H*)*	Less than 1.5%	N/A	N/A	N/A
Residual tilting towards the sea	Less than 3 degrees	N/A	N/A	N/A

* d : residual horizontal displacement at the top of the wall
H : height of sheet pile wall from mudline

Table 1.19- Acceptable Extent of Damage in Performance-based Design

PIANC Guideline WG34 Table 3.1

Acceptable extent of damage	Structural	Operational
Degree I: Serviceable	Minor or no damage	Little or no loss of serviceability
Degree II: Repairable	Controlled damage	Short-term loss of serviceability
Degree III: Near collapse	Extensive damage in near collapse	Long-term or complete loss of serviceability
Degree IV: Collapse	Complete loss of structure	Complete loss of serviceability

Table 1.20- Performance Grades

PIANC Guideline WG34 Table 3.2

Performance grade	Design earthquake	
	Level 1 (L1)	Level 2 (L2)
Grade S	Degree I: Serviceable	Degree I: Serviceable
Grade A	Degree I: Serviceable	Degree II: Repairable
Grade B	Degree I: Serviceable	Degree III: Near collapse
Grade C	Degree II: Repairable	Degree IV: Collapse

Source: PIANC Guideline WG34

2. Design Example

2-1. Typical Section for Design Example

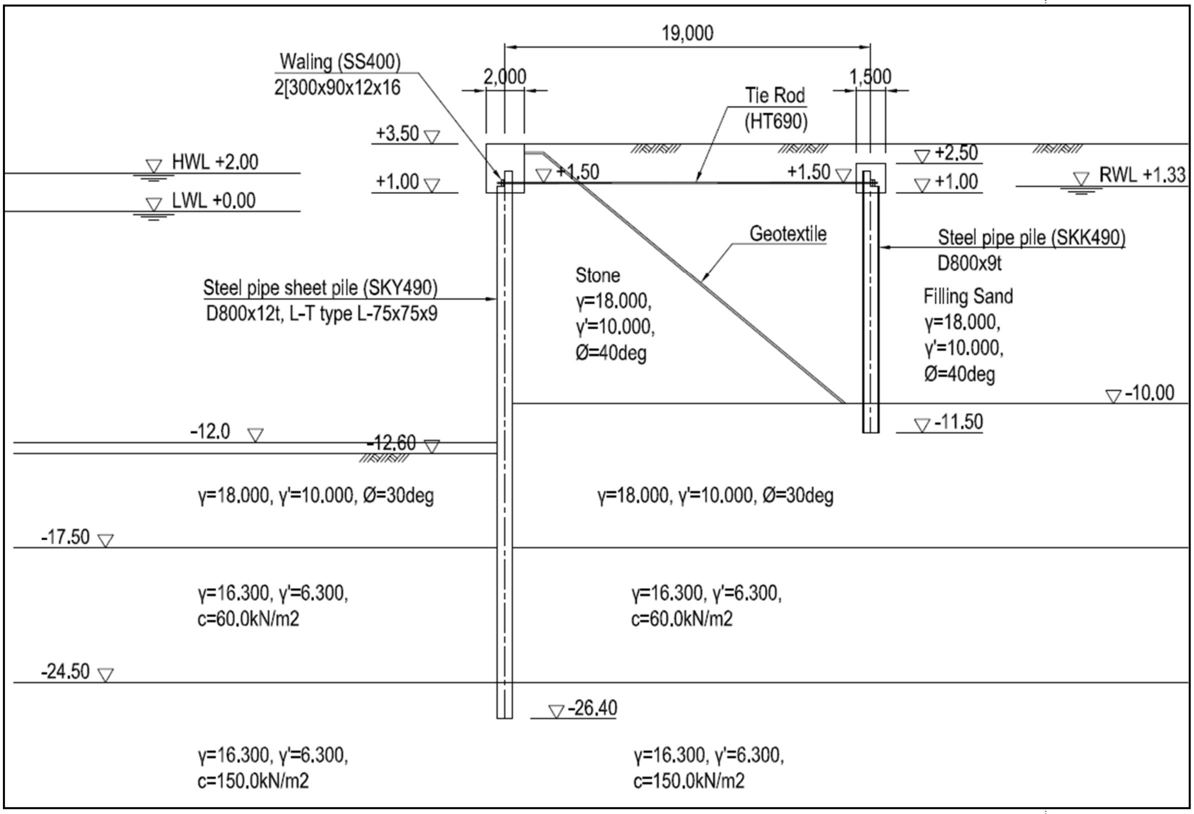


Figure 2.1- Typical Section for Design Example

2-2. Design Conditions

(1) Specifications of Quaywall

- ✓ Crown height of quaywall: +3.50 m
- ✓ Top level of sheet pile: +2.10 m
- ✓ Tie rod installation height: +1.50 m
- ✓ Planned water depth: –12.00 m
- ✓ Design water depth for verification: –12.60 m
- ✓ Seabed slope in front of the quaywall: 0.0°

The design water depth for verification, described above, shall be -12.60 m relative to the planned depth of -12.0 m by considering the tolerance of dredging (extra-dredging).

(2) Natural Condition

1) Tide level

- 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

Figure 2.1 presents the soil conditions

3) Unit weight of seawater

w_w= 10.1 kN/m^3

(3) Use Conditions

1) Target vessel

30,000 DWT

2) Surcharge load (characteristic value)

Permanent state: 30.0 kN/m^2
Variable state: 15.0 kN/m^2

3) Mooring force (characteristic value)

Mooring force (characteristic value): 700.0 kN/post
Service height: +3.87 m

4) Design service life

50 years

5) Allowable displacement

The allowable displacement tolerance is determined for this verification example as follows while considering various relevant factors, including the conditions at the location where the facility is to be constructed, and the functions required of the facility:

Variable state related to Level 1 earthquake ground motion: Assumed 15 cm or less

(4) Seismic Coefficient

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 coefficient is assumed in this design example.

(5) Corrosion Allowance for Steels (treated with corrosion protection)

Service life: 50 years
Corrosion rate μ: 90%

1) Steel-pipe sheet pile

Corrosion speed: 1 - μ
t_1 = 0.10 mm/year × 0.1 × 50 years = 0.50 mm (seaside)
t_2 = 0.02 mm/year × 50 years = 1.00 mm (landside)

2) Tie rod

t = 2 × 0.03 mm/year × 50 years = 3.00 mm

3) Waling (to be provided in the superstructure)

t = 0.00 mm

4) Anchorage pile

t = 0.02 mm/year × 50 years = 1.00 mm

(6) Characteristic Values of Steel Yield Stress

Table 2.1 presents the characteristic values for steel yield stress.

Table 2.1- Characteristic Values of Steel Yield Stress

Item	Material	Unit	Yield stress
Steel-pipe sheet pile	SPSP490	N/mm^2	315.0
General steel (waling)	SS400	N/mm^2	235.0
Anchor pile	SPP490	N/mm^2	315.0

2-3. Outline of Performance Verification

In this verification example, performance verification of a sheet pile quaywall is conducted for design states of permanent state and variable state due to earthquake ground motion depending on the service condition of the quaywall and the related action conditions.

Verification items for the permanent and variable states are shown below:

- ✓ Verification of embedded length of sheet piles for the front wall
- ✓ Verification of stress of sheet piles for the front wall
- ✓ Verification of stress of tie members
- ✓ Verification of stress of waling
- ✓ Verification of stress, embedded length, and installation position of anchorage structure
- ✓ Verification of circular slip failure for the front wall (for permanent state only)

2-4. Performance Verification of Sheet Pile Quaywalls in the Permanent State

(1) Performance Verification of the Embedded Length of Sheet pile

1) Earth Pressure and Residual Water Pressure

Tables 2.2 and 2.3 show the calculation results for active earth pressure, residual water pressure and passive earth pressure in the permanent situation.

Table 2.2- Active Earth Pressure (Permanent State)

(β= 0.0° δ= 15.0°)

Layer (m)	h (m)	φ (°)	c (kN/m²)	γ (kN/m³)	γh (kN/m²)	$\Sigma\gamma h$ (kN/m²)	w (kN/m²)	ζ (°)	$K_a\cos\delta$	P_1 (kN/m²)	P_2 (kN/m²)	Pa (kN/m²)
3.5			——			0	30	63.2	0.1942	——	——	5.826
	2.0	40		18	36							
1.5			——			36.00	30	63.2	0.1942	——	——	12.817
1.5			——			36.00	30	63.2	0.1942	——	——	12.817
	0.17	40		18	3.06							
1.33			——			39.06	30	63.2	0.1942	——	——	13.411
1.33			——			39.06	30	63.2	0.1942	——	——	13.411
	11.33	40		10	113.3							
-10			——			152.36	30	63.2	0.1942	——	——	35.414
-10			——			152.36	30	56.9	0.2911	——	——	53.085
	2.6	30		10	26							
-12.6			——			178.36	30	56.9	0.2911	——	——	60.654
-12.6			——			178.36	30	56.9	0.2911	——	——	60.654
	4.9	30		10	49							
-17.5			——			227.36	30	56.9	0.2911	——	——	74.917
-17.5			60			227.36	30	45	——	137.36	——	137.36
	5.1	——		6.3	32.13							
-22.6			60			259.49	30	45	——	169.49	——	169.49
-22.6			60			259.49	30	45	——	169.49	——	169.49
	1.9	——		6.3	11.97							
-24.5			60			271.46	30	45	——	181.46	——	181.46
-24.5			150			271.46	30	45	——	1.46	——	1.46
	25.5	——		7.7	196.35							
-50			150			467.81	30	45	——	197.81	——	197.81

Table 2.3- Passive Earth Pressure (Permanent State)

(β= 0.0° δ=-15.0°)

Lay (m)	h (m)	φ (°)	c (kN/m²)	γ (kN/m³)	γh (kN/m²)	Layer (m)	$\Sigma\gamma h$ (kN/m²)	w (kN/m²)	ζ (°)	$K_P cos\delta$	Pp (kN/m²)
-12.6			——			-12.6	0	0	20.7	4.8069	0
	4.9	30		10	49						
-17.5			——			-17.5	49	0	20.7	4.8069	235.538
-17.5			60			-17.5	49	0	45	——	169.0
	7.0	——		6.3	44.1						
-24.5			60			-24.5	93.1	0	45	——	213.1
-24.5			150			-24.5	93.1	0	45	——	393.1
	25.5	——		7.7	196.35						
-50			150			-50	289.45	0	45	——	589.45

Table 2.4- Summary of Earth Pressure and Residual Water Pressure (Permanent State)

Layer (m)	Active Pressure (kN/m²) $Pa + Pw$	Passive Pressure (kN/m²) Pp
3.50	5.826+ 0.000= 5.826	——————
1.50	12.817+ 0.000= 12.817	——————
1.50	12.817+ 0.000= 12.817	——————
1.33	13.411+ 0.000= 13.411	——————
1.33	13.411+ 0.000= 13.411	——————
0.00	15.994+ 13.433= 29.427	——————
0.00	15.994+ 13.433= 29.417	——————
-10.00	35.414+ 13.433= 48.847	——————
-10.00	53.085+ 13.433= 66.518	——————
-12.60	60.654+ 13.433= 74.087	——————
-12.60	60.654+ 13.433= 74.087	0.000
-17.50	74.917+ 13.433= 88.350	235.538
-17.50	137.360+ 13.433= 150.793	169.000
-22.60	169.490+ 13.433= 182.923	201.130
-22.60	169.490+ 13.433= 182.923	201.130
-24.50	181.460+ 13.433= 194.893	213.100
-24.50	1.460+ 13.433= 14.893	393.100
-50.00	197.810+ 13.433= 211.243	589.450

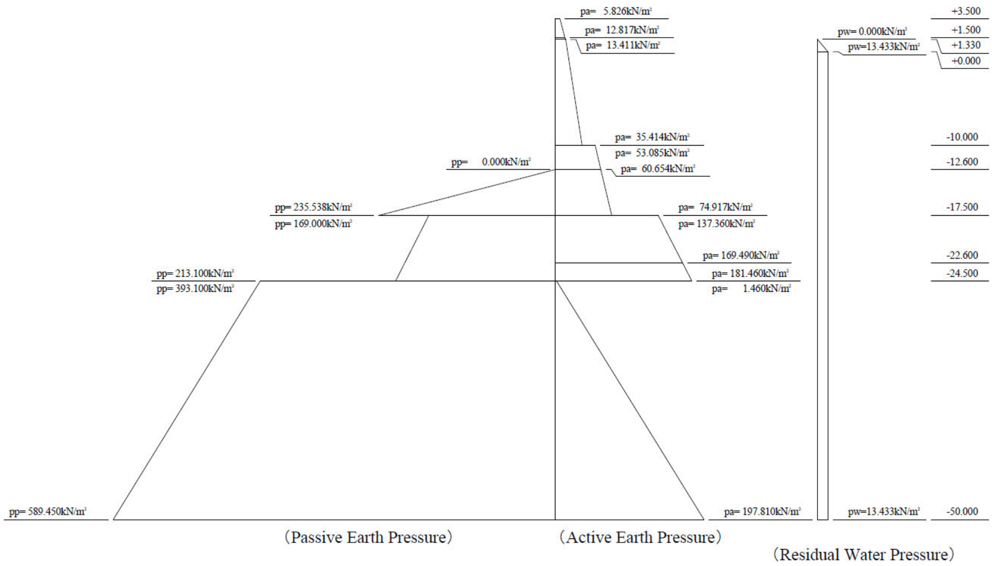


Figure 2.2- Distribution of Active Earth Pressure, Residual Water Pressure and Passive Earth Pressure (Permanent state)

2) Verification of the Embedded Length of Sheet-pile Wall

The embedded length of the steel sheet-pile wall in the permanent state is verified using the free earth support method (Equation (2.1)) and Rowe's method.

$$m \cdot S_d / R_d \leq 1.0,\ R_d = \gamma_R \cdot R_k,\ S_d = \gamma_S \cdot S_k$$
$$R_k = a \cdot P_{pk} \tag{2.1}$$
$$S_k = b \cdot P_{ak} + c \cdot P_{wk} + d \cdot P_{dwk}$$

Where:

R	:	resistance term(kN/m)
S	:	load term (kN/m)
P_p	:	resultant passive earth pressure acting on the sheet-pile wall (kN/m)
P_a	:	resultant active earth pressure acting on the sheet-pile wall (kN/m)
P_w	:	resultant residual water pressure acting on the wall structure (kN/m)
P_{dw}	:	resultant dynamic water pressure acting on the wall body (kN/m) (only during earthquake)
a to d	:	distance between the position of installation of the tie member and the point of action of resultant force (m)
γ_R	:	partial factor multiplied by resistance term
γ_S	:	partial factor multiplied by load term
m	:	adjustment factor

3) Verification by the Free Earth Support Method

i) Active moment

Table 2.5 presents the calculation results for the active moment of the active earth pressure and residual water pressure at the tie rod installation point.

