**Part 5    Dolphin (Draft Final)**

**1. Technical Notes**

**1-1. Characteristics of Dolphins**

    Dolphins are facilities that divide and collaborate on the functions typically possessed by pier structures, which include berthing, mooring, and cargo handling functions. The verification of dolphin structures includes the application of the situations during various actions, including when ship berthing, mooring, and during cargo handling operations, as well as during storms and Level 1 earthquake ground motion. Since there are many examples of dolphins that are accompanied by LNG and oil facilities, when considering accidental situations associated with Level 2 earthquake ground motion, the extent of damage should be verified using dynamic analysis.

    Since there is only a 500-year return period earthquake in Vietnam, the designer must thoroughly consider the earthquake motion or performance to be applied.

    In addition, depending on the type of cargo handled, the port facilities shall also satisfy performance requirements stipulated by relevant regulations other than the port facility standards described herein. Therefore, when undertaking specific design tasks, it is necessary to confirm whether the facility is subject to such additional regulatory requirements.

    This casebook will introduce piled-type dolphins among structural types such as piled-type, cell-type, and caisson-type structures. The typical facilities that make up the dolphins, including the Breasting Dolphin (BD), Mooring Dolphin (MD), and cargo handling dolphin (Working Platform: WP), will be introduced. Then, as an example of the verification of each type of dolphin, a performance verification during variable actions will be presented for BD.

**1-2. Basic Policy for Performance Verification**

**(1)  Verification Procedures**

    When performing a performance verification of dolphins, it is necessary to determine the layout of individual facilities. The layout of breasting and mooring dolphins is set as follows:
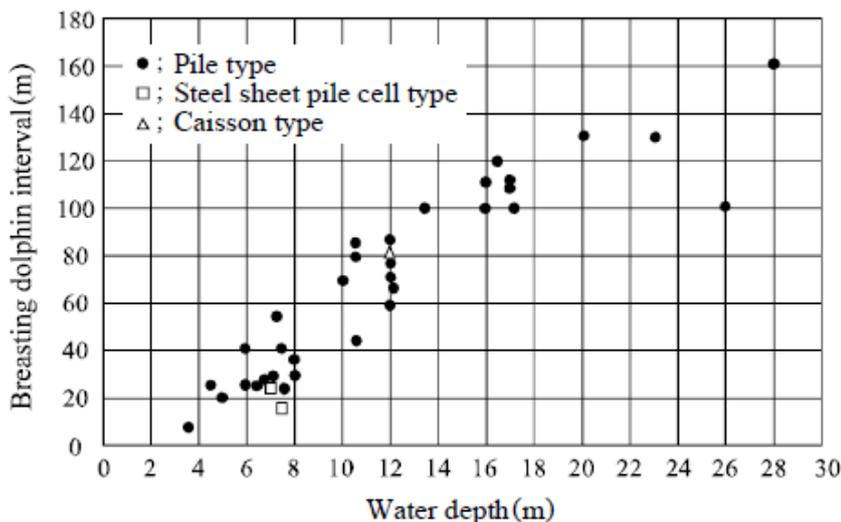
  **1)  Layout of Dolphins**

    When special cargo handling equipment is required for oil handling, a cargo handling platform dolphin is installed in the middle of the breasting dolphins. In this case, it is preferable to place the working platform slightly behind the breasting dolphin so that the berthing force is not directly applied to the working platform.

    Dolphins are designed so that their longitudinal direction is parallel to the prevailing direction of wind, waves and currents. This arrangement makes it easier to maneuver the ship during berthing and reduces the external forces acting on the dolphin during mooring. It is also advantageous to position the dolphin so that the planned depth can be secured within the existing water depth.

  **2)  Breasting Dolphin (BD)**

    A typical ship has curved sections at the bow and stern that are approximately 1/8 of the ship's overall length $L$, and approximately 3/4 of the central section is straight. It is desirable to install dolphins on these straight sections to improve berthing when coming alongside. The wider the spacing between breasting dolphins, the more advantageous it is

for berthing force, but it also makes parallel berthing more difficult and there is less room to move around to adjust position after coming alongside. Conversely, if the spacing is too narrow, the ship's center of gravity will be closer to the breasting dolphins, which is disadvantageous for berthing force. Therefore, the best dolphin spacing is considered to be approximately 1/3 of the ship's overall length. The distance between breasting dolphins is closely related to the overall length ($L$) of the design ships. Figure 1.1 presents the relationship between the breasting dolphin interval and the water depth derived from past construction data for reference.
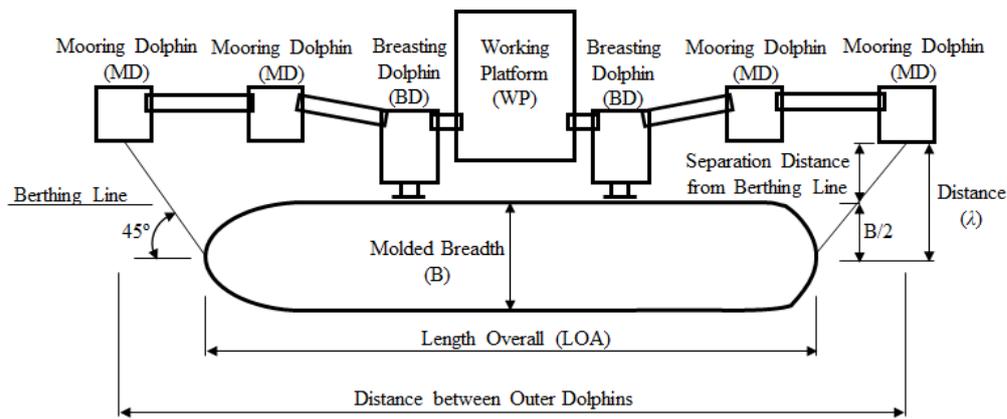
Source: TCVN 11820-5-2021

**Figure 1.1- Distance between Breasting Dolphins**

### 3) Mooring Dolphin (MD)

Mooring dolphins are generally arranged with an outer dolphin for the mooring lines (head line and stern line) from the bow and stern sides, and an inner dolphin for the breast line and spring line. The positioning of each dolphin is set as follows.

Mooring dolphins are normally set at a 45° angle from the rope bitts on the bow and stern of a ship and with a certain setback from the front face of the breasting dolphins. The number of mooring dolphins is decided on the basis of the tractive force of ships, but two to four units are usually adopted based on previous construction examples. It is also possible to install mooring posts on breasting dolphins and use them for the mooring of design ships and small ships.

As shown in Figure 1.2, the distance and angle from the mooring position to the MD are taken into consideration.

✓ Distance ($\lambda$): B/2 + Separation distance from berthing line
✓ Distance between outer dolphins: Length overall (LOA) +$\lambda$/tan45×2

When mooring at an angle smaller than 45°, the stability during mooring and dolphins arrangement shall be verified by mooring analysis.

**Figure 1.2- Arrangement of Mooring Dolphins**

### 4) Working Platform (WP)

Working platform (WP) is a facility primarily used for loading and unloading cargo and will not fall under the category of "port facilities" unless the facility is also equipped with berthing or mooring functions. However, here it is considered as part of a series of port facilities and is included as a design example.

### 5) Catwalk

The dolphins are facilities specialized for individual functions, and each facility is connected by catwalks. The reaction forces of the catwalk's supports act on each dolphin as vertical and horizontal loads.

## 1-3. Performance Verification
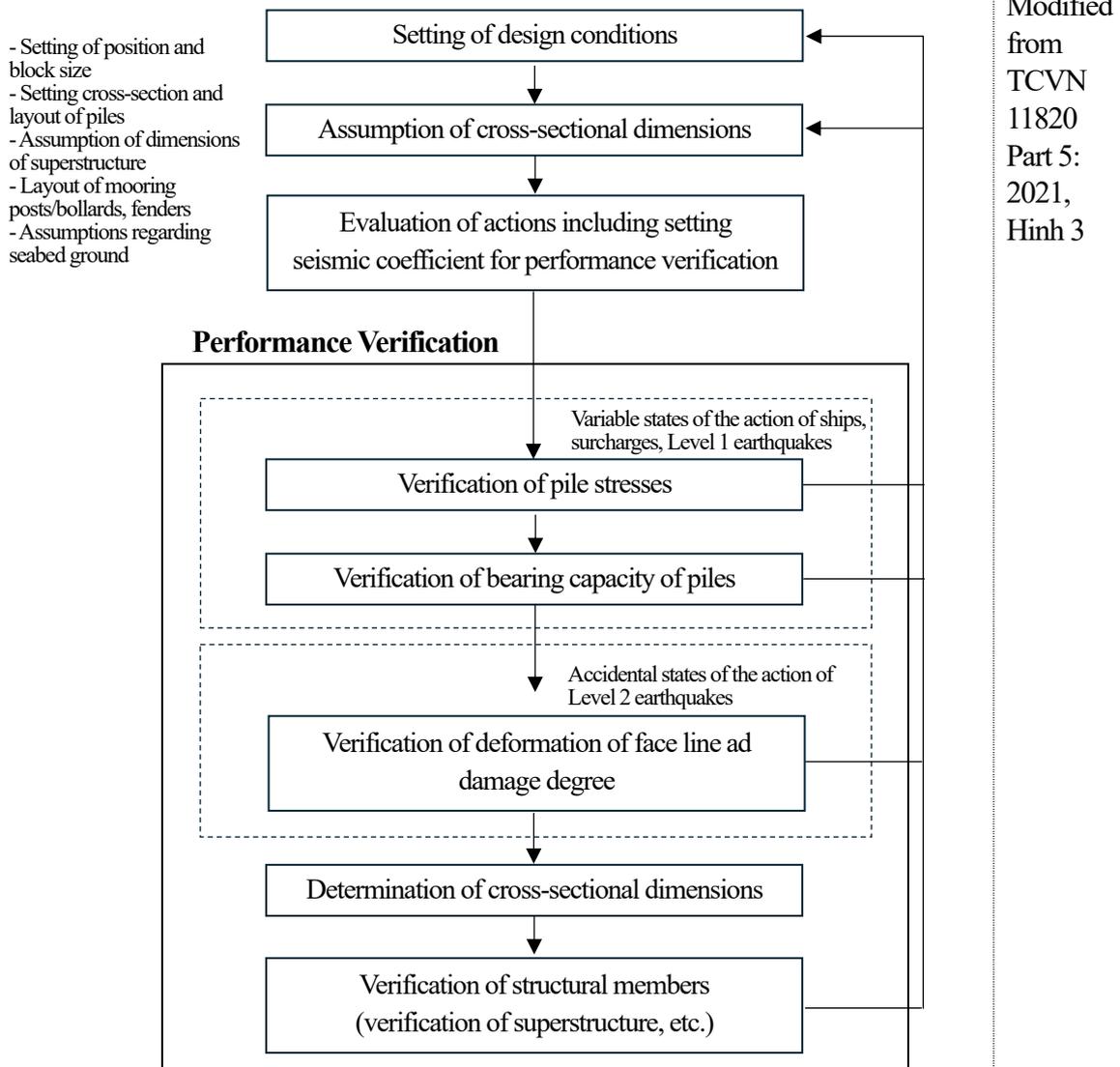### (1) Verification Items and Methods

The classification of design situation and verification items for the performance verification of dolphins shall be as shown in Table 1.1, in accordance with the verification contents for Part 2 Open-type Wharves on Piles.

**Table 1.1- Verification Items**

| Design situation | Cross-sectional failure of superstructure | Fatigue failure of superstructure | Yeilding of piles (Pile stress) | Axial forces in piles (Bearing capacity) | Deformation of face line |
|---|---|---|---|---|---|
| Variable situation (Berthing, tractive force by ships, Level 1 earthquake ground motion) | ○ | | ○ | ○ | |
| Variable situation (Surcharge load) | ○ | | ○ | | |
| Variable situation (Repeated surcharge load) | ○ | ○ | ○ | ○ | |
| Variable situation (Variable waves) | ○ | ○ | | ○ | |
| Accidental situation (Level 2 earthquake ground motion | ○ | | ○ | ○ | ○ |

Part 1 Performance-based Design Approach includes classification of Level 1 and Level 2 earthquake ground motions.

Structural verification of individual dolphins is carried out following the procedure shown in Figure 1.3, although there are differences in the loads subject to verification.

- Setting of position and block size
- Setting cross-section and layout of piles
- Assumption of dimensions of superstructure
- Layout of mooring posts/bollards, fenders
- Assumptions regarding seabed ground

```
┌─────────────────────────────────────┐
│     Setting of design conditions     │◄────┐
└─────────────────────────────────────┘     │
                  │                          │
                  ▼                          │
┌─────────────────────────────────────┐     │
│  Assumption of cross-sectional dimensions │◄────┤
└─────────────────────────────────────┘     │
                  │                          │
                  ▼                          │
┌─────────────────────────────────────┐     │
│  Evaluation of actions including setting  │    │
│  seismic coefficient for performance      │    │
│  verification                             │    │
└─────────────────────────────────────┘     │
```

**Performance Verification**

Variable states of the action of ships, surcharges, Level 1 earthquakes

```
┌─────────────────────────────────────┐
│      Verification of pile stresses    │───┐
└─────────────────────────────────────┘   │
                  │                        │
                  ▼                        │
┌─────────────────────────────────────┐   │
│ Verification of bearing capacity of piles │───┤
└─────────────────────────────────────┘   │
```

Accidental states of the action of Level 2 earthquakes

```
┌─────────────────────────────────────┐
│ Verification of deformation of face line ad │───┤
│            damage degree                   │   │
└─────────────────────────────────────┘   │
                  │                        │
                  ▼                        │
┌─────────────────────────────────────┐   │
│ Determination of cross-sectional dimensions │  │
└─────────────────────────────────────┘   │
                  │                        │
                  ▼                        │
┌─────────────────────────────────────┐   │
│    Verification of structural members     │──┘
│    (verification of superstructure, etc.) │
└─────────────────────────────────────┘
```

Note: Evaluation of the effect of liquefaction and settlement is not shown in the diagram; therefore, it is necessary these effects separately.