Table 2.5- Active Moment of the Active Earth Pressure and Residual Water Pressure at the tie Member Installation Point (Permanent State)

No.	Layer (m)	Formula	S (kN/m)	l (m)	M (kN·m/m)	M_a (kN·m/m)
1	3.50	1/2× 5.826× 2.000	5.826	-1.333	-7.766	
2	1.50	1/2× 12.817× 2.000	12.817	-0.667	-8.549	-16.315
3	1.50	1/2× 12.817× 0.170	1.089	0.057	0.062	
4	1.33	1/2× 13.411× 0.170	1.140	0.113	0.129	-16.124
5	1.33	1/2× 13.411× 1.330	8.918	0.613	5.467	
6	0.00	1/2× 29.427× 1.330	19.569	1.057	20.684	10.027
7	0.00	1/2× 29.427× 10.000	147.135	4.833	711.103	
8	-10.00	1/2× 48.847× 10.000	244.235	8.167	1,994.667	2,715.797
9	-10.00	1/2× 66.518× 2.600	86.473	12.367	1,069.412	
10	-12.60	1/2× 74.087× 2.600	96.313	13.233	1,274.510	5,059.719
11	-12.60	1/2× 74.087× 4.900	181.513	15.733	2,855.744	
12	-17.50	1/2× 88.350× 4.900	216.458	17.367	3,759.226	11,674.689
13	-17.50	1/2× 150.793× 5.100	384.522	20.700	7,959.605	
14	-22.60	1/2× 182.923× 5.100	466.454	22.400	10,448.570	30,082.864
15	-22.60	1/2× 182.923× 1.900	173.777	24.733	4,298.027	
16	-24.50	1/2× 194.893× 1.900	185.148	25.367	4,696.649	39,077.540
17	-24.50	1/2× 14.893× 25.500	189.886	34.500	6,551.067	
18	-50.00	1/2× 211.243× 25.500	2,693.348	43.000	115,813.964	161,442.571

Where:

S : horizontal force (kN/m)
l : distance from tie member installation point (m)
M_a : moment at tie point (kN·m/m)

ii) Resisting moment

Table 2.6 presents the calculation results for the resisting moment of the passive earth pressure at the tie member installation point.

Table 2.6- Resisting Moment of the Passive Earth Pressure at the tie Member Installation Point (Permanent State)

No.	Layer (m)	Formula	S (kN/m)	l (m)	M (kN·m/m)	M_p (kN·m/m)
—	-12.60	1/2× 0.000× 4.900	0.000	15.733	0.000	
1	-17.50	1/2× 235.538× 4.900	577.068	17.367	10,021.940	10,021.940
2	-17.50	1/2× 169.000× 5.100	430.950	20.700	8,920.665	
3	-22.60	1/2× 201.130× 5.100	512.882	22.400	11,488.557	30,431.162
4	-22.60	1/2× 201.130× 1.900	191.074	24.733	4,725.833	
5	-24.50	1/2× 213.100× 1.900	202.445	25.367	5,135.422	40,292.417
6	-24.50	1/2× 383.100× 25.500	5,012.025	34.500	172,914.862	
7	-50.00	1/2× 589.450× 25.500	7,515.488	43.000	323,165.984	536,373.263

Where:

S : horizontal force (kN/m)
l : distance from tie member installation point (m)
M_p : moment at tie point (kN·m/m)

iii) Verification of the embedded length

Table 2.7 presents the verification results for the embedded length. The partial factors related to the embedded length of sheet piles (γ_R and γ_S) shall be 0.77 and 1.11, respectively, since cohesive soil is partly included in the soil composition from the ground surface to the bottom of the embedded length. The adjustment factor (m) shall be 1.00.

Table 2.7- Verification Results for Embedded Length (Permanent State)

m = 1.00

Layer (m)	M_a (kN·m/m)	γ_S	$m \cdot S_d$ (kN·m/m)	M_p (kN·m/m)	γ_R	R_d (kN·m/m)	
-24.50	39,077.540	1.11	43,376.069	40,292.417	0.77	31,025.161	NO
-50.00	161,442.571	1.11	179,201.253	536,373.263	0.77	413,007.413	OK

The above calculation therefore shows that the required embedded length for a sheet-pile wall is in the range from -24.50 to -50.0 m.

Then, performance verification will be conducted with the bottom level of embedded length using free earth support methods set to -26.122 m.

4) Verification of the Embedded Length of Sheet Pile using Rowe's Method

The following is verification method of the embedded length of the sheet-pile wall using Rowe's method.

$$\delta_N = D_F/H_T \geq 4.9510 \times \omega^{-0.2} - 0.2486 \qquad (2.2)$$

Where:

δ_N : ratio of the embedded length of a sheet pile wall to the height from a tie member installation position to a seafloor surface

(Permanent state)

D_F	:	embedded length of a sheet pile wall (m)
H_T	:	height from a tie member installation position to a seafloor surface (m)
ω	:	similarity number (ρl_h)
ρ	:	flexibility number (H_T^4/EI) (m^3/MN)
E	:	Young's modulus of a sheet pile wall (MN/m^2)
I	:	geometrical moment of the inertia per unit width of the cross section of the sheet pile (m^4/m)
l_h	:	coefficient of subgrade reaction of a sheet pile wall (MN/m^3)

i) Structural details

Table 2.8 presents the specifications of steel pipe sheet pile wall.

Table 2.8- Specifications of Steel Pipe Sheet Pile Wall (Permanent State)

Item	Unit	Value
Type of sheet pile		D800×t12
Young's modulus (E)	MN/m^2	2.00E+5
Geometrical moment of inertia of cross-section (I)	m^4/m	2.30632E-03
Embedded length of sheet pile (D_F) (free earth support method)	m	13.522
Sheet-pile wall height (H_T)	m	14.100
Average N-value of ground under seabed surface		19

ii) Modulus of the subgrade reaction of sheet-pile wall (l_h)

Since ground improvement is not conducted on the seaside of the quaywall, determine the modulus of the subgrade reaction l_h based on the soil specifications of the original ground.

Also, since the average N-value from the seabed surface is assumed to be 19, φ=30 degree sandy soil, using Figure 2.3, l_h should have a value of 28.0 MN/m^3.

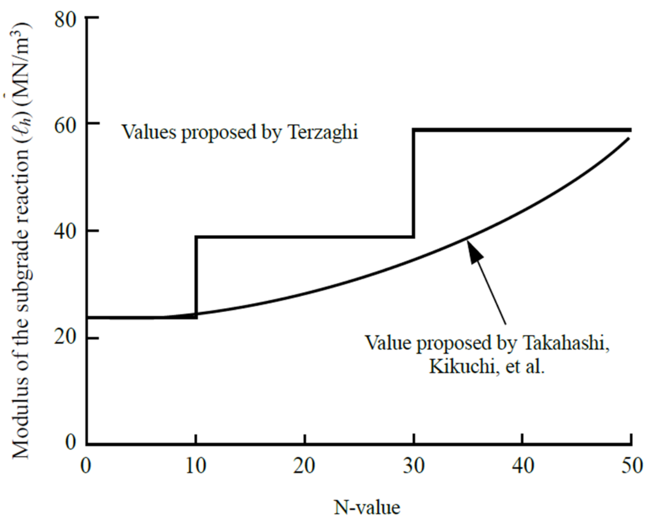


Figure 2.3- Relationship between Modulus of the Subgrade Reaction and N-value

iii) Calculation of δ_N, ρ, and ω

Table 2.9 presents the calculation results for δ_N, ρ, and ω.

Table 2.9- Calculation Results for δ_N, ρ, and ω (Permanent State)

Item	Unit	Value
Type of sheet pile		D800×t12
Young's modulus (E)	MN/m^2	2.00E+5
Geometrical moment of inertia of cross-section (I)	m^4/m	2.64E-03
(EI)	$MN/m^2/m$	528
Embedded length of sheet pile (D_F) (free earth support method)	m	13.522
Sheet-pile wall height (H_T)	m	14.100
$\delta_N=D_F/H_T$		0.9590
$\rho=H_T^4/EI$	m^3/MN	74.859
l_h	MN/m^3	28
$\omega=\rho\times l_h$		2,096.052

iv) Verification of the embedded length of sheet pile using Rowe's method

Calculate the required embedded length using Equation (2.2) as follows:

$$\delta_N = D_F/H_T = 0.9590 \;\geq\; 4.9510 \times (2{,}096.052)^{-0.2} - 0.2486 = 0.8239 \quad \text{O.K}$$

The above calculation presents that the embedded length by the free earth support method satisfies the requirement when verified using Rowe's method.

Therefore, the height of the bottom of the required embedded length of the sheet-pile wall in the permanent state is thus -26.122 m.

(2) Performance Verification of Stress of Sheet pile Wall

1) Moment and Reaction Force by Equivalent Beam Method

Table 2.10 presents the calculation results for the active earth pressure and residual water pressure in the permanent state.

The calculation results of Table 2.10 are shown in Figure 2.4.

Table 2.10- Active Earth Pressure and Residual Water Pressure (Permanent State)

Layer (m)	$Pa + Pw$ (kN/m^2)	Pp (kN/m^2)	$Pa - Pp$ (kN/m^2)
3.50 1.50	5.826 12.817	― ―	5.826 12.817
1.50 1.33	12.817 13.411	― ―	12.817 13.411
1.33 0.00	13.411 29.427	― ―	13.411 29.427
0.00 -10.00	29.427 48.847	― ―	29.427 48.847
-10.00 -12.60	66.518 74.087	― ―	66.518 74.087
-12.60 -17.50	74.087 88.350	0.000 235.538	74.087 -147.188
-17.50 -22.60	150.793 182.923	169.000 201.130	-18.207 -18.207
-22.60 -24.50	182.923 194.893	201.130 213.100	-18.207 -18.207
-24.50 -50.00	14.893 211.243	393.100 589.450	-387.207 -387.207

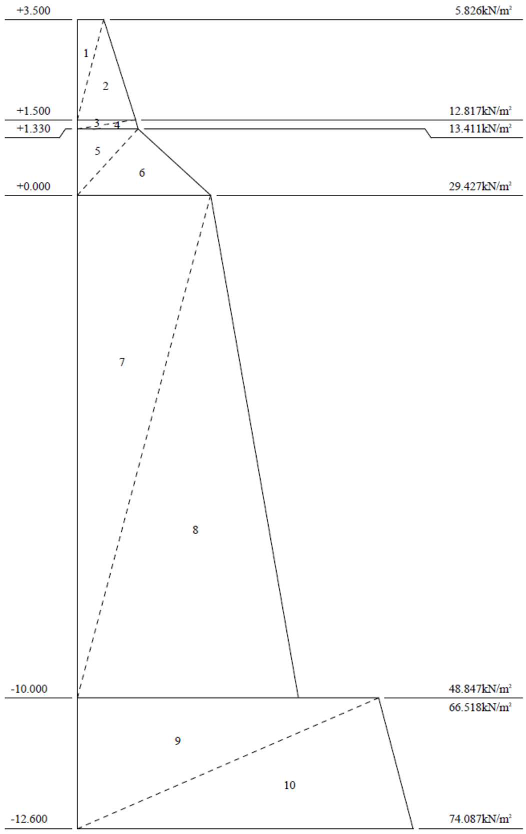


Figure 2.4- Distribution of the Active Earth Pressure, Residual Water Pressure (Permanent State)

i) Maximum moment and reaction force generated at the tie member installation point

Calculate the moment acting on the sheet-pile wall and the reaction at the tie member installation point assuming that the tie member installation point and the seabed surface serve as the support, and that this system is a simple beam with the earth pressure above the seabed surface and the residual water pressure acting on the beam.

ii) Moment at the tie member installation point

Table 2.11 presents the calculation results for the moment at the tie member installation point.

Table 2.11- Moment at the Tie Member Installation Point (Permanent State)

No.	Formula	S_a (kN/m)	l (m)	M_a (kN･m/m)
1	1/2× 5.826× 2.000	5.826	-1.333	-7.766
2	1/2× 12.817× 2.000	12.817	-0.667	-8.549
3	1/2× 12.817× 0.170	1.089	0.057	0.062
4	1/2× 13.411× 0.170	1.140	0.113	0.129
5	1/2× 29.427× 10.000	147.135	4.833	711.103
6	1/2× 48.847× 10.000	244.235	8.167	1,994.667
7	1/2× 66.518× 2.600	86.473	12.367	1,069.412
8	1/2× 74.087× 2.600	96.313	13.233	1,274.510
Total		623.515	――――	5,059.719

Where:

S_a : horizontal force (kN/m)

l : distance from tie member installation point (m)
M_a : moment at tie point (kN·m/m)

iii) Reaction force at the tie member installation point

Reaction at the support on the seabed surface: R_0
$R_0 = \sum M_a \div l = 5{,}059.719 \div 14.10 = 358.845$ kN/m

Reaction force at the tie member installation point: A_P
$A_P = \sum S_a - R_0 = 623.515 - 358.845 = 264.670$ kN/m

iv) Maximum bending moment of sheet-pile wall

The maximum bending moment acting on the sheet-pile wall occurs at a location where shear force Q is zero. The shear force is calculated using $Q = A_P - \sum P$, giving the position of Q = zero as -6.092 m

The calculation results are shown in Table 2.12.

Table 2.12- Shear force Q = 0 Position (Permanent State)

Layer (m)	Force P (kN/m)	ΣP (kN/m)	Tie Force A_p (kN/m)	Shear Force Q (kN/m)
3.50 1.50	5.826 12.817	18.643	264.670	246.027
1.50 1.33	1.089 1.140	20.872	264.670	243.798
1.33 0.00	8.918 19.569	49.359	264.670	215.311
0.00 -10.00	147.135 244.235	440.729	264.670	-176.059
-10.00 -12.60	86.473 96.313	623.515	264.670	-358.845

The bending moment related to the position of $Q = 0$ for the earth pressure and residual water pressure from the quaywall top, or +3.50 m, to -6.092 m, is calculated as in Table 2.13.

Table 2.13- Bending Moment at Shear Force Q = 0 Position (Permanent State)

No.	Formula	S (kN/m)	l (m)	M (kN·m/m)
1	1/2× 5.826× 2.000	-5.826	8.925	-51.997
2	1/2× 12.817× 2.000	-12.817	8.259	-105.856
3	1/2× 12.817× 0.170	-1.089	7.535	-8.206
4	1/2× 13.411× 0.170	-1.140	7.479	-8.526
5	1/2× 13.411× 1.330	-8.918	6.979	-62.239
6	1/2× 29.427× 1.330	-19.569	6.535	-127.883
7	1/2× 29.427× 6.092	-89.635	4.061	-364.008
8	1/2× 41.258× 6.092	-125.672	2.031	-255.240
Total		———	———	-983.955

The calculation shown in Table 2.13 gives the bending moment at the position in the sheet pile wall with shear force $Q = 0$ (–6.092 m) as follows:

Distance from the tie member installation point to the zero shear-force point
$h = 1.500 - (-6.092) = 7.592$ m

Maximum bending moment

$M_{a(Q=0)} = \Sigma M_a = A_P \times h - \Sigma M = 264.670 \times 7.592 - 983.955 = 1{,}025.420$ kN·m/m

2) Moment and Reaction Force by Rowe's Method

Figure 2.5 and Table 2.14 indicate the calculation results for the maximum bending moment and the reaction force at the tie member installation point using the virtual beam method.

Table 2.14- Maximum Moment and Reaction Force at the Tie Member Installation Point (Permanent State)

Item	Sign	Unit	Permanent state
Maximum bending moment	M_{max}	kN·m/m	1,025.420
Location of occurrence		DL.m	-6.092
Reaction force at tie member installation point	Ap	kN/m	264.670

Given the above results, the maximum bending moment and the reaction force at the tie member installation point are corrected using Equation (2.3) and Equation (2.4) as follows:

Permanent state, correction of the maximum bending moment:

$$\mu_N = M_F/M_T = 3.8625 \times \omega^{-0.2} + 0.2255 \quad (2.3)$$

Permanent state, correction of the reaction force at the tie member installation point:

$$\tau_N = T_F/T_T = 1.8259 \times \omega^{-0.2} + 0.6232 \quad (2.4)$$

Where:

μ_N : correction factor for the maximum moment (permanent state)

M_F : maximum bending moment after correction

M_T : maximum bending moment before correction

τ_N : correction factor for the reaction force at the tie member installation point (permanent state)

T_F : reaction force at the tie member installation point after correction

T_T : reaction force at the tie member installation point before correction

ω : similarity number $(\rho \times l_h)$

$\omega = 74.859 \times 28.0 = 2{,}096.052$

The correction results are given as follows:

Correction factor for maximum bending moment: $\mu_N = 1.0622$

$M_F = 1.0622 \times 1{,}025.420 = 1{,}089.201$ kN·m/m

Correction factor for the reaction force at the installation point: $\tau_N = 1.0187$

$T_F = 1.0187 \times 264.670 = 269.619$ kN/m

The above corrected values will be used for verification of the sheet-pile stress, the tensile stress of tie members, the stress of waling, and the anchorage.

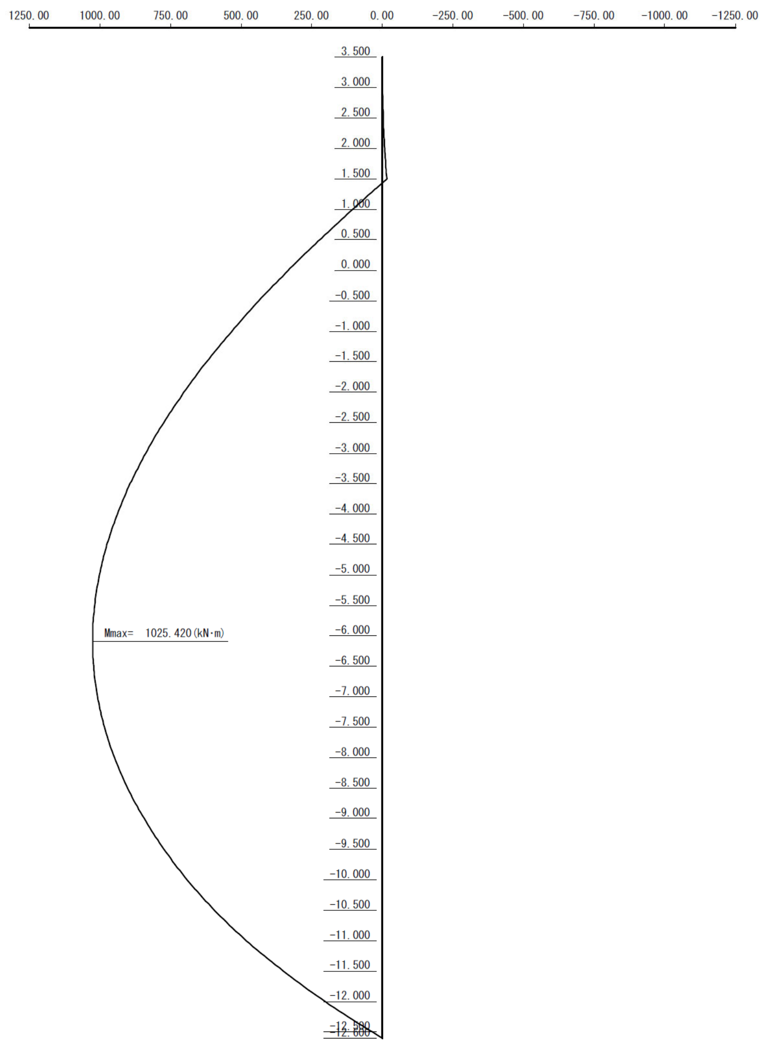


Figure 2.5- Maximum Bending Moment (Permanent State)

3) Performance Verification of Stress of Sheet Piles

i) Cross-sectional properties of sheet piles

The cross-sectional properties of steel-pipe sheet pile shown in Table 2.15 are as follows:

Table 2.15- Specifications of Steel Pipe Sheet Pile

Item	Unit	Value	Remarks
Type of sheet pile		D800 x t12	
Material		SPSP490	
Section modulus (Zo)	cm^3/m	6,590	Before corrosion
Section modulus (Z)	cm^3/m	6,084	After corrosion
Bending yield stress of steel (σ_{yd})	N/mm^2	315.0	

Corrosion allowance for steel-pipe sheet pile (corrosion rate μ: 90%)

$1 - \mu = 0.1$

Seaside: t_1= 0.100 mm/year × 0.1 × 50 years = 0.50 mm

Landside: t_2 = 0.020 mm/year × 50 years = 1.00 mm

The cross-sectional properties of steel-pipe sheet pile after corrosion as shown above are the result of calculations done with 0.75 mm given as the average corrosion allowance.

ii) Performance Verification of Stress of Sheet Pile Wall

$$m \cdot S_d / R_d \leq 1.0,\ R_d = \gamma_R \cdot R_k,\ S_d = \gamma_s \cdot S_k$$
$$R_k = \sigma_{yk} \tag{2.5}$$
$$S_k = M_{max\,k} / Z$$

Where:

σ_y	:	bending yield stress of the steel material (N/mm^2)
M_{max}	:	maximum bending moment in the sheet-pile wall (N·mm/m)
Z	:	section modulus of steel material (mm^3/m)
R	:	resistance term(kN/m)
S	:	load term (kN/m)
γ_R	:	partial factor multiplied by resistance term (=0.84)
γ_s	:	partial factor multiplied by load term (=1.18)
m	:	adjustment factor (=1.0)

The verification result for bending stress of a sheet pile is shown below:

$$m \cdot \frac{S_d}{R_d} = m \cdot \frac{\gamma_S S_k}{\gamma_R R_k} = 1.0 \times \frac{1.18 \times 1{,}072.639 \times 10^6 / (6{,}084 \times 10^3)}{0.84 \times 315.0} = 0.786 \leq 1.0$$

(3) Performance Verification of Tie Members

1) Tie Member Specifications

The specifications of tie member are presented in Table 2.16 as follows:

Table 2.16- Specifications of Tie Rod (Permanent State)

Item	Unit	Value
Type of tie member		Tie Rod
Material		High Tension 690
Yield stress of steel (σ_{yk})	N/mm^2	440.0
Corrosion rate (Δd)	mm	3.0

2) Tension Force of Tie Rod

The tension force of the tie rod is calculated by the following Equation:

$$T_k = A_{pk}\, l \sec\theta \tag{2.6}$$

Where:

T	:	tension force of the tie member (kN)
A_p	:	reaction at the tie member installation point (kN) (=269.19 kN/m)
l	:	tie member installation interval (m)
θ	:	inclination angle of the tie member to the line perpendicular to the sheet pile wall (°)

The calculation result for tie rod tension force T is shown as follows:

$T_d = 269.619 \times 2.321 \times \sec(0.0°) = 625.786$ kN/pcs

3) Verification of the Stress of Tie Members

$$m \cdot S_d / R_d \leq 1.0,\ R_d = \gamma_R \cdot R_k,\ S_d = \gamma_s \cdot S_k$$
$$R_k = \sigma_{yk}$$
$$S_k = T_k / A$$

Where:

σ_y	:	tensile yield stress of a tie member (N/mm^2)
T	:	tension force on a tie member (N)
A	:	cross section area of a tie member (mm^2)
R	:	resistance term (N/mm^2)
S	:	the load term (N/mm^2)
γ_R	:	partial factor multiplied by resistance term (=0.64)
γ_s	:	partial factor multiplied by load term (=1.29)
m	:	adjustment factor (=1.0)

The required diameter of the tie rod is calculated as follows:

$$d = 2\times\sqrt{\frac{m\cdot\gamma_s\cdot T_k}{\pi\cdot\gamma_R\cdot\sigma_{yk}}} + \Delta d = 2\times\sqrt{\frac{1.0\times1.29\times625.786\times10^3}{\pi\times0.64\times440.0}} + 3.0 = 63.42\text{mm}$$

Therefore, the diameter of the tie rod is assumed to be 70mm.
The verification results for the stress of tie rod are then given as follows:

$$m\cdot\frac{S_d}{R_d} = m\cdot\frac{\gamma_S S_k}{\gamma_R R_k} = 1.0\times\frac{1.29\times625.786\times10^3/3{,}525.65}{0.64\times440} = 0.813\leq1.0\quad \text{O.K}$$

(4) Performance Verification of Waling Member

1) Waling Member Specifications

The specifications of waling member are shown in Table 2.17.