Source: Modified from TCVN 11820-5-2021

**Figure 1.3- Flow Chart of Performance Verification**

The performance verification of dolphins should be performed by considering the following items. For other items, it is preferable to appropriately perform performance verification in accordance with each structural form.

✓ The direction of actions on dolphins is not necessarily a constant direction; hence, the verification should be performed for several directions as necessary.

✓ Torsion in the case of pile-type structures and rotation in the case of caisson-type structures have not been examined comprehensively. However, these factors may affect the stability of structures in certain cases; therefore, it is necessary to carefully consider these aspects.

✓ The superstructure of the dolphin should have a height that shall not be affected by waves, and the crown height of the dolphin shall be appropriately set in accordance with its function. In this connection, the position of installation of the fenders for

breasting dolphins, the level of the deck of the ship for mooring dolphins, and the working range of the loading arm for working platforms should be taken into consideration. For connecting bridge, its height should be sufficient to ensure that it is not affected by the action of waves.

### 1) Classification of Design Conditions

The design conditions for dolphins can be broadly categorized into the following two states:

✓ Variable actions (actions due to ship berthing/tractive force, surcharge load, Level 1 earthquake ground motion, and wave action)
✓ Accidental actions (Level 2 earthquake ground motion)

While performance verification for general dolphins is typically conducted under the variable actions mentioned above, facilities designed with enhanced seismic resistance such as LNG dolphins may also be verified under the accidental actions.

### 2) Verification Items

Verification of dolphins typically requires evaluation of the following items:

✓ Verification of pile stress
✓ Verification of bearing capacity
✓ Verification of superstructure

For facilities requiring enhanced seismic resistance, the following two additional verifications are necessary under accidental action involving Level 2 earthquake ground motion:

✓ Perform dynamic analyses to verify potential damage of piles.
✓ When displacement limits are specified for working platforms, perform analyses using the effective stress method to verify the displacement of the dolphin and the structural capacity of the piles

### (2) Performance Verification of Pile Stress

Performance verification for the stresses in the piles under variable situation (surcharge, ship berthing force, tractive force by ship, and Level 1 earthquake ground motion). The stresses occurring in the piles of a dolphin may be verified using Equation (1.1). In the following equations, denotes the partial factor corresponding to the suffix, where the suffixes k and d indicate the characteristic value and the design value, respectively. As for the partial factors in the relevant equations, the values shown in Table 1.2 can be used. The values shown as "-" in Table 1.2 indicate that the values may be verified using the values enclosed in parentheses ( ) to ensure convenience. If the axial forces are tensile, $S_k$ and $R_k$ can be calculated using Equations (1.1 (b)) and (1.1 (c)), respectively, and each value should satisfy Equation (1.1).

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \qquad (1.1)$$

✓ When the axial forces are compressive,

$$S_k = \left(\frac{\sigma_{c_k}}{red} + \sigma_{bc_k}\right) \quad R_k = \sigma_{by_k} \qquad (1.1 \text{ (a)})$$

✓ When the axial forces are tensile,

$$S_k = \sigma_{t_k} + \sigma_{bt_k} \qquad R_k = \sigma_{ty_k} \qquad (1.1\ (b))$$

$$S_k = -\sigma_{t_k} + \sigma_{bc_k} \qquad R_k = \sigma_{by_k} \qquad (1.1\ (c))$$

Where:

*red* : coefficient defined as the value of the axial compressive yield stress (refer to Table 1.4) divided by the characteristic value of the yield stress

$\sigma_t$ and $\sigma_c$ : tensile stress due to the axial tensile forces acting on the cross-section and compressive stress due to the axial compressive forces, respectively (N/mm$^2$)

$\sigma_{bt}$ and $\sigma_{bc}$ : maximum tensile stress and maximum compressive stress because of the flexural moment acting on the cross-section, respectively (N/mm$^2$)

$\sigma_{ty}$ and $\sigma_{cy}$ : axial tensile yield stress and axial compressive yield stress, respectively (N/mm$^2$)

$\sigma_{by}$ : bending compressive yield stress (N/mm$^2$)

$R$ : resistance term (N/mm$^2$)

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

$\gamma_R$ : partial factor that is to be multiplied with the resistance term

$\gamma_S$ : partial factor that is to be multiplied with the load term

$m$ : adjustment factor

**Table 1.2- Partial Factor Used for Verification of the Stresses Occurring in the Piles**

OCDI
2020
Part III
Chapter 5
Table 5.2.1
Table 5.3.1

| Verification target | Installation water depth | Partial factor to be multiplied with resistance term: $\gamma_R$ | Partial factor to be multiplied with load term: $\gamma_S$ | Adjustment factor: $m$ |
|---|---|---|---|---|
| Stress occurring in the piles (variable action due to surcharge (during work)) | All water depth | - (1.00) | - (1.00) | 1.67 |
| Stress occurring in the piles (variable action due to surcharge (during storm)) | All water depth | - (1.00) | - (1.00) | 1.12 |
| Stress occurring in the piles (variable action due to tractive force by ship) | All water depth | - (1.00) | - (1.00) | 1.67 |
| Compressive stress occurring in the batter piles (variable action due to ship berthing force) | All water depth | - (1.00) | - (1.00) | 1.67 |
| Compressive stress occurring in the vertical piles (variable action due to ship berthing force) | Less than 12.0m | 0.97 | 1.34 | - (1.00) |
|  | 12.0m and above | 1.01 | 1.29 |  |
| Tensile stress occurring in the piles (variable action due to ship berthing force) | All water depth | - (1.00) | - (1.00) | 1.67 |
| Stress occurring in the piles (variable action due to Level 1 earthquake ground motion) | All water depth | - (1.00) | - (1.00) | 1.12 |

Source: Modified from OCDI 2020

The partial factors are used for the verification of the compressive stresses occurring

in the piers at the time when a ship berth in Table 1.2 is the coefficient obtained by the conducted code calibrations so that the obtained dimensions are averagely equivalent as the cross-sections of open-type wharves on vertical piles designed using the previous design methods.

Therefore, when applying other verification formulas or partial factors, appropriate code calibration shall be performed and it shall be considered whether the performance is sufficiently satisfactory.

When analyzing using a two-dimensional frame and coupled batter piles are laid out in the cross section direction, it is preferable that the stress of each pile in the cross section direction is 20 to 30% lower than the yield stress based on performance verification regarding open-type wharves on vertical piles, in order to deal with the flexural moment or secondary stress not considered in the verification.

In addition, partial factors related to other design states set by referring to the allowable stresses of the steel members in the previous design methods.

The design values in Equation (1.1) are calculated according to Equation (1.2).

$$\sigma_{t_k} = \frac{P_k}{A}, \quad \sigma_{c_k} = \frac{P_k}{A}$$

$$\sigma_{bt_k} = \frac{M_k}{Z}, \quad \sigma_{bc_k} = \frac{M_k}{Z}$$

(1.2)

Where:

$A$ : cross-sectional area of the piles (mm$^2$)
$P$ : axial force on the pile (N)
$Z$ : section modulus of the piles (mm$^3$)
$M$ : flexural moment of the piles (N·mm)

**Table 1.3- Characteristic Values of Yield Strength for Steel Pile**

| Steel ⟍ Grade / Type of Stress | TCVN 9245: 2012 SPP400 | TCVN 9246: 2012 SPP490 |
|---|---|---|
| Axial tensile stress (per net cross-sectional area) | 235 | 315 |
| Bending tensile stress (per net cross-sectional area) | 235 | 315 |
| Bending compression stress (per total cross-sectional area) | 235 | 315 |
| Shear stress (per total cross-sectional area) | 136 | 182 |

Source: Modified from TCVN 11820-4-1-2020

Reference:

The axial compressive yield stress of typical steel pipe piles in Vietnam may be calculated by the equation given in Table 1.4 from OCDI 2020. Since dolphins are often installed far from land, it is rare to consider a virtual ground surface. However, as for the effective buckling length of the members, the distance from the lower end of the superstructure to $1/\beta$ under the virtual ground surface may be used as denoted in Figure 1.4.

**Table 1.4- The Axial Compressive Yield Stresses**

| SPP400 | SPP490 |
|---|---|
| a) When $\frac{l}{r} \leq 19$   235 | a) When $\frac{l}{r} \leq 16$   315 |

| | |
|---|---|
| b) When $19 < \dfrac{l}{r} \leq 93$    $235 - 1.4\left(\dfrac{l}{r} - 19\right)$ | b) When $16 < \dfrac{l}{r} \leq 80$    $315 - 2.1\left(\dfrac{l}{r} - 16\right)$ |
| c) When $\dfrac{l}{r} > 93$    $\dfrac{2.0 \times 10^6}{6.7 \times 10^3 + \left(\dfrac{l}{r}\right)^2}$ | c) When $\dfrac{l}{r} > 80$    $\dfrac{2.0 \times 10^6}{5.0 \times 10^3 + \left(\dfrac{l}{r}\right)^2}$ |

Source: Modified from OCDI 2020

Where:

$l$  :  effective buckling length of the member (mm)

$r$  :  radius of gyration of the member gross cross-section (mm)

When bedrock is present, high-strength piles may be used such as socketed piles (Refer to Part 2 Open-type Wharves on Piles, Figure 1.3 ) to resist pull-out forces, allowing for a reduced number of piles. Table 1.5 provides examples of axial compressive yield stresses for high-strength piles.

**Table 1.5- The Axial Compressive Yield Stresses for High-strength Materials**

| SM490Y | SM570 |
|---|---|
| a) When $\dfrac{l}{r} \leq 15$   355 | a) When $\dfrac{l}{r} \leq 13$   450 |
| b) When $15 < \dfrac{l}{r} \leq 76$    $355 - 2.6\left(\dfrac{l}{r} - 15\right)$ | b) When $13 < \dfrac{l}{r} \leq 67$    $450 - 3.7\left(\dfrac{l}{r} - 13\right)$ |
| c) When $\dfrac{l}{r} > 76$    $\dfrac{2.0 \times 10^6}{4.4 \times 10^3 + \left(\dfrac{l}{r}\right)^2}$ | c) When $\dfrac{l}{r} > 67$    $\dfrac{2.0 \times 10^6}{3.5 \times 10^3 + \left(\dfrac{l}{r}\right)^2}$ |

Source: OCDI 2020
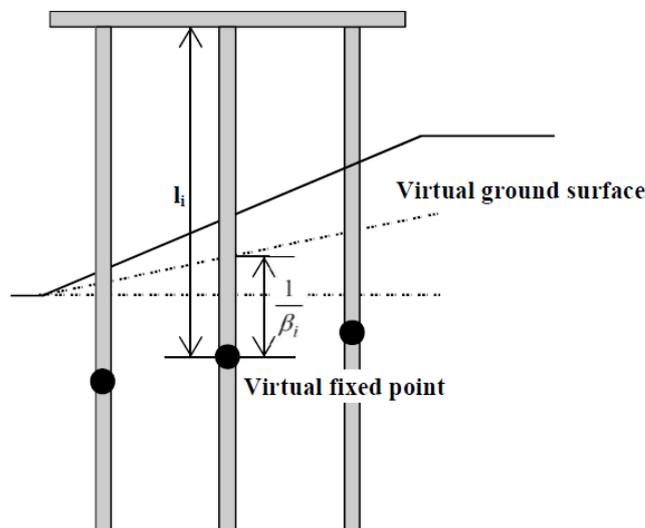
Source: OCDI 2020

**Figure 1.4- Setting of the Effective Buckling Length**

**(3) Performance Verification of the Bearing Capacity in Piles**

The bearing capacity of the piles can be verified using Equation (1.3). The symbol is the partial factor corresponding to the suffix, where the suffixes $k$ and d indicate the characteristic value and the design value, respectively. As for the partial factors in the relevant equation, the values shown in Table 1.6 can be used. The values denoted as "-" in Table 1.6 indicate that the values may be verified using the values enclosed in parentheses

( ) to ensure convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \tag{1.3}$$

Where:

| | | |
|---|---|---|
| $m$ | : | adjustment factor |
| $S$ | : | characteristic value of load term (kN/m) |
| $R$ | : | characteristic value of resistance term (kN/m) |
| $\gamma_S$ | : | partial factor that is to be multiplied with the load term |
| $\gamma_R$ | : | partial factor that is to be multiplied with the resistance term |

The pile lateral resistance force can be calculated as an effective soil layer below the virtual ground surface. If waves are expected to hit the deck, the pull-out resistance of the piles against the uplift pressure of the waves and the damage caused by flooding of the equipment on the deck and other damages shall be considered.

The axial resistance force should be determined by the pile loading test, but if it is difficult to perform the loading test in advance, it can be estimated based on the formula in Part 2 Open-type Wharves on Piles 1-4. (3). However, even if the formula is used, a load test of the pile should be carried out during the construction stage.