Table 2.17- Waling Specifications (Permanent State)

Item	Unit	Value
Type of waling (channel steel)		2[-300×90×12.0×16.0
Material		SS400
Section modulus (Z) (after corrosion)	cm^3	525.0
Bending yield stress of steel (σ_{yk})	N/mm^2	235.0
Tie rod installation interval (L)	m	2.321

2) Maximum Bending Moment

Calculate the maximum bending moment acting on waling $M_{max\,k}$ using the following Equation:

$$M_{max\,k} = T_k\, l / 10$$

Where:

M_{max}	:	maximum bending moment in the waling (N·mm/m)
T	:	tension force of a tie member (kN)
l	:	tie member installation interval (m)

The calculation results for the maximum bending moment acting on waling $M_{max\,k}$ are shown below:

$$M_{max\,k} = 625.786 \times 2.321 \div 10 = 145.245 \text{ kN·m}$$

3) Verification of the Stress of Waling

$$m \cdot S_d / R_d \leq 1.0,\ R_d = \gamma_R \cdot R_k,\ S_d = \gamma_s \cdot S_k$$
$$R_k = \sigma_{yk}$$
$$S_k = M_{max\,k} / Z$$

Where:

σ_y	:	bending yield stress in the waling (N/mm^2)
M_{max}	:	maximum bending moment in the waling (N·mm/m)
Z	:	section modulus of the waling (mm^3)
R	:	resistance term (N/mm^2)
S	:	the load term (N/mm^2)
γ_R	:	partial factor multiplied by resistance term (=1.0)
γ_s	:	partial factor multiplied by load term (=1.0)
m	:	adjustment factor (=1.67)

The verification results for the stress of waling (channel section) are shown below:

$$m \cdot \frac{S_d}{R_d} = m \cdot \frac{\gamma_S S_k}{\gamma_R R_k} = 1.67 \times \frac{1.0 \times 145.245 \times 10^6 / (2 \times 525.0 \times 10^3)}{1.0 \times 235.0} = 0.983 \leq 1.0 \text{ O.K}$$

(5) Performance Verification of Anchorage

1) Anchorage Specifications

The specifications of anchorage members are shown in Table 2.18.

Table 2.18- Anchorage Specifications and Tie Rod Tension (Permanent State)

Item	Unit	Value	Remark
Anchorage installation height	m	+2.50	
Tie rod installation height	m	+1.50	
Type of anchorage (steel-pipe pile)		D800×t9.0	
Effective width of anchorage (*B*)	mm	800	
Material		SPP490	
Young's modulus (*E*)	kN/m^2	200	
Geometrical moment of inertia (*Io*)	cm^4	175,000	Before corrosion
Geometrical moment of inertia (*I*)	cm^4	154,909	After corrosion
Section modulus (*Zo*)	cm^3	4,370	Before corrosion
Section modulus (*Z*)	cm^3	3,882	After corrosion
Tie rod tension force (*T*)	kN	625.786	

Note: Corrosion allowance for steel-pipe pile

$t = 1 \times 0.020$ mm/year × 50 years = 1.00 mm

2) Lateral resistance constant k_c

The soil condition where anchor piles are installed is sandy soil, and the N- value is considered to be constant in the depth direction. Therefore, it is taken as C-type ground.

Average N-value: 10

Calculate the lateral resistance constant using Figure. 2.7.

$$k_c = 540N^{0.648} = 540 \times 10^{0.648} = 2{,}401 \text{ kN/m}^{2.5}$$

3) Maximum Bending Moment, Displacement, and Embedded Length

The maximum bending moment, displacement, and embedded length calculated using the "Koken Method (PHRI Method)" are shown in Table 2.19.

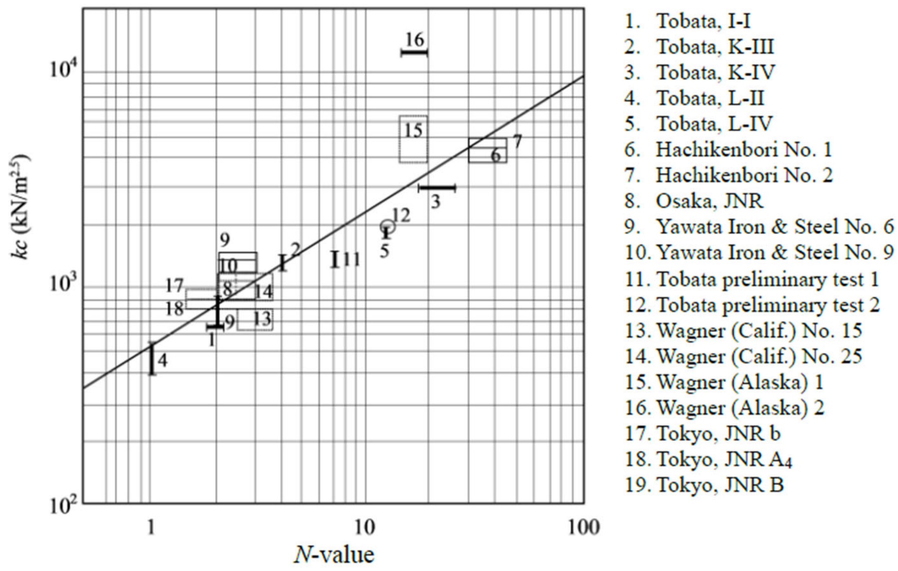


Figure 2.7- Relationship between N-value and *kc*

Table 2.19- Anchor Pile Design Values (Permanent State)

Item	Symbol	After corrosion	Unit
Anchor Pile displacement	Y_{top}	2.833	cm
Ground surface displacement	Y_0		cm
Pile top moment	M_{top}	0.000	kN·m
Maximum underground moment	M_{max}	712.057	kN·m
Depth of moment $M = 0$	l_{m1}	8.068	m
Angle of deflection at pile top	i_{top}		rad
Angle of deflection on ground surface	i_0		rad
$l_{m1}/3$		2.689	m
$1.5 \times l_{m1}$		12.102	m

4) Verification of the Stress of Anchor Pile

Use the following Equation to verify the stress of an anchor pile for the result after corrosion.

$$m \cdot S_d / R_d \leq 1.0,\ R_d = \gamma_R \cdot R_k,,\ S_d = \gamma_s \cdot S_k$$
$$R_k = \sigma_{yk}$$
$$S_k = M_{max\,k} / Z$$

Where:

σ_y : bending yield stress of a pile anchorage (N/mm^2)

M_{max} : maximum bending moment in a pile anchorage (N·mm/m)

Z : section modulus of a pile anchorage (mm^3/m)

R : resistance term (N/mm^2)

S	:	the load term (N/mm^2)
γ_R	:	partial factor multiplied by resistance term (=1.0)
γ_S	:	partial factor multiplied by load term (=1.0)
m	:	adjustment factor (=1.67)

The verification results for the stress of anchor pile are shown below:

$$m \cdot \frac{S_d}{R_d} = m \cdot \frac{\gamma_S S_k}{\gamma_R R_k} = 1.67 \times \frac{1.0 \times 712.057 \times 10^6/(3{,}882 \times 10^3)}{1.0 \times 315.0} = 0.972 \leq 1.0 \text{ O.K}$$

5) Bottom Level of Anchor Pile

The bottom level of anchor pile using the pre-corrosion result is as follows:

Bottom level of the anchor pile = height of tie rod installation – 1.5 × l_{m1}
= +1.50 -12.102 = -10.602 m (permanent state)

6) Anchorage Installation Position

Place a vertical anchor pile at a location where the active failure plane of the front sheet pile drawn from the seabed surface does not intersect the passive failure plane of the anchorage drawn from the position of $l_{m1}/3$ down the anchorage-side tie rod installation point to the tie rod installation height.

- Foundation ground height (design height): -12.60 m
- Tie rod installation height (DL): +1.50 m
- Anchorage $l_{m1}/3$ length: 2.689m

The anchorage installation position is determined by the angle of failure surface of each layer. The distance between sheet pile quaywall and the anchorage pile is minimum 15.025m.

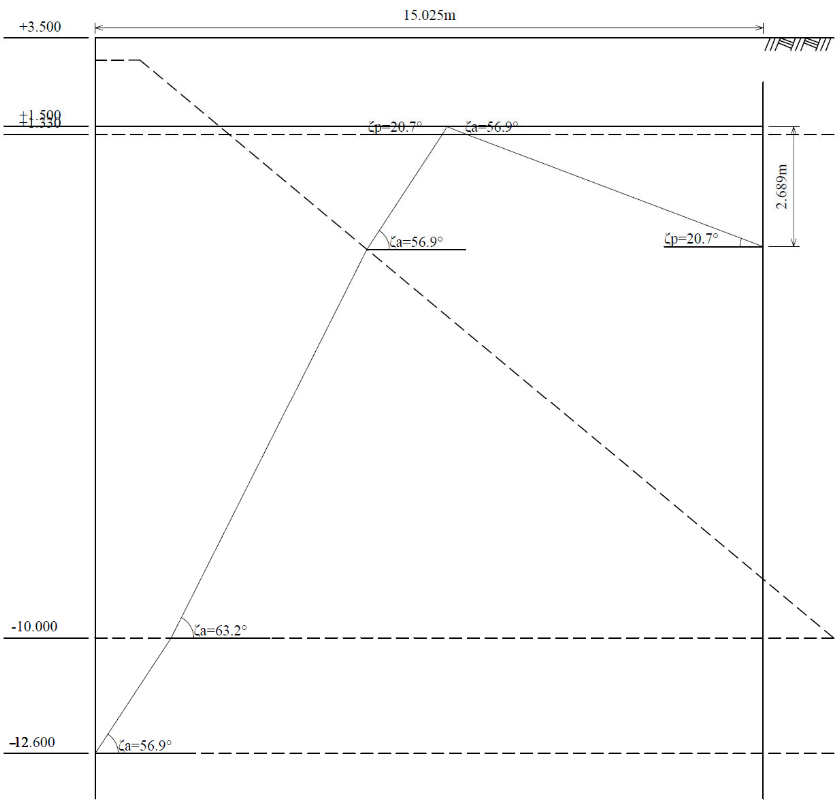


Figure 2.8- Anchorage Installation Position (Permanent State)

(6) Performance Verification of Circular Slip Failure

A circular slip failure result is shown in Figure 2.9.

The partial factor is verified with a coefficient of variation (CV) greater than 0.25, the load factor γ_S being 1.0, the resistance factor γ_R being 1.0, and the adjustment factor m being 1.30.

The verification results of circular slip failure of the foundation ground of the sheet pile quaywall are given as follows:

- Characteristic value of load (acting moment)
  S_k = 118,087.781 (kN·m/m)

- Characteristic value of resistance (resisting moment)
  R_k= 260,154.089 (kN·m/m)

$$m \cdot \frac{S_d}{R_d} = 1.30 \times \frac{1.0 \times 118{,}087.781}{1.0 \times 260{,}154.089} = 0.590 \leq 1.0 \quad \text{O.K}$$

The stability verification gives a value of not more than 1.0, which therefore satisfies the performance requirement.

Table 2.20- Soil Properties for Circular Slip Failure (Permanent State)

Block	Saturated weight $W1$ (kN/m³)	Wet weight $W2$ (kN/m³)	Weight in water W' (kN/m³)	Angle of internal friction φ (°)	Standard cohesion Co (kN/m²)	Cohesion gradient K	Cohesion reference height Yo (m)
1	20.0	18.0	10.0	40	0.0	0.0	0.0
2	20.0	18.0	10.0	30	0.0	0.0	0.0
3	20.0	18.0	10.0	30	0.0	0.0	0.0
4	16.3	16.3	6.3	0	60.0	0.0	0.0
5	16.3	16.3	6.3	0	150.0	0.0	0.0

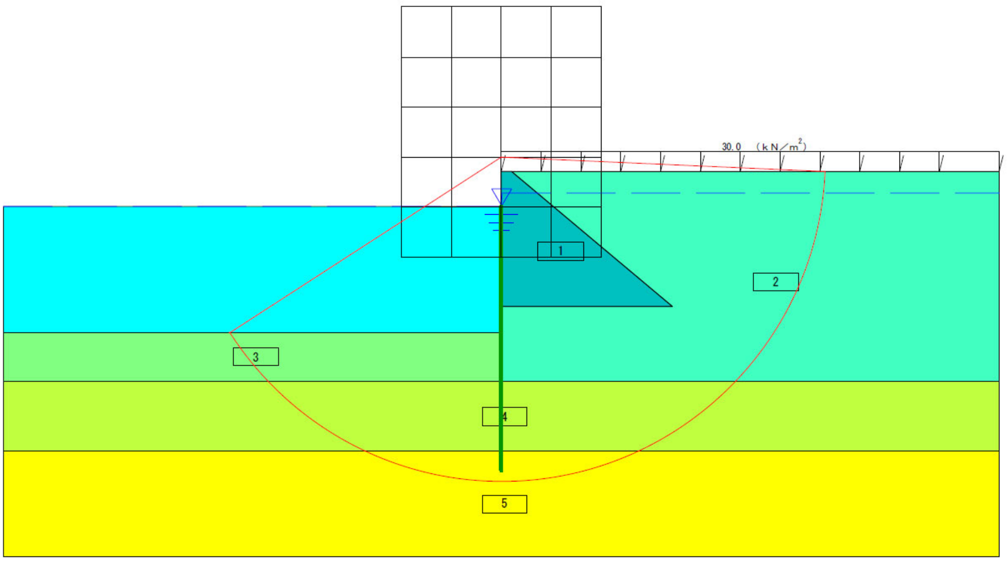


Figure 2.9- Verification Results for Circular Slip Failure

2-5. Performance Verification of Sheet Pile Quaywalls in the Variable State of Level 1 Earthquake Ground Motion

(1) Performance Verification of the Embedded Length of Sheet pile

1) Earth Pressure and Residual Water Pressure

The acting force and moment caused by the dynamic water pressure at the tie rod installation point should be calculated as follows.