**Table 1.6- Partial Factors Used for Performance Verification Regarding the Bearing Capacity in Piles**

| Verification target | Type of piles | Partial factor to be multiplied with resistance term: $\gamma_R$ | Partial factor to be multiplied with load term: $\gamma_S$ | Adjustment factor: $m$ |
|---|---|---|---|---|
| Bearing capacity of the piles (variable actions due to surcharges, and during ship actions) | Pulling pile | - (1.00) | - (1.00) | 3.00 |
| | Pushing pile | - (1.00) | - (1.00) | 2.50 |
| Bearing capacity of the piles (variable actions due to storm, high waves, and Level 1 earthquake ground motion) | Pulling pile | - (1.00) | - (1.00) | 2.50 |
| | Pushing pile (bearing pile) | - (1.00) | - (1.00) | 1.50 |
| | Pushing pile (friction pile) | - (1.00) | - (1.00) | 2.00 |

Source: Modified from TCVN 11820-5-2021, OCDI 2020

## 1-4. Loads and Actions
### (1) Loads to be Considered

The loads acting on the dolphins need to be set according to operation conditions. The typical design loads and actions to be considered are as shown in Table 1.7.

**Table 1.7- Design Loads and Actions**

| Vertical load and action | Horizontal load and action |
|---|---|
| 1) Self-weight (Superstructure and piles) | 1) Earthquake forces acting on the superstructure and piles |
| 2) The loads on the superstructure:<br>  - Surcharge | 2) Earthquake forces acting on the surcharge, vehicle load, cargo handling |

| - Vehicle load | equipment and catwalk load |
|---|---|
| - Cargo handling equipment load | 4) Wave loads and current loads |
| - Catwalk load | 5) Reaction force of the fender |
| 3) Tractive force of vessel | 6) Tractive force of vessel |
| 4) Uplift | |
| 5) Negative friction forces on piles | |

**(2) Fender Reaction Force**

For the reaction force of the fendering structure, refer to "Part 2 Open-type Wharves on Piles 1.4 (2) Fender Reaction Force".

When a vessel approaches for berthing, either the bow or stern typically makes initial contact with the fender system; therefore, the berthing energy can be calculated assuming it is absorbed by a single berthing dolphin.

Large vessels, such as tankers, generally berth at an angle to the quay. In such cases, caution is required, as the performance characteristics of the fenders vary depending on the angle of approach.

Additionally, fenders shall safely withstand shear forces generated by frictional resistance acting perpendicular (longitudinal direction) to the berthing structure during angled berthing. Typically, these shear forces can be calculated using the formula proposed by Vasco Costa. For berthing angles ranging approximately from 6° to 14° relative to the normal direction of the mooring facility, the shear force typically amounts to around 10–25% of the berthing force.

**(3) Tractive Force by Ships**

For the tractive force by ships, refer to Table 1.8.

**Table 1.8- Standard Values of Tractive Force by Ships**

| GT of ship (ton) | | | | | Tractive force acting on a bollard (kN) | Tractive force acting on a mooring post (kN) |
|---|---|---|---|---|---|---|
| Over | 200 | and | not more than | 500 | 150 | 150 |
| Over | 500 | and | not more than | 1,000 | 250 | 250 |
| Over | 1,000 | and | not more than | 2,000 | 250 | 350 |
| Over | 2,000 | and | not more than | 3,000 | 350 | 350 |
| Over | 3,000 | and | not more than | 5,000 | 350 | 500 |
| Over | 5,000 | and | not more than | 10,000 | 500 | 700 |
| Over | 10,000 | and | not more than | 20,000 | 700 | 1,000 |
| Over | 20,000 | and | not more than | 50,000 | 1,000 | 1,500 |
| Over | 50,000 | and | not more than | 100,000 | 1,000 | 2,000 |

Source: TCVN 11820-2-2025

**(4) Seismic Coefficient**

For the seismic coefficient of dolphins, refer to "Part 2 Open-type Wharves on Piles 1.4 (3) Seismic Coefficient".

**(5) Wave Force**

For the wave force of dolphins, refer to "Part 2 Open-type Wharves on Piles 1.4 (4) Wave Force".

**(6) Current Force**

For the current force of dolphins, refer to "Part 2 Open-type Wharves on Piles 1.4 (5) Current Force".

**(7) Uplift Force**

For the uplift force of dolphins, refer to "Part 2 Open-type Wharves on Piles 1.4 (6) Uplift Acting on Piled Pier".

## 1-5. Frame Analysis

**(1) Virtual Ground Surface**

When the seabed surface where dolphin piles are installed is sloped due to dredging or other factors, the reference elevation for verifying pile stress and bearing capacity (the "virtual ground surface") may be set at a point halfway between the existing seabed and the dredged seabed elevation at the pile centerline.

**(2) Inclination Angle of Batter Piles**

The inclination angle of batter piles should be determined by considering the required performance of the dolphin structure, pile length, and pile weight, as well as positional relationships with other piles and construction constraints such as the capabilities of pile-driving equipment.

**(3) Node Placement**

Structural analysis of dolphins is often conducted using three-dimensional frame analysis. In frame analysis, nodes are used to define the structural geometry and each member. Consequently, it is necessary to place nodes at locations where structural cross-sectional properties, load conditions, and boundary conditions change, in order to accurately define the members.

**(4) Input of Member Properties**

The main input parameters for each member's properties are as follows:

**1) Position and Length of members**

The position and length of a member are defined using the coordinates of the nodes.

**2) Material Conditions**

The main material properties for main structural members include the following four physical properties.

✓ Modulus of elasticity (Young's modulus): $E$ (kN/m$^2$)
✓ Shear modulus (elastic shear modulus): $G$
  $G = E / (2 \times (1 + \nu))$ (kN/m$^2$)

Note: Shear modulus $G$ is not required for analyses involving two-dimensional planar structural models.

✓ Linear expansion coefficient: $\alpha$ (1/°C)

Note: If stress due to temperature change is not considered, $\alpha = 0$ can be assumed.

✓ Poisson's ratio: $\nu$

**3) Member Connection Conditions**

Member connection conditions vary depending on the structural requirements and the purpose of the analysis. In principle, if the transmission of moments between members is allowed, the connection should be rigid. If moments are not transmitted, the connection should be pinned. For example, joints in a frame structure are generally rigid connections, while joints in a truss are typically pinned connections.

**4) Input of Member Cross-Section Properties**

For frame analysis, the cross-sectional area, moment of inertia, and, for 3D frame analysis, the torsional constant ($J=2\times$(moment of inertia)) are applied, without considering the corrosion.

### 5) Determination of Coefficient of Horizontal Subgrade Reaction

When conducting frame analysis, the following considerations regarding coefficient of horizontal subgrade reaction should be noted:

Derived from soil conditions, the coefficient of horizontal subgrade reaction ($k_{CH}$) has units of kN/m³ (or N/cm³). It serves as the basis for determining the horizontal spring constant of piles ($K_H$) used in frame analysis.

$$K_H = k_{CH} \times D \qquad (1.4)$$

Where:

$K_H$ : horizontal spring constant of piles (kN/m²)
$k_{CH}$ : coefficient of horizontal subgrade reaction (kN/m³)
$D$ : pile diameter (m)

When piles are inclined, the coefficient of lateral subgrade reaction perpendicular to the pile's axis changes depending on the inclination. Therefore, when calculating the deflection of inclined piles, it is necessary to adjust the coefficient of horizontal subgrade reaction ($k_{CH}$) to account for the influence of the pile's inclination angle.

The lateral resistance force of a pile in the direction perpendicular to the axis generally tends to become smaller when load acts in the direction to raise a pile. Conversely, it tends to become larger when load acts in the direction of pushing a pile to the ground. Therefore, the influence of the angle of a pile's inclination shall be considered by correcting the coefficient of horizontal subgrade reaction in calculating the deflection of batter piles. Figure 1.5 shows the relation between the angle of a pile's inclination and the ratio of the lateral resistance coefficient. Here, the ratio of lateral resistance coefficient of the ground means that of the coefficient of lateral subgrade reaction used for calculation of batter piles to the coefficient of horizontal subgrade reaction used for calculation of vertical piles.
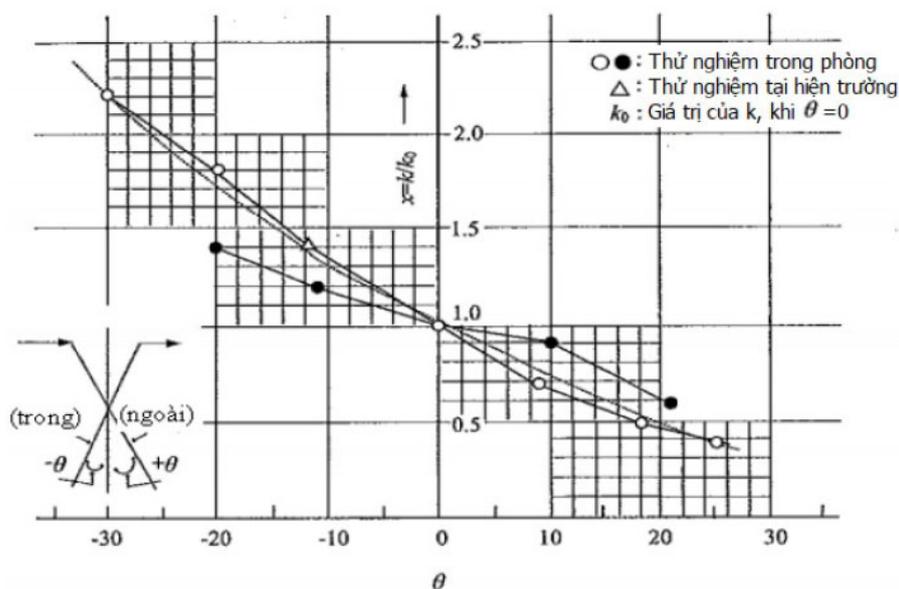


TCVN 11820 Part 1: 2025, Hình I.20

TCVN 11820 Part 4-1: 2020, Hình C.18

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

**Figure 1.5- Ratio of Angle of Pile's Inclination to the Lateral Resistance Coefficient of the Ground**

In dolphin design, the load direction often varies, and pile layouts are frequently adjusted to avoid interference with adjacent piles. Adjusting the coefficient of lateral subgrade reaction for each angle can become excessively complex. Considering that the effects of such angle corrections are typically limited to near-surface zones (within a depth of around $1/\beta$) and that pile groups are generally expected to carry axial forces, the above angle-based corrections are sometimes omitted.

## (5) Design Load
### 1) Horizontal Load

In traditional manual calculations, the horizontal load distribution ratio for the pile group is determined based on pile layout, with the assumption that all horizontal forces acting on the pile group are carried solely by the battered piles. This assumption implies that horizontal forces are divided and designed separately for vertical piles and battered piles.

In frame analysis, the method for calculating the horizontal load distribution ratio among the pile group remains the same as in manual calculations. However, the assumption that only the battered piles resist all horizontal forces does not apply. Instead, the horizontal forces are resisted by the entire pile group as a whole, with vertical and battered piles functioning as a unified system to share the horizontal loads.

### 2) Vertical Load

In traditional manual calculations, for the sake of simplicity, the vertical load distribution is divided among the piles by assigning a specific contribution area for each pile. The vertical distributed loads are then replaced with equivalent concentrated loads acting on the individual piles.

In contrast, frame analysis does not require the assignment of contribution areas for each individual pile within a pile group. Instead, the distribution width for each pile row is defined, and the vertical distributed loads are not converted into concentrated loads. Rather, they are handled as distributed loads throughout the analysis. For example, the self-weight of the slab, uniformly distributed live loads, and the self-weight of the piles themselves are all treated as vertically distributed loads.

Given the approaches outlined above, the handling of loads in frame analysis more closely represents the actual loading conditions than conventional manual calculations. This allows frame analysis to more accurately reflect the structural behavior under real-world conditions.

## 2. Design Example

The design example is shown within the layout of Figure 2.1, indicating the breasting dolphin designated for the oil tanker. The typical layout and section for the breasting dolphin are indicated in Figure 2.2 and 2.3.
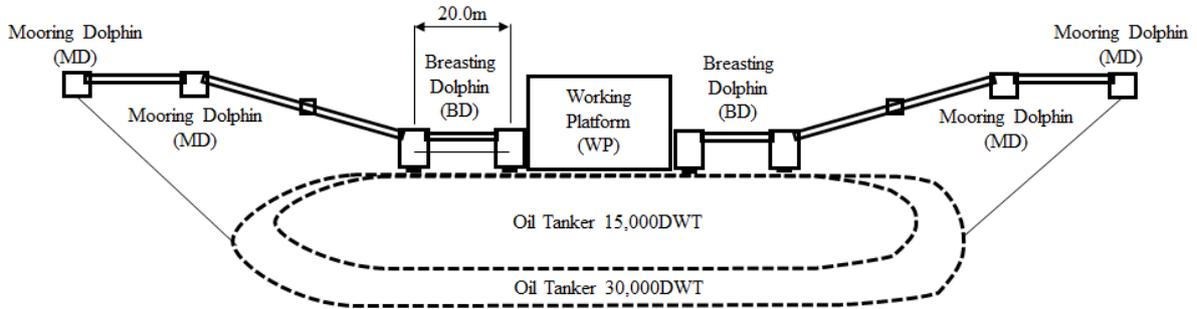
## 2-1. Layout for Performance Verification
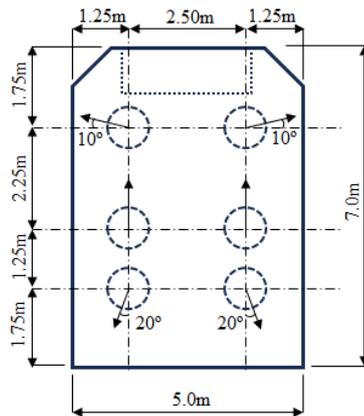


**Figure 2.1- Layout of Design Example**



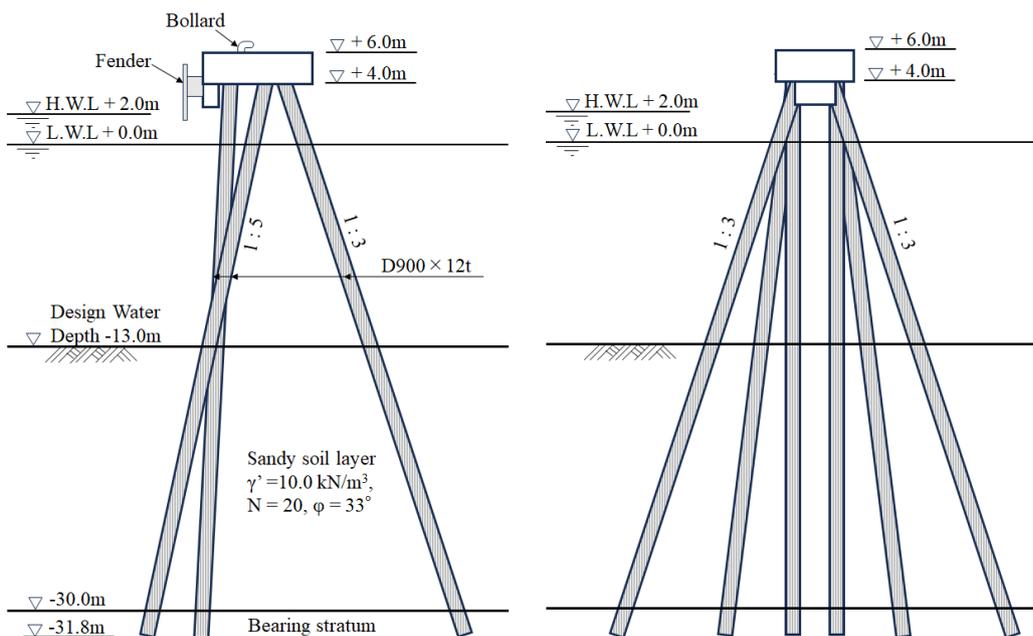**Figure 2.2- Typical Layout of Breasting Dolphin**



**Figure 2.3- Typical Section of Breasting Dolphin**

## 2-2. Design Conditions

### (1) Planning Conditions
- Planned water depth: $D.L. - 12.5$ m
  The planned water depth was set at -12.5m, considering a loaded draft of design vessel and wave motions
- Design water depth: $D.L. - 13.0$ m
  The design water depth was set at -13.0m, considering an over-dredging depth of 0.5m below the planned water depth
- Crown height of deck: $D.L. +6.0$ m
- Bottom height of deck: $D.L. +4.0$ m

### (2) Natural Conditions
#### 1) Tide Levels
- Design High Water Level: $D.L. +2.0$ m
- Mean Sea Level: $D.L. +1.0$ m
- Design Low Water Level: $D.L. \pm 0.0$ m

#### 2) Ground Conditions
$\underline{D.L. -12.5 \text{ m (Planned water depth)}}$
Sandy soil layer
$\gamma' = 10.0$ kN/m$^3$, $N = 20$, $\varphi = 33°$

$\underline{D.L. -30.0 \text{ m}}$
Gravel sand layer (bearing stratum)
$\gamma' = 10.0$ kN/m$^3$, $N = 50$, $\varphi = 40°$

#### 3) Wave Conditions
- $H_{1/3} = 1.8$ (m): Storm
- $H_{max} = 1.8 \times 1.8 = 3.24$ (m)
- $T = 7.0$ (sec)

#### 4) Current Flow Conditions
- $U = 0.25$ (m/s)

### (3) Use Conditions
#### 1) Dimensions of Design Vessel
- Oil tanker. 30,000 *DWT* (15,000 to 30,000 *DWT)*
- Length overall: $Loa = 177.0$ m
- Length between perpendiculars $L_{pp} = 168.0$ m
- Beam: $B = 26.9$ m
- Loaded draft: $d = 10.50$ m
- Displacement tonnage: $DT = 1.688DWT^{0.976} = 39,540$ t
- Gross tonnage: $GT = 0.523DWT = 0.523 \times 30,000 = 15,690$ t

#### 2) Design Service Life and Corrosion Countermeasures
- Design service life: 50 years
- Corrosion countermeasures

Cathodic protection and protective coating: 50 years is assumed. The assumed

corrosion control ratio is 90 %.

Heavy anticorrosion coating is applied to the intertidal zone of steel pipe piles from L.W.L. -1.00 m to the bottom of the deck beams. The assumed corrosion protection efficiency in this case is 100 %.

The corrosion thickness of piles is assumed to be 1.5mm in this example.

### 3) Surcharge Loads
- Permanent state: 5.0 kN/m$^2$
- Variable situation at mooring and berthing: 5.0 kN/m$^2$
- Variable situation at storm and Level 1 earthquake: 2.5 kN/m$^2$

### 4) Mooring Force
- Tractive force: 700 kN type (Bollard)

## (4) Seismic Condition
- Ground type: Type C
- Importance coefficient: 1.25 (Oil facility)
- Background acceleration: 0.10g
- Behavior factor: 1.0 (Batter piled structure)

The seismic coefficient for verification in the variable situation is set by natural period estimated and shape of the elastic response spectrum shown in Figure 2.4.

### 1) Horizontal Spring Constant
The horizontal spring constant ($k_{CH}$) for sandy soil layer is calculated as follows:

In calculating the natural period at Level 1 earthquake ground motion, a horizontal spring constant can be doubled, but a value that is not doubled is used to be on the conservative approach. In this design example, the natural period is the same whether the coefficient of horizontal subgrade reaction ($k_{CH}$) is doubled or left at 1.0. Therefore, this example shows that a conservative approach was adopted in calculating the horizontal spring constant without doubling and without considering the inclination angle of the batter piles.
- $k_{CH} = 1500 N_{avg} = 1500 \times 20 = 30,000$ kN/m³

### 2) Seismic Coefficient
- Natural period of structure: 0.40 sec, estimated by three-dimensional analysis
- Importance coefficient: 1.25
- Ground type: Type C
- Seismic coefficient: $k_h = 0.10g \times 2.875 \times 1.25 / q$ (=1.0, behavior factor) = 0.36
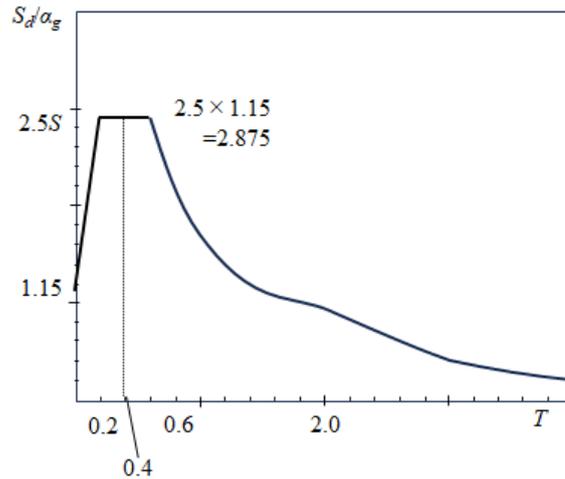
**Figure 2.4- Shape of the Elastic Response Spectrum**

## (5) Berthing Force

### 1) Calculation of Berthing Energy

The characteristic value ($E_{fk}$) of the berthing energy of a vessel is obtained by the following equation (refer to Part 2 Equation (1.4)).

$$E_{f_k} = \frac{1}{2} M_{s_k} V_{b_k}^2 C_{m_k} C_{e_k} C_{s_k} C_{c_k}$$

Where:

- $E_f$ : berthing energy of the ship (kJ)
- $M_s$ : mass of the ship (t)
- $V_b$ : berthing velocity of the ship (m/s)
- $C_m$ : virtual mass factor
- $C_e$ : eccentricity factor
- $C_s$ : flexibility factor =1.0
- $C_c$ : berth configuration factor =1.0

- Mass of ship
  $M_{sk} = DT = 1.688 DWT^{0.976} = 39,540 \text{ t}$

- Berthing velocity of the ship
  $V_{bk} = 0.15 \text{ m/s}$

- Block coefficient
  Length between perpendiculars $L_{pp} = 168.0 \text{ m}$
  The block coefficient ($C_b$) of the vessel is obtained by the following equation.

  $$C_{bk} = \frac{\nabla}{L_{bb} B d} = \frac{39,540/1.03}{168.0 \times 26.9 \times 10.5} = 0.809$$

- Virtual mass factor

  $$C_{mk} = 1 + \frac{\pi}{2 C_{bk}} \frac{d}{B} = 1 + \frac{\pi}{2 \times 0.809} \times \frac{10.5}{26.9} = 1.76$$

- Eccentricity factor
  Radius of gyration $r$
  $r = (0.19 C_{bk} + 0.11) L_{pp} = (0.19 \times 0.809 + 0.11) \times 168.0 = 44.3 \text{ m}$

Distance $L_i$ from berthing point of vessel along the normal line of the mooring facility to the center of gravity of the vessel

$$L_1 = \{0.5\alpha + e\,(1-k)\}\,L_{pp}\cos\theta$$
$$\quad = \{0.5 \times 0.50 + 0.120 \times (1-0.5)\} \times 168.0 \times \cos 6° = 51.79 \text{ m}$$
$$L_2 = (0.5\alpha - ek)\,L_{pp}\cos\theta$$
$$\quad = (0.5 \times 0.50 - 0.120 \times 0.5) \times 195.0 \times \cos 6° = 51.79 \text{ m}$$

$e$ = Fender pitch (here 20.0m) / $L_{pp}\cos\theta$
$\quad = 20.0\,/\,(168.0 \times \cos 6°) = 0.120$

Where:

$\theta$ : berthing angle 6°

$e$ : ratio of fender interval and length between perpendiculars

$\alpha$ : ratio of length of parallel side and length between perpendiculars (0.50)

When it is assumed that the length of the parallel sides of a vessel is generally $\alpha L$ ($L$: ship length), a guideline of approximately 1/2 may be used for the parallel factor $\alpha$ when the design ship is a cargo or tanker ship.

Therefore, $\alpha = 0.50$ was used here.

$k$ : parameter (0.50)

$$C_{ek} = \cfrac{1}{1+\left(\dfrac{l}{r}\right)^2} = \cfrac{1}{1+\left(\dfrac{51.79}{44.30}\right)^2} = 0.423$$
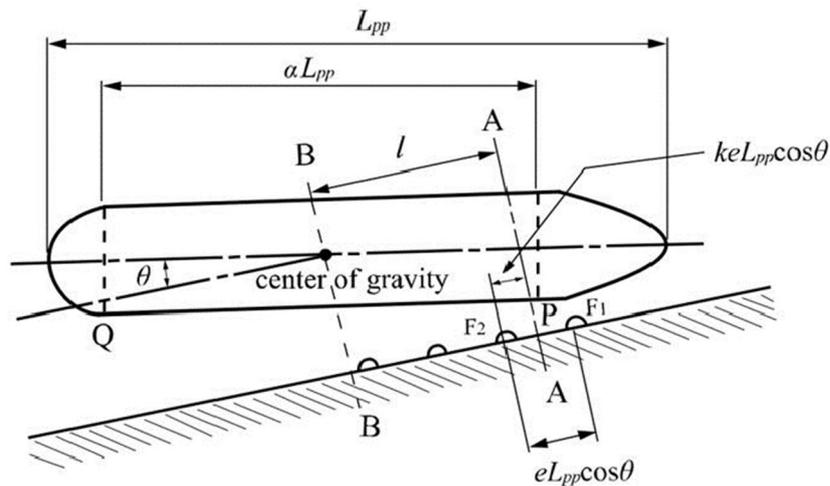


Source: TCVN 11820-2-2025

**Figure 2.5- Schematic Illustration of Ship Berthing**

Accordingly, the berthing energy $E_f$ of the vessel is as follows.

$$E_{f_k} = \frac{1}{2}M_{s_k}V_{b_k}{}^2 C_{m_k}C_{e_k}C_{s_k}C_{c_k}$$
$$\quad = \frac{1}{2} \times 39{,}540 \times 0.15^2 \times 1.76 \times 0.423 \times 1.0 \times 1.0$$
$$\quad = 331.2 \text{ kJ (kN·m)}$$

## 2) Selection of Rubber Fender

Here, Cell-type fender (natural rubber material) is used.
- Fender size: Cell-type 1250H
- Rubber content: E1.5
- Energy absorption: 393 kN·m
- Reaction force: 716 kN
- Design reaction force: $716 \times 1.1 = 787.6$ kN
- Design shear force: $787.6 \times 0.2 = 157.5$ kN

The shear force is applied to X-direction in this example.