The dynamic water pressure distribution, dynamic water pressure service force, and service position are calculated as the following equations:

$$p_{dwk}=\pm\frac{7}{8}ck_{hk}\rho_w g\sqrt{Hy}$$

$$P_{dw}=\pm\frac{7}{12}k_{hk}\rho_w gh^2,\ h_{dw}=\frac{3}{5}h$$

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 $L/H \leq 1.5$, $c = L/1.5H$; when $L/H \leq 1.5$, $c = 1.0$)
L : length of the space occupied by water in the direction of vibration (m)
P_{dwk} : resultant force of dynamic water pressure (kN/m)
h_{dw} : depth of the acting point of the dynamic water pressure resultant force from the still water level (m)

Table 2.21 presents the apparent seismic coefficient for estimating the active earth pressure and passive earth pressure. Table 2.22 presents the summary of earth pressure and residual water pressure including dynamic water pressure, and Table 2.23 presents the calculation results of the dynamic water pressure.

Seismic coefficient for verification: 0.10
Unit weight of seawater: 10.10 kN/m³
Location of tie rod installation point: +1.50 m

Table 2.21- Apparent Seismic Coefficient

For active earth pressure
(Seismic Coefficient k= 0.10 , $\Sigma\gamma h_i$ = 30.060 (kN/m²))

Layer (m)	h (m)	γ_t (kN/m³)	γ (kN/m³)	$\Sigma\gamma_t h_j$ (kN/m²)	$\Sigma\gamma h_j$ (kN/m²)	w (kN/m²)	k'	θ (deg)
3.50							———	5.7
	2.17	20.000	18.000	———	———	15.000		
1.33							———	5.7
1.33							0.15	8.5
	11.33	20.000	10.000	0.000	0.000	15.000		
-10.00							0.15	8.5

-10.00 -17.50	7.50	20.000	10.000	266.600	133.300	15.000	0.17 0.17	9.6 9.6
-17.50 -24.50	7.00	16.300	6.300	376.600	188.300	15.000	——— 0.00	——— 0.0
-24.50 -50.00	25.50	17.700	7.700	490.700	232.400	15.000	0.00 0.00	0.0 0.0

For passive earth pressure
(Seismic Coefficient k= 0.10 , $\Sigma\gamma h_i$ = 0 (kN/m^2)

Layer (m)	h (m)	γ_t (kN/m^3)	γ (kN/m^3)	$\Sigma\gamma_t h_j$ (kN/m^2)	$\Sigma\gamma h_j$ (kN/m^2)	w (kN/m^2)	k'	θ (deg)
-12.60 -17.50	4.90	20.000	10.000	0.000	0.000	0.000	0.20 0.20	11.3 11.3
-17.50 -24.50	7.00	16.300	6.300	98.000	49.000	0.000	0.22 0.22	12.4 12.4
-24.50 -50.00	25.50	17.700	7.700	212.100	93.100	0.000	0.23 0.23	13.0 13.0

Table 2.22- Dynamic Water Pressure

y (m)	H (m)	k	γ_w (kN/m^3)	p_{dw} (kN/m^2)
0.00	12.60	0.10	10.10	0.000
1.00	12.60	0.10	10.10	3.137
2.00	12.60	0.10	10.10	4.436
3.00	12.60	0.10	10.10	5.433
4.00	12.60	0.10	10.10	6.274
5.00	12.60	0.10	10.10	7.015
6.00	12.60	0.10	10.10	7.684
7.00	12.60	0.10	10.10	8.300
8.00	12.60	0.10	10.10	8.873
9.00	12.60	0.10	10.10	9.411
10.00	12.60	0.10	10.10	9.920
11.00	12.60	0.10	10.10	10.404
12.00	12.60	0.10	10.10	10.867
12.60	12.60	0.10	10.10	11.135

Table 2.23- Summary of Earth Pressure, Residual Water Pressure, and Dynamic Water Pressure (Variable State: Level 1 Earthquake Ground Motion)

Layer	Active Pressure (kN/m^2)	Passive Pressure (kN/m^2)
(m)	Pa + Pw + p_{dw}	Pp
3.50 1.50	3.664+ 0.000+ 0.000= 3.664 12.459+ 0.000+ 0.000= 12.459	——— ———
1.50 1.33	12.459+ 0.000+ 0.000= 12.459 13.207+ 0.000+ 0.000= 13.207	——— ———
1.33 0.00	14.775+ 0.000+ 0.000= 14.775 18.410+ 13.433+ 0.000= 31.843	——— ———
0.00 -1.00	18.410+ 13.433+ 0.000= 31.843 21.143+ 13.433+ 3.137= 37.713	——— ———
-1.00 -2.00	21.143+ 13.433+ 3.137= 37.713 23.876+ 13.433+ 4.436= 41.745	——— ———
-2.00 -3.00	23.876+ 13.433+ 4.436= 41.745 26.609+ 13.433+ 5.433= 45.475	——— ———

-3.00	26.609+	13.433+	5.433=	45.475	
-4.00	29.342+	13.433+	6.274=	49.049	
-4.00	29.342+	13.433+	6.274=	49.049	
-5.00	32.075+	13.433+	7.015=	52.523	
-5.00	32.075+	13.433+	7.015=	52.523	
-6.00	34.808+	13.433+	7.684=	55.924	
-6.00	34.808+	13.433+	7.684=	55.924	
-7.00	37.541+	13.433+	8.300=	59.274	
-7.00	37.541+	13.433+	8.300=	59.274	
-8.00	40.274+	13.433+	8.873=	62.580	
-8.00	40.274+	13.433+	8.873=	62.580	
-9.00	43.006+	13.433+	9.411=	65.850	
-9.00	43.006+	13.433+	9.411=	65.850	
-10.00	45.739+	13.433+	9.920=	69.092	
-10.00	68.500+	13.433+	9.920=	91.853	
-11.00	72.593+	13.433+	10.404=	96.430	
-11.00	72.593+	13.433+	10.404=	96.430	
-12.00	76.686+	13.433+	10.867=	100.986	
-12.00	76.686+	13.433+	10.867=	100.986	
-12.60	79.142+	13.433+	11.135=	103.710	
-12.60	79.142+	13.433+	0.000=	92.575	0.000
-17.50	99.198+	13.433+	0.000=	112.631	195.461
-17.50	125.982+	13.433+	0.000=	139.415	169.000
-22.60	136.360+	13.433+	0.000=	149.793	201.130
-22.60	154.490+	13.433+	0.000=	167.923	201.130
-24.50	166.460+	13.433+	0.000=	179.893	213.100
-24.50	0.000+	13.433+	0.000=	13.433	393.100
-26.26	0.000+	13.433+	0.000=	13.433	406.640
-26.26	0.000+	13.433+	0.000=	13.433	406.640
-50.00	182.810+	13.433+	0.000=	196.243	589.450

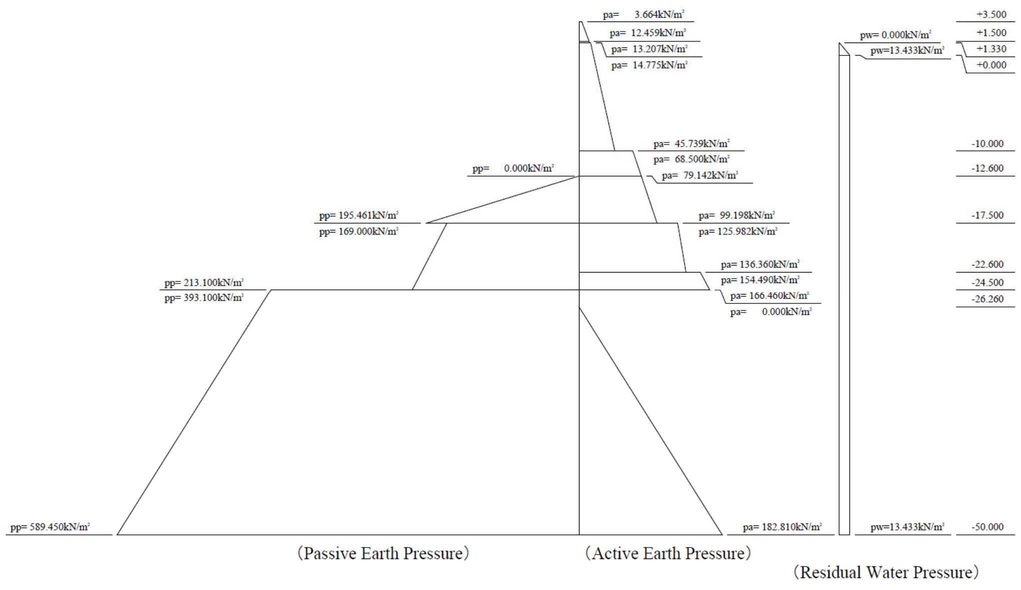


Figure 2.10- Distribution of Active Earth Pressure, Residual Water Pressure and Passive Earth Pressure (Variable State: Level 1 Earthquake Ground Motion)

3) Verification by the Free Earth Support Method

i) Calculation of the active moment of the active earth pressure and residual water pressure at the tie rod installation point

Table 2.24 presents the calculation results for the active moment of the active earth pressure and residual water pressure at the tie rod installation point.

Table 2.24- Active Moment of the Active Earth Pressure and Residual Water Pressure at Tie Member Installation Point (Variable State: Level 1 Earthquake Ground Motion)

No.	Layer (m)	Formula	S (kN/m)	l (m)	M (kN·m/m)	M_a (kN·m/m)
1	3.50	1/2× 3.664× 2.000	3.664	-1.333	-4.864	
2	1.50	1/2× 12.459× 2.000	12.459	-0.667	-8.310	-13.194
3	1.50	1/2× 12.459× 0.170	1.059	0.057	0.060	
4	1.33	1/2× 13.207× 0.170	1.123	0.113	0.127	-13.007
5	1.33	1/2× 14.775× 1.330	9.825	0.613	6.023	
6	0.00	1/2× 31.843× 1.330	21.176	1.057	22.383	15.399
7	0.00	1/2× 31.843× 1.000	15.922	1.833	29.185	
8	-1.00	1/2× 37.713× 1.000	18.856	2.167	40.861	85.445
9	-1.00	1/2× 37.713× 1.000	18.856	2.833	53.419	
10	-2.00	1/2× 41.745× 1.000	20.872	3.167	66.102	204.966
11	-2.00	1/2× 41.745× 1.000	20.872	3.833	80.002	
12	-3.00	1/2× 45.475× 1.000	22.738	4.167	94.749	379.717
13	-3.00	1/2× 45.475× 1.000	22.738	4.833	109.893	
14	-4.00	1/2× 49.049× 1.000	24.524	5.167	126.716	616.326
15	-4.00	1/2× 49.049× 1.000	24.524	5.833	143.048	
16	-5.00	1/2× 52.523× 1.000	26.262	6.167	161.958	921.332
17	-5.00	1/2× 52.523× 1.000	26.262	6.833	179.448	
18	-6.00	1/2× 55.925× 1.000	27.962	7.167	200.404	1,301.184
19	-6.00	1/2× 55.925× 1.000	27.962	7.833	219.026	
20	-7.00	1/2× 59.274× 1.000	29.637	8.167	242.045	1,762.255
21	-7.00	1/2× 59.274× 1.000	29.637	8.833	261.784	
22	-8.00	1/2× 62.580× 1.000	31.290	9.167	286.835	2,310.874
23	-8.00	1/2× 62.580× 1.000	31.290	9.833	307.675	
24	-9.00	1/2× 65.850× 1.000	32.925	10.167	334.748	2,953.297
25	-9.00	1/2× 65.850× 1.000	32.925	10.833	356.677	
26	-10.00	1/2× 69.092× 1.000	34.546	11.167	385.775	3,695.749
27	-10.00	1/2× 91.853× 1.000	45.926	11.833	543.442	
28	-11.00	1/2× 96.430× 1.000	48.215	12.167	586.632	4,825.823
29	-11.00	1/2× 96.430× 1.000	48.215	12.833	618.743	
30	-12.00	1/2× 100.986× 1.000	50.493	13.167	664.841	6,109.407
31	-12.00	1/2× 100.986× 0.600	30.296	13.700	415.055	
32	-12.60	1/2× 103.710× 0.600	31.113	13.900	432.471	6,956.933
33	-12.60	1/2× 92.575× 4.900	226.809	15.733	3,568.386	
34	-17.50	1/2× 112.631× 4.900	275.946	17.367	4,792.354	15,317.673
35	-17.50	1/2× 139.415× 5.100	355.508	20.700	7,359.016	
36	-22.60	1/2× 149.793× 5.100	381.972	22.400	8,556.173	31,232.862
37	-22.60	1/2× 167.923× 1.900	159.527	24.733	3,945.581	
38	-24.50	1/2× 179.893× 1.900	170.898	25.367	4,335.170	39,513.613
39	-24.50	1/2× 13.433× 1.760	11.821	26.587	314.285	
40	-26.26	1/2× 13.433× 1.760	11.821	27.173	321.212	40,149.110
41	-26.26	1/2× 13.433× 23.740	159.450	35.673	5,688.060	
42	-50.00	1/2× 196.243× 23.740	2,329.404	43.587	101,531.732	147,368.902

Where:

S : horizontal force (kN/m)
l : distance from tie member installation point (m)
M_a : moment at tie point (kN·m/m)

ii) Resisting moment of the passive earth pressure at the tie rod installation point

Table 2.25 presents the calculation results for the resisting moment of the passive earth pressure at the tie member installation point.