## (6) Mooring Force
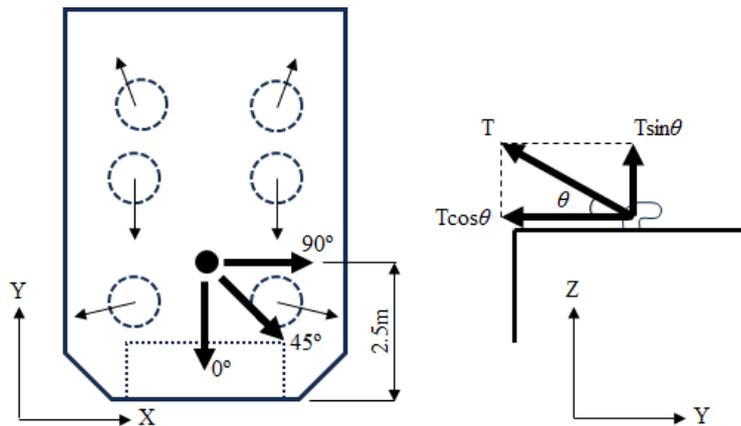


**Figure 2.6- Various Cases for Examining Mooring Forces**

**Table 2.1- Various Cases for Examining Mooring Forces**

| Case | Horizontal Angle (º) | Vertical Angle (º) | Tractive Force (kN) | X (kN) | Y (kN) | Z (kN) |
|------|------|------|------|------|------|------|
| M1 | 0 | 0 | | 0.0 | -700.0 | 0.0 |
| M2 | 45 | 0 | | 495.0 | -495.0 | 0.0 |
| M3 | 90 | 0 | 700 | 700.0 | 0.0 | 0.0 |
| M4 | 0 | 45 | | 0.0 | -495.0 | 495.0 |
| M5 | 45 | 45 | | 350.0 | -350.0 | 495.0 |
| M6 | 90 | 45 | | 495.0 | 0.0 | 495.0 |

## (7) Wave Force

The wave force is automatically calculated by SAP 2000 analysis using the Morison's formula as shown in Equation (2.1).

$$\vec{f_n} = \frac{1}{2}\,C_D\rho_0|\vec{u_n}|\vec{u_n}D\Delta S + C_M\rho_0\vec{a_n}A\Delta S \qquad (2.1)$$

Where:

$\vec{f_n}$ : force that acts on a small length $\Delta S$ (m) in the axial direction of the member, where the direction of this force lies in the plane containing the member axis and the direction of motion of the water particles and is perpendicular to the member axis (kN)

$\vec{u_n}$ , $\vec{a_n}$ : components of the water particle velocity (m/s) and acceleration (m/s$^2$), respectively, in the direction perpendicular to the

member axis that lies within the plane containing the member axis and the direction of motion of the water particles (i.e., the same direction as $\rightarrow f_n$) (these components are for incident waves that are not disturbed by the presence of member)

$|\vec{u_n}|$ : absolute value of $\rightarrow u_n$ (m/s)

$C_D$ : drag coefficient

$C_M$ : inertia coefficient

$D$ : width of the member in the direction perpendicular to the member axis as viewed from the direction of $\rightarrow f_n$(m)

$A$ : cross-sectional area of the member along a plane perpendicular to member axis (m²)

$\rho_0$ : density of seawater (normally 1.03 t/m³)

## (8) Current Force

$$F_D = \frac{1}{2} \, C_D \rho_0 A U^2$$

Where:

$F_D$ : drag force acting on the object in the flow direction (kN)

$C_D$ : drag coefficient = 1.0

$\rho_0$ : density of seawater (normally 1.03 t/m³)

$A$ : projected area of the object in the direction of flow (m²)
  HWL to Design water depth = 2.0 + 13.0 = 15.0 (m)
  $A = 15.0 \times 0.9 = 13.5$ (m²)

$U$ : flow velocity (m/s) = 0.25 (m/s)

$$F_D = \frac{1}{2} \, C_D \rho_0 A U^2$$

$$= \frac{1}{2} \times 1.0 \times 1.03 \times 13.5 \times 0.25^2$$

$$= 0.43 \text{ (kN/pile)}$$

## (9) Dynamic Water Pressure

For the verification of dynamic water pressure on piles, the method is referred to the dynamic water pressure formulation for column-type structures in the 'Road Bridge Design Specifications in Japan'. This formulation incorporates the decrease in pressure due to flow separation and bypass around the column. The dynamic water pressure is organized by the ratio of the submerged member width to the water depth. The formulation with a pile diameter b = 900 mm and a water depth h = 13.0 m (at low tide) is presented in the following equation.

$$P = \frac{3}{4} k_h w_0 A_0 h \left(\frac{b}{a}\right)\left(1 - \frac{b}{4h}\right)$$

$$h_g = \frac{3}{7} h$$

Where:

$P$ : resultant dynamic water pressure acting on the structure (kN)

$k_h$ : seismic coefficient at Level 1 earthquake ground motion (=0.36)

$w_0$ : unit weight of seawater (=10.1 kN/m³)

$h$ : water depth (= 15.0 m)

$h_g$ : distance from the seabed surface to the point of the resultant dynamic water pressure (m)

$b$ : structural member width perpendicular to the direction of dynamic water pressure (m)

$a$ : structural member width in direction of dynamic water pressure (m)

$A_0$ : cross-sectional area of the structure ($=0.636 \text{ m}^2$)

Note that, because the foundation piles are installed at an incline, the effective pile diameter in the direction parallel to the seismic loading and in the direction perpendicular to it differs slightly; however, this difference is negligible. Therefore, $a=b=D$ (pile diameter) is assumed.

$$
\begin{aligned}
P &= \frac{3}{4} k_h w_0 A_0 h \left(\frac{b}{a}\right)\left(1-\frac{b}{4h}\right) \\
&= \frac{3}{4} \times 0.36 \times 10.1 \times 0.636 \times 15.0 \times \left(\frac{0.9}{0.9}\right)\left(1-\frac{0.9}{4 \times 15.0}\right) \\
&= 25.6 \text{ (kN)} \\
h_g &= \frac{3}{7} h \\
&= \frac{3}{7} \times 15.0 \\
&= 6.43 \text{ (m)}
\end{aligned}
$$

## 2-3. Structural Analysis
### (1) Analysis Model
#### 1) Three-dimensional model

Dolphin's structural calculation is carried out using a three-dimensional model, with horizontal spring constant applied to the piles buried into the ground. The analysis model is shown in Figure 2.7. In this calculation example, the following items are not considered.

- The weight of water inside the pile and its inertia force.

Furthermore, subsequent sections will present performance assessment results, using the steel pipe as a representative case after corrosion.
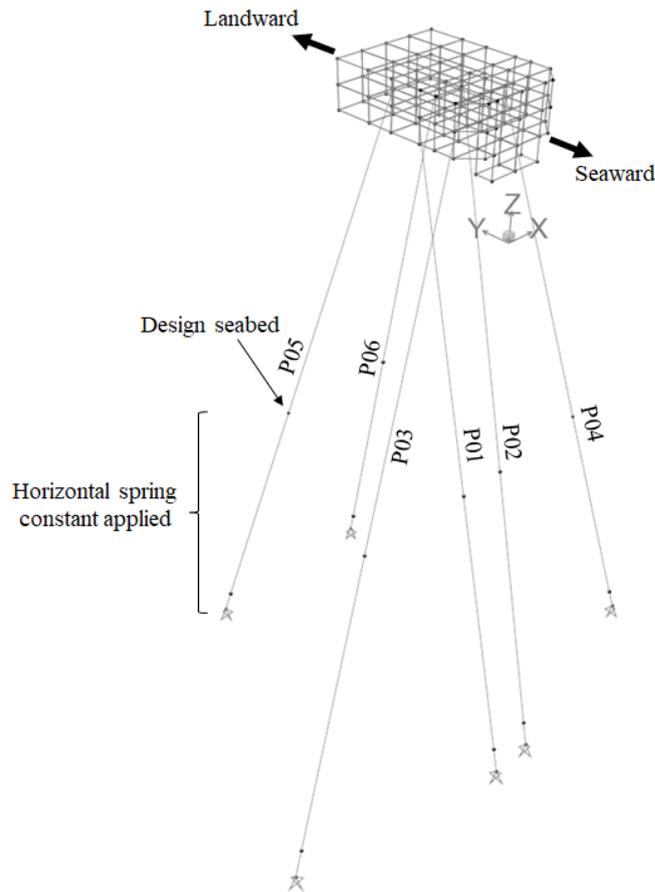
**Figure 2.7- Three-dimensional Model**

## 2) Material Specifications

The specifications of pile members are shown in Table 2.2.

**Table 2.2- Pile Specifications**

| Item | Unit | Value | Remark |
|---|---|---|---|
| Dimension | | D900×t12.0 | |
| Material | | SPP400 | |
| Young's modulus ($E$) | kN/m$^2$ | 200 | |
| Geometrical moment of inertia ($I_o$) | cm$^4$ | 330,000 | Before corrosion |
| Geometrical moment of inertia ($I$) | cm$^4$ | 287,000 | After corrosion |
| Section modulus ($Z_o$) | cm$^3$ | 7,330 | Before corrosion |
| Section modulus ($Z$) | cm$^3$ | 6,410 | After corrosion |
| Internal Radius ($r$) | cm | 31.345 | After corrosion |

The effective buckling length and yield strength of pile members are shown in Table 2.3.

**Table 2.3- Effective Buckling Length and Yield Strength of Pile**

| Item | Unit | P01 | P02 | P03 | P04 | P05 | P06 |
|---|---|---|---|---|---|---|---|
| N-Value of embedded part | | 20 | 20 | 20 | 20 | 20 | 20 |
| Horizontal spring constant: $k_{CH}$ | kN/m$^3$ | 30,000 | 30,000 | 30,000 | 30,000 | 30,000 | 30,000 |

| characteristic value of the pile: $\beta$ | m⁻¹ | 0.3292 | 0.3292 | 0.3292 | 0.3292 | 0.3292 | 0.3292 |
|---|---|---|---|---|---|---|---|
| Virtual fixed point: $1/\beta$ | m | 3.04 | 3.04 | 3.04 | 3.04 | 3.04 | 3.04 |
| Pile Top Level | m | 4.00 | 4.00 | 4.00 | 4.00 | 4.00 | 4.00 |
| Design Seabed Level | m | -13.00 | -13.00 | -13.00 | -13.00 | -13.00 | -13.00 |
| Effective Buckling Length: $l$ | m | 20.43 | 20.43 | 21.12 | 21.12 | 21.12 | 21.12 |
| Internal Radius: $r$ | m | 0.31345 | 0.31345 | 0.31345 | 0.31345 | 0.31345 | 0.31345 |
| $l/r$ | | 65.19 | 65.19 | 67.38 | 67.38 | 67.38 | 67.38 |
| Axial compressive yield strength: $\sigma_{cy}$ | MPa | 170.35 | 170.35 | 167.27 | 167.27 | 167.27 | 167.27 |
| *red* | | 0.725 | 0.725 | 0.712 | 0.712 | 0.712 | 0.712 |

## (2) Load Case and Combination

LC 1 : Dead weight load
LC 2 : Superimposed dead load (Bollard, Fender, etc)
LC 3.1 : Live load (permanent state)
LC 3.2 : Live load (variable situation)
LC 4 : Berthing force (shear force is applied to X(+) direction)
LC 5 : Mooring force (Figure 2.6)
LC 6 : Wave load (variable situation, in storm condition)
LC 7 : Current load (variable situation, in storm condition)
LC 8 : Dynamic water pressure (variable situation, in earthquake condition)
LC 9 : Earthquake load (Level 1 earthquake ground motion)

**Table 2.4- Load Factor and Load Combinations**

| Load Combination | Dead Load | Superimposed Dead Load | Live Load | | Berthing Force | Mooring Force | Wave Load | Current Load | Dynamic Water Pressure | Earthquake Load | Condition |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Normal | Abnormal | | | Storm | | | | |
| | LC1 | LC2 | LC3.1 | LC3.2 | LC4 | LC5 | LC6 | LC7 | LC8 | LC9 | |
| B1 | 1.0 | 1.0 | 1.0 | | 1.0 | | | | | | Berthing |
| M1 | 1.0 | 1.0 | 1.0 | | | 1.0 | | | | | Mooring (Table 2.1) |
| M2 | 1.0 | 1.0 | 1.0 | | | 1.0 | | | | | Mooring (Table 2.1) |
| M3 | 1.0 | 1.0 | 1.0 | | | 1.0 | | | | | Mooring (Table 2.1) |
| M4 | 1.0 | 1.0 | 1.0 | | | 1.0 | | | | | Mooring (Table 2.1) |
| M5 | 1.0 | 1.0 | 1.0 | | | 1.0 | | | | | Mooring (Table 2.1) |
| M6 | 1.0 | 1.0 | 1.0 | | | 1.0 | | | | | Mooring (Table 2.1) |
| S1 | 1.0 | 1.0 | | 1.0 | | | 1.0 | 1.0 | | | Storm |
| S2 | 1.0 | 1.0 | | 1.0 | | | 1.0 | 1.0 | | | Storm |
| S3 | 1.0 | 1.0 | | 1.0 | | | 1.0 | 1.0 | | | Storm |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| E1 | 1.0 | 1.0 | | 1.0 | | | | | 1.0 | 1.0 | Earthquake (±X-DIR) |
| E2 | 1.0 | 1.0 | | 1.0 | | | | | 1.0 | 1.0 | Earthquake (±Y-DIR) |

## (3) Sectional Forces

### 1) Berthing Condition

#### Table 2.5- Sectional Force (B1: Berthing Condition)

| Pile Row | Locaiton | Sectional Force | | | Verification |
|---|---|---|---|---|---|
| | | $P$ | $M_2$ | $M_3$ | |
| | | kN | kN·m | kN·m | ≤ 1.0 |
| P01 | Max P | -165.6 | 105.9 | 272.4 | 0.364 |
| | Min P | -73.3 | 0.0 | 0.0 | 0.018 |
| | Max $M_2$ | -165.6 | 105.9 | 272.4 | 0.364 |
| | Min $M_2$ | -119.3 | -36.0 | -152.1 | 0.202 |
| | Max $M_3$ | -165.6 | 105.9 | 272.4 | 0.364 |
| | Min $M_3$ | -119.3 | -36.0 | -152.1 | 0.202 |
| P02 | Max P | -492.8 | 105.9 | 320.2 | 0.494 |
| | Min P | -400.5 | 0.0 | 0.0 | 0.097 |
| | Max $M_2$ | -165.6 | 105.9 | 272.4 | 0.364 |
| | Min $M_2$ | -119.3 | -36.0 | -152.1 | 0.202 |
| | Max $M_3$ | -492.8 | 105.9 | 320.2 | 0.494 |
| | Min $M_3$ | -446.5 | -36.0 | -185.5 | 0.318 |
| P03 | Max P | 300.1 | -159.1 | 115.8 | 0.321 |
| | Min P | 392.4 | 0.0 | 0.0 | 0.134 |
| | Max $M_2$ | 346.4 | 97.2 | -88.6 | 0.264 |
| | Min $M_2$ | 300.1 | -159.1 | 115.8 | 0.321 |
| | Max $M_3$ | 300.1 | -159.1 | 115.8 | 0.321 |
| | Min $M_3$ | 346.4 | 97.2 | -88.6 | 0.264 |
| P04 | Max P | 709.2 | 417.2 | -50.7 | 0.708 |
| | Min P | 801.6 | 0.0 | 0.0 | 0.274 |
| | Max $M_2$ | 709.2 | 417.2 | -50.7 | 0.708 |
| | Min $M_2$ | 755.6 | -204.6 | -2.8 | 0.485 |
| | Max $M_3$ | 736.6 | 32.6 | 12.0 | 0.290 |
| | Min $M_3$ | 709.2 | 417.2 | -50.7 | 0.708 |
| P05 | Max P | 910.8 | -21.6 | -272.8 | 0.615 |
| | Min P | 1,003.2 | 0.0 | 0.0 | 0.343 |
| | Max $M_2$ | 954.8 | 17.7 | 113.9 | 0.454 |
| | Min $M_2$ | 910.8 | -21.6 | -272.8 | 0.615 |
| | Max $M_3$ | 954.8 | 17.7 | 113.9 | 0.454 |
| | Min $M_3$ | 910.8 | -21.6 | -272.8 | 0.615 |
| P06 | Max P | 981.1 | 178.5 | -334.0 | 0.755 |
| | Min P | 1,073.4 | 0.0 | 0.0 | 0.366 |
| | Max $M_2$ | 981.1 | 178.5 | -334.0 | 0.755 |
| | Min $M_2$ | 1,027.4 | -87.3 | 144.9 | 0.538 |
| | Max $M_3$ | 1,025.0 | -85.4 | 145.5 | 0.537 |
| | Min $M_3$ | 981.1 | 178.5 | -334.0 | 0.755 |

Notes:

$M_2$, $M_3$: moment in local axis 2 and axis 3 direction of the pile cross section

## Figure 2.8- Analysis of Sectional Forces (B1: Berthing Condition)

| Load combination B1: Berthing Condition | | |
|---|---|---|
| Axial forces, $P$ (kN) | Moment, $M_2$ (kN·m) | Moment, $M_3$ (kN·m) |
|  |  |  |
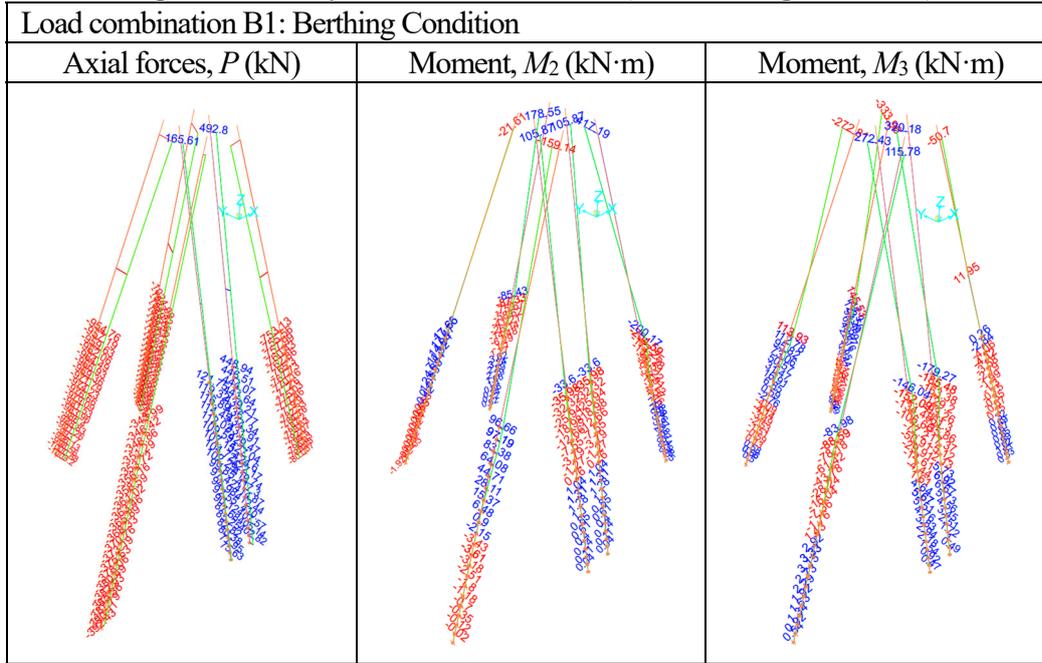
Notes:

The sign is reversed due to the analysis program setting

### 2) Mooring Condition
#### Table 2.6- Sectional Force (M1-M6: Mooring Condition)

| Pile Row | Location | Sectional Force | | | Verification |
|---|---|---|---|---|---|
| | | $P$ | $M_2$ | $M_3$ | |
| | | kN | kN·m | kN·m | ≤1.0 |
| P01 | Max P | 367.8 | 162.5 | -20.5 | 0.305 |
| | Min P | 1,036.0 | 0.0 | 0.0 | 0.347 |
| | Max M₂ | 450.9 | 229.8 | 18.6 | 0.407 |
| | Min M₂ | 497.2 | -145.1 | -24.9 | 0.330 |
| | Max M₃ | 987.6 | 0.0 | 94.1 | 0.436 |
| | Min M₃ | 943.7 | 0.0 | -173.6 | 0.509 |
| P02 | Max P | 273.1 | 162.5 | -43.0 | 0.278 |
| | Min P | 1,036.0 | 0.0 | 0.0 | 0.347 |
| | Max M₂ | 450.9 | 229.8 | 18.6 | 0.407 |
| | Min M₂ | 497.2 | -145.1 | -24.9 | 0.330 |
| | Max M₃ | 987.6 | 0.0 | 94.1 | 0.436 |
| | Min M₃ | 943.7 | 0.0 | -173.6 | 0.509 |
| P03 | Max P | -437.9 | -43.3 | 200.5 | 0.334 |
| | Min P | 558.1 | 0.0 | 0.0 | 0.191 |
| | Max M₂ | 465.7 | 151.3 | -49.8 | 0.336 |
| | Min M₂ | 512.0 | -97.6 | 0.9 | 0.283 |
| | Max M₃ | -437.9 | -43.3 | 200.5 | 0.334 |
| | Min M₃ | -391.6 | 8.5 | -144.3 | 0.256 |
| P04 | Max P | 296.6 | 133.4 | -49.3 | 0.259 |
| | Min P | 1,330.6 | 0.0 | 0.0 | 0.454 |
| | Max M₂ | 512.0 | 97.6 | 0.9 | 0.283 |
| | Min M₂ | 465.7 | -151.3 | -49.8 | 0.336 |
| | Max M₃ | 1,282.1 | -0.1 | 112.3 | 0.562 |
| | Min M₃ | 1,238.2 | -14.4 | -240.7 | 0.690 |
| P05 | Max P | -331.9 | 52.6 | 117.0 | 0.223 |
| | Min P | 209.8 | 0.0 | 0.0 | 0.072 |
| | Max M₂ | 147.0 | 170.7 | -56.0 | 0.249 |
| | Min M₂ | 100.6 | -310.9 | 38.5 | 0.382 |
| | Max M₃ | -293.1 | -180.6 | 126.1 | 0.316 |

| | | P | M₂ | M₃ | Verification |
|---|---|---|---|---|---|
| | Min M3 | -246.7 | 96.1 | -109.8 | 0.222 |
| | Max P | -331.9 | 52.6 | 117.0 | 0.223 |
| | Min P | 617.1 | 0.0 | 0.0 | 0.211 |
| P06 | Max M2 | 571.1 | 167.9 | 33.3 | 0.385 |
| | Min M2 | 524.7 | -300.9 | -116.2 | 0.537 |
| | Max M3 | -331.9 | 52.6 | 117.0 | 0.223 |
| | Min M3 | 524.7 | -300.9 | -116.2 | 0.537 |

Notes:

$M_2$, $M_3$: moment in local axis 2 and axis 3 direction of the pile cross section

**Figure 2.9- Analysis of Sectional Forces (M1-M6: Mooring Condition)**

| Load combination M1-M6: Mooring Condition | | |
|---|---|---|
| Axial forces, $P$ (kN) | Moment, $M_2$ (kN·m) | Moment, $M_3$ (kN·m) |



Notes:

The sign is reversed due to the analysis program setting

### 3) Storm Condition

**Table 2.7- Sectional Force (S1-S3: Storm Condition)**
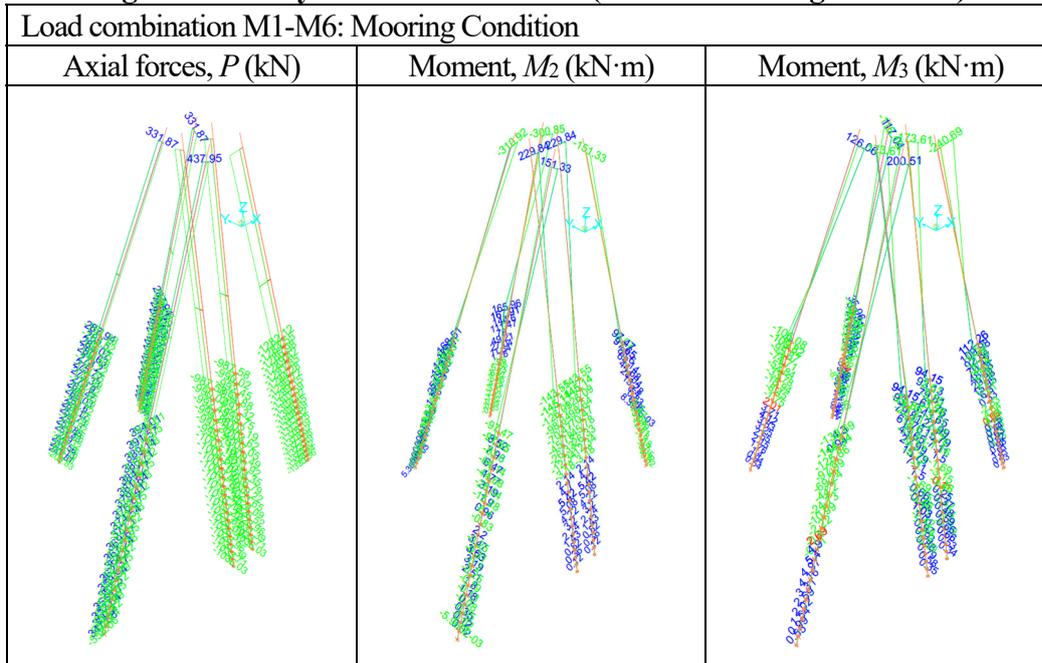
| Pile Row | Location | Sectional Force | | | Verification |
|---|---|---|---|---|---|
| | | $P$ | $M_2$ | $M_3$ | |
| | | kN | kN·m | kN·m | ≤1.0 |
| P01 | Max P | 294.7 | 0.1 | 14.8 | 0.077 |
| | Min P | 476.9 | 0.0 | 0.0 | 0.107 |
| | Max M2 | 375.4 | 23.6 | -7.1 | 0.103 |
| | Min M2 | 405.5 | -32.5 | -15.4 | 0.118 |
| | Max M3 | 309.6 | 0.5 | 23.1 | 0.087 |
| | Min M3 | 320.5 | -0.8 | -36.2 | 0.099 |
| P02 | Max P | 293.7 | 0.1 | 14.5 | 0.077 |
| | Min P | 475.9 | 0.0 | 0.0 | 0.107 |
| | Max M2 | 404.1 | 23.5 | -13.2 | 0.111 |
| | Min M2 | 368.3 | -32.8 | -5.4 | 0.108 |
| | Max M3 | 308.6 | 0.5 | 23.1 | 0.087 |
| | Min M3 | 319.5 | -0.8 | -35.9 | 0.099 |
| P03 | Max P | 328.6 | -43.2 | 13.2 | 0.109 |
| | Min P | 487.6 | 0.0 | 0.0 | 0.112 |
| | Max M2 | 432.4 | 29.4 | -14.8 | 0.123 |
| | Min M2 | 328.6 | -43.2 | 13.2 | 0.109 |
| | Max M3 | 343.4 | -3.8 | 19.2 | 0.093 |
| | Min M3 | 343.4 | 11.1 | -36.4 | 0.107 |

| | | | | | |
|---|---|---|---|---|---|
| P04 | Max P | 346.2 | 29.4 | 2.6 | 0.101 |
| | Min P | 513.4 | 0.0 | 0.0 | 0.118 |
| | Max $M_2$ | 351.6 | 33.6 | 4.3 | 0.106 |
| | Min $M_2$ | 435.1 | -29.1 | -13.3 | 0.123 |
| | Max $M_3$ | 461.6 | 3.4 | 21.4 | 0.122 |
| | Min $M_3$ | 369.4 | -7.5 | -29.3 | 0.107 |
| P05 | Max P | 278.6 | -4.9 | -17.3 | 0.077 |
| | Min P | 399.3 | 0.0 | 0.0 | 0.091 |
| | Max $M_2$ | 323.1 | 48.7 | -13.8 | 0.112 |
| | Min $M_2$ | 307.3 | -57.3 | -17.7 | 0.115 |
| | Max $M_3$ | 353.3 | 11.6 | 22.1 | 0.099 |
| | Min $M_3$ | 294.1 | -9.5 | -24.1 | 0.087 |
| P06 | Max P | 278.1 | 3.0 | -17.4 | 0.077 |
| | Min P | 398.9 | 0.0 | 0.0 | 0.091 |
| | Max $M_2$ | 307.5 | 47.2 | 7.4 | 0.106 |
| | Min $M_2$ | 291.9 | -42.9 | -22.1 | 0.103 |
| | Max $M_3$ | 352.9 | -9.3 | 22.6 | 0.099 |
| | Min $M_3$ | 300.3 | -1.1 | -24.3 | 0.087 |