Table 2.25- Resisting Moment of the Passive Earth Pressure at the Tie Member Installation Point (Variable State: Level 1 Earthquake Ground Motion)

No.	Layer (m)	Formula	S (kN/m)	l (m)	M (kN·m/m)	M_p (kN·m/m)
—	-12.60	1/2× 0.000× 4.900	0.000	15.733	0.000	
1	-17.50	1/2× 195.461× 4.900	478.879	17.367	8,316.692	8,316.692
2	-17.50	1/2× 169.000× 5.100	430.950	20.700	8,920.665	
3	-22.60	1/2× 201.130× 5.100	512.882	22.400	11,488.557	28,725.914
4	-22.60	1/2× 201.130× 1.900	191.074	24.733	4,725.833	
5	-24.50	1/2× 213.100× 1.900	202.445	25.367	5,135.422	38,587.169
6	-24.50	1/2× 393.100× 1.760	345.928	26.587	9,197.188	
7	-26.26	1/2× 406.640× 1.760	357.843	27.173	9,723.668	57,508.025
8	-26.26	1/2× 406.640× 23.740	4,826.817	35.673	172,187.043	
9	-50.00	1/2× 589.450× 23.740	6,996.772	43.587	304,968.301	534,663.369

Where:

S : horizontal force (kN/m)

L : distance from tie member installation point (m)

M_p : moment at tie point (kN·m/m)

iii) Verification of the embedded length

Table 2.26 shows the verification results for the embedded length. The partial factors related to the embedded length of sheet piles (γ_R and γ_S) shall be 1.0 and 1.0, respectively, since cohesive soil is partly included in the soil composition from the ground surface to the bottom of the embedded length. The adjustment factor (m) shall be 1.20.

Table 2.26- Verification Results for Embedded Length (Variable State: Level 1 Earthquake Ground Motion)

m = 1.20

Layer (m)	Ma (kN·m/m)	γ_s	$m.S_d$ (kN·m/m)	Mp (kN·m/m)	γ_R	R_d (kN·m/m)	
-12.60	6,956.933	1.0	8,348.318	0.000	1.0	0.000	NO
-17.50	15,317.673	1.0	18,381.206	8,316.692	1.0	8,316.692	NO
-22.60	31,232.862	1.0	37,479.433	28,725.914	1.0	28,725.914	NO
-24.50	39,513.613	1.0	47,416.334	38,587.169	1.0	38,587.169	NO
-26.26	40,149.110	1.0	48,178.930	57,508.025	1.0	57,508.025	OK

The above calculation therefore shows that the required embedded length for a sheet-pile wall is in the range from -24.50 to -26.260 m.

Then, performance verification will be conducted with the bottom of embedded length being -25.378 m.

4) Verification of the Embedded Length of Sheet Pile using Rowe's Method

The following is verification of the embedded length of the sheet-pile wall using Rowe's method.

Table 2.27- Calculation Results for δ_S, ρ, and ω (Variable State: Level 1 Earthquake Ground Motion)

Item	Unit	value
Type of sheet pile		D800×t12
Young's modulus (E)	MN/m^2	2.00E+5
Geometrical moment of inertia of cross-section (I)	m^4/m	2.64E-03
(EI)	MN/m^2/m	528

Embedded length of sheet pile (D_F) (free earth support method)	m	12.778
Sheet-pile wall height (H_T)	m	14.100
$\delta_s=D_F/H_T$		0.9062
$\rho=H_T^4/EI$	m^3/MN	74.859
l_h	MN/m^3	28
$\omega=\rho\times l_h$		2,096.052

$$\delta_S = D_F/H_T = 0.9062 \ \geq \ 5.0916 \times (2{,}096.052)^{-0.2} - 0.2591 = 0.8439 \quad \text{O.K}$$

The above calculation presents that the embedded length by the free earth support method satisfies the requirement when verified using Rowe's method.

(2) Performance Verification of Stress of Sheet Pile Wall

1) Moment and Reaction Force by Equivalent Beam Method

Table 2.28 presents the calculation results for the earth pressures and residual water pressure in the Level 1 earthquake ground motion.

The calculation results of Table 2.28 are presented in Figure 2.11.

Table 2.28- Calculation Results for the Earth Pressures, Residual Water Pressure, and Dynamic Water Pressure (Variable State: Level 1 Earthquake Ground Motion)

Layer (m)	$Pa + Pw + p_{dw}$ (kN/m²)	Pp (kN/m²)	$Pa + Pw + p_{dw} - Pp$ (kN/m²)
3.50	3.664	———	3.664
1.50	12.459	———	12.459
1.50	12.459	———	12.459
1.33	13.207	———	13.207
1.33	14.775	———	14.775
0.00	31.843	———	31.843
0.00	31.843	———	31.843
-1.00	37.713	———	37.713
-1.00	37.713	———	37.713
-2.00	41.745	———	41.745
-2.00	41.745	———	41.745
-3.00	45.475	———	45.475
-3.00	45.475	———	45.475
-4.00	49.049	———	49.049
-4.00	49.049	———	49.049
-5.00	52.523	———	52.523
-5.00	52.523	———	52.523
-6.00	55.925	———	55.925
-6.00	55.925	———	55.925
-7.00	59.274	———	59.274
-7.00	59.274	———	59.274
-8.00	62.580	———	62.580
-8.00	62.580	———	62.580
-9.00	65.850	———	65.850
-9.00	65.850	———	65.850
-10.00	69.092	———	69.092
-10.00	91.853	———	91.853
-11.00	96.430	———	96.430
-11.00	96.430	———	96.430
-12.00	100.986	———	100.986

-12.00 -12.60	100.986 103.710	——— ———	100.986 103.710
-12.60 -17.50	92.575 112.631	0.000 195.461	92.575 -82.830
-17.50 -22.60	139.415 149.793	169.000 201.130	-29.585 -51.337
-22.60 -24.50	167.923 179.893	201.130 213.100	-33.207 -33.207
-24.50 -26.26	13.433 13.433	393.100 406.640	-379.667 -393.207
-26.26 -50.00	13.433 196.243	406.640 589.450	-393.207 -393.207

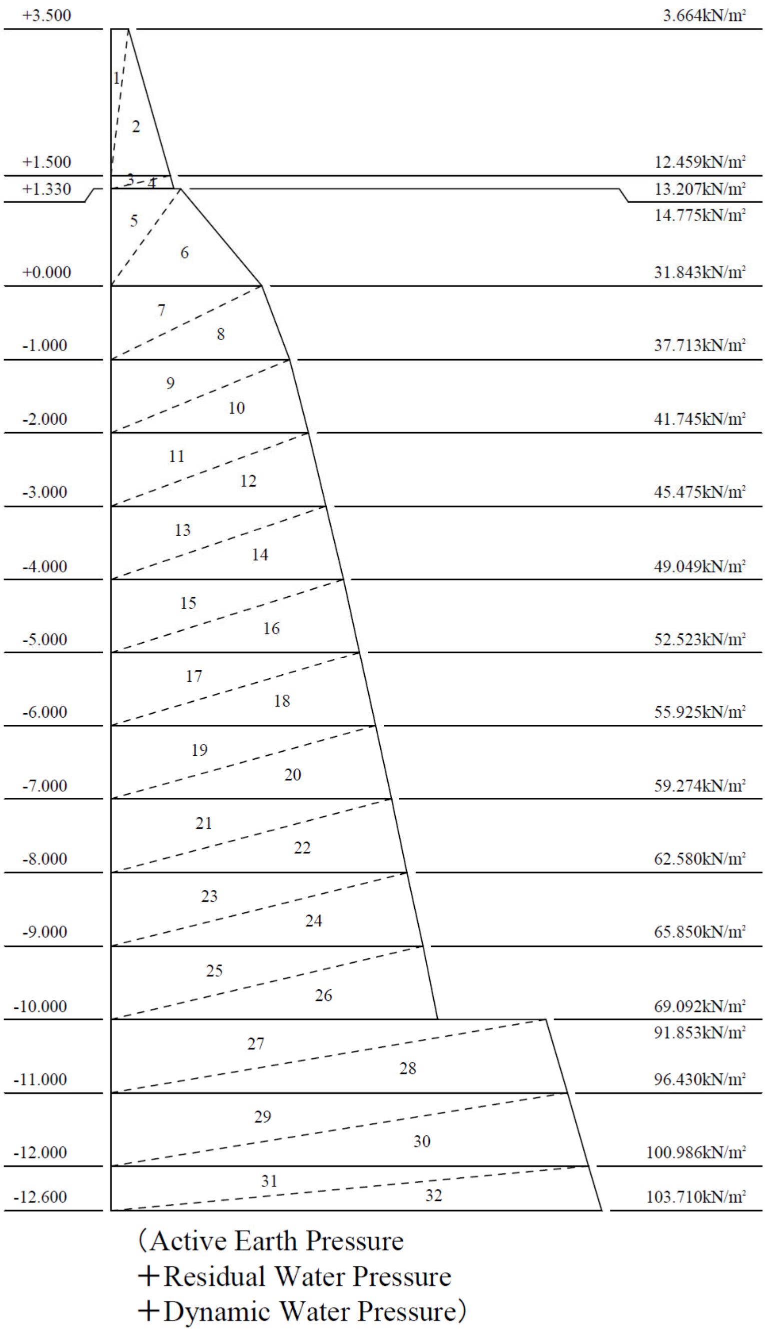


Figure 2.11- Distribution of the Active Earth Pressure, Residual Water Pressure, and Dynamic Water Pressure (Variable State: Level 1 Earthquake Ground Motion)

i) Moment at the tie member installation point

Table 2.29 presents the calculation results for the moment at the tie member installation point.

Table 2.29- Moment at the Tie Member Installation Point (Variable State: Level 1 Earthquake Ground Motion)

No.	Formula	S_a (kN/m)	l (m)	M_a (kN·m/m)
1	1/2× 3.664× 2.000	3.664	-1.333	-4.884
2	1/2× 12.459× 2.000	12.459	-0.667	-8.310
3	1/2× 12.459× 0.170	1.059	0.057	0.060
4	1/2× 13.207× 0.170	1.123	0.113	0.127
5	1/2× 14.775× 1.330	9.925	0.613	6.023
6	1/2× 31.843× 1.330	21.176	1.057	22.383
7	1/2× 31.843× 1.000	15.922	1.833	29.185
8	1/2× 37.713× 1.000	18.856	2.167	40.861
9	1/2× 37.713× 1.000	18.856	2.833	53.419
10	1/2× 41.745× 1.000	20.872	3.167	66.102
11	1/2× 41.745× 1.000	20.872	3.833	80.002
12	1/2× 45.475× 1.000	22.738	4.167	94.749
13	1/2× 45.475× 1.000	22.738	4.833	109.893
14	1/2× 49.049× 1.000	24.524	5.167	126.716
15	1/2× 49.049× 1.000	24.524	5.833	143.048
16	1/2× 52.523× 1.000	26.262	6.167	161.958
17	1/2× 52.523× 1.000	26.262	6.833	179.448
18	1/2× 55.925× 1.000	27.962	7.167	200.404
19	1/2× 55.925× 1.000	27.962	7.833	219.026
20	1/2× 59.274× 1.000	29.637	8.167	242.045
21	1/2× 59.274× 1.000	29.637	8.833	261.784
22	1/2× 62.580× 1.000	31.290	9.167	286.835
23	1/2× 62.580× 1.000	31.290	9.833	307.675
24	1/2× 65.850× 1.000	32.925	10.167	334.748
25	1/2× 65.850× 1.000	32.925	10.833	356.677
26	1/2× 69.092× 1.000	34.546	11.167	385.775
27	1/2× 91.853× 1.000	45.926	11.833	543.442
28	1/2× 96.430× 1.000	48.215	12.167	586.632
29	1/2× 96.430× 1.000	48.215	12.833	618.743
30	1/2× 100.986× 1.000	50.493	13.167	664.841
31	1/2× 100.986× 0.600	30.296	13.700	415.055
32	1/2× 103.710× 0.600	31.113	13.900	432.471
Total		824.164	———	6,956.933

Where:

S_a : horizontal force (kN/m)

l : distance from tie member installation point (m)

M_a : moment at tie point (kN·m/m)

ii) Reaction force at the tie member installation point

Reaction at the support on the seabed surface: R_0

$R_0 = \sum M_a \div l = 6{,}956.933 \div 14.10 = 493.400$ kN/m

Reaction force at the tie member installation point: A_P

$A_P = \sum S_a - R_0 = 824.164 - 493.400 = 330.764$ kN/m

iii) Maximum bending moment of sheet-pile wall

The maximum bending moment acting on the sheet-pile wall occurs at a location where shear force Q is zero. The shear force is calculated using $Q = A_P - \sum P$, giving the position of Q = zero as -6.197 m

The calculation results are shown in Table 2.30.