Notes:

$M_2$, $M_3$: moment in local axis 2 and axis 3 direction of the pile cross section

**Figure 2.10- Analysis of Sectional Forces (S1-S3: Storm Condition)**

| Load combination S1-S3: Storm Condition | | |
|---|---|---|
| Axial forces, $P$ (kN) | Moment, $M_2$ (kN·m) | Moment, $M_3$ (kN·m) |
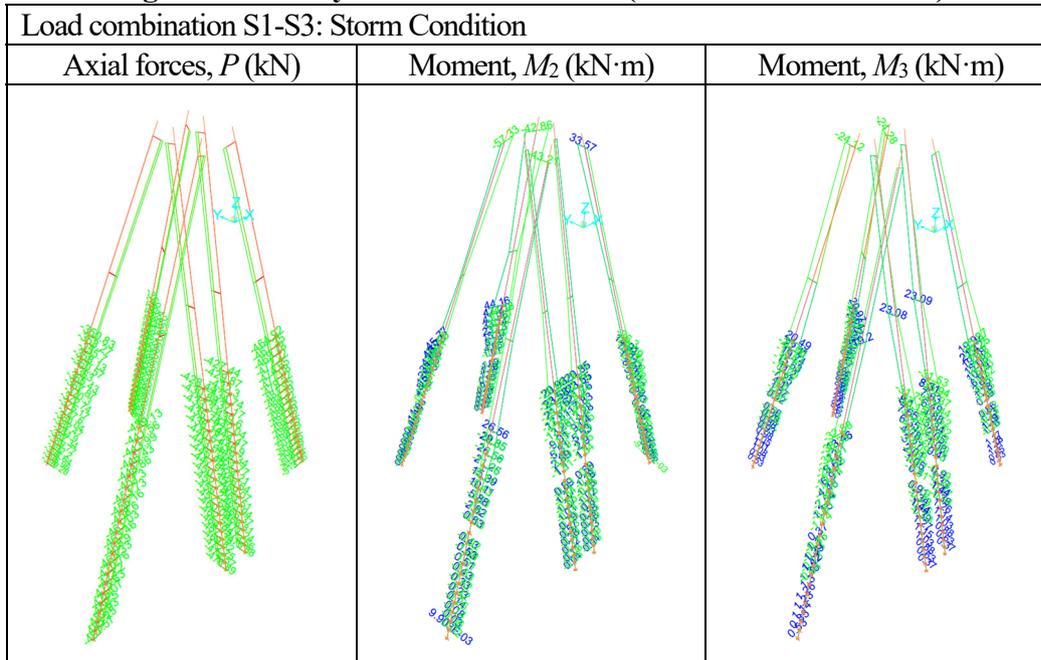


Notes:

The sign is reversed due to the analysis program setting

### 4) Earthquake Condition

**Table 2.8- Sectional Force (E1-E2: Earthquake Condition)**

| Pile Row | Location | Sectional Force | | | Verification |
|---|---|---|---|---|---|
| | | $P$ | $M_2$ | $M_3$ | |
| | | kN | kN·m | kN·m | ≤ 1.0 |
| P01 | Max P | -371.6 | 0.0 | 233.5 | 0.234 |
| | Min P | 1,036.8 | 0.0 | 0.0 | 0.233 |
| | Max $M_2$ | 41.6 | 203.1 | 104.4 | 0.179 |
| | Min $M_2$ | 767.6 | -215.8 | -84.2 | 0.345 |
| | Max $M_3$ | -371.6 | 0.0 | 233.5 | 0.234 |
| | Min $M_3$ | 947.5 | 0.0 | -255.1 | 0.403 |
| P02 | Max P | -371.6 | 0.0 | 233.5 | 0.234 |

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
|  | Min P | 1,036.8 | 0.0 | 0.0 | 0.233 |
|  | Max M$_2$ | 41.6 | 203.1 | 104.4 | 0.179 |
|  | Min M$_2$ | 767.6 | -215.8 | -84.2 | 0.345 |
|  | Max M$_3$ | -371.6 | 0.0 | 233.5 | 0.234 |
|  | Min M$_3$ | 947.5 | 0.0 | -255.1 | 0.403 |
| P03 | Max P | -264.2 | 382.3 | 178.4 | 0.357 |
|  | Min P | 998.3 | 0.0 | 0.0 | 0.229 |
|  | Max M$_2$ | -264.2 | 382.3 | 178.4 | 0.357 |
|  | Min M$_2$ | 912.1 | -480.1 | -221.1 | 0.602 |
|  | Max M$_3$ | -264.2 | 382.3 | 178.4 | 0.357 |
|  | Min M$_3$ | 912.1 | -480.1 | -221.1 | 0.602 |
| P04 | Max P | -143.3 | 410.7 | 180.3 | 0.357 |
|  | Min P | 1,135.2 | 0.0 | 0.0 | 0.260 |
|  | Max M$_2$ | -143.3 | 410.7 | 180.3 | 0.357 |
|  | Min M$_2$ | 1,033.1 | -451.7 | -219.2 | 0.610 |
|  | Max M$_3$ | -143.3 | 410.7 | 180.3 | 0.357 |
|  | Min M$_3$ | 1,033.1 | -451.7 | -219.2 | 0.610 |
| P05 | Max P | -326.8 | 80.7 | 192.5 | 0.209 |
|  | Min P | 1,091.7 | 0.0 | 0.0 | 0.250 |
|  | Max M$_2$ | 209.6 | 624.6 | 31.4 | 0.513 |
|  | Min M$_2$ | 429.5 | -714.8 | -108.9 | 0.636 |
|  | Max M$_3$ | -326.8 | 80.7 | 192.5 | 0.209 |
|  | Min M$_3$ | 989.6 | -78.0 | -244.8 | 0.418 |
| P06 | Max P | -326.8 | 80.7 | 192.5 | 0.209 |
|  | Min P | 1,091.7 | 0.0 | 0.0 | 0.250 |
|  | Max M$_2$ | 169.5 | 634.4 | 31.3 | 0.511 |
|  | Min M$_2$ | 389.4 | -705.0 | -109.0 | 0.620 |
|  | Max M$_3$ | -326.8 | 80.7 | 192.5 | 0.209 |
|  | Min M$_3$ | 989.6 | -78.0 | -244.8 | 0.418 |

Notes:

$M_2$, $M_3$: moment in local axis 2 and axis 3 direction of the pile cross section

**Figure 2.11- Analysis of Sectional Forces (E1-E2: Earthquake Condition)**

| Load combination E1-E2: Earthquake Condition | | |
| --- | --- | --- |
| Axial forces, $P$ (kN) | Moment, $M_2$ (kN·m) | Moment, $M_3$ (kN·m) |
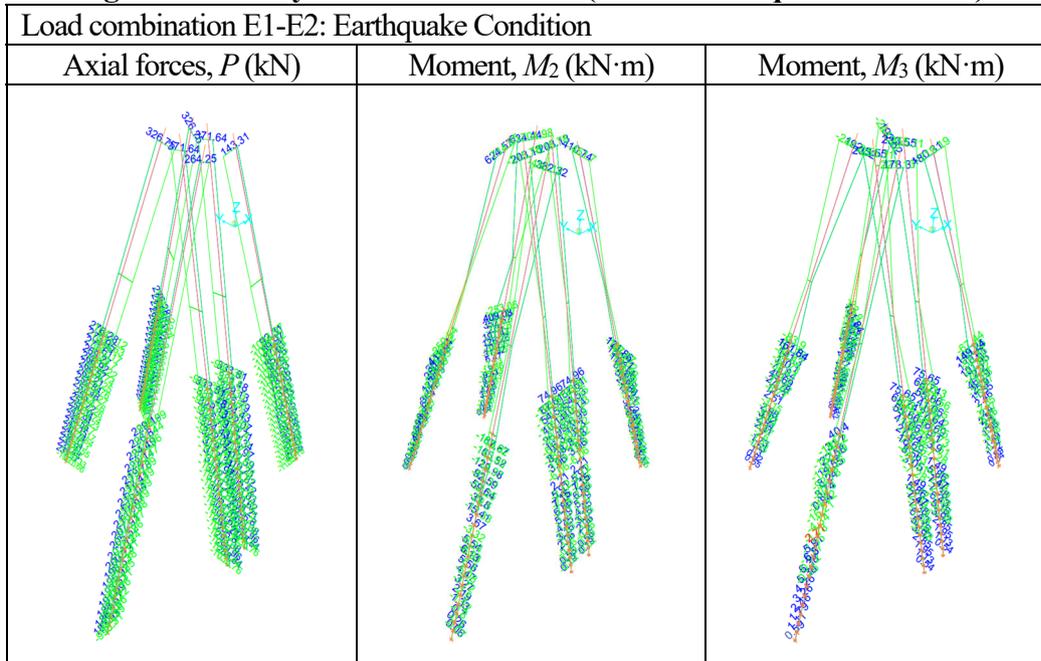


## 2-4. Verification of Pile Stress and Bearing Capacity

### (1) Verification of Pile Stress

**Table 2.9- Summary of Pile Stress (Berthing and Mooring Conditions)**

| Content | Equation | Unit | Conditions | |
|---|---|---|---|---|
| | | | Berthing | Mooring |
| Axial Force | $P_k$ | kN | 981.1 | 1,238.2 |
| Bending Moment | $M_{2k}$ | kN·m | 178.5 | 14.4 |
| Bending Moment | $M_{3k}$ | kN·m | 334.0 | 240.7 |
| Outer Diameter | $D_o$ | mm | 900 | 900 |
| Wall Thickness | $t$ | mm | 12 | 12 |
| Corrosion Thickness | $\delta$ | mm | 1.5 | 1.5 |
| Buckling Length | $l$ | m | 21.12 | 21.12 |
| Outer Diameter after Corrosion | $D = D_o - 2 \cdot \delta$ | mm | 897 | 897 |
| Inner Diameter | $d = D_o - 2 \cdot t$ | mm | 876 | 876 |
| Cross Section Area | $A = \pi(D^2 - d^2)/4$ | m² | 0.02924 | 0.02924 |
| Section Inertial Moment | $I = \pi(D^4 - d^4)/64$ | m⁴ | 0.00287 | 0.00287 |
| Section Modulus | $Z = \pi(D^4 - d^4)/(32D)$ | m³ | 0.00641 | 0.00641 |
| Inertial Radius | $r = \mathrm{sqrt}(I/A)$ | m | 0.313 | 0.313 |
| Slenderness Ratio | $l/r$ | - | 67.38 | 67.38 |
| Axial Tensile Stress | $\sigma_{tk} = P_k/A$ | kN/m² | --- | --- |
| Axial Compressive Stress | $\sigma_{ck} = P_k/A$ | kN/m² | 33,550 | 42,342 |
| Bending Tensile Stress | $\sigma_{btk} = -\mathrm{sqrt}[(M_{2k}/Z)^2 + (M_{3k}/Z)^2]$ | kN/m² | -59,118 | -37,642 |
| Bending Compressive Stress | $\sigma_{bck} = \mathrm{sqrt}[(M_{2k}/Z)^2 + (M_{3k}/Z)^2]$ | kN/m² | 59,118 | 37,642 |
| Tensile Yield Stress | $\sigma_{tyk}$ | kN/m² | 235,000 | 235,000 |
| Compressive Yield Stress | $\sigma_{cyk}(=235,000-1.4(l/r-19))$ | kN/m² | 167,268 | 167,268 |
| Bending Yield Stress | $\sigma_{byk}$ | kN/m² | 235,000 | 235,000 |
| Reduction Factor | $red = \sigma_{cy}/\sigma_y$ | - | 0.712 | 0.712 |
| When $P_k$ is compression | $S_k = \sigma_{ck}/red + \sigma_{bck}$ | kN/m² | 106,253 | 97,129 |
| | $R_k = \sigma_{byk}$ | kN/m² | 235,000 | 235,000 |
| Partial Factor for Load Term | $\gamma_S$ | - | 1.00 | 1.00 |
| Partial Factor for Resistance Term | $\gamma_R$ | - | 1.00 | 1.00 |
| Adjustment Factor | $m$ | - | 1.67 | 1.67 |
| Load Term | $S_d = \gamma_S \cdot S_k$ | kN/m² | 106,253 | 97,129 |
| Resistance Term | $R_d = \gamma_R \cdot R_k$ | kN/m² | 235,000 | 235,000 |
| Verification | $m \cdot S_d/R_d$ | - | 0.755 | 0.690 |
| Judgement | $m \cdot S_d/R_d \leq 1.0$ | | OK | OK |

**Table 2.10- Summary of Pile Stress (Storm and Earthquake Conditions)**

| Content | Equation | Unit | Conditions | |
|---|---|---|---|---|
| | | | Storm | Earthquake |
| Axial Force | $P_k$ | kN | 435.1 | 429.5 |
| Bending Moment | $M_{2k}$ | kN·m | 29.1 | 714.8 |
| Bending Moment | $M_{3k}$ | kN·m | 13.3 | 108.9 |
| Outer Diameter | $D_o$ | mm | 900 | 900 |
| Wall Thickness | $t$ | mm | 12 | 12 |
| Corrosion Thickness | $\delta$ | mm | 1.5 | 1.5 |
| Buckling Length | $l$ | m | 21.12 | 21.12 |
| Outer Diameter after Corrosion | $D = D_o - 2 \cdot \delta$ | mm | 897 | 897 |

| Inner Diameter | $d = D_o - 2 \cdot t$ | mm | 876 | 876 |
|---|---|---|---|---|
| Cross Section Area | $A = \pi(D^2 - d^2)/4$ | m² | 0.02924 | 0.02924 |
| Section Inertial Moment | $I = \pi(D^4 - d^4)/64$ | m⁴ | 0.00287 | 0.00287 |
| Section Modulus | $Z = \pi(D^4 - d^4)/(32D)$ | m³ | 0.00641 | 0.00641 |
| Inertial Radius | $r = \text{sqrt}\,(I/A)$ | m | 0.313 | 0.313 |
| Slenderness Ratio | $l/r$ | - | 67.38 | 67.38 |
| Axial Tensile Stress | $\sigma_{tk} = P_k/A$ | kN/m² | --- | --- |
| Axial Compressive Stress | $\sigma_{ck} = P_k/A$ | kN/m² | 14,879 | 14,687 |
| Bending Tensile Stress | $\sigma_{btk} = -\sqrt{[(M_{2k}/Z)^2 + (M_{3k}/Z)^2]}$ | kN/m² | -4,995 | -112,871 |
| Bending Compressive Stress | $\sigma_{bck} = \sqrt{[(M_{2k}/Z)^2 + (M_{3k}/Z)^2]}$ | kN/m² | 4,995 | 112,871 |
| Tensile Yield Stress | $\sigma_{tyk}$ | kN/m² | 235,000 | 235,000 |
| Compressive Yield Stress | $\sigma_{cyk} = 235,000 - 1.4(l/r - 19)$ | kN/m² | 167,268 | 167,268 |
| Bending Yield Stress | $\sigma_{byk}$ | kN/m² | 235,000 | 235,000 |
| Reduction Factor | $red = \sigma_{cy}/\sigma_y$ | - | 0.712 | 0.712 |
| When $P_k$ is compression | $S_k = \sigma_{ck}/red + \sigma_{bck}$ | kN/m² | 25,898 | 133,506 |
| | $R_k = \sigma_{byk}$ | kN/m² | 235,000 | 235,000 |
| Partial Factor for Load Term | $\gamma_S$ | - | 1.00 | 1.00 |
| Partial Factor for Resistance Term | $\gamma_R$ | - | 1.00 | 1.00 |
| Adjustment Factor | $m$ | - | 1.12 | 1.12 |
| Load Term | $S_d = \gamma_S \cdot S_k$ | kN/m² | 25,898 | 133,506 |
| Resistance Term | $R_d = \gamma_R \cdot R_k$ | kN/m² | 235,000 | 235,000 |
| Verification | $m \cdot S_d/R_d$ | - | 0.123 | 0.636 |
| Judgement | $m \cdot S_d/R_d \leq 1.0$ | | OK | OK |

## (2) Verification of Pile Bearing Capacity

The verification of the pile bearing capacity under seismic ground motion will be conducted refer to the Equation (1.3).

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

Where:

$m$ : adjustment factor

$S$ : characteristic value of load term (kN/m)

$R$ : characteristic value of resistance term (kN/m)

$\gamma_S$ : partial factor that is to be multiplied with the load term

$\gamma_R$ : partial factor that is to be multiplied with the resistance term

| Verification target | Type of piles | Partial factor to be multiplied with resistance term: $\gamma_R$ | Partial factor to be multiplied with load term: $\gamma_S$ | Adjustment factor: $m$ |
|---|---|---|---|---|
| Bearing capacity of the dolphin (variable situation for ship actions) | Pulling pile | - (1.00) | - (1.00) | 3.00 |
| | Pushing pile | - (1.00) | - (1.00) | 2.50 |
| Bearing capacity of the dolphin (variable situation for storm, and Level 1 earthquake ground motion) | Pulling pile | - (1.00) | - (1.00) | 2.50 |
| | Pushing pile (bearing pile) | - (1.00) | - (1.00) | 1.50 |
| | Pushing pile (friction pile) | - (1.00) | - (1.00) | 2.00 |

$$R_k = R_{pk} + R_{fk}$$

Where:

$R_{tk}$ : characteristic value of the pushing resistance force of a pile in its axial direction (kN)

$R_{pk}$ : characteristic value of the base resistance force of a pile (kN)

$R_{fk}$ : characteristic value of the skin friction force of a pile (kN)

## 1) Characteristic Value of End Bearing Resistance ($R_{pk}$)

$$R_{pk} = 300N \times \alpha \times A_p$$

Where:

$N_1 = 50$

$N_2 = (1.80 \times 20 + 1.80 \times 50) / (4 \times 0.90) = 35$

$N = (N_1 + N_2) / 2 = (50 + 35) / 2 = 42.5$

$A_p = \pi \times 0.90^2 / 4 = 0.636 \ (m^2)$

$\alpha$: pile toe closure ratio (Based on adjacent work experience, assumed as 0.60)

Therefore,

$R_{pk} = 300 \times 42.5 \times 0.6 \times 0.636 = 4,865.4 \ (kN)$

## 2) Characteristic Value of Skin Friction Resistance ($R_{fk}$)

$$R_{fk} = \sum \overline{\gamma_{fki}} \times A_{si} = 2 \sum \overline{N}_i \times A_{si}$$

Where:

$A_{si} = \pi \times 0.90 \times l_i = 2.827 \times l_i \ (m^2)$

- Frictional resistance of P01, P02

$R_{fk} = 2 \times 20 \times 17.34 \times 2.827 + 2 \times 50 \times 1.84 \times 2.827$
$\quad = 2,481.0 \ (kN)$

- Frictional resistance of P03, P04, P05, P06

$R_{fk} = 2 \times 20 \times 17.92 \times 2.827 + 2 \times 50 \times 1.90 \times 2.827$
$\quad = 2,563.5 \ (kN)$

## 3) Characteristic Value of Axial Resistance Force of Piles ($R_{tk}$)

- Axial resistance of P01, P02: (compression)

$R_{tk} = 4,865.4 + 2,481.0 = 7,346.4 \ (kN)$

- Axial resistance of P03, P04, P05, P06: (compression)

$R_{tk} = 4,865.4 + 2,563.5 = 7,428.9 \ (kN)$

## 4) Verification of Pile Bearing Capacity

The design values of axial resistance force are calculated from the base bearing resistance and skin friction resistance of each pile, and the verification results regarding pile capacity are indicated in Table 2.11.

**Table 2.11- Verification Results regarding the Bearing Capacity of Piles**

| Conditions | Pile | Case | $m$ | Load Term (kN) | Resistance Term (kN) | $m(S_d/R_d)$ | Judgement |
|---|---|---|---|---|---|---|---|
| Berthing | P01 | Push | 2.5 | --- | 7,346.4 | --- | O.K. |
| | P02 | Push | 2.5 | --- | 7,346.4 | --- | O.K. |
| | P03 | Push | 2.5 | 392.4 | 7,428.9 | 0.132 | O.K. |
| | P04 | Push | 2.5 | 801.6 | 7,428.9 | 0.270 | O.K. |
| | P05 | Push | 2.5 | 1003.2 | 7,428.9 | 0.338 | O.K. |
| | P06 | Push | 2.5 | 1073.4 | 7,428.9 | 0.361 | O.K. |
| | P01 | Pull | 3.0 | 165.6 | 2,481.0 | 0.200 | O.K. |
| | P02 | Pull | 3.0 | 492.8 | 2,481.0 | 0.596 | O.K. |
| | P03 | Pull | 3.0 | --- | 2,563.5 | --- | O.K. |
| | P04 | Pull | 3.0 | --- | 2,563.5 | --- | O.K. |
| | P05 | Pull | 3.0 | --- | 2,563.5 | --- | O.K. |
| | P06 | Pull | 3.0 | --- | 2,563.5 | --- | O.K. |
| Mooring | P01 | Push | 2.5 | 1036.0 | 7,346.4 | 0.353 | O.K. |
| | P02 | Push | 2.5 | 1036.0 | 7,346.4 | 0.353 | O.K. |
| | P03 | Push | 2.5 | 558.1 | 7,428.9 | 0.188 | O.K. |
| | P04 | Push | 2.5 | 1330.6 | 7,428.9 | 0.448 | O.K. |
| | P05 | Push | 2.5 | 209.8 | 7,428.9 | 0.071 | O.K. |
| | P06 | Push | 2.5 | 617.1 | 7,428.9 | 0.208 | O.K. |
| | P01 | Pull | 3.0 | --- | 2,481.0 | --- | O.K. |
| | P02 | Pull | 3.0 | --- | 2,481.0 | --- | O.K. |
| | P03 | Pull | 3.0 | 437.9 | 2,563.5 | 0.513 | O.K. |
| | P04 | Pull | 3.0 | --- | 2,563.5 | --- | O.K. |
| | P05 | Pull | 3.0 | 331.9 | 2,563.5 | 0.388 | O.K. |
| | P06 | Pull | 3.0 | 331.9 | 2,563.5 | 0.388 | O.K. |
| Storm | P01 | Push | 1.5 | 476.9 | 7,346.4 | 0.097 | O.K. |
| | P02 | Push | 1.5 | 475.9 | 7,346.4 | 0.097 | O.K. |
| | P03 | Push | 1.5 | 487.6 | 7,428.9 | 0.098 | O.K. |
| | P04 | Push | 1.5 | 513.4 | 7,428.9 | 0.104 | O.K. |
| | P05 | Push | 1.5 | 399.3 | 7,428.9 | 0.081 | O.K. |
| | P06 | Push | 1.5 | 398.9 | 7,428.9 | 0.081 | O.K. |
| | P01 | Pull | 2.5 | --- | 2,481.0 | --- | O.K. |
| | P02 | Pull | 2.5 | --- | 2,481.0 | --- | O.K. |
| | P03 | Pull | 2.5 | --- | 2,563.5 | --- | O.K. |
| | P04 | Pull | 2.5 | --- | 2,563.5 | --- | O.K. |
| | P05 | Pull | 2.5 | --- | 2,563.5 | --- | O.K. |
| | P06 | Pull | 2.5 | --- | 2,563.5 | --- | O.K. |
| Earthquake | P01 | Push | 1.5 | 1036.8 | 7,346.4 | 0.212 | O.K. |
| | P02 | Push | 1.5 | 1036.8 | 7,346.4 | 0.212 | O.K. |
| | P03 | Push | 1.5 | 998.3 | 7,428.9 | 0.202 | O.K. |
| | P04 | Push | 1.5 | 1135.2 | 7,428.9 | 0.229 | O.K. |
| | P05 | Push | 1.5 | 1091.7 | 7,428.9 | 0.220 | O.K. |
| | P06 | Push | 1.5 | 1091.7 | 7,428.9 | 0.220 | O.K. |
| | P01 | Pull | 2.5 | 371.6 | 2,481.00 | 0.374 | O.K. |
| | P02 | Pull | 2.5 | 371.6 | 2,481.00 | 0.374 | O.K. |
| | P03 | Pull | 2.5 | 264.2 | 2,563.50 | 0.258 | O.K. |

| | P04 | Pull | 2.5 | 143.3 | 2,563.50 | 0.140 | O.K. |
| --- | --- | --- | --- | --- | --- | --- | --- |
| | P05 | Pull | 2.5 | 326.8 | 2,563.50 | 0.319 | O.K. |
| | P06 | Pull | 2.5 | 326.8 | 2,563.50 | 0.319 | O.K. |

## 2-5. Verification of Structural Members

The superstructure of a dolphin is typically constructed using mass concrete. In designing the reinforcement of the superstructure, it is necessary not only to perform calculations based on stresses in accordance with Part 2, but also to ensure that the reinforcement meets the minimum reinforcement requirements for reinforced concrete structures.

## 2-6. Additional Notes

Dolphins are commonly used in tanker berths and LNG berths. Depending on the type of cargo handled, it may be necessary to ensure performance in accordance with relevant standards and/or regulations other than TCVN or OCDI 2020.

- End -