Table 2.30- Shear force Q = 0 Position (Variable State: Level 1 Earthquake Ground Motion)

Layer (m)	Force P (kN/m)	ΣP (kN/m)	Tie Force A_P (kN/m)	Shear Force Q (kN/m)
3.50 1.50	3.664 12.459	16.123	330.764	314.641
1.50 1.33	1.059 1.123	18.305	330.764	312.459
1.33 0.00	9.825 21.176	49.306	330.764	281.458
0.00 -1.00	15.922 18.856	84.084	330.764	246.680
-1.00 -2.00	18.856 20.872	123.812	330.764	206.952
-2.00 -3.00	20.872 22.738	167.422	330.764	163.342
-3.00 -4.00	22.738 24.524	214.684	330.764	116.080
-4.00 -5.00	24.524 26.262	265.470	330.764	65.294
-5.00 -6.00	26.262 27.962	319.694	330.764	11.070
-6.00 -7.00	27.962 29.637	377.293	330.764	-46.529
-7.00 -8.00	29.637 31.290	438.220	330.764	-107.456
-8.00 -9.00	31.290 32.925	502.435	330.764	-171.671
-9.00 -10.00	32.925 34.546	569.906	330.764	-239.142
-10.00 -11.00	45.926 48.215	664.047	330.764	-333.283
-11.00 -12.00	48.215 50.493	762.755	330.764	-431.991
-12.00 -12.60	30.296 31.113	824.164	330.764	-493.400

The bending moment related to the position of Q = 0 for the earth pressure and residual water pressure from the quaywall top, or +3.50 m, to -6.197 m, is calculated as in Table 2.31.

Table 2.31- Bending Moment at Shear Force Q = 0 Position (Variable State: Level 1 Earthquake Ground Motion)

No.	Formula	S (kN/m)	l (m)	M (kN・m/m)
1	1/2× 3.664× 2.000	-3.664	9.030	-33.086
2	1/2× 12.459× 2.000	-12.459	8.364	-104.207
3	1/2× 12.459× 0.170	-1.059	7.640	-8.091
4	1/2× 13.207× 0.170	-1.123	7.584	-8.517
5	1/2× 14.775× 1.330	-9.825	7.084	-69.600
6	1/2× 31.843× 1.330	-21.176	6.640	-140.609
7	1/2× 31.843× 1.000	-15.922	5.864	-93.367
8	1/2× 37.713× 1.000	-18.856	5.530	-104.274
9	1/2× 37.713× 1.000	-18.856	4.864	-91.716

10	1/2× 41.745× 1.000	-20.872	4.530	-94.550
11	1/2× 41.745× 1.000	-20.872	3.864	-80.649
12	1/2× 45.475× 1.000	-22.738	3.530	-80.265
13	1/2× 45.475× 1.000	-22.738	2.864	-65.122
14	1/2× 49.049× 1.000	-24.524	2.530	-62.046
15	1/2× 49.049× 1.000	-24.524	1.864	-45.713
16	1/2× 52.523× 1.000	-26.262	1.530	-40.181
17	1/2× 52.523× 1.000	-26.262	0.864	-22.690
18	1/2× 55.925× 1.000	-27.962	0.530	-14.820
19	1/2× 55.925× 0.197	-5.509	0.131	-0.722
20	1/2× 56.585× 0.197	-5.574	0.066	-0.368
Total		————	————	-1,160.593

The calculation shown in Table 2.31 gives the bending moment at the position in the sheet-pile wall with shear force $Q = 0$ (–6.197 m) as follows:

Distance from the tie member installation point to the zero shear-force point

$h = 1.500 - (-6.197) = 7.697$ m

Maximum bending moment

$M_{a(Q=0)} = \Sigma M_a = A_P \times h - \Sigma M = 330.764 \times 7.697 - 1{,}160.593 = 1{,}385.298$ kN·m/m

2) Moment and Reaction Force by Rowe's Method

Figure 2.12 and Table 2.32 show the calculation results for the maximum bending moment and the reaction force at the tie member installation point using the virtual beam method.

Table 2.32- Maximum Moment and Reaction Force at the Tie Member Installation Point (Variable State: Level 1 Earthquake Ground Motion)

Item	Sign	Unit	Variable state
Maximum bending moment	M_{max}	kN·m/m	1,385.298
Location of occurrence		DL.m	-6.197
Reaction force at tie member installation point	Ap	kN/m	330.764

Given the above results, the maximum bending moment and the reaction force at the tie member installation point are corrected using Equation (2.7) and Equation (2.8) as follows:

Variable state, correction of the maximum bending moment:

$$\mu_S = M_F/M_T = 4.5647 \times \omega^{-0.2} + 0.1329 \qquad (2.7)$$

Variable state, correction of the reaction force at the tie member installation point:

$$\tau_S = T_F/T_T = 2.3174 \times \omega^{-0.2} + 0.5514 \qquad (2.8)$$

Where:

μ_S : correction factor for the maximum moment (variable state)
M_F : maximum bending moment after correction
M_T : maximum bending moment before correction

τ_S : correction factor for the reaction force at the tie member installation point (variable state)

T_F : reaction force at the tie member installation point after correction

T_T : reaction force at the tie member installation point before correction

ω : similarity number $(\rho \times l_h)$

$\omega = 74.859 \times 28.0 = 2{,}096.052$

The correction results are given as follows:

Correction factor for maximum bending moment: $\mu_s = 1.1218$

$M_F = 1.1218 \times 1{,}385.298 = 1{,}554.027$ kN·m/m

Correction factor for the reaction force at the installation point: $\tau_s = 1.0534$

$T_F = 1.0534 \times 330.764 = 348.427$ kN/m

The above corrected values will be used for verification of the sheet-pile stress, the tensile stress of tie members, the stress of waling, and the anchorage.

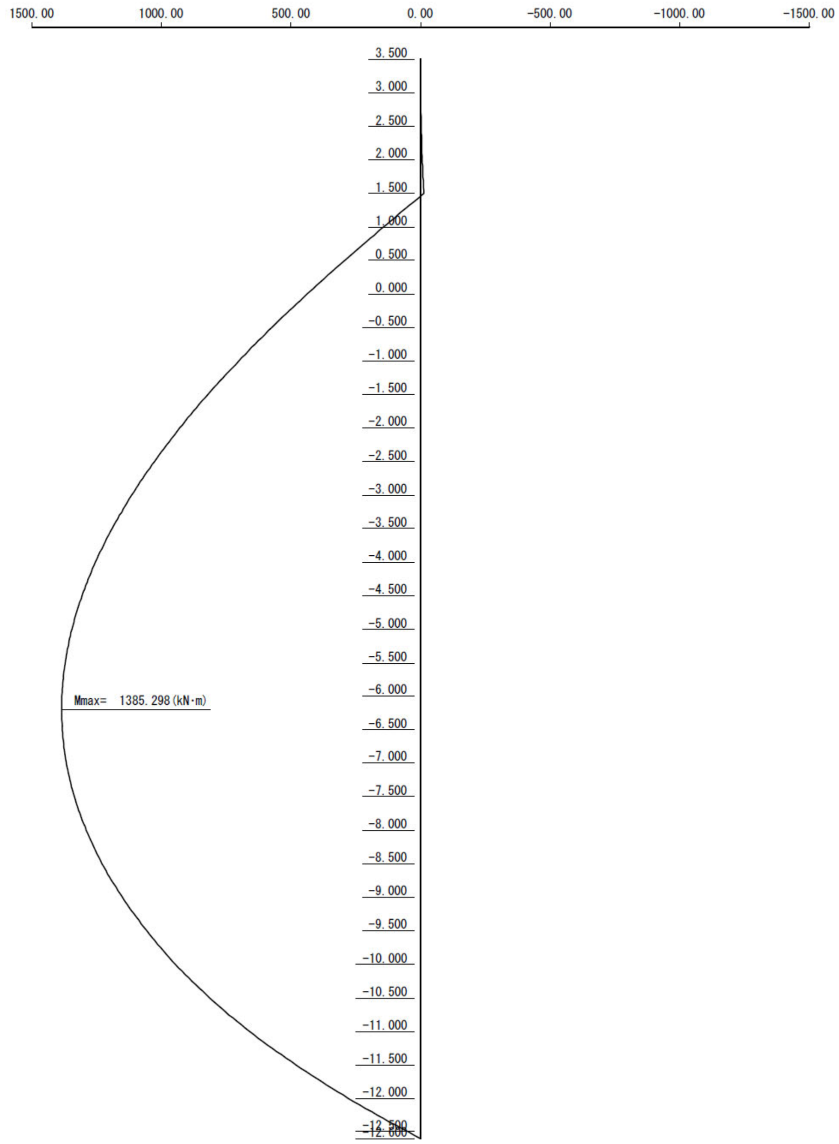


Figure 2.12- Maximum Bending Moment (Variable State: Level 1 Earthquake Ground Motion)

3) Performance Verification of Stress of Sheet Piles

i) Cross-sectional properties of sheet piles

The cross-sectional properties of steel-pipe sheet pile shown in Table 2.33 as follows:

Table 2.33: Specifications of Steel-pipe Sheet Pile

Item	Unit	Value	Remarks
Type of sheet pile		D800 x t12	
Material		SPSP490	
Section modulus (Zo)	cm^3/m	6,590	Before corrosion
Section modulus (Z)	cm^3/m	6,084	After corrosion
Bending yield stress of steel (σ_{yd})	N/mm^2	315.0	

Corrosion allowance for steel-pipe sheet pile (corrosion rate μ: 90%)

$1-\mu=0.1$

Seaside: t_1= 0.100 mm/year × 0.1 × 50 years = 0.50 mm

Landside: t_2 = 0.020 mm/year × 50 years = 1.00 mm

The cross-sectional properties of steel-pipe sheet pile after corrosion as shown above are the result of calculations done with 0.75 mm given as the average corrosion allowance.

ii) Performance Verification of Stress of Sheet Pile Wall

$$m \cdot S_d / R_d \leq 1.0,\ R_d = \gamma_R \cdot R_k,\ S_d = \gamma_s \cdot S_k$$
$$R_k = \sigma_{yk}$$
$$S_k = M_{max\,k} / Z$$

Where:

σ_y : bending yield stress of the steel material (N/mm^2)

M_{max} : maximum bending moment in the sheet-pile wall (N·mm/m)

Z : section modulus of steel material (mm^3/m)

R : resistance term(kN/m)

S : load term (kN/m)

γ_R : partial factor multiplied by resistance term (=1.0)

γ_s : partial factor multiplied by load term (=1.0)

m : adjustment factor (=1.12)

The verification result for bending stress of a sheet pile is shown below:

$$m \cdot \frac{S_d}{R_d} = m \cdot \frac{\gamma_S S_k}{\gamma_R R_k} = 1.12 \times \frac{1.0 \times 1{,}554.027 \times 10^6 / (6{,}084 \times 10^3)}{1.0 \times 315.0} = 0.908 \leq 1.0$$

(3) Performance Verification of Tie Members

1) Tie Member Specifications

The specifications of tie member are presented in Table 2.34 as follows:

Table 2.34- Specifications of Tie Rod (Variable State: Level 1 Earthquake Ground Motion)

Item	Unit	Value
Type of tie member		Tie Rod
Material		High Tension 690
Yield stress of steel (σ_{yk})	N/mm^2	440.0
Corrosion rate (Δd)	mm	3.0

2) Tension Force of Tie Rod

The tension force of the tie rod is calculated by the following Equation:

$$T_k = A_{pk}\, l \sec\theta$$

Where:

T	:	tension force of the tie member (kN)
A_p	:	reaction at the tie member installation point (kN) (=348.427 kN/m)
l	:	tie member installation interval (m)
θ	:	inclination angle of the tie member to the line perpendicular to the sheet pile wall (°)

The calculation result for tie rod tension force T is shown as follows:

$T_d = 348.427 \times 2.321 \times \sec(0.0°) = 808.699$ kN/pcs

As reference, the calculation result for tie rod tension force T at variable state (mooring condition) is shown as follows:

$T_d = (269.619 \times 2.321 + 700.0 / 4) \times \sec(0.0°) = 800.786$ kN/pcs
< 808.699 kN/pcs

3) Verification of the Stress of Tie Members

$$m \cdot S_d / R_d \leq 1.0,\ R_d = \gamma_R \cdot R_k,\ S_d = \gamma_s \cdot S_k$$
$$R_k = \sigma_{yk}$$
$$S_k = T_k / A$$

Where:

σ_y	:	tensile yield stress of a tie member (N/mm^2)
T	:	tension force on a tie member (N)
A	:	cross section area of a tie member (mm^2)
R	:	resistance term (N/mm^2)
S	:	the load term (N/mm^2)
γ_R	:	partial factor multiplied by resistance term (=1.0)
γ_s	:	partial factor multiplied by load term (=1.0)
m	:	adjustment factor (=1.67)

The required diameter of the tie rod is calculated as follows:

$$d = 2\times\sqrt{\frac{m\cdot\gamma_s\cdot T_k}{\pi\cdot\gamma_R\cdot\sigma_{yk}}} + \Delta d = 2\times\sqrt{\frac{1.67\times1.0\times808.699\times10^3}{\pi\times1.0\times440.0}} + 3.0 = 65.51\text{mm}$$

Therefore, the diameter of the tie rod is assumed to be 70mm.
The verification results for the stress of tie rod are then given as follows:

$$m\cdot\frac{S_d}{R_d} = m\cdot\frac{\gamma_S S_k}{\gamma_R R_k} = 1.67\times\frac{1.0\times808.699\times10^3/3{,}525.65}{1.0\times440} = 0.871 \leq 1.0 \quad \text{O.K}$$

(4) Performance Verification of Waling Member

1) Waling Member Specifications

The specifications of waling member are shown in Table 2.35.

Table 2.35- Waling Specifications (Variable State)

Item	Unit	Value
Type of waling (channel steel)		2[-300×90×12.0×16.0
Material		SS400
Section modulus (Z) (after corrosion)	cm^3	525.0
Bending yield stress of steel (σ_{yk})	N/mm^2	235.0
Tie rod installation interval (L)	m	2.321

2) Maximum Bending Moment

Calculate the maximum bending moment acting on waling $M_{max\,k}$ using the following Equation:

$$M_{max\,k} = T_k\, l / 10$$

Where:

M_{max} : maximum bending moment in the waling (N·mm/m)
T : tension force of a tie member (kN)
l : tie member installation interval (m)

The calculation results for the maximum bending moment acting on waling $M_{max\,k}$ are shown below:

$M_{max\,k} = 808.699 \times 2.321 \div 10 = 187.699$ kN·m

The calculation results for the maximum bending moment acting on waling $M_{max\,k}$ at variable state (mooring condition) are shown below:

$M_{max\,k} = 800.786 \times 2.321 \div 10 = 185.862$ kN·m < 187.699 kN·m

3) Verification of the Stress of Waling

$$m \cdot S_d / R_d \leq 1.0,\ R_d = \gamma_R \cdot R_k,\ S_d = \gamma_s \cdot S_k$$
$$R_k = \sigma_{yk}$$
$$S_k = M_{max\,k} / Z$$

Where:

σ_y : bending yield stress in the waling (N/mm^2)
M_{max} : maximum bending moment in the waling (N·mm/m)
Z : section modulus of the waling (mm^3)
R : resistance term (N/mm^2)
S : the load term (N/mm^2)
γ_R : partial factor multiplied by resistance term (=1.0)
γ_s : partial factor multiplied by load term (=1.0)
m : adjustment factor (=1.12)

The verification results for the stress of waling (channel section) are shown below:

$$m \cdot \frac{S_d}{R_d} = m \cdot \frac{\gamma_S S_k}{\gamma_R R_k} = 1.12 \times \frac{1.0 \times 187.699 \times 10^6 / (2 \times 525.0 \times 10^3)}{1.0 \times 235.0} = 0.852 \leq 1.0 \text{ O.K}$$

(5) Performance Verification of Anchorage

1) Anchorage Specifications

The specifications of anchorage members are shown in Table 2.36.

Table 2.36- Anchorage Specifications and Tie Rod Tension (Variable State)

Item	Unit	Value	Remark
Anchorage installation height	m	+2.50	
Tie rod installation height	m	+1.50	
Type of anchorage (steel-pipe pile)		D800×t9.0	
Effective width of anchorage (*B*)	mm	800	
Material		SPP490	
Young's modulus (*E*)	kN/m^2	200	
Geometrical moment of inertia (*Io*)	cm^4	175,000	Before corrosion
Geometrical moment of inertia (*I*)	cm^4	154,909	After corrosion
Section modulus (*Zo*)	cm^3	4,370	Before corrosion
Section modulus (*Z*)	cm^3	3,882	After corrosion
Tie rod tension force (*T*)	kN	808.699	

Note: Corrosion allowance for steel-pipe pile

$t = 1 \times 0.020$ mm/year $\times$ 50 years = 1.00 mm

2) Lateral resistance constant k_c

The soil condition where anchor piles are installed is sandy soil, and the N- value is considered to be constant in the depth direction. Therefore, it is taken as C-type ground.

Average N-value: 10

Calculate the lateral resistance constant using Figure. 2.13.

$$k_c = 540N^{0.648} = 540 \times 10^{0.648} = 2{,}401 \text{ kN/m}^{2.5}$$

3) Maximum Bending Moment, Displacement, and Embedded Length

The maximum bending moment, displacement, and embedded length calculated using the "Koken Method (PHRI Method)" are shown in Table 2.37.

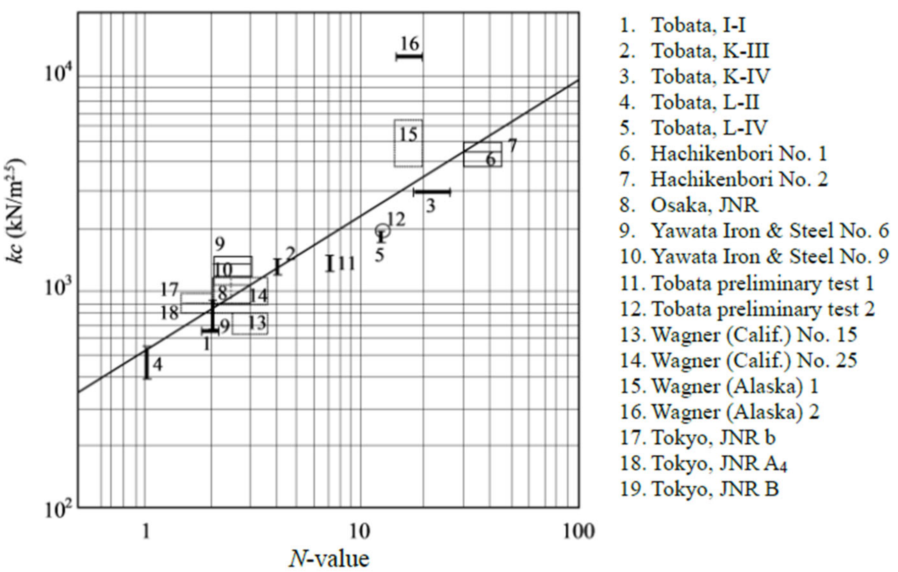


Figure 2.13- Relationship between N-value and *kc*

Table 2.37- Anchor Pile Design Values (Variable State: Level 1 Earthquake Ground Motion)

Item	Symbol	After corrosion	Unit
Anchor Pile displacement	Y_{top}	4.270	cm
Ground surface displacement	Y_0		cm
Pile top moment	M_{top}	0.000	kN·m
Maximum underground moment	M_{max}	968.608	kN·m
Depth of moment $M = 0$	l_{m1}	8.492	m
Angle of deflection at pile top	i_{top}		rad
Angle of deflection on ground surface	i_0		rad
$l_{m1}/3$		2.831	m
$1.5\times l_{m1}$		12.738	m

4) Verification of the Stress of Anchor Pile

Use the following Equation to verify the stress of an anchor pile for the result after corrosion.

$$m\cdot S_d/R_d \leq 1.0,\ R_d = \gamma_R\cdot R_k,\ S_d = \gamma_S\cdot S_k$$

$$R_k = \sigma_{yk}$$

$$S_k = M_{max\,k}/Z$$

Where:

- σ_y : bending yield stress of a pile anchorage (N/mm^2)
- M_{max} : maximum bending moment in a pile anchorage (N·mm/m)
- Z : section modulus of a pile anchorage (mm^3/m)
- R : resistance term (N/mm^2)
- S : the load term (N/mm^2)
- γ_R : partial factor multiplied by resistance term (=1.0)
- γ_s : partial factor multiplied by load term (=1.0)
- m : adjustment factor (=1.12)

The verification results for the stress of anchor pile are shown below:

$$m\cdot\frac{S_d}{R_d}=m\cdot\frac{\gamma_S S_k}{\gamma_R R_k}=1.12\times\frac{1.0\times968.608\times10^6/(3{,}882\times10^3)}{1.0\times315.0}=0.887 \leq 1.0\ \text{O.K}$$

5) Bottom Level of Anchor Pile

The bottom level of anchor pile using the pre-corrosion result is as follows:

Bottom level of the anchor pile = height of tie rod installation – 1.5 × l_{m1}
= +1.50 -12.738 = -11.238 m = -11.50m (variable state)

6) Anchorage Installation Position

Place a vertical anchor pile at a location where the active failure plane of the front sheet pile drawn from the seabed surface does not intersect the passive failure plane of the anchorage drawn from the position of $l_{m1}/3$ down the anchorage-side tie rod installation point to the tie rod installation height.

- Foundation ground height (design height): -12.60 m
- Tie rod installation height (DL): +1.50 m
- Anchorage $l_{m1}/3$ length: 2.831m

The anchorage installation position is determined by the angle of failure surface of each layer. The distance between sheet pile quaywall and the anchorage pile is minimum 18.998m = 19.0m.

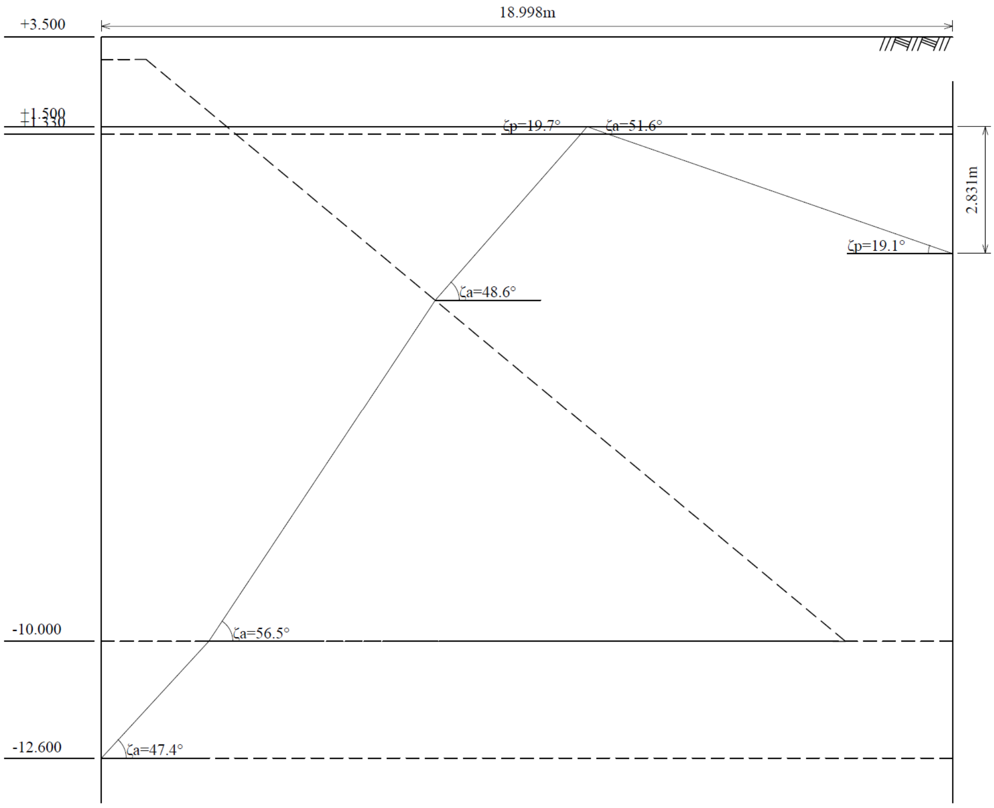


Figure 2.14- Anchorage Installation Position (Variable State: Level 1 Earthquake Ground Motion)

- End